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30 July 2018

[redacted]

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RE Official information request CDHB 9873

We refer to your email dated 13 June 2018 requesting the following information under section 12 of the Official Information Act (the 'Act') from Canterbury DHB.

1. **Can you provide the most recent detailed engineering evaluations or seismic assessments for the Christchurch Women's Hospital and the Clinical Services Building?**

Please find attached as **Appendix 1** the Earthquake Seismic Assessment Report for Christchurch Women's Hospital October 2013 and **Appendix 2** the Detailed Seismic Assessment Report for the Clinical Services Building October 2013.

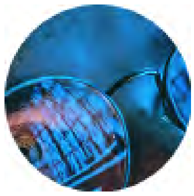
Please note these assessments were obtained under the previous assessment regime and the buildings may not have been reassessed in accordance with the current assessment regime (*"The Seismic Assessment of Existing Buildings, Technical Guidelines for Engineering Assessments, July 2017* Version 1 ("NZSEE, 2017")). We also note that earthquake remediation works may have since been completed, or are currently being undertaken, for these buildings. Please let us know if you have any further questions in respect of these specific buildings.

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 ten working days after your receipt of this response.

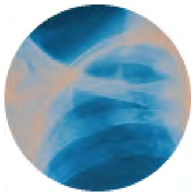
Yours sincerely

Carolyn Gullery
Executive Director
Planning, Funding & Decision Support



EARTHQUAKE SEISMIC ASSESSMENT REPORT

STRUCTURAL AND CIVIL ENGINEERS



CHRISTCHURCH HOSPITAL

REPORT 6 - CHRISTCHURCH WOMEN'S HOSPITAL

PREPARED FOR

CANTERBURY DISTRICT HEALTH BOARD

106186.72

REV 2 - 7 OCTOBER 2013

CANTERBURY DISTRICT HEALTH BOARD

CHRISTCHURCH HOSPITAL

REPORT 6 – CHRISTCHURCH WOMEN'S HOSPITAL

Prepared For:

CANTEBURY DISTRICT HEALTH BOARD

Date: 7 October 2013
Project No: 106168.72
Revision No: 2

Prepared By:

Didier Pettinga
PROJECT ENGINEER

Reviewed By:

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TECHNICAL DIRECTOR

Holmes Consulting Group Limited
Christchurch Office

REPORT ISSUE REGISTER

DATE	REV. NO.	REASON FOR ISSUE
04/05/12	1	First issue incl. initial slab crack inspection
7/10/13	2	Incl. developed slab/interspan inspections, rebar tests, on going inspections

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Holmes Consulting Group has been engaged by Canterbury District Health Board to complete a full structural review of the Christchurch City 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 Canterbury District Health Board's Christchurch Women's Hospital, as a result of the series of Earthquakes that includes the Darfield Earthquake that struck at 4:36am on 4th September 2010, the Lyttelton Earthquake at 12.51 pm on the 22nd of February 2011, the June 13th 2011 (2:20pm) earthquake and December 23rd 2011 (1:58pm) event. The Darfield Earthquake produced force demands in the isolator system equal to Maximum Considered Earthquake (MCE for an IL2 building), or ultimate limit state (ULS), conditions for an Importance Level 4 building. The Lyttelton Earthquake by comparison did not induce such large horizontal forces, but likely took the structure through larger displacement demands at the isolator level. Consequently it is important that a full evaluation is performed.

The information available for the review included: the original structural drawings, the levels survey, the façade damage survey and the geotechnical report.

Christchurch Women's Hospital was designed in 2001/2002 and construction was completed in 2004. The building is adjacent to the west end of the Parkside building complex, with a 550 mm seismic gap between the structures. The two buildings are connected via drop-in plates at each of the floors from Lower Ground to Level Four.

The primary structure consists of precast pre-stressed floor ribs (spanning NS) and 100 mm thick topping slab on timber infill planks. The floor is supported on precast beams (EW) that span onto cast insitu interior and exterior columns. The lateral force resisting system in the NS direction from the lower ground floor to underside of level three is a dual system using reinforced concrete moment-frames at the ends of the building and eccentric K-braced frames forming the sides of the stair/service shafts. From Level Three to the roof the reinforced concrete moment-frame forms the lateral force resisting system. The EW direction lateral system is full height moment-frames on the north and south faces of the building. The entire building is supported both for vertical gravity loads and lateral seismic shears at the underside of the Lower Ground floor on lead-rubber isolator bearings that are connected with a grid of stiff transfer beams.

The stair, lift and service shafts are framed with structural steel beams and posts, with Hi-bond steel deck and concrete topping forming the floors in these areas. The staircases are precast concrete seated on steel beams and tied into the floor topping slabs with reinforcement.

Above Level Six there are two mechanical/service floors, covered by a structural steel portal frame and lightweight roof system.

The block is currently designated as an Importance Level 4 building. Comparison of the original seismic design spectrum against the current code design spectrum indicates that the structure can be considered to have 100% of NBS. However this will need to be reviewed once the revised Christchurch seismic demands are published in the near future.

In general the structural damage above the isolator level is limited to cracking of the floor slab, intermittent cracking of the precast floor rib units and cracking of some stair landings. In some locations the cracking of the slabs is consistent with shrinkage crack patterns that would have been pre-existing, however their extent and width may have been increased as a result of earthquake movements. In other locations the slab cracks are clearly new and a result of the earthquake induced shear forces. Some minor cracks in the concrete columns that form the lateral force resisting moment-frames at the ends of the building were observed, however they did not indicate that significant ductile action had occurred in the upper levels. Similarly the structural steel braced frames in the north-south direction of the building showed no signs of high demand.

Observations in the basement showed there were a number of locations that developed cracks as a result of the building movement and forces in the transfer grid forming the Lower Ground level. Damage in the transfer beams mainly related to the bending demands induced by the suspended elevator shafts on the beams, as well as the post-tensioned tie-downs at selected locations around the perimeter of the building. Extensive cracks were noted in the precast concrete ribs forming the Lower Ground floor joists that span between the transfer beams. These cracks ranged in size from 0.4 mm to 1.5 mm, and were a result of the infill detail used in the region of the seating.

Evaluation of the structural drawings and observations from site do not suggest that any critical structural weaknesses exist in the lateral force resisting system. However the cracks in the precast ribs forming the Lower Ground floor can be considered a significant weakness requiring immediate attention.

A further critical structural weakness is the detailing of the stair mid-landings. Based on the structural drawings it appears that the preferred allowance for relative movement between the floors levels can not be accommodated by the landing and detailing used, and as such will need to be remediated to ensure that no further damage occurs under large earthquake demands.

Based on the following description of observed damage and structural weaknesses, the majority of the remediation work required for earthquake induced damage will centre on epoxy injection of cracks in the floor slabs at most levels. Some minor injection may be required in the concrete columns and beam ends around the perimeter of the building. Non-destructive testing of the slab reinforcement at selected locations has indicated limited or negligible strain-hardening in the bars. Based on these findings, and our back-analysis of the frame behaviour, additional retrofit strengthening to tie the slab across the frames is not considered necessary.

A significant portion of the epoxy injection in the basement has already been carried out, but is noted here for reference.

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

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

1. INTRODUCTION

Holmes Consulting Group has been engaged by Canterbury District Health Board to complete a full structural review of the Christchurch City 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.

The 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 and is referred to as required in the specific building reports.

1.1 SCOPE OF WORK

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

This report covers the structural damage sustained by the Canterbury District Health Board's Christchurch Women's Hospital, as a result of the series of Earthquakes that includes the Darfield Earthquake that struck at 4:36am on 4th September 2010, the Lyttelton Earthquake at 12.51 pm on the 22nd of February 2011, the June 13th 2011 (2:20pm) earthquake and December 23rd 2011 (1:58pm) event. The Darfield Earthquake produced force demands in the isolator system equal to Maximum Considered Earthquake (MCE for an IL2 building), or ultimate limit state (ULS), conditions for an Importance Level 4 building. The Lyttelton Earthquake by comparison did not induce such large horizontal forces, but likely took the structure through larger displacement demands at the isolator level.

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

1.2 LIMITATIONS

Our observations have been restricted to structural aspects only. Waterproofing elements, electrical and mechanical equipment, fire protection and safety systems, service connections, water supplies and sanitary fittings have not been inspected or reviewed. Secondary elements, such as windows and fittings, have not generally been reviewed. This report is considered a live document and will be updated throughout the course of the project with the final report issued once the repairs and/or strengthening of the building have been completed.

2.1 BUILDING FORM

Christchurch Women's Hospital was designed in 2001/02 and finished construction in 2004. It was designed as a Category I structure as defined in NZS4203:1992 [3]. NZS 1170.0:2002 [4] redefines the building categories such that post disaster structures, that were previously Category I, are now referred to as Importance Level 4 (IL4)

The building is adjacent to the west end of the Parkside building complex, with a 550 mm seismic gap between the structures. The two buildings are connected by drop-in plates at each of the floors from the Basement to Level Four.

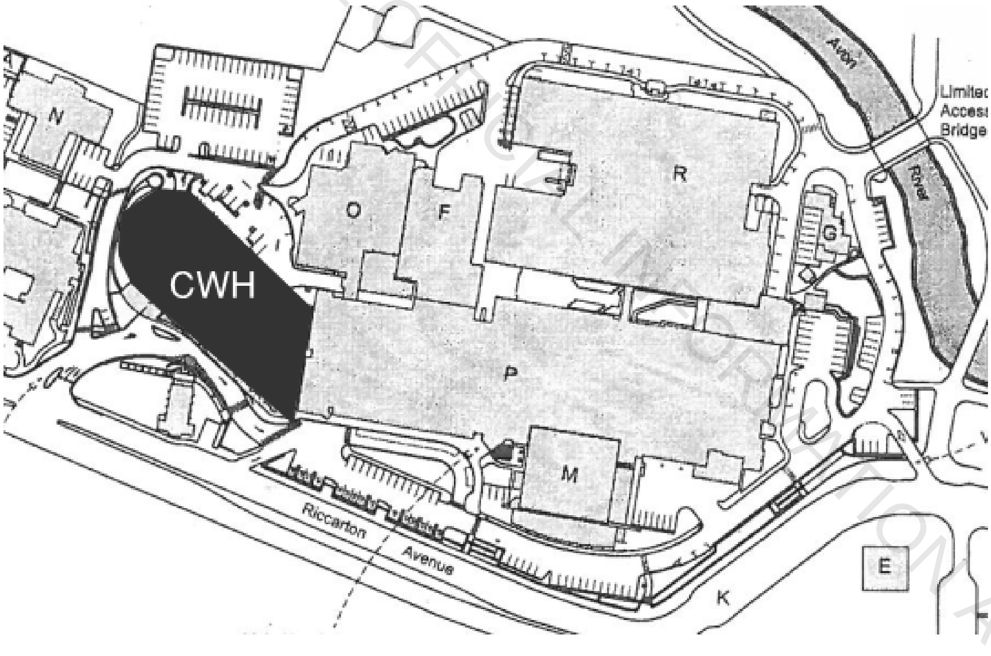

Figure 2-1: Location of Christchurch Women's Hospital

The primary structure consists of precast pre-stressed floor ribs (spanning north-south, NS) and 100 mm thick concrete topping slab on timber infill planks. The floor is supported on precast beams (spanning east-west, EW) that span onto cast insitu interior and exterior columns. The lateral force resisting system in the NS direction from the Lower Ground floor to underside of Level Three is a dual system using reinforced concrete moment-frames at the ends of the building and eccentric K-braced frames forming the sides of the stair/service shafts. The EW direction lateral system is full height moment-frames on the north and south faces of the building. The entire building is supported both for vertical gravity loads and lateral seismic shears at the underside of the lower ground floor on lead-rubber isolator bearings that are connected with a grid of stiff transfer beams.

The stair, lift and service shafts are framed with structural steel beams and posts, with *Hi-bond* steel deck and concrete topping forming the floors in these areas. The staircases are precast concrete seated on steel beams and tied into the concrete floor topping slabs with reinforcement.

Above Level Six there are two mechanical/service floors, covered by a structural steel portal frame and lightweight roof system.

Figure 2-2: Photo of Christchurch Women's Hospital

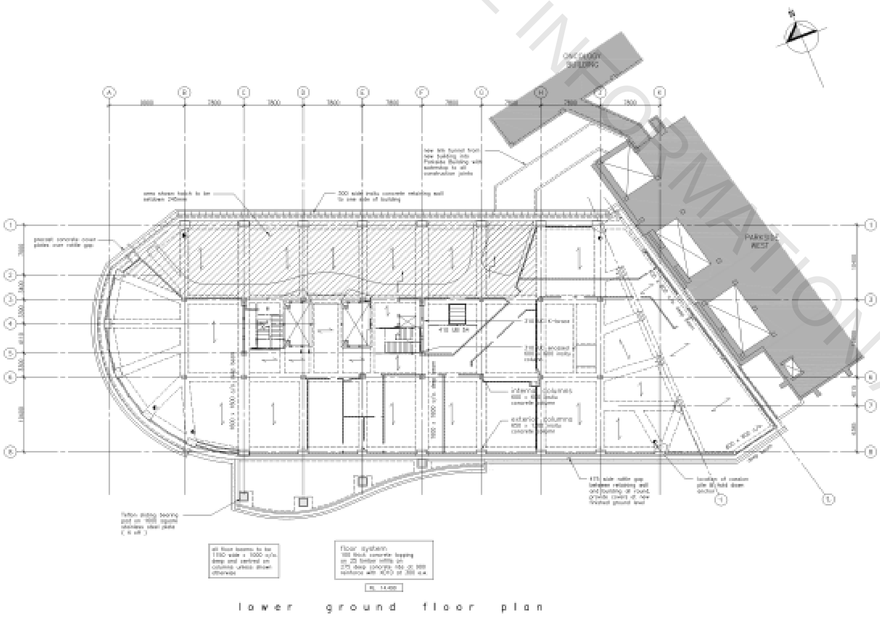

Figure 2-3: Structural plan of CWH

2.2 PRE-EARTHQUAKE BUILDING CAPACITY

Christchurch Women's Hospital was designed following NZS 3101:1995 [5] (concrete), NZS 3404:1997 [6] (steel) and NZS 4203:1992 [3] (loadings), the predecessor to the current structural seismic design actions code NZS 1170.5:2004 [7]. The design did however acknowledge the draft version of the current loading code, called DR902: Draft New Zealand Loadings Standard [8]. Allowance was made by comparing the ultimate limit state design accelerations from both NZS4203:1992 and DR902. In doing so it is noted that the draft standard used a 2000 year return period for the ULS design of Category II (redefined from Category I) buildings with a 50 year design life, while NZS4203:1992 used a return period of 1000 years for a Category I (post disaster) building. Because of the soil conditions and because the 2000 year return period earthquake was not defined by the legal standard at the time of design, therefore a site specific design acceleration spectrum for the 2000 year return period event was generated by Tonkin & Taylor (2001) [9].

NZS 1170.0:2002 redefines the building categories such that post disaster structures are now referred to as Importance Level 4 (IL4) with a 2500 year return period. Comparing the 2000 year and 2500 year return periods the difference in design acceleration is less than 1.5%, which is relatively insignificant.

The response of the building to ground motion is significantly more complicated than standard structures designed to sustain seismic demands through yielding structural deformation over the building height. The presence of the isolator plane below the Lower Ground floor produces a phased response defined by:

1. Building response before the isolators reach their yield base-shear. In this phase the structure above the isolator level deforms elastically with limited displacement in the isolators themselves.

2. Yield of the isolators but elastic response of the building above the Lower Ground floor. Once the isolators yield they are significantly more flexible than the structural frame above. The majority of the building displacement demands are therefore concentrated at the isolator level, while the structure above experiences very limited deformation.

3. Continued yield of the isolators with minor yield of the reinforced concrete frames and structural steel frames. If the seismic demands continue to increase then additional forces may be generated in the upper structure that induce a limited amount of yield in the reinforced concrete and steel frames.

Without in-depth numerical modelling and analyses to follow the step-by-step response through the time-history of the earthquake, it is not possible to accurately predict the full yielding response of the building. However general indications of the likely building response and performance can be obtained by comparing the recorded ground motion acceleration spectra and the original design spectrum with the expected periods of vibration of the structure at each of the three phases noted above.

The earthquake shaking experienced at the hospital site is outlined in the Base Report [1] for the Christchurch Hospital Campus.

2.2.1 Comparison of Earthquake Demand

Reference to the original design documentation allows a comparison between the original site specific design spectrum provided by Tonkin & Taylor [8] and the current NZS 1170.5:2004 design spectrum using the factors give in Table 2-1. Figure 2-4 shows the site specific spectrum at damping levels of 30% and 22% which reflects the energy absorption by the isolators at the originally defined Design-Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE) demand levels respectively. This is compared to the NZS1170.5:2004 spectrum at the same levels of damping.

Table 2-1: NZS1170.5:2004 Design spectrum factors

Design Life:	50 years
Zone factor, Z:	0.30
Subsoil Class:	D
Importance Level:	4
Risk Factor, R:	1.8
Ductility, μ:	1.25
Structural Performance Factor, $S_{p:}$	1.0

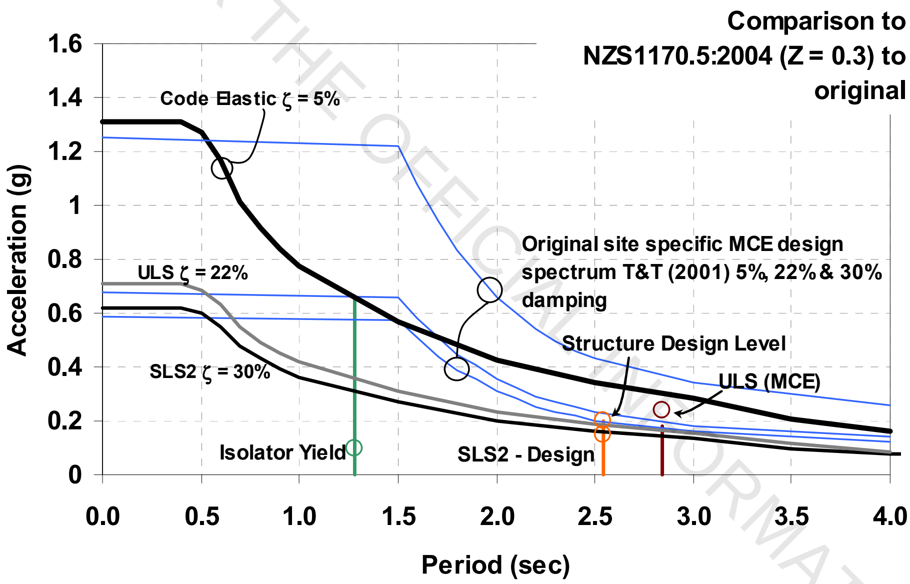

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Figure 2-4: NZS 1170:5:2004 acceleration spectra at damping levels of 5%, 30% and 22% which correspond to Serviceability (SLS), Design Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE) respectively. Circles indicate original design forces for each limit state.

The key points to draw from Figure 2-4 are that the original design spectrum exceeds the current NZS1170.5:2004 spectrum for periods over 0.6 seconds. The fundamental period of the structure prior to the isolators yielding is 1.28 seconds. Once the isolators have yielded the effective period of the building becomes 2.54 seconds under DBE (now referred to as SLS2 under current code definition) displacements, and 2.84 seconds under MCE (now referred to as Ultimate Limit State, ULS under current code definition) displacements. Subsequent discussion will refer to current code definitions of SLS2 and ULS for consistency.

Currently there is no design spectrum for Christchurch that includes structural periods of 1.5 seconds. Therefore the previous code design spectrum has been used to provide an idea of spectral demand. From this the building can be considered to have capacity up to 100% of New Building Standard. Once a design spectrum has been confirmed for Christchurch the building capacity will need to be re-evaluated against this updated demand.

It is noted in the design features report for this building that the as-designed overstrength of the structure resulted in governing design forces that were capped by the ULS (MCE) level demands. The implication of this is that while the structure was designed assuming a design ductility demand of 1.4 (SLS2) and 1.8 (ULS), which correspond to minor amounts of yielding, the actual building behaviour would be essentially elastic. Thus while the lateral force-resisting system is capacity designed to have a weak-beam strong-column ductile mechanism above the isolator level, it is expected that there would be minimal damage to the structure.

3. POST EARTHQUAKE BUILDING CONDITION

This section covers the structural damage sustained by the Christchurch Women's Hospital building as a result of the Darfield Earthquake (4th September 2010) and the Lyttelton Earthquake (22nd of February 2011), as well as the subsequent aftershock sequence in the Christchurch region. Sections 3.1 and 3.2 provided specific comments on probable building response during each of these events.

3.1 THE DARFIELD EARTHQUAKE

The Darfield earthquake had stronger ground motion in the north-south direction (N01W), than in the east-west (S89W). Figure 3-1 shows this response when comparison of the record spectra is made between (a) and (b). It is not possible to interpret the exact demands that the building experienced from these spectra, and in particular the behaviour of the building after the isolator units yield can only be generally interpreted. To this extent the indications are that the isolators would have yielded in both building principal axis directions when the structural period was approximately 1.28 seconds and apparent damping approximately 5%. Following the isolator yield the effective period of the building moved to 2.5 seconds at which point the next performance level is the Ultimate Limit State.

At an SLS2 level the seismic demand shown by the "SLS2 ζ = 30%" curves ("ζ" represents damping) suggest that the building could have developed the SLS2 and DBE/ULS isolator base-shear levels and the design base-shear demand expected for upper structure. However as noted in Section 2.2.1 the as-designed overstrength has led to a structure that responds in an essentially elastic manner up to ULS levels. Thus even with these near-design level forces it is unlikely that significant structural damage would have occurred in the seismic system. Observations of the structural members suggest that this is in fact the case as the damage noted in the log does not correspond to significant demands in the upper structure.

The east-west demands were comparatively low with respect to the north-south demands. Beyond the isolator yield point, the spectra at 30% damped (SLS2) and 22% damped (ULS/MCE) were below the design level base shears which would indicate that the upper structure was not subject to significant forces along the length of the building.

The large demands in the north-south response indicate that the isolators would have accommodated significant displacements, a point reflected in the displacement spectra for each direction of motion. The displacement demands on the isolators were also indicated by the permanent offset of the isolator top-plate from the bottom-plate of 25 mm, in the north direction [14]. Displacement induced damage to non-structural components at the isolator level was also noted at some locations around the perimeter of the building. In particular the seismic gap between the Parkside and Women's Hospital, and the "moat" or "rattle-space" around the exterior of the building suffered some damage where coverings impacted the external pit walls, though displacements are not believed to have reached design levels.

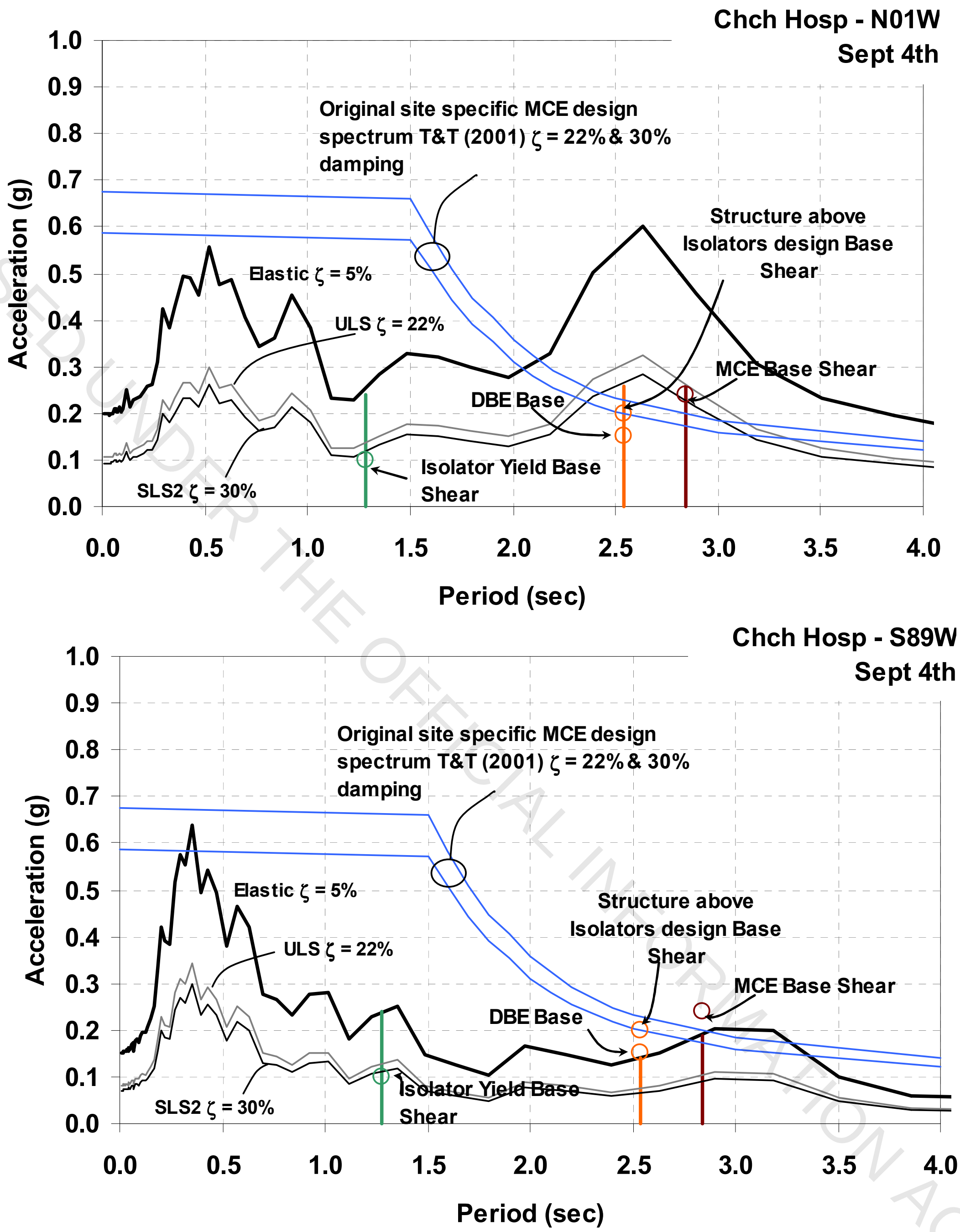


Figure 3-1: N01W and S89W components of the CHHC spectra for the Darfield Earthquake. Shown are the original site specific design spectrum curves for 30% and 22% damping levels corresponding to DBE and MCE earthquake events. The design base shears for each performance level are shown at their respective periods.

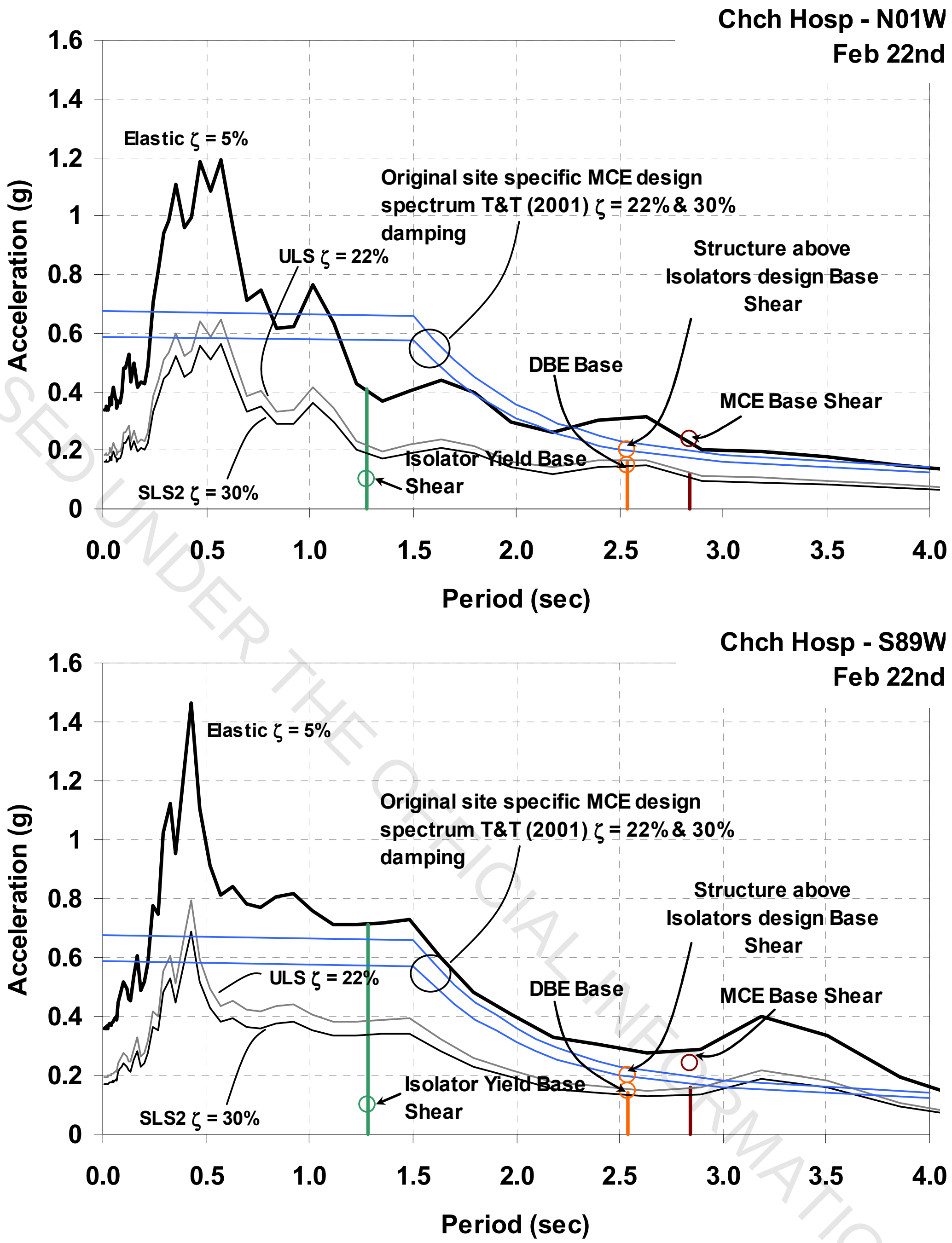


Figure 3-2: N01W and S89W components of the CHHC spectra for the Lyttelton Earthquake. Shown are the original site specific design spectrum curves for 30% and 22% damping levels corresponding to DBE and MCE earthquake events. The design base shears for each performance level are shown at their respective periods.

3.2 THE LYTTELTON EARTHQUAKE

The apparent spectral response to the February 22nd earthquake is markedly different to the September 4th event. Similar to the discussion in Section 3.1 the sequential response of the building can be approximately interpreted from

Figure 3-2(a) & (b).

The comparison of design base shear values to the appropriately damped acceleration spectra suggests that the isolators would have yielded in both directions and could have then generated SLS2 level base shears, but not ULS level shears. Also it seems that this event did not induce ductility demand on the structure above the isolation level.

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 approximately 10 seconds. Rupture of the Alpine Fault is expected to contain 60 seconds or more of strong motion.

3.3 PRELIMINARY INVESTIGATIONS

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 [10]
- damage observed after the Darfield Earthquake
- damage observed after the Lyttelton Earthquake

In conjunction with a review of the structural drawings and previous seismic assessment work associated with this building the following areas were identified for potential damage;

- flexural cracking of the columns/piers
- shear cracking of beams and columns
- damage to the active links of the steel braced frames
- damage to the brace/beam/column joints of the steel brace frames
- damage to plant-room structure
- possible pounding at seismic joint to the Parkside building and perimeter "moat" at ground level.
- floor slab cracking
- damage to the precast stairs and cast-in-place landings
- damage to precast floor ribs

Preliminary observations were carried out following the 4th September 2010 and 22nd February 2011 earthquakes. These identified the following primary areas of deformation or damage;

- Permanent displacement of the isolator bearing pads
- Finishes damage around seismic joints at the isolator level (Lower Ground floor)
- Cracking of the exterior precast concrete façade panels

In general, the building appears to have behaved in the manner anticipated by the original design intent, with the majority of the seismic deformation occurring in the isolators in the basement and only limited structural and non-structural deformation above the isolator plane.

3.4 DETAILED OBSERVATIONS

A detailed assessment of the building was carried out in February 2012, with an initial inspection followed by additional inspections as particular areas of the structure could be opened up for viewing.

A full record of the observations from these inspections is provided in Section 4, with reference plans describing the location labelling used, included in Appendix B. A full photographic record of the observations is available electronically on request.

3.5 SUMMARY OF BUILDING DAMAGE

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

In general there has been very little structural damage to the vertical elements of building as a result of the earthquake demands placed on the building as a result of the Canterbury earthquakes and aftershock sequence. This is in keeping with the philosophy behind the seismic base-isolator system incorporated in the basement that concentrates the earthquake induced deformations to the isolated level of the building. The isolator pads themselves show no signs of excessive deformation and similarly the connections of the isolators to the foundation raft, and to the transfer beam grid above, do not show any damage.

Some diagonal cracking has been observed in the transfer beams (forming part of the stiff grid of the Lower Ground floor) that support the elevator pit and span back to adjacent isolator bearings. Given that cracks where not extensively observed in other transfer beams, it is possible that this cracking has occurred as a result of the elevator shaft mass being vertically accelerated during the February 22nd earthquake.

The only other transfer beam locations at the Lower Ground floor that showed signs of movement were over the tension tie-downs located near the perimeter of the western end of the building. These tie-downs comprise of post-tensioned cables dead-end anchored into the ground below the level of the raft, and live-end anchored into the top of the transfer beams making up the Lower Ground floor system. The transfer beams inspected at gridlines A.5/7, B/7.5 and B/1 have a developed cracks of 0.2-0.3 mm in width. Further investigation is needed to assess whether the same damage exists in the transfer beams at J/1 and K/8 as these were not accessible during the current inspection phase. Given this is considered an exterior environment these cracks will need epoxy injection if 0.2 mm or wider. One anchor head has been inspected at the Lower Ground floor and did not show indications of loss of anchorage strength. Comparison of the transfer beams at this location and elsewhere suggests that the other anchorages have not suffered damage.

Inspection of the caissons containing the tie-down anchors found water had collected on top of the concrete plug in some locations. Further inspection of the tie-down encasement found water inside the pipe housing the post-tensioned tie-down strands, but not within the post-tensioned cable strand sheaths. A remediation detail for the encasement pipes, and concrete plug has been provided and indications are that the water ingress has been mitigated (see Appendix C. In the south-west shaft (referred to as Shaft 2 in site reports) some further water ingress was noted although at markedly slower rates, for which a further instruction to apply a concrete top seal over a hydrophilic-type perimeter strip that is embedded in a Xypex bonding agent (to help bond with the steel caisson) been issued.

The precast concrete floor ribs that make up the Lower Ground floor and are observable from the basement have shown a consistent amount of damage in the western half of the building.

Single cracks have developed in a number of the units near the seating with cracks widths ranging up to 1.5 mm in size. A remediation Site Instruction and detail has been issued for the locations considered to be critical, where epoxy injection will not be sufficient to ensure the units perform with their original strength. In locations at other floor levels where such damage is observed with crack widths of 0.5 mm or more (in internal spaces), it is suggested that the cracks be epoxy injected and their location noted as part of a full building survey. Where the ribs are in an external environment we recommend that cracks over 0.5mm in width are epoxy injected, and all other cracks less than 0.5mm be painted over with a flexible industrial paint coating.

No inelastic deformation of the structural steel braced-frames was observed, however one of the concrete stubs providing connection of the adjacent concrete floor to the braced-frame was damaged with a corner of concrete having spalled off. Not all stub locations were observable due to the mechanical risers beside the frames, hence further investigation is required to confirm if other transfer stubs require repair.

At Levels Three and Four a series of cracks have been found parallel to the beam edges along Grids 3 and 6 in the vicinity of the stair and lift shafts (see Appendix D). In some cases these cracks have already been epoxy injected and finished off, while others are yet to be remediated. The consistent observation of these cracks and their size indicates that further investigation is required to confirm their full extent across the length of the building. This extent of investigation and repair has been directed as part of the HCG Site Report 46 (issued 10/6/2013). The locations of these cracks, and observed cracking in other areas of the slabs suggest they are likely to be pre-existing shrinkage cracks that have been worked open by the earthquake movements. Although no carpet or vinyl finishing damage was observed, this further investigation should include Level Five as it is likely that similar cracks exist at this level and further investigation is required to confirm and possibly remediate these (this extent of investigation and repair has been included as part of HCG Site Report 46.

Further cracking of the slabs at Levels Three, Four and Five has been observed at the west end of the building between grids A to C (see Appendix D). The crack patterns are consistent in width and extents from one location to another, and in some cases are considered to be significant enough that the slab reinforcement may have yielded. Representative locations at Level Three and Four have had in-situ testing of the reinforcement [15] to assess the residual steel capacity (see Appendix E). A computer model of the Level Four perimeter moment frame was also developed to investigate the extent of frame deformation and hence potential for slab damage under earthquake loading. Combined with the reinforcement testing, the information from the computer model and visual observations suggests that there is a mixture of new cracks and existing shrinkage cracks that may have opened further under the earthquake demands, or have not changed significantly as a result of the earthquakes. In the locations selected for reinforcement testing, limited or no yield of the reinforcement was observed which indicates that the floor slab diaphragms have not suffered a significant loss of strength or deformation capacity.

Some cracking has also been found at the east end of Level Four, which has provided indication that further locations at the corners of the floor slab should be investigated when possible. Instruction has been given for this continued work (see Appendix F).

Given the cracking found in the topping slab it is recommended that the precast floor ribs be investigated throughout the building. As noted above there may be further cracks to the rib units which may need repair following the directive provided by HCG (see Appendix F).

Beam-column joints of the seismic-resisting frames at Level Three were examined. In all three locations considered, only minor cracks were observed in the columns at their midheight. One beam exhibited a crack although this was partly obscured by the flooring glue. Based on observations of floor cracks, the fact that flooring glue is present over the crack suggests this is an existing shrinkage crack.

At the plant-room Levels Six, Mezzanine and Seven, floor cracking has been noted in a number of locations. In particular sets of cracks fanning out from columns are in the range of 0.5 to 0.7 mm. The north edge of the level six mezzanine is cantilevered, and has developed diagonal cracks at the base of the cantilever visible at the east edge of the slab projection. These are probably the result of the vertical accelerations during the earthquake exciting the mechanical equipment and thus flexing the slabs.

The beams along Grid D & E supporting the Level Seven mechanical service floor have developed a series of cracks 0.3 – 0.4 mm in width at regular spacing along the length of the beam. Cracks are present at the mezzanine support beam over the column at D3 (seen at level six). Full depth cracking of the slab around the penetrations through the floor slab underneath the lift machines has been observed with widths from 0.4 - 0.8 mm. Based on their location it is likely that the effect of earthquake vertical accelerations on the lift machines have caused these cracks.

The two main staircases are precast reinforced concrete. The western staircase (between Grids C & D) has developed a number of cracks (some minor and some significant) in the landings at various levels where saw-cuts have not been provided to separate each half of the landing. From the structural drawings it appears that saw-cuts were expected, however their presence is inconsistent over the building height. Cracking has been induced by the upper and lower stair flights working against each other and therefore forcing the landings to transfer shear forces as the floor levels move relative to one-another. As has been directed, these will need remedial work carried out to them in order to prevent this happening again under strong seismic demands.

The undersides of the western and eastern stairs, at most levels, were observed to have a series of transverse cracks (0.3 mm) across their width at approximately 0.1 – 0.3 mm in width and 400 mm spacing.

The Level Seven slab has a number of parallel cracks (0.3 mm width) at a spacing of approximately 2.5 m.

3.6 LEVELS SURVEY

A levels survey was carried out by Fox & Associates on 16 June 2011 and the results are summarised in their report dated 28 June 2011 [11].

The results of the verticality survey do not indicate any permanent lean of the Christchurch Women's Hospital building.

3.7 GEOTECHNICAL INVESTIGATION

A geotechnical investigation was carried out by Tonkin & Taylor Ltd in August/ September 2011 and the results are summarised in their report dated September 2011 [12].

The investigation did not specifically address the Christchurch Women's Hospital building as no significant land damage had been observed around the building and no significant verticality issues had been identified. The investigation specifically addressed the Riverside and Parkside buildings which are to the east of Christchurch Women's Hospital. From the investigations carried out it can be concluded that the ground conditions Christchurch Women's Hospital are likely to be similar to that for the Riverside and Parkside buildings, i.e. a non-liquefiable gravel layer present from basement level to 4-5m below basement level with a dense sand layer approximately 2.5m deep below the gravel layer which is believed to have liquefied during the 22 February 2011 earthquake.

The geotechnical report concluded that for both Parkside and Riverside 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 the dynamic loads that were applied to the building foundation during the earthquakes.

3.8 FAÇADE SURVEY AND ASSESSMENT

A survey was carried out on the exterior of the building by Goleman and the earthquake damage observed is outlined in their report dated 25 October 2011 [13].

The damage recorded included cracking and spalling of the corners and edges of the precast concrete cladding panels, damage to sealant and membranes, plus damage to flashings.

3.9 MATERIALS TESTING

Given the generally limited crack widths observed and their locations, along with the lack of evidence for structural steel damage in the braced frames, in-situ materials testing was only carried out to confirm that the topping slab reinforcement had sufficient remaining strain capacity.

3.10 POST EARTHQUAKE BUILDING CAPACITY

Based on the observations up to the date of this report, in its current state following the earthquakes, we do not consider the Christchurch Women's Hospital building to have any significant reduction in gravity load resistance at levels above the Lower Ground floor.

It is possible that with the minor cracks observed around the structure there is some reduction in the lateral stiffness of the building. With the application of pressure epoxy at noted locations the building will have, in our opinion, close to its original stiffness.

As noted in Section 2.2.1 the original site specific design spectrum exceeds the previously accepted NZS1170.5:2004 spectrum, and thus the building can be considered to have capacity sufficient to meet new building standard. It is likely that the Christchurch design spectrum will be revised in the near future to reflect observed site response characteristics in the area of the hospital. Once available the current seismic lateral-force resisting capacity will need to be re-visited and confirmed again.

4. RECORD OF OBSERVATIONS

The observed damage to Christchurch Women's Hospital as described in the previous section will need a level of repair applied. Following a complete detailed investigation to confirm the full extent of cracks beyond that observed in sample locations, the repairs will help maintain the structural capacity and integrity of the building such that its performance in future seismic events will be close to the original design intent. As part of this investigation it needs to be estimated which cracks are the result of or have been opened further by the earthquakes, and which were pre-existing but unknown.

The majority of the work required is epoxy injection of the cracks, of which a number of locations have already been repaired in this manner. Table 3.1 summarises the locations of observed damage and typical repairs required, with reference to Appendix A Record of Observations and Appendix B Reference Plans. The Repair Specification [2] referred to in the Table 4-1 has been issued separately.

The aim of any earthquake repair work 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.

Recommended remediation of critical structural weaknesses, to improve the buildings performance during earthquake motions, are outlined in Section 4.

4.1 FURTHER INVESTIGATION REQUIRED

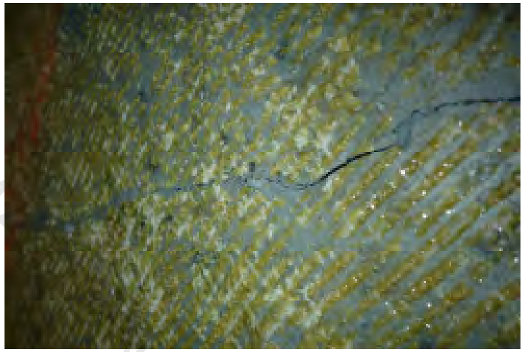
Based on the following observations, further investigation work that should be completed now includes:

- Check of the precast floor rib units at their supports in the above-ground levels.
- Check of topping slab for cracking in Ground Floor, Level 1 and Level 2 at similar locations to the Level 3, 4 and 5 inspections already completed.

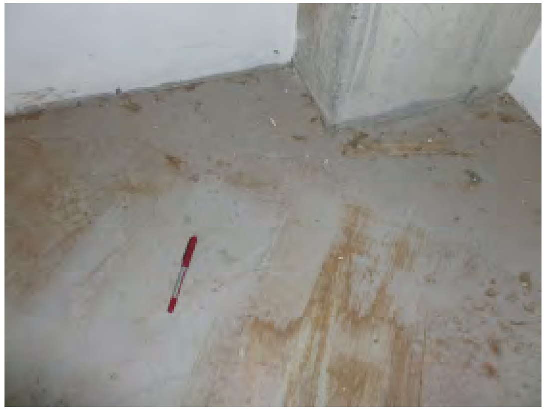
Investigation work to be carried out as repairs are carried out includes:

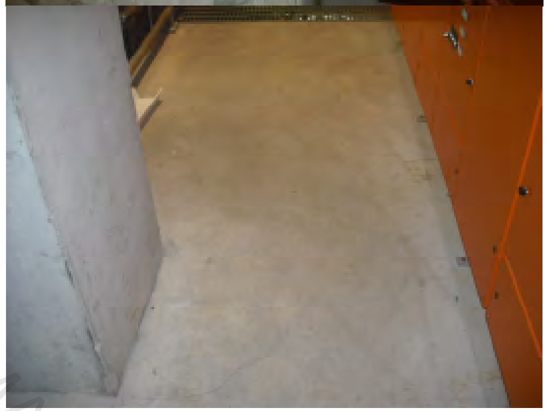
- Extent of cracks in the topping slab in rooms and corridors at all levels where such cracks have been identified previously or from future investigation work.
- Extent of precast stair-unit cracks and seating or support structure connections damage.

Table 4-1: Record of Observations

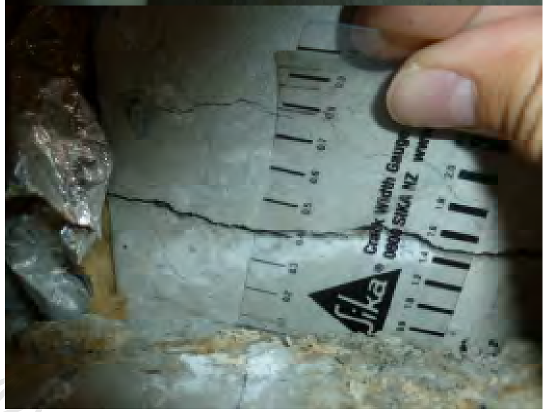
Damage	Locations	Recommendation	Example
1. Floor slabs			
1.1. Cracking between 0.2mm and 0.5mm	BASEMENT: Cracking at various locations throughout basement walk and crawl spaces.	Epoxy inject cracks in slab and raft greater than 0.2mm in width where external and 0.3mm in width where internal. Refer to HCG Specification	No photo
1.2. Inspection	LEVEL 1 & 2 Topping slab cracks as observed on other floor levels	See App F	No photo
1.3. Cracking up to 0.6mm in topping slab + Inspection	LEVEL 3: Cracking in topping slab parallel to beams on GL 3 & 6. Cracks observed on north and south sides of beams.	See App F	
1.4. Cracking up to 0.6mm in	LEVEL 4:	See App F	See above

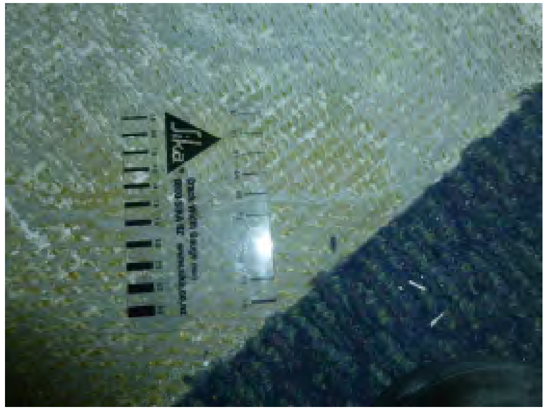
Damage	Locations	Recommendation	Example
topping slab + Inspection	Cracking in topping slab parallel to beams on GL 3 & 6. Cracks observed on north and south sides of beams.		
1.5. Cracking up to 1.2mm in topping slab + Inspection	LEVEL 4:	See App F	
1.6. Inspection	LEVEL 5:	See App F	

Damage	Locations	Recommendation	Example
1.7. Inspection	LEVEL 5: Possible cracking in topping slab parallel to beams on GL 3 & 6 per Level 3 and 4. Cracks observed on north and south sides of beams.	See App F	See above
1.8. Slab cracks radiating from column	LEVEL 6: South-west corner column	Epoxy inject cracks greater than 0.2 mm in width. Refer to HCG Specification	

Damage	Locations	Recommendation	Example
1.9. Slab cracks up to 0.7 mm	LEVEL 6 MEZZANINE: Cracks in slab observed in soffit of landing, cantilevered slab and radiating from columns	Epoxy inject cracks greater than 0.2 mm in width. Refer to HCG Specification	
1.10. Slab cracks up to 0.4 mm	LEVEL 7: Parallel cracks in slab observed at regular spacing along length of slab	Epoxy inject cracks greater than 0.2 mm in width. Refer to HCG Specification	No photo

Damage	Locations	Recommendation	Example
1.11. Slab cracks up to 0.8 mm	LIFT MACHINE ROOM: Full slab depth cracks observed around/beneath lift machines and central area of floor.	Epoxy inject cracks greater than 0.2 mm in width. Refer to HCG Specification	
2. Beams and Precast Floor Ribs			
2.1. Flexure and shear cracks up to 0.4 mm + Inspection	BASEMENT: Cracks in transfer beams spanning around the elevator pits GLs C, 6 & E. Also at locations where post-tensioned tie-downs are anchored around perimeter of building	Epoxy inject cracks greater than 0.2 mm in width. Inspect beams with tie-down anchors passing through at east end of building. Refer to HCG Specification	

Damage	Locations	Recommendation	Example
2.2. Shear cracks in precast concrete rib joists up to 1.5 mm wide	BASEMENT: Cracks in precast ribs near seating at multiple locations as indicated on plan provided Appendix B	Epoxy inject cracks greater than 0.2 mm in width. Where cracks are wider than 0.8 mm provide steel seating detail. See concept sketch SKS-C1 in App. C. Refer to HCG Specification	 

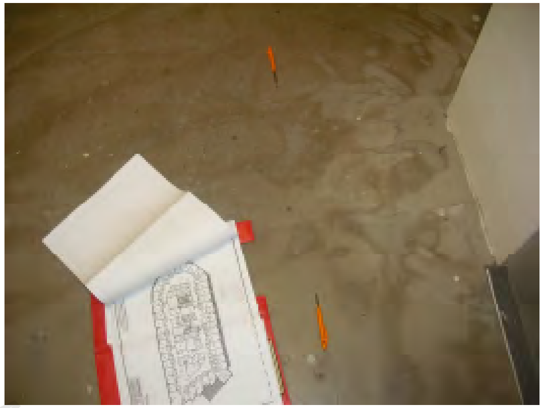
Damage	Locations	Recommendation	Example
2.3. Cracks up to 0.5 mm wide + Inspection	LEVEL 3: Crack noted in slab/top of beam in Rm 3094 on SW side of column	Further inspection required on beams around perimeter of building. Suggest beams are exposed at every 2nd column by lifting flooring, and removing ceiling tiles. If cracks are consistently noted then similar for Level 4 and 5. Epoxy inject cracks greater than 0.3 mm. Refer to HCG Specification	
2.4. Precast rib joists	ALL LEVELS: Possible cracks near supports of precast floor ribs.	Investigation of precast floor ribs at all levels required to confirm if similar cracks near the supports is present (as seen in basement). See App. F	

Damage	Locations	Recommendation	Example
2.5. Cracks up to 0.6 mm wide	LEVEL 6: Support beam to mezzanine above as seen at L6 landing.	Epoxy inject cracks greater than 0.2 mm in width. Refer to HCG Specification	
2.6. Cracks up to 0.4 mm wide	LEVEL 6 MEZZANINE: Beams supporting Level 7 have diagonal cracks	Epoxy inject cracks greater than 0.2 mm in width. Refer to HCG Specification	No Photo
3. Columns			

Damage	Locations	Recommendation	Example
3.1. Cracks <0.2 mm. + Inspection	LEVEL 3: Columns inspected in three locations Rm 3009, 3094 & 3096. Minor cracks at mid-height observed.	See item 2.3. Further inspection of every 2nd column required. Epoxy inject cracks greater than 0.2 mm Refer to HCG Specification	

Damage	Locations	Recommendation	Example
3.2. Diagonal cracks up to 0.4 mm	LEVEL 6 MEZZANINE: Crack all way through column at landing GL D/3	Epoxy inject cracks greater than 0.3 mm Refer to HCG Specification	
4. Basement Walls			
4.1. Cracks in perimeter walls + Inspection	Some locations already noted/repaired. Confirm locations with Fletcher.	Inspect all walls around basement including tunnel through to Parkside and epoxy inject all cracks that are greater than 0.2mm in width. Refer to HCG specification.	

Damage	Locations	Recommendation	Example
4.2. Diagonal cracks up to 1.2 mm wide	Elevator shaft pit walls	Epoxy inject cracks greater than 0.2 mm. Refer to HCG Specification	
4.3. Water ingress into tie-down caisson shafts	Inspection by Goleman indicated Shafts 1, 2 and 4 had water present on top of the concrete plug at the bottom of the shafts.	See App. C for repair carried out	No photo.
5. Seismic Gaps			
5.1. Damage to cover plates and linings	Seismic gaps to Parkside	Make good finishes and cover plates	No photo

Damage	Locations	Recommendation	Example
5.2. Exterior covers have pounded perimeter wall	Perimeter "moat" around exterior of building at Lower Ground floor	Contact locations to be repaired per original specifications. See revised details issued previously.	
6. Staircases			
6.1. Damage to landings noted in west service stair with cracks up to 0.8 mm + Inspection	L3 mid-landing & L5 mid-landing. Confirm if present at other levels as vinyl may be hiding cracks.	Remediation of stair connections similar to concept sketch SKS-C2 App C. Epoxy inject all cracks that are greater than 0.3mm in width. Refer to HCG specification	
6.2. Inspection	East stair	Inspect landings for concrete damage when carrying out remediation per concept SKS-C2 App C.	
6.3. Transverse cracks up to 0.4 mm wide in underside	Both east and west stairs. Nurses have noted that stair vibrations	Epoxy inject cracks greater than 0.3 mm	

Damage	Locations	Recommendation	Example
of stair case	are noticeable since Sept 4th earthquake.		
7. Cladding	From Goleman Survey		
7.1. General damage to cladding and flashing elements	Refer to Goleman Report	Epoxy inject all cracks that are greater than 0.2mm in width. Refer to HCG specification or by others where appropriate	

5. REMEDIATION OF CRITICAL STRUCTURAL ELEMENTS

As a result of observations made during site inspections and review of the structural drawings, two particular critical structural weaknesses have been identified. These are addressed in a subsequent section, with recommendations as to how effective remediation can be carried out.

5.1 REMEDIATION OF CRITICAL STRUCTURAL WEAKNESSES

Observations from the basement of the precast concrete ribs supporting the Lower Ground floor slab noted a number of cracks, of varying width, through the concrete ribs near or at the seating locations. In order to ensure the gravity load carrying capacity of these units is maintained, it is recommended that the cracks be epoxy injected in all cases. Where the cracks exceed 0.8 mm in width the unit shall be supported with an additional seating steel angle fixed to the main concrete beams with mechanical or chemical anchors. Figure B2 in Appendix B, as provided by RCP, indicates locations and crack widths. A scheme for additional seating angles (that has been issued) is provided in sketch SKS-C1 in Appendix C.

At floors above the Lower Ground floor cracks in the precast floor ribs greater than 0.5mm in width require epoxy injection, but do not require consideration for additional seating. See instruction provided in Appendix F for further details.

As noted in Table 3.1 some of the stair landings developed cracks both parallel and perpendicular to the precast stair flights. Review of the structural drawings indicates that the detailing of the landings and connection to the stairs may not allow for adequate relative movement of the stairs and landings during a major earthquake. This condition is common to both the east and west stairs at all levels. Our recommendation is that the issue be remediated by introducing a separation between the upper and lower stair flights at the mid-landing, while providing a revision to the connection details between the landing steel framing and mid-landing slab. A preliminary scheme for this detail is provided in sketch SKS-C2 in Appendix C.

Remediation of the cracking noted in the floor slab of Level Three, Four and Five would require epoxy injection, which would provide near equivalent integrity as an uncracked slab.

5.2 FURTHER REMEDIATION DETAILS TO BE ISSUED

Currently there are no further remediation details to be issued by Holmes Consulting Group.

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. NZS4203:1992*, Code of Practice for General Structural Design and Design Loadings for Buildings,* Standards New Zealand, 1992.

4. NZS1170.0:2002*, Structural Design Actions : General,* Standards New Zealand, 2002.

5. NZS3101:1995, Concrete St*ructures Standard*, Standards New Zealand, 1995.

6. NZS3404:1997, *Steel Structures Standard*, Standards New Zealand, 1997.

7. NZS1170.5:2004*, Structural Design Actions Part 5: Earthquake actions – New Zealand,* Standards New Zealand, 2004.

8. DR902, *Draft – Structural Design General requirements and design actions Part 4: Earthquake actions*, New Zealand Standards Authority, 2000.

9. Tonkin and Taylor Ltd, *Christchurch Woman's and Day Surgery Unit – Site Specific Seismic Assessment*, Unpublished Report, 2001.

10. *Canterbury District Health Board New Women's Hospital and Day Surgery Unit Chch. Hosptial, Riccarton Avenue - Structural Drawings*, Holmes Consulting Group, 2002.

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

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

13. Goleman, *Earthquake Inspection Christchurch Base Hospital Park Side,* 25 October 2011.

14. Gavin, H.P. and Wilkinson, G. (2010) *Preliminary Observations of the Effects of the 2010 Darfield Earthquake on the Base-Isolated Christchurch Women's Hospital*, Bull. NZSEE Vol 43. No. 4, 2010.

15. Holmes Solutions, *Non-Destructive Testing of the Reinforcing Steel in Floor Slabs of Christchurch Women's Hospital*, Report 108281 (v1.0), May 2012.

APPENDIX A

Record of Observations

APPENDIX A – RECORD OF OBSERVATIONS & REPAIRS - CDHB Christchurch Women's Hospital

Inspection dates: 19/12/2011 10/2/2012 16/2/2012 28/2/2012 6/3/2012. Numerous follow-up inspections for each room of slab inspections

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
B/M	General			Contech have been doing crack injection work. Generally on the floors in the walk space, on the raft slab in the crawl space and vertical walls. Work has been started to inject transfer beams.	Y	Epoxy inject	11-12-20: 004, 005, 007, 006, 008 20120216: 001, 001a, 003, 004, 005, 006, 007, 009, 010
B/M		Lift Pit	East Lift Shaft	Diagonal cracking up to 1.2mm	Y	Epoxy inject	11-12-20: 004, 005. 20120216: 004
B/M		Lift Pit	West Lift Shaft	Diagonal cracking - less than east	Y	Epoxy inject	11-12-20: 007
B/M		Beam		Shear crack 0.4mm rooted from penetration	Y	Epoxy inject	11-12-20: 006
B/M		Lift Pit area	Transfer beams	Flexural cracks around beam connections and bearing pad locations.	Y	Epoxy inject	20120216: 007, 008
B/M	General	Floor ribs	Grid B to D/1 to 8	A number of precast rib units have full depth cracks at/near seatings (ref. plan provided by RCP). Crack widths range from 0.2 to 1.5 mm. Noted that cracks have grown for example 0.5mm (Dec) 0.5 to 1.0mm (Feb).	Y	Epoxy inject & add steel angle seating	11-12-20: 008, 016 20120228: 003, 004

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
B/M	General	Floor ribs	Grid F to G/4 to 5	A seating of precast ribs cracks around rib/to rib end noted	**Y**	Epoxy inject	
B/M	General		Rubber bearing isolators	Current permanent offset approx 6-10mm in NE direction	**N**		011, 012, 013, 014, 015
B/M	General	SW crnr: G.L. A.5/7	Caisson Tie Down Anchorage to beam	Vertical crack 0.3mm in transfer beam. It is recommended that the anchor heads of the post-tensioned tie-down be inspected from the Lower Ground floor in order to confirm that no loss of pre-tension has occurred	**Y**	Epoxy inject	20120228: 001, 002
B/M	General	SW crnr: G.L. B/7.5	Caisson Tie Down Anchorage to beam	Vertical cracks 0.2-0.3 mm in transfer beam. It is recommended that the anchor heads of the post-tensioned tie-down be inspected from the Lower Ground floor in order to confirm that no loss of pre-tension has occurred	**Y**	Epoxy inject	20120228: 001 sim, 002 sim
B/M	General	SW crnr: G.L. B/7.5	Caisson Tie Down concrete plug	Water collected in 3 of the caissons that could be inspected.	**Y**	See Appendix C	20120808
L Grnd	L047		Tie Down anchorage	No indication of damage or loss of tensioning	**N**		20120327: Anchorhead_1
L Grnd	General		Precast floor rib units	Vertical or diagonal cracks at various ends of the precast units. Not a consistent distribution but found in a significant number of locations	**Y**	See Appendix F	
Grnd		Entry curb G8	Apron slab	Pounding/uplift due to incorrectly constructed frame detail	**Y**		20111010: 001
L1	Drive-Thru Entry		Beam-Col connection	Double height column and beam connection has flexed causing damage to existing sealant at each beam-col interface	**Y**	Epoxy inject	20120427: 001
L3	3023		Floor Slab	Main crack with branching off cracks. Widths 0.4-0.6mm	**Y**	Epoxy inject	20120417: 001, 002, 003, 004, 005, 006

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
L3	3058		Floor Slab	Crack parallel to beam at beam edge. 0.5/0.6 mm slightly spalled	**Y**	Epoxy inject	11-12-20: 026
L3	3070		Floor Slab	Cracks 0.4-0.6mm	**Y**	Epoxy inject	20120423: 001, 002, 003
L3	3101		Floor Slab	Numerous old cracks already filled 0.4mm	**C**		11-12-20: 006, 025
L3	3035		Floor Slab	Crack parallel to beam at beam edge. 0.5/0.6 mm slightly spalled	**Y**	Epoxy inject	11-12-19 RCP: 041
L3	3052		Floor Slab	Crack parallel to beam at beam edge. 0.5/0.6 mm slightly spalled. ID by RCP 11/12/19	**Y**	Epoxy inject	11-12-19 RCP: IMG-C26
L3	3036		Floor Slab	Floor deformed under carpet tile but not lifted for inspection	**F**	Epoxy inject	
L3			Floor Slab	Consistent cracking parallel to either side of beams on GL 3 & 6 indicates that this might be present along entire length even though not showing through carpet/vinyl in all areas	**F**	Epoxy inject	
L3	3071		Stair mid landing	Crack (0.3mm) across landing parallel to stair case and crack across landing parallel to first tread up/down	**Y**	Revise stair detailing to allow slip	
L3	3009		Column SW corner	Exposed at top/bott of column + L3 beam at column face to NW side. Minor horizontal cracks 0.1-0.2 mm at midheight of column. No beam cracks observed through vinyl glue	**N**		20120228: 005
L3	3089		Floor Slab	Multiple cracks at varying angles and widths 0.4mm - 1.0mm	**Y**	Epoxy inject	20120412: 022, 023, 024, 025
L3	General		Floor Slab	Multiple cracks similar to above descriptions. Noted pre-grind as being new, existing but further damaged with movement, or existing.	**Y**	Ref. Appendix F	

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
L3	3094		Column	Exposed at bott of column + L3 beam at column face SW side. Minor horizontal cracks 0.1-0.2 mm at midheight of column. Beam crack 0.4-0.5 mm running diagonally away from column from edge of beam towards centre-line.	F	Epoxy inject	20120228: 007, 008, 009 (beam)
L3	3096		Column NW corner	Exposed at bott of column + L3 beam at column face to E side. Minor horizontal cracks 0.1-0.2 mm at midheight of column. No beam cracks observed through carpet glue	F	Epoxy inject	20120228: 009
L3	3072	North end	Steel braced frame	Concrete stub connecting concrete east-west floor beam (Grid 3) shows damage with spalling of stub concrete. Confirm if similar damage at all floor levels and both ends of steel beam making up brace frame	F	Epoxy/High-strength grout patch of damaged/lost concrete	20120216: 021
L4	4028		Floor Slab	Old crack already filled 0.5 mm	C		
L4	4051		Floor Slab	Crease in vinyl inside N double doors	F		
L4	4061	Corridor C3	Floor Slab	Old crack already filled 0.5mm	C		20120210 004, 005
L4			Floor Slab	Consistent cracking parallel to either side of beams on GL 3 & 6 indicates that this might be present along entire length even though not showing through carpet/vinyl in all areas	F	Epoxy inject	
L4	4001		Floor Slab	Multiple cracks widths 0.5-0.8mm	Y	Epoxy inject	20120412: 026, 027, 028
L4	4086		Floor Slab	Multiple cracks at varying angles and widths 0.4mm - 0.8mm	Y	Epoxy inject	20120327: 001, 002, 003, 004, 005, 006
L4	4080		Floor Slab	Multiple cracks at varying angles and widths 0.4mm - 1.0mm	Y	Epoxy inject	20120412: 002, 003, 004, 005, 006, 007

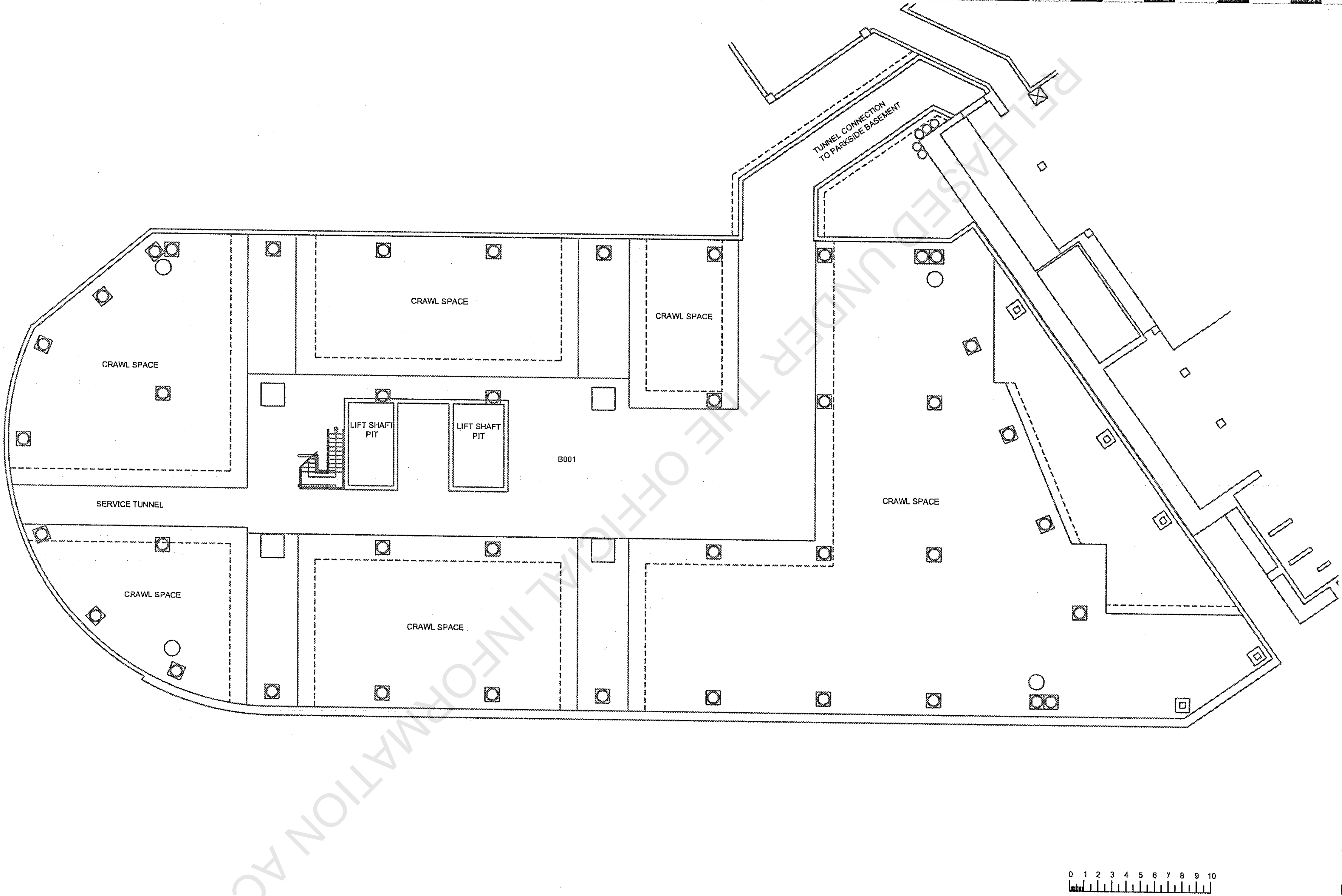
Refer to Table 3.1 and HCG Specification for repair details

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
L4	4084		Floor Slab	Multiple cracks at varying angles and widths 0.4mm - 1.0mm	Y	Epoxy inject	20120412: 009, 010, 011
L4	4072		Floor Slab	Multiple cracks at varying angles and widths 0.4mm - 0.6mm	Y	Epoxy inject	20120412: 013, 014, 015
L4	4069		Floor Slab	Multiple cracks parallel to floor rib joists (below). Widths 0.4 - 0.8mm	Y	Epoxy inject	20120412: 017, 018, 019
L3	General		Floor Slab	Multiple cracks similar to above descriptions. Noted pre-grind as being new, existing but further damaged with movement, or existing.	Y	Ref. Appendix F	
L5	5052		Stair mid landing	Crack 0.7-0.8 mm	Y	See full stair repair desc.	11-12-20: 011, 012
L5			Floor Slab	Consistent cracking parallel to either side of beams on GL 3 & 6 indicates that this might be present along entire length even though not showing through carpet/vinyl in all areas. Confirm if present in similar locations along grid line.	F	Epoxy inject	
L5	5080		Floor Slab	Multiple cracks at varying angles and widths 0.4mm - 1.0mm	Y	Epoxy inject	20120501: 002 to 012
L3	General		Floor Slab	Multiple cracks similar to above descriptions. Noted pre-grind as being new, existing but further damaged with movement, or existing.	Y	Ref. Appendix F	
L6		u/s Stair up to L7	Stair case	Transverse cracks 0.3 mm @ 400 crs underside p.c. stair case. Confirm if present at all other levels as nurses commented on stair vibrations sinse September 4th earthquake.	F	Epoxy inject	11-12-20: 009, 010
L6		Top of stair landing	Beam	Mezzanine support beam diagonal crack	Y	Epoxy inject	11-12-20: 014
L6	8013	SW corner column	Floor Slab	Cracks in slab fanning from column	Y	Epoxy inject	024
L6 Mezz	8013	Soffit	Floor Slab	Crack 0.5mm	Y	Epoxy inject	11-12-20: 013

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
L6 Mezz	8013	Column by door	Column	Diagonal crack in column all way through 0.3/0.4mm	**Y**	Epoxy inject	11-12-20: 021, 022
L6 Mezz	8015	Lift motor rm	Main beams supporting L7	Diagonal cracks 0.3/0.4 mm	**Y**	Epoxy inject	
L6 Mezz	8015 ext of NE crnr	E+/3	Cantilever flr	Transverse crack in landing beside mech bolt + flexural crack in supporting beam	**Y**	Epoxy inject	11-12-20: 017, 031
L6 Mezz	8016	Both columns	Floor Slab	Cracking away from column up to 0.7mm	**Y**	Epoxy inject	11-12-20:018, 019, 020
Lift Room	8015		Floor Slab	Multiple cracks, predominantly under/around lift machines. Widths 0.4 to 0.6	**Y**	Epoxy inject	20120501: Lift2 - Lift8
L7	9002		Floor Slab	Regular cracks across slab 0.3mm @ 2.5 m crs	**Y**	Epoxy inject	
L1 - L5		East Stair	Stair case	Nurses have commented on stair vibrations since September 4th earthquake. Transverse cracks 0.3 mm @ 400 crs underside p.c. stair case flights at all levels	**F**	Epoxy inject	

APPENDIX B

Reference Plans



CHRISTCHURCH HOSPITAL
WOMENS - BASEMENT
SPACE NUMBERS

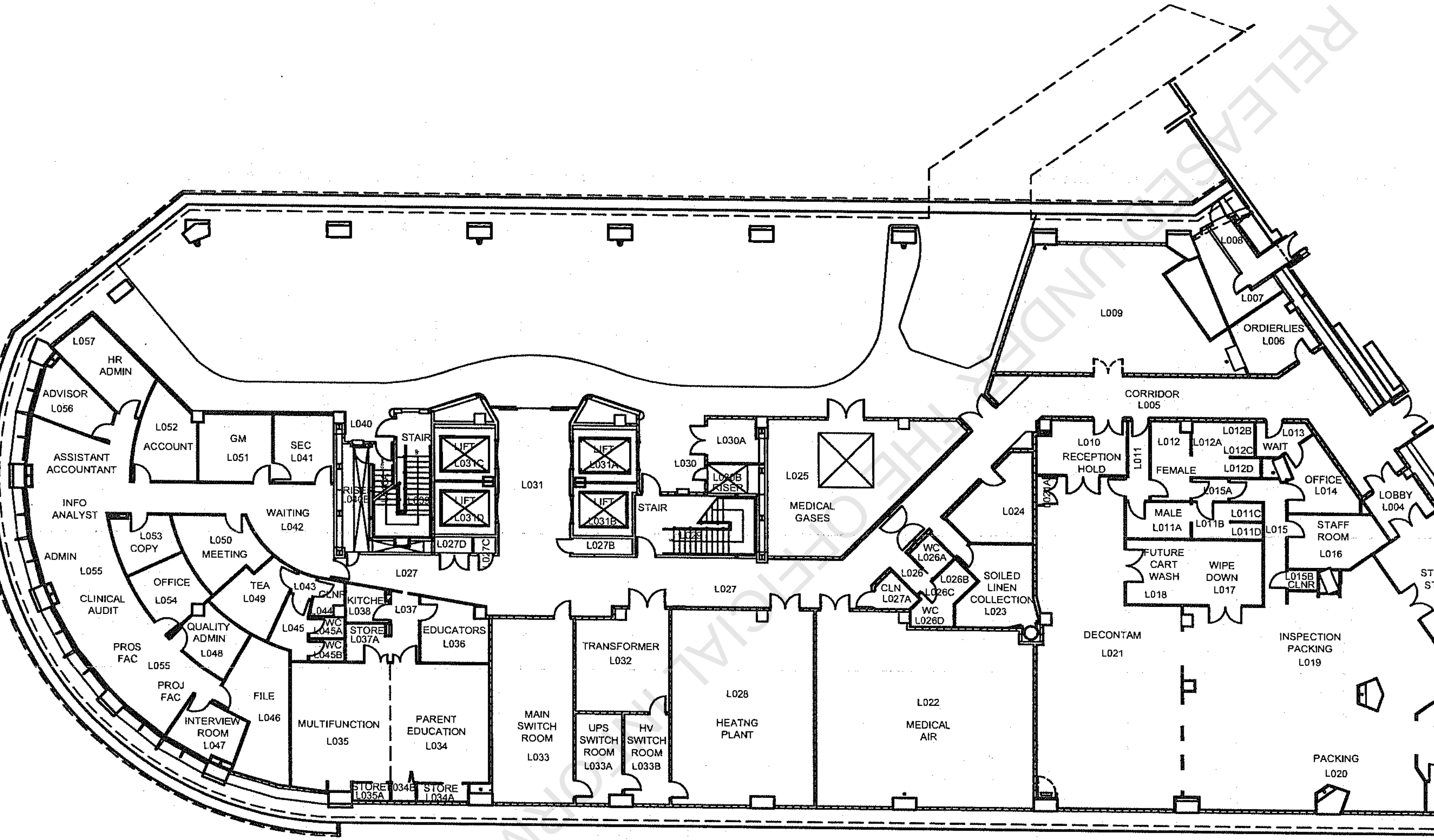
Canterbury
District Health Board
Te Poari Hauora ō Waitaha

DESIGNED:
DRAWN: B.J.Taylor
CHECKED:
DATE: 21-01-09

DWG No. 01291007
SCALE 1:250 AT A3
REVISED -

XREFED DRAWING/S: 01291006

Maintenance and Engineering Department - Christchurch Hospital



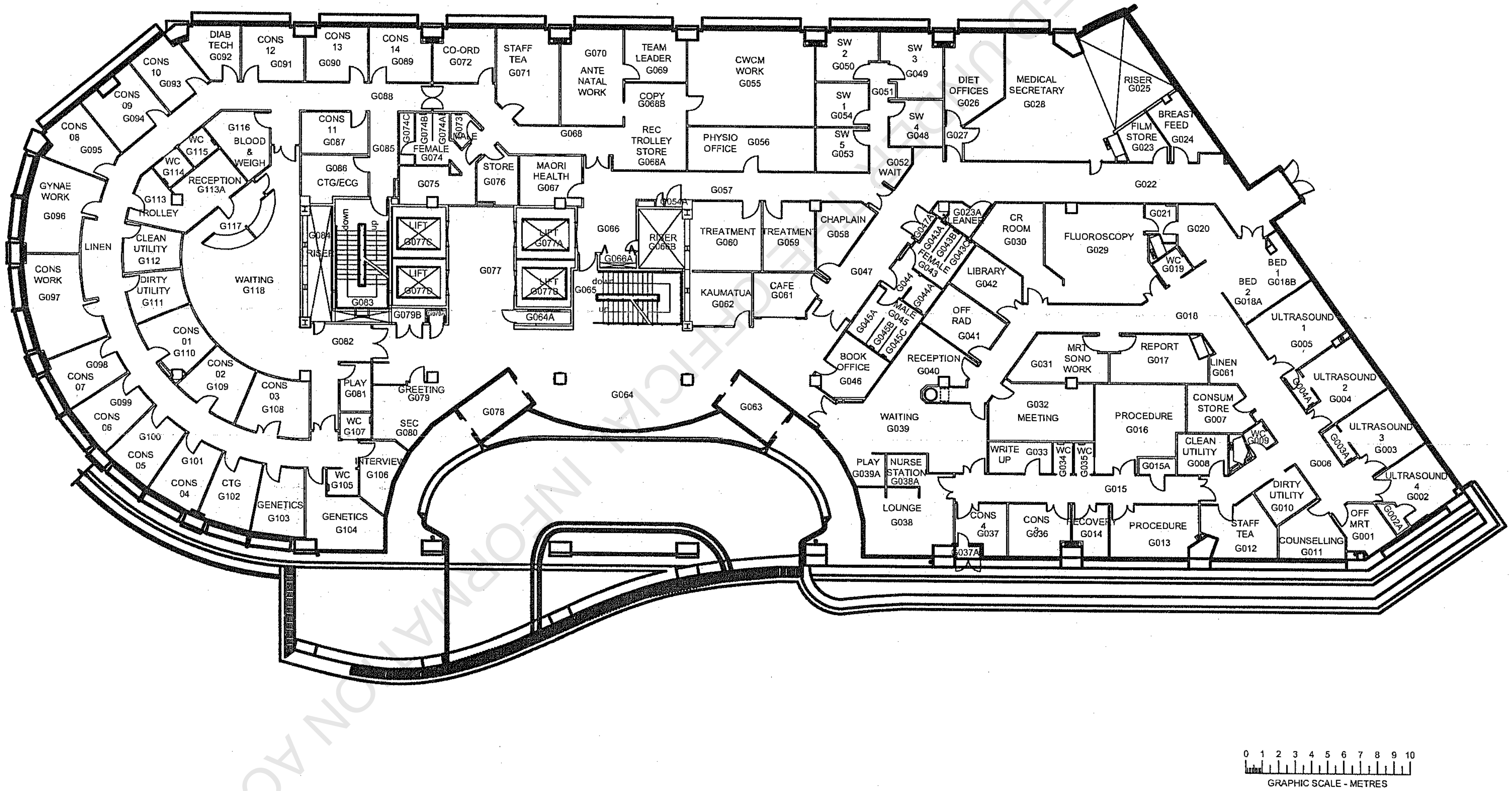
0 1 2 3 4 5 6 7 8 9 10
GRAPHIC SCALE - METRES

CHRISTCHURCH HOSPITAL
WOMENS - LOWER GROUND FLOOR
SPACE NUMBERS

Canterbury
District Health Board
Te Poari Hauora ō Waitaha

DESIGNED:		DWG No	01291002
DRAWN:	B.Tudehope	SCALE 1:250 AT A3	REVISED 26-10-05
CHECKED:			
DATE:	26-10-05		
XREFED DRAWING/S:	01291001		

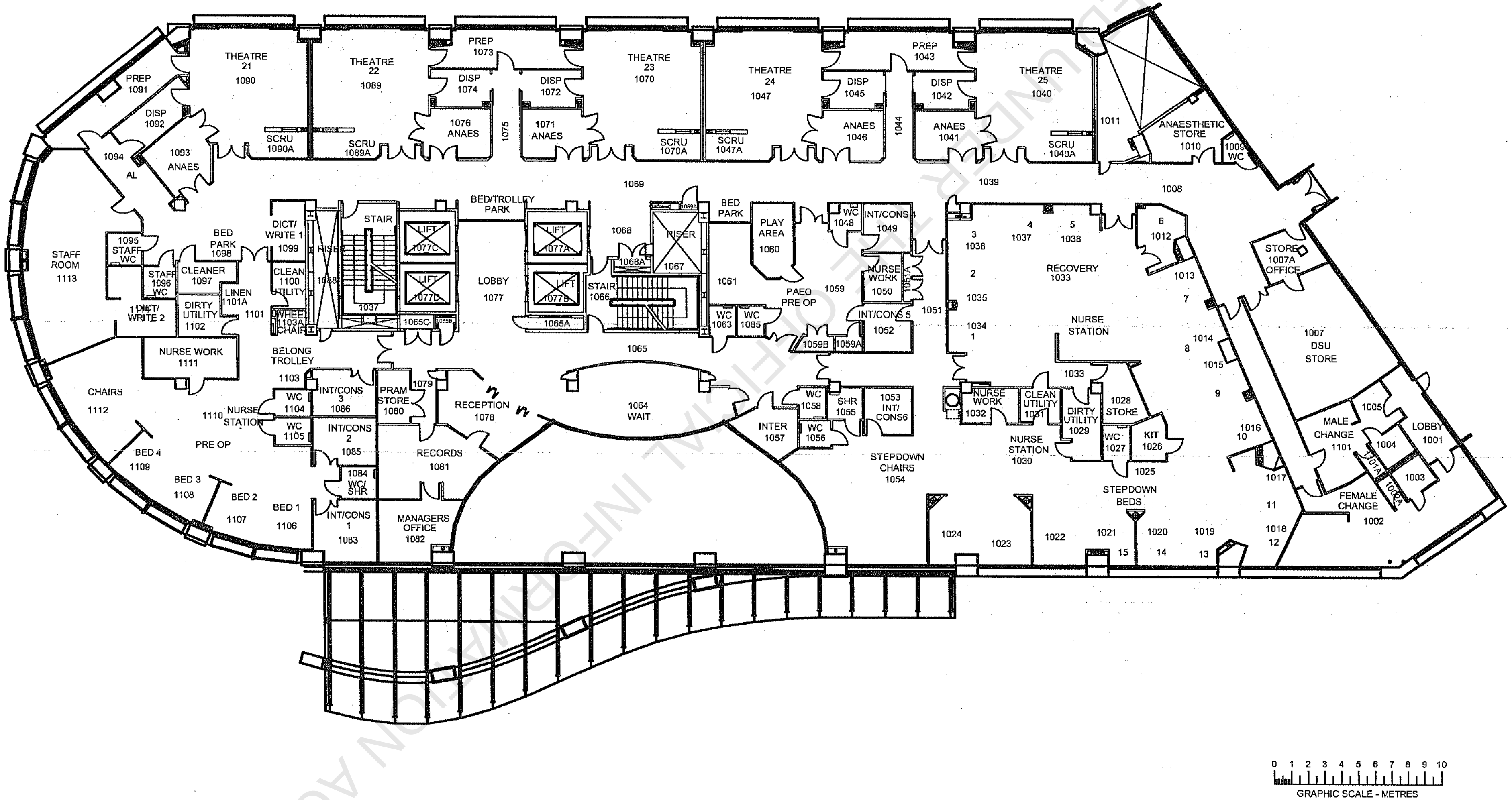
Maintenance and Engineering Department - Christchurch Hospital



CHRISTCHURCH HOSPITAL
WOMENS - GROUND FLOOR
SPACE NUMBERS

Canterbury District Health Board Te Poari Hauora ō Waitaha	DESIGNED: DRAWN: B.Tudehope CHECKED: DATE: 26-10-05	DWG No. 01292002 SCALE 1:250 AT A3 REVISED 23-09-09
	XREFED DRAWING/S: 01292001	

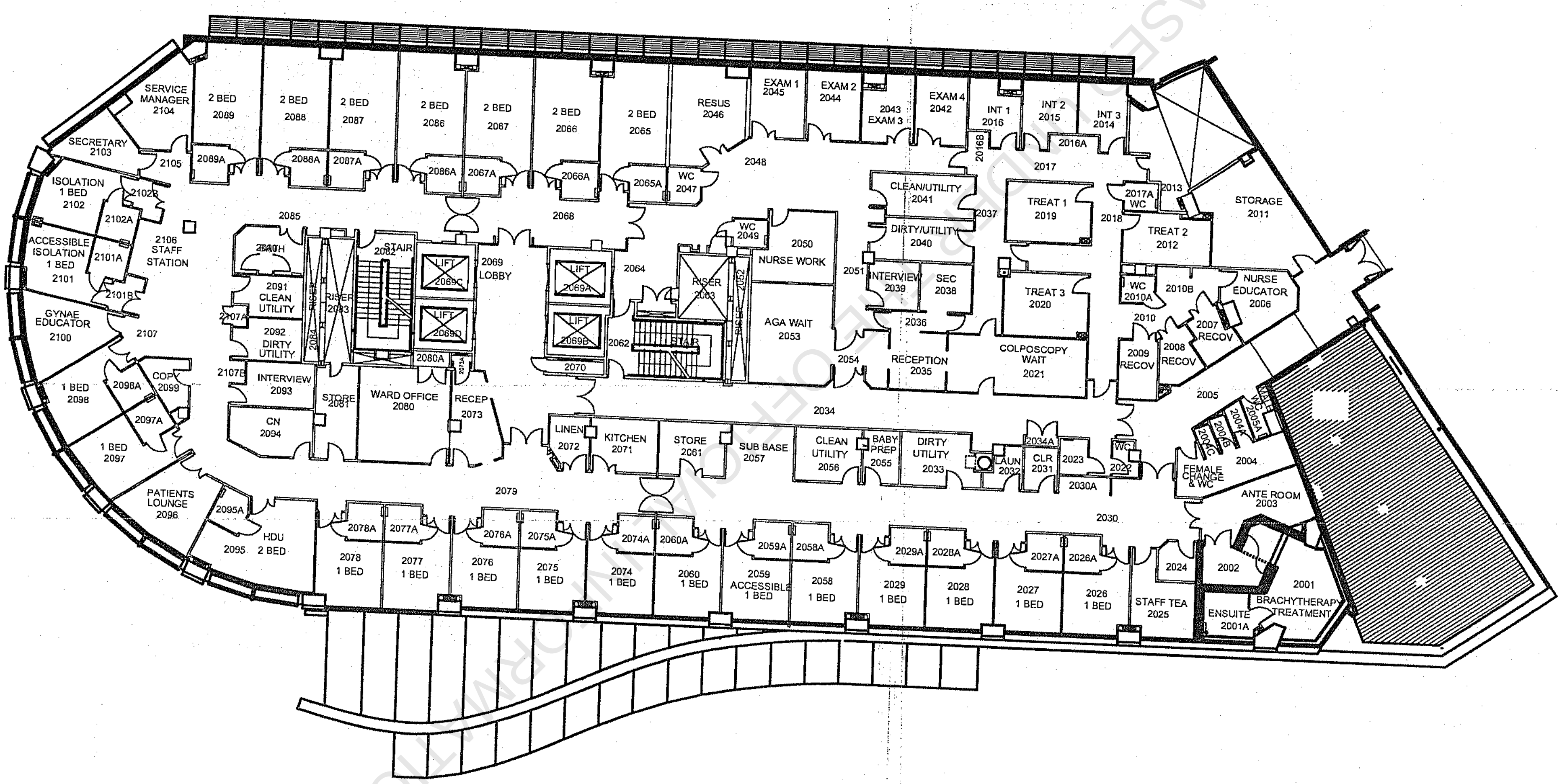
Maintenance and Engineering Department - Christchurch Hospital



CHRISTCHURCH HOSPITAL
WOMENS - FIRST FLOOR
SPACE NUMBERS

Canterbury District Health Board Te Poari Hauora ō Waitaha	DESIGNED: DRAWN: B.Tudehope CHECKED: DATE: 27-10-05	DWG No. 01293002 SCALE 1:250 AT A3 REVISED 30-09-08
	XREFED DRAWING/S: 01293001	

Maintenance and Engineering Department - Christchurch Hospital

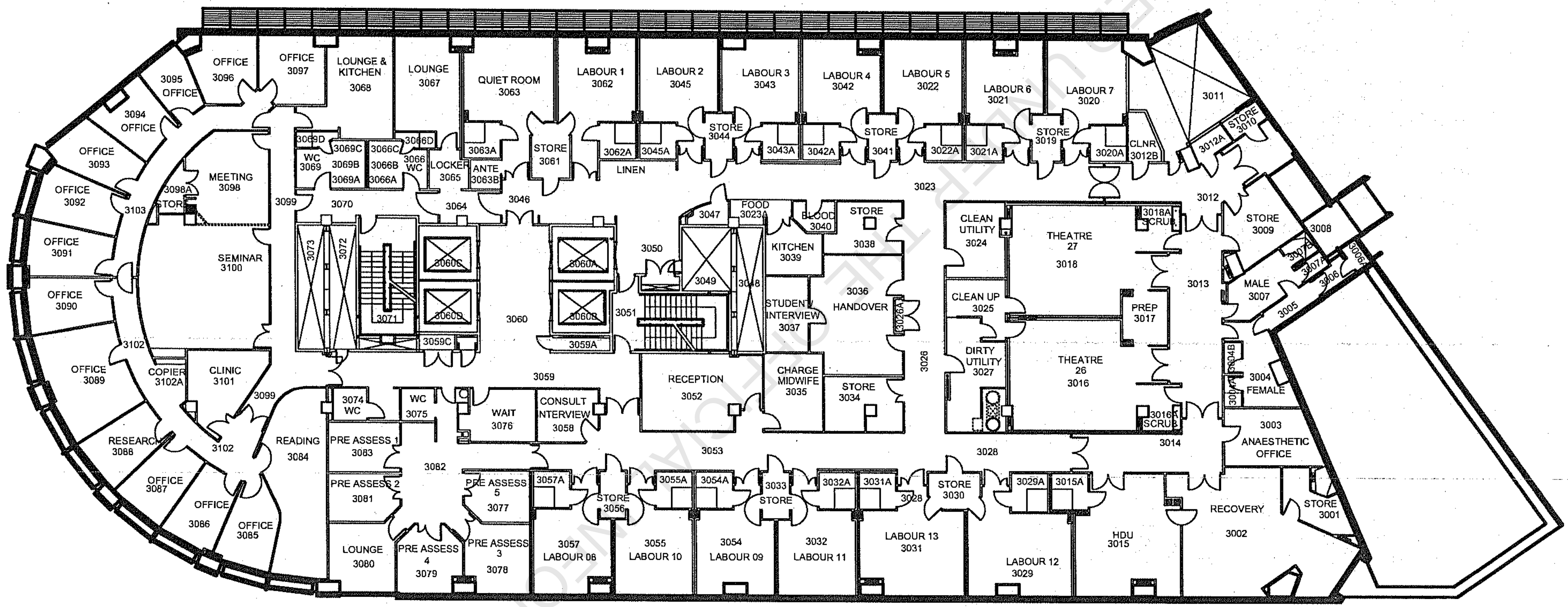


CHRISTCHURCH HOSPITAL WOMENS - SECOND FLOOR SPACE NUMBERS

0 1 2 3 4 5 6 7 8 9 10
GRAPHIC SCALE - METRES

Canterbury District Health Board Te Poari Hauora o Waitaha	DESIGNED: DRAWN: B.Tudehope CHECKED: DATE: 09-11-05	DWG No. 01294002 SCALE 1:250 AT A3 REVISED 27-08-08
	XREFED DRAWING/S: 01294001	

Maintenance and Engineering Department - Christchurch Hospital

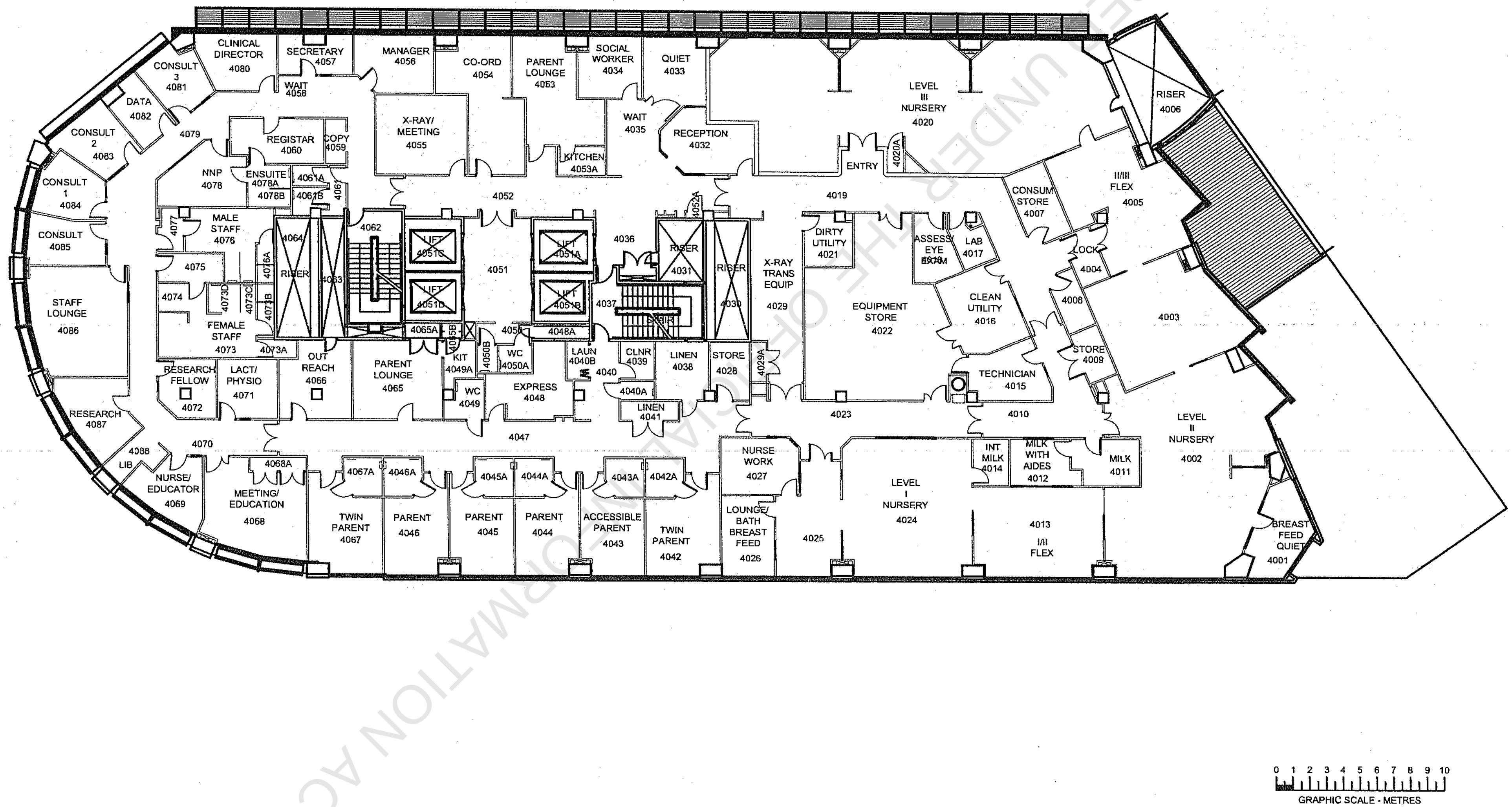


CHRISTCHURCH HOSPITAL
WOMENS - THIRD FLOOR
SPACE NUMBERS

Canterbury
District Health Board
Te Poari Hauora ō Waitaha

DESIGNED:		DWG No. 01295002
DRAWN:	B.Tudehope	SCALE 1:250 AT A3
CHECKED:		REVISED 12-10-05-10
DATE:	16-11-05	
XREFED DRAWING/S:	01295001	

Maintenance and Engineering Department - Christchurch Hospital



CHRISTCHURCH HOSPITAL
WOMENS - FOURTH FLOOR
SPACE NUMBERS

Canterbury District Health Board Te Poari Hauora ō Waitaha	DESIGNED: DRAWN: S.Tudehope CHECKED: DATE: 16-11-05	DWG No. 01296002 SCALE 1:250 AT A3 REVISED 18-11-05
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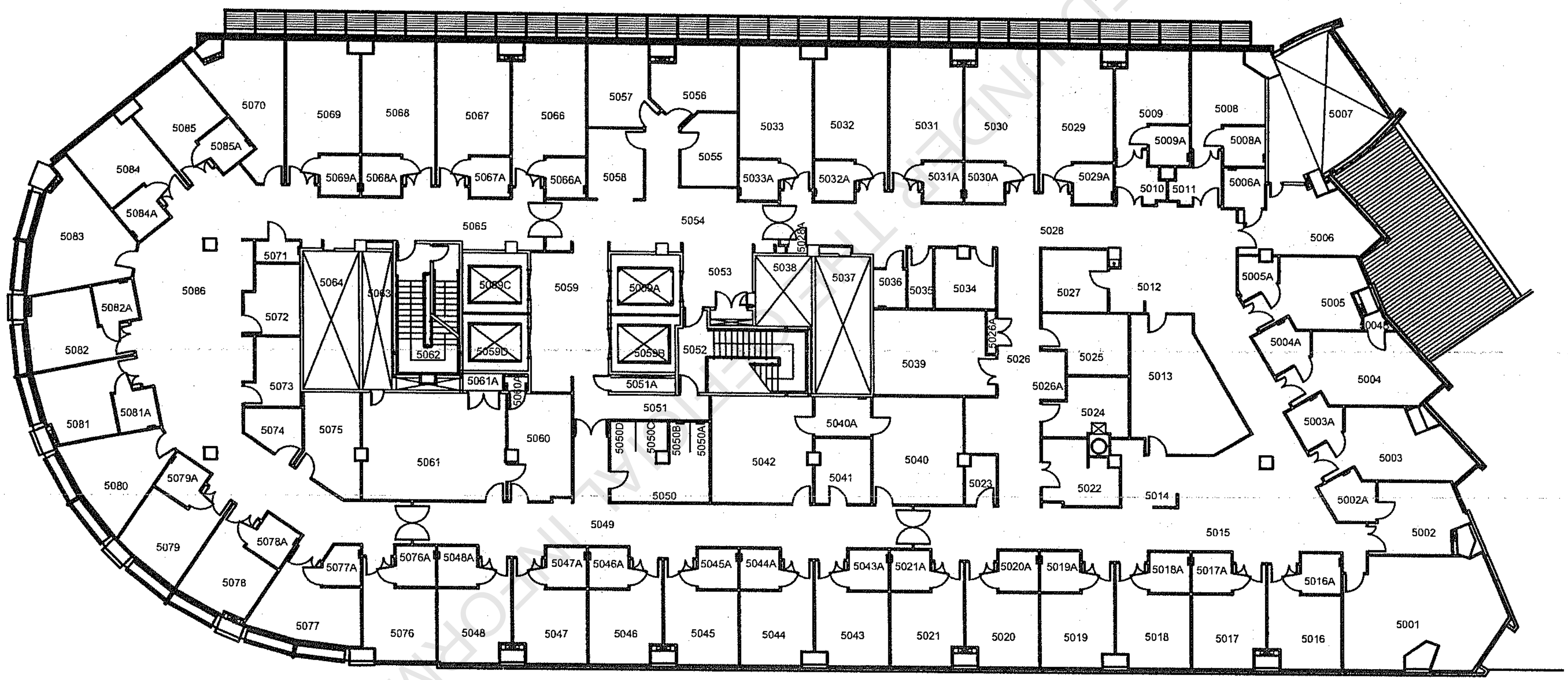
Maintenance and Engineering Department - Christchurch Hospital



0 1 2 3 4 5 6 7 8 9 10
GRAPHIC SCALE - METRES

CHRISTCHURCH HOSPITAL
WOMENS - FIFTH FLOOR
SPACE NUMBERS

Canterbury District Health Board Te Poari Hauora ō Waitaha	DESIGNED: DRAWN: B.Tudehope CHECKED: DATE: 18-11-05	DWG No. 01297002	
		SCALE 1:250 AT A3	REVISED 18-11-05
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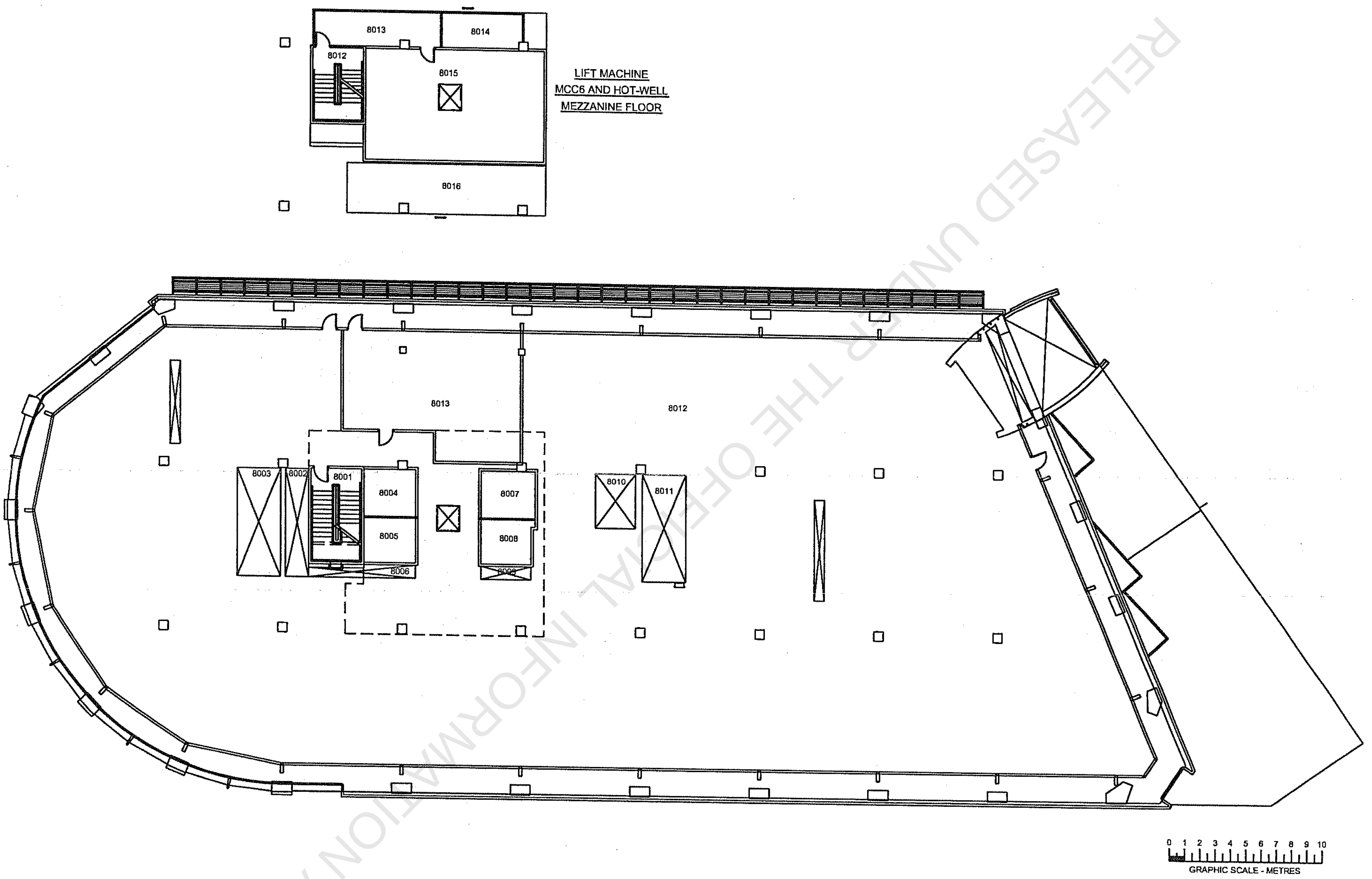
Maintenance and Engineering Department - Christchurch Hospital



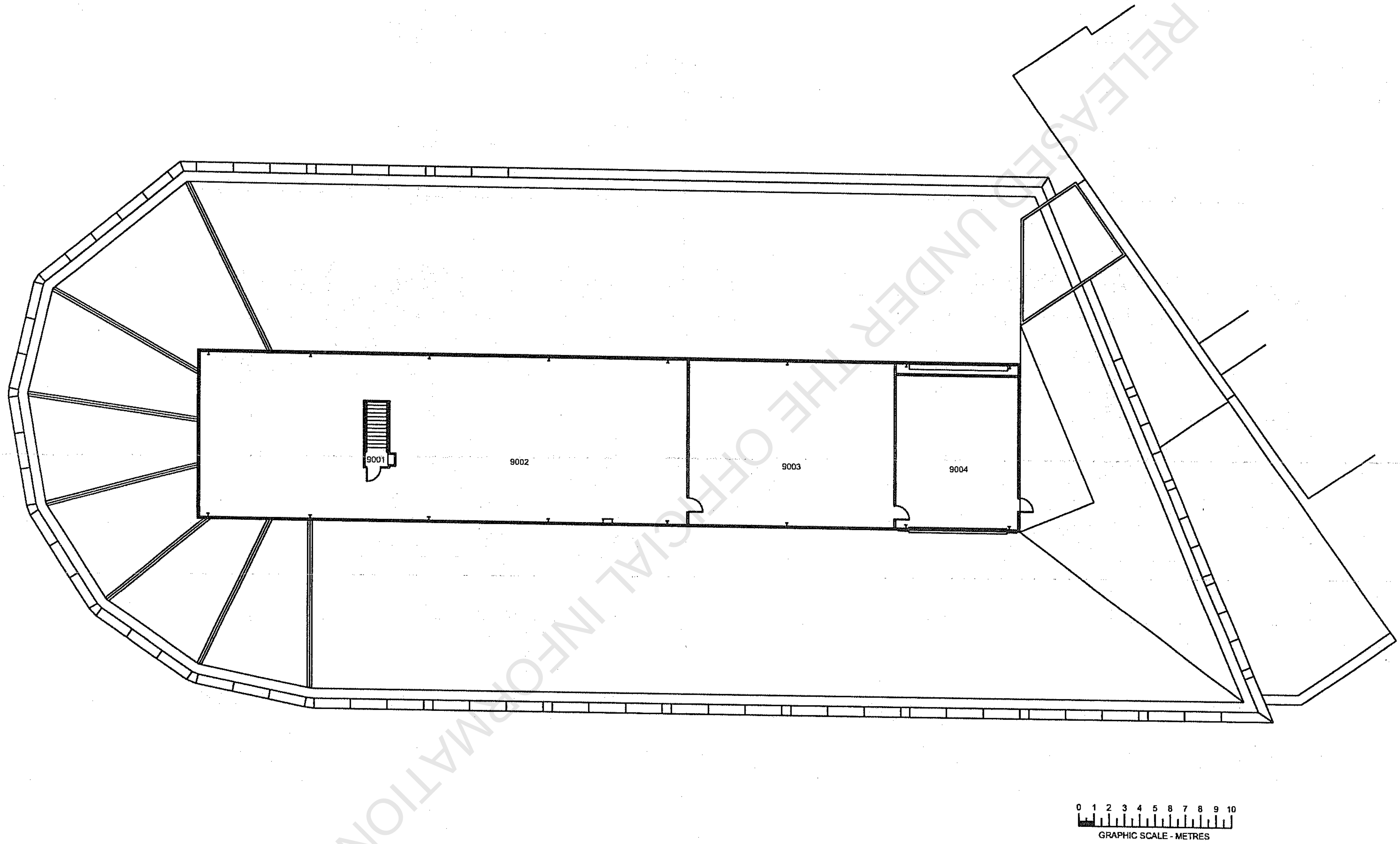
CHRISTCHURCH HOSPITAL
WOMENS - SIXTH FLOOR
SPACE NUMBERS

Canterbury District Health Board
Te Poari Hauora ō Waitaha

DESIGNED:
DRAWN: B.Tudehope
CHECKED:
DATE: 11-12-06
XREFED DRAWING/S: 01298001

DWG No. 01298002
SCALE 1:250 AT A3
REVISED 11-12-06

Maintenance and Engineering Department - Christchurch Hospital



CHRISTCHURCH HOSPITAL
WOMENS - SEVENTH FLOOR
SPACE NUMBERS

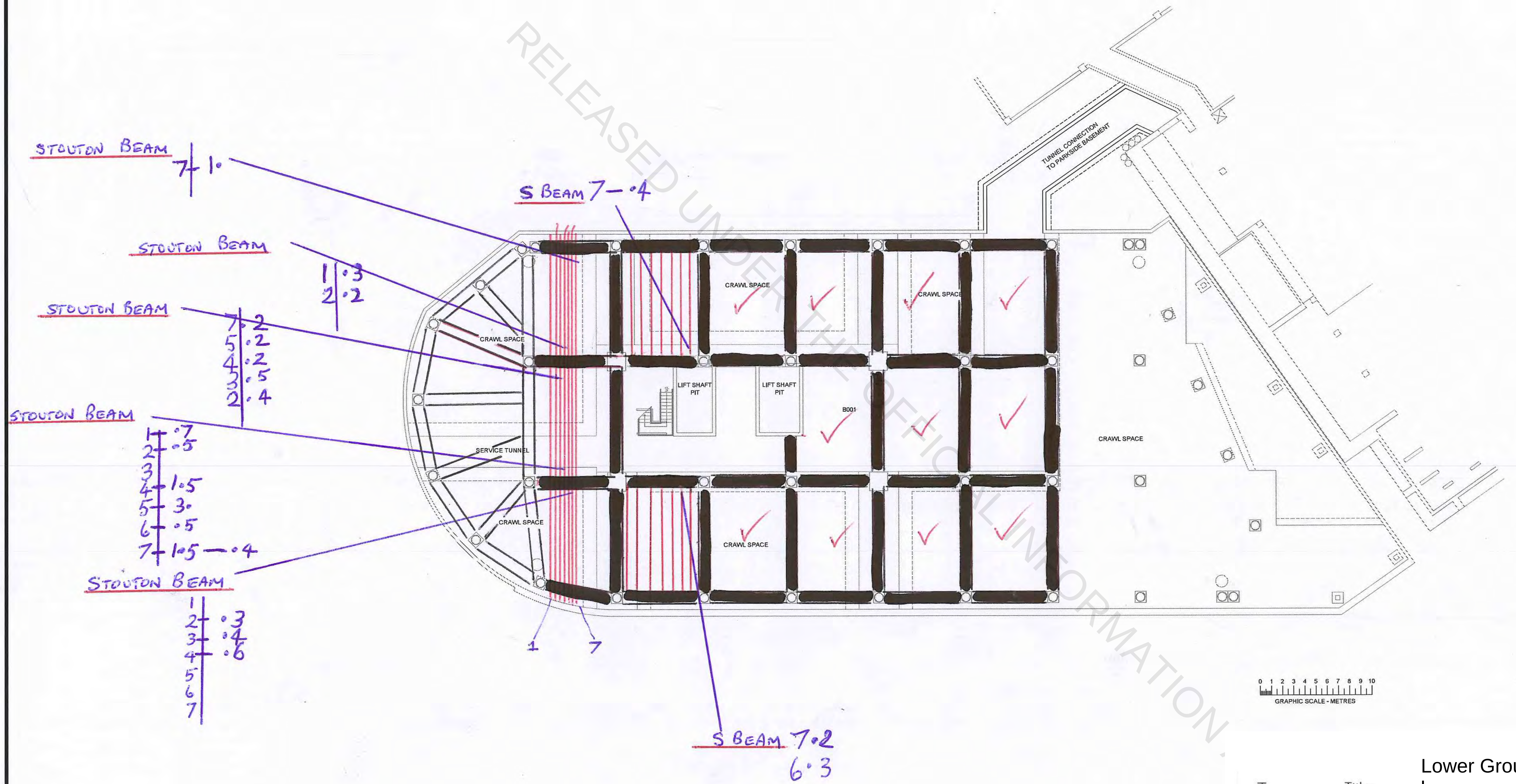
Canterbury District Health Board Te Poari Hauora ō Waitaha	DESIGNED: DRAWN: B.Tudehope CHECKED: DATE: 11-12-06	DWG No. 01299002 SCALE 1:250 AT A3 REVISED 11-12-06
XREFED DRAWING/S: 01299001		

Maintenance and Engineering Department - Christchurch Hospital

APPENDIX C

Remediation Sketches

SKS-C1, SKS-C2, SKS-C3



Title: Lower Ground Floor Stahlton rib crack locs

Job Name: CWH

Job #: 106186.72 SSK#: SKS-C1b

Date: 19/3/12 Rev: 1

HolmesConsultingGroup

AMENDMENT	DRAWN	CHECKED	DATE
AS BUILT	Architect		07-10-05

CHRISTCHURCH HOSPITAL
WOMENS - BASEMENT
FLOOR PLAN

Canterbury
District Health Board
Te Poari Hauora ō Waitaha

DESIGNED: Chow Hill
DRAWN: Opus
CHECKED:
DATE: 07-10-05

DWG No. 01291006
SCALE 1: 200 AT A1
REVISED 15-11-05

XREFED DRAWING/S:

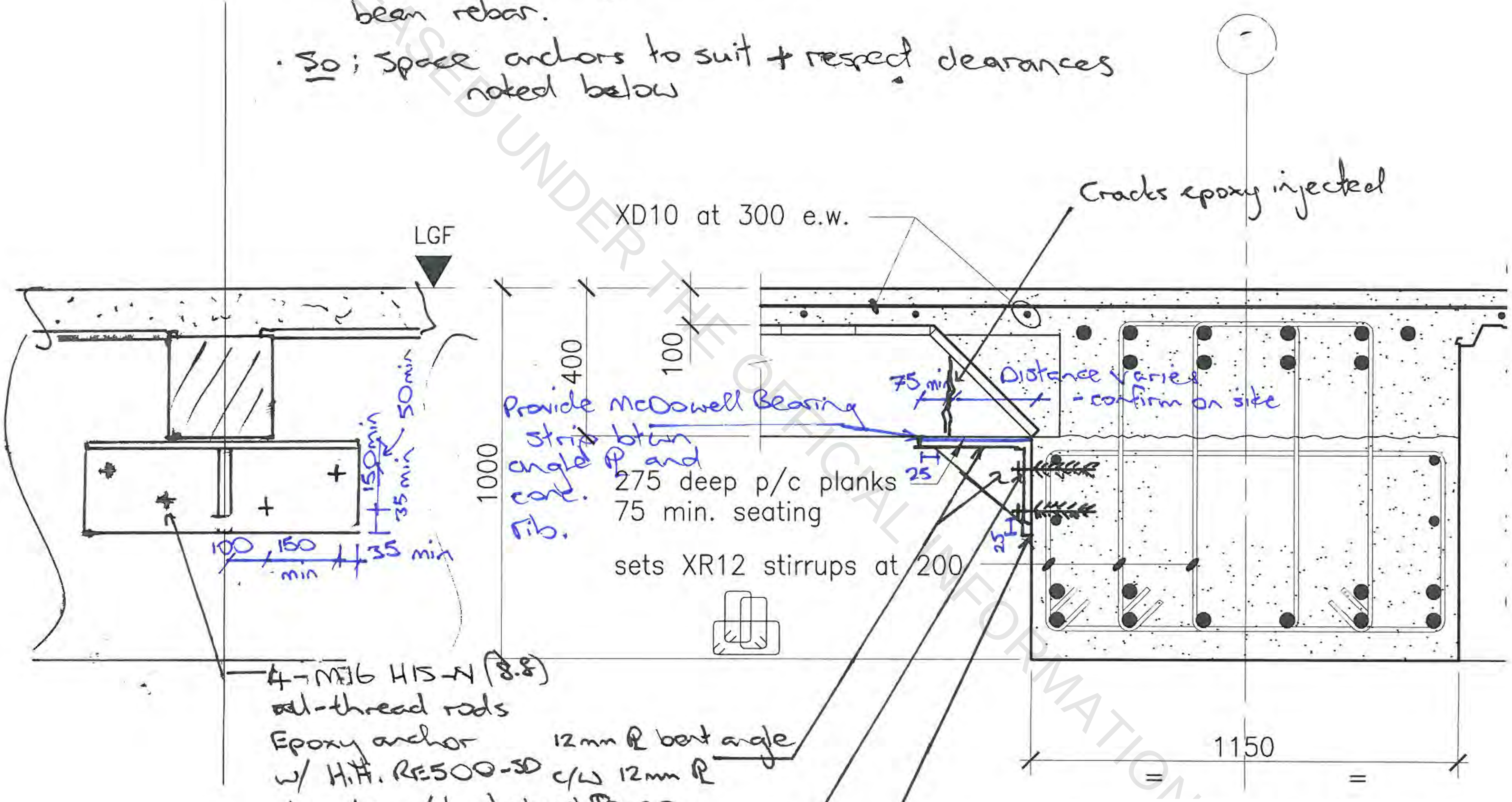
Maintenance and Engineering Department - Christchurch Hospital

19/3/12.

Joe

NB; Prescan existing conc. beam & locate rebar.
- position epoxyed anchors to not cut beam rebar.

· So; space anchors to suit + respect clearances noted below



4-M16 HIS-N (8.8) all-thread rods
Epoxy anchor w/ Hilti RE500-SD into clean/dry hole
Embed 200 mm min.

12mm PL bent angle c/w 12mm PL stiffener

5

Non-shrink grout between vert face of PL and conc beam (f'_c = 40MPa

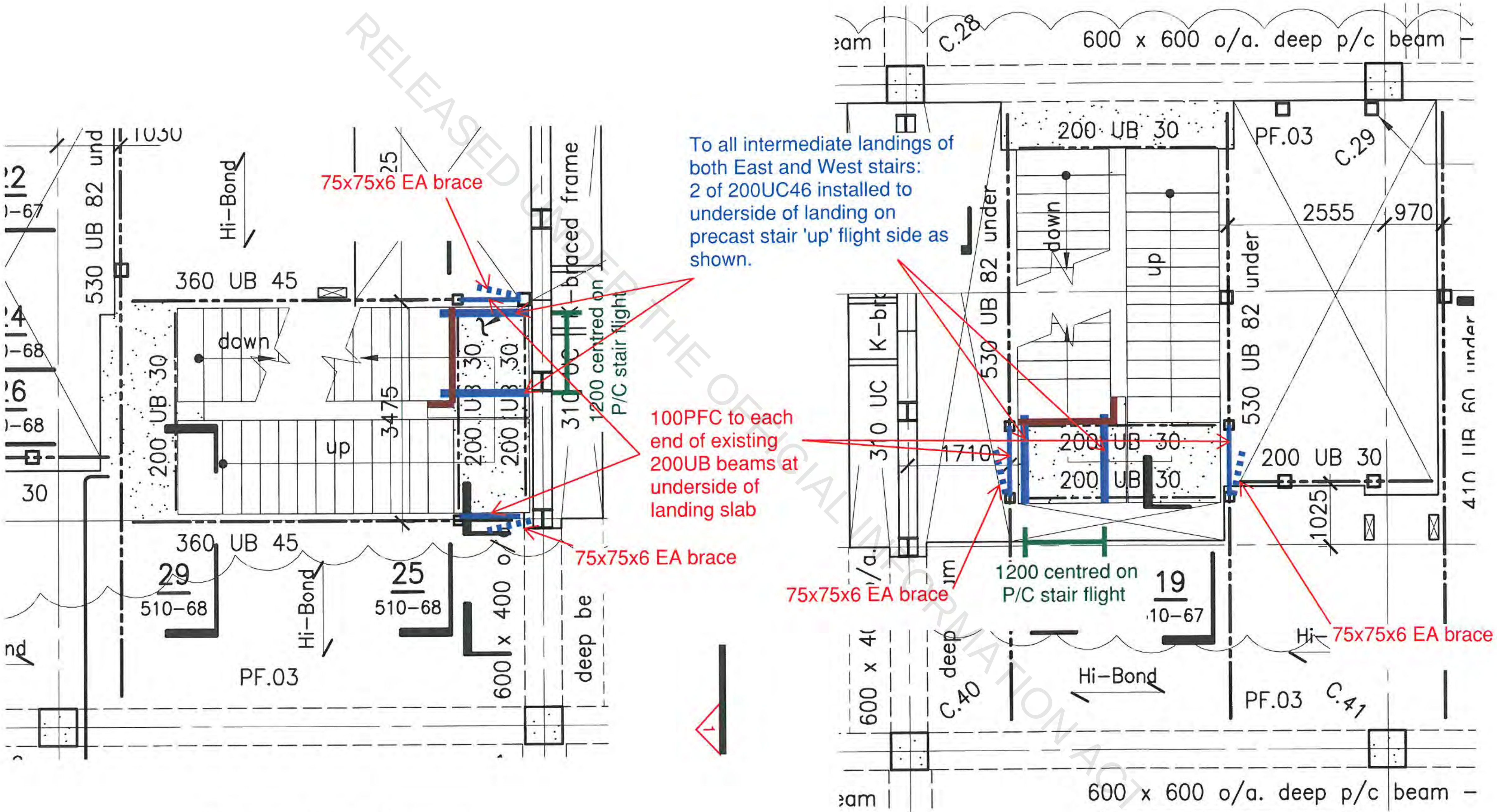
HolmesConsultingGroup

Title: Lower Ground Floor Stahlton rib seat

Job Name: CWH

Job #: 106186.72 SSK#: SKS-C1b

Date: 19/3/12 Rev: 1



East Stair Plan

West Stair Plan

N.T.S

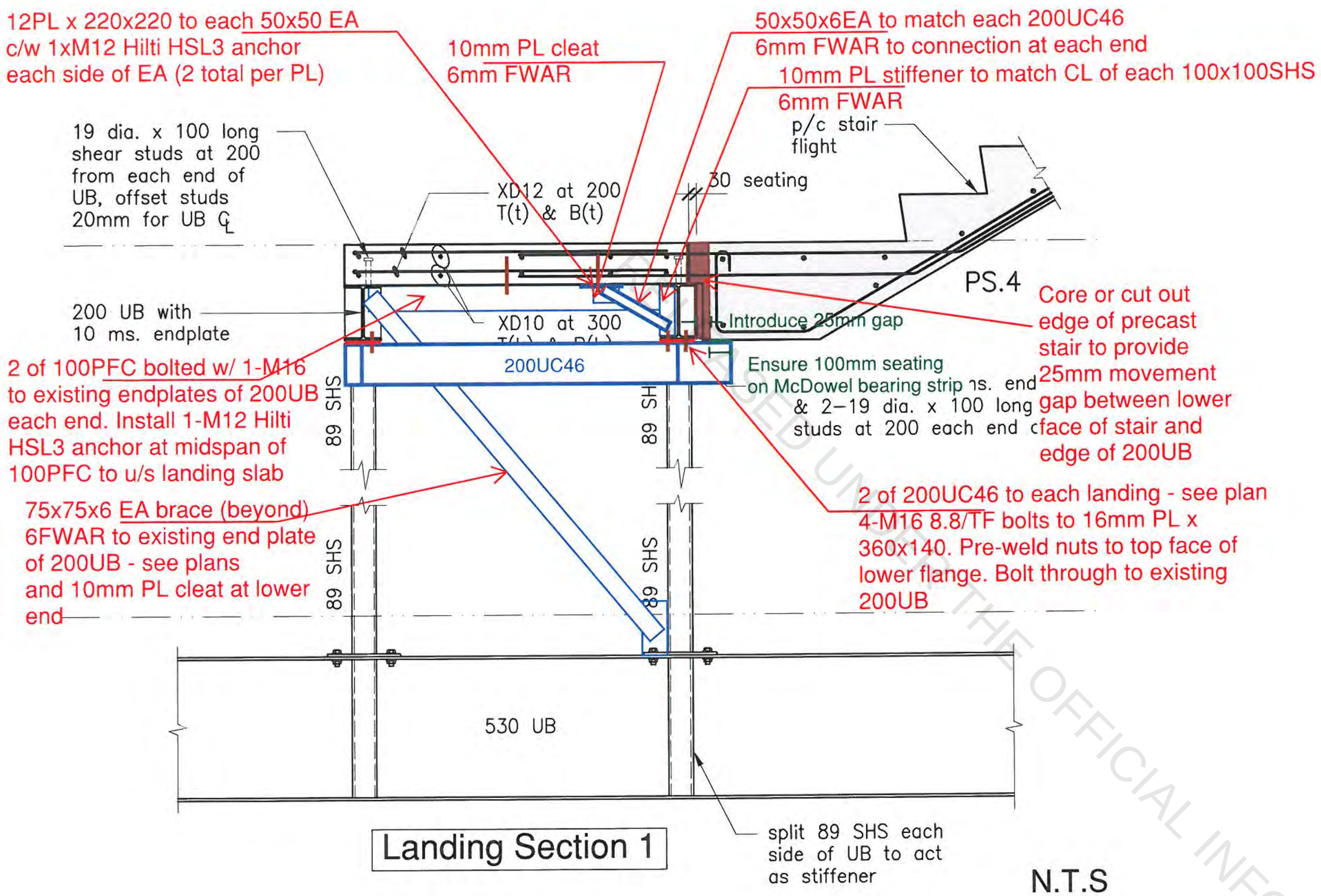
Holmes Consulting Group

Job Name: CWH Stair Remediation

Title: East and West Stair Plans

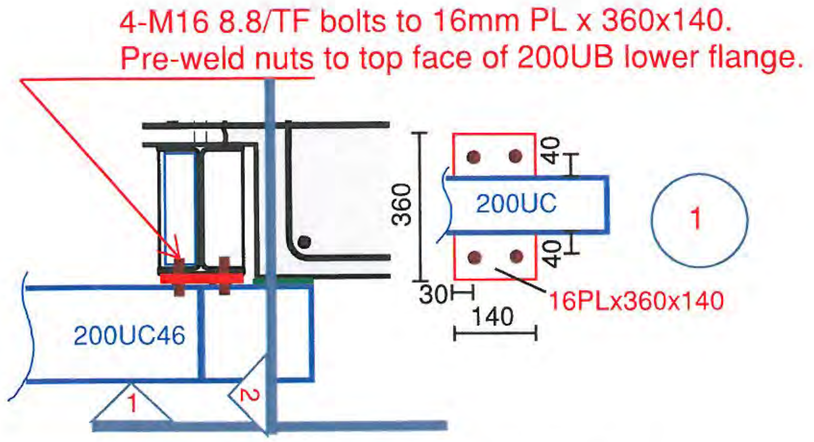
Job #: 106186.72 Sketch#: SKS-C2a

Date: 20/9/12 Drawn: JDP Rev: 1

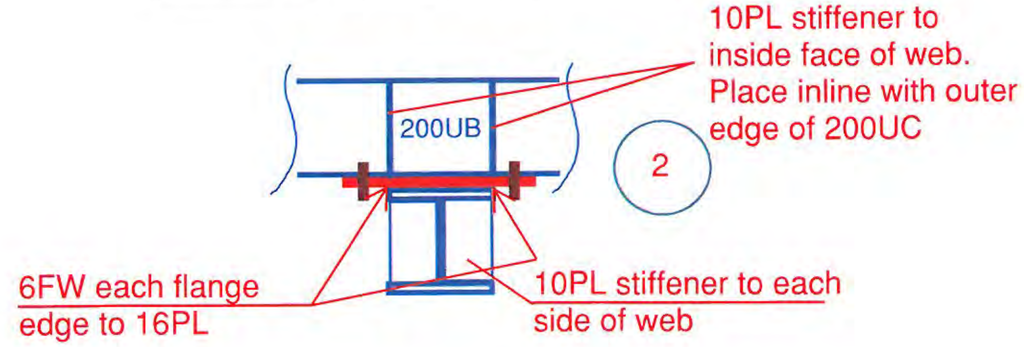


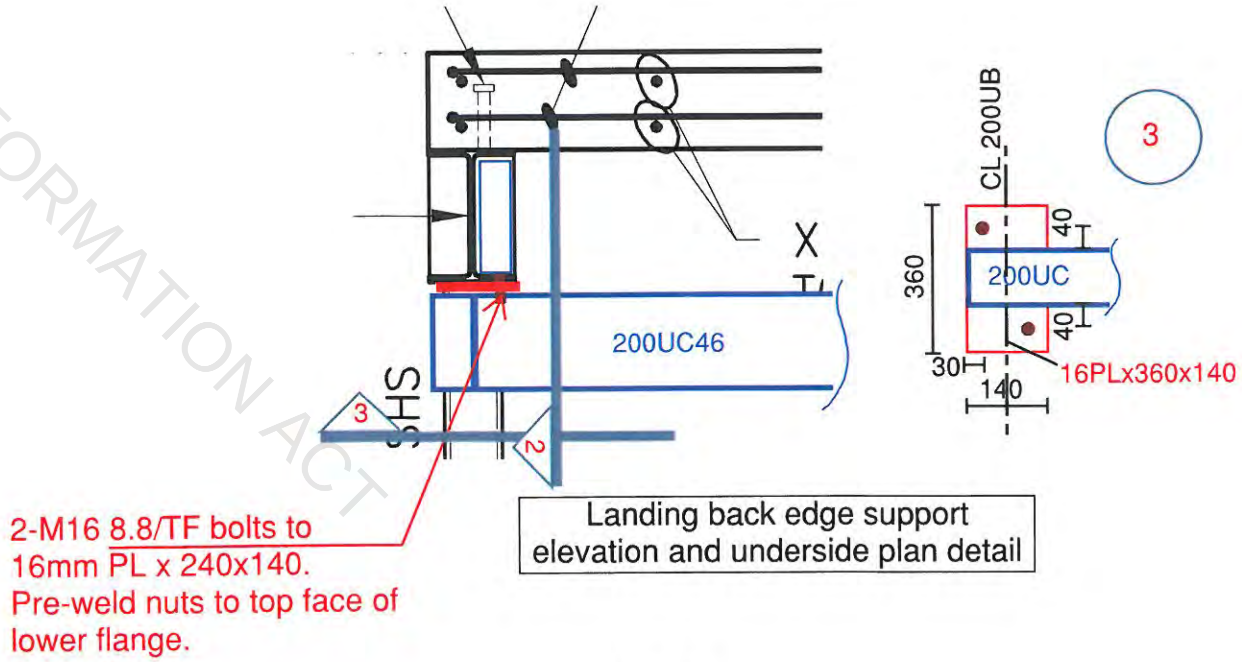
Landing Section 1

N.T.S



P/C stair end support elevation and underside plan detail





Landing back edge support elevation and underside plan detail

N.T.S

Notes:

1. Consider providing temporary propping to share stair load and maintain even support as drilling is carried out.
2. Ensure that new seating 200UC beams + bearing strip are snug to underside of PC stairs to prevent settlement when stairs are released is transferred. Weld shims to top of 200UC if additional fitting required to maintain stair level/elevation.
3. Scan landing slab prior to drilling for mechanical anchors to ensure that reinforcement is not cut
4. Provide covering protection of new movement gap to ensure that gap can never be filled at a later time
5. If bolting options are available for braces then these will be considered with review by the structural engineer
6. 75x75 EA braces are to fit within wall cavity each side of landing
7. Provide for leveling compound/finish to provide even surface from PC stair to landing
8. Tighten bolts to friction-tight fit.
9. All web stiffeners 10PL c/w 6FWAR
10. Site measure and confirm all dimensions and locations

HolmesConsultingGroup

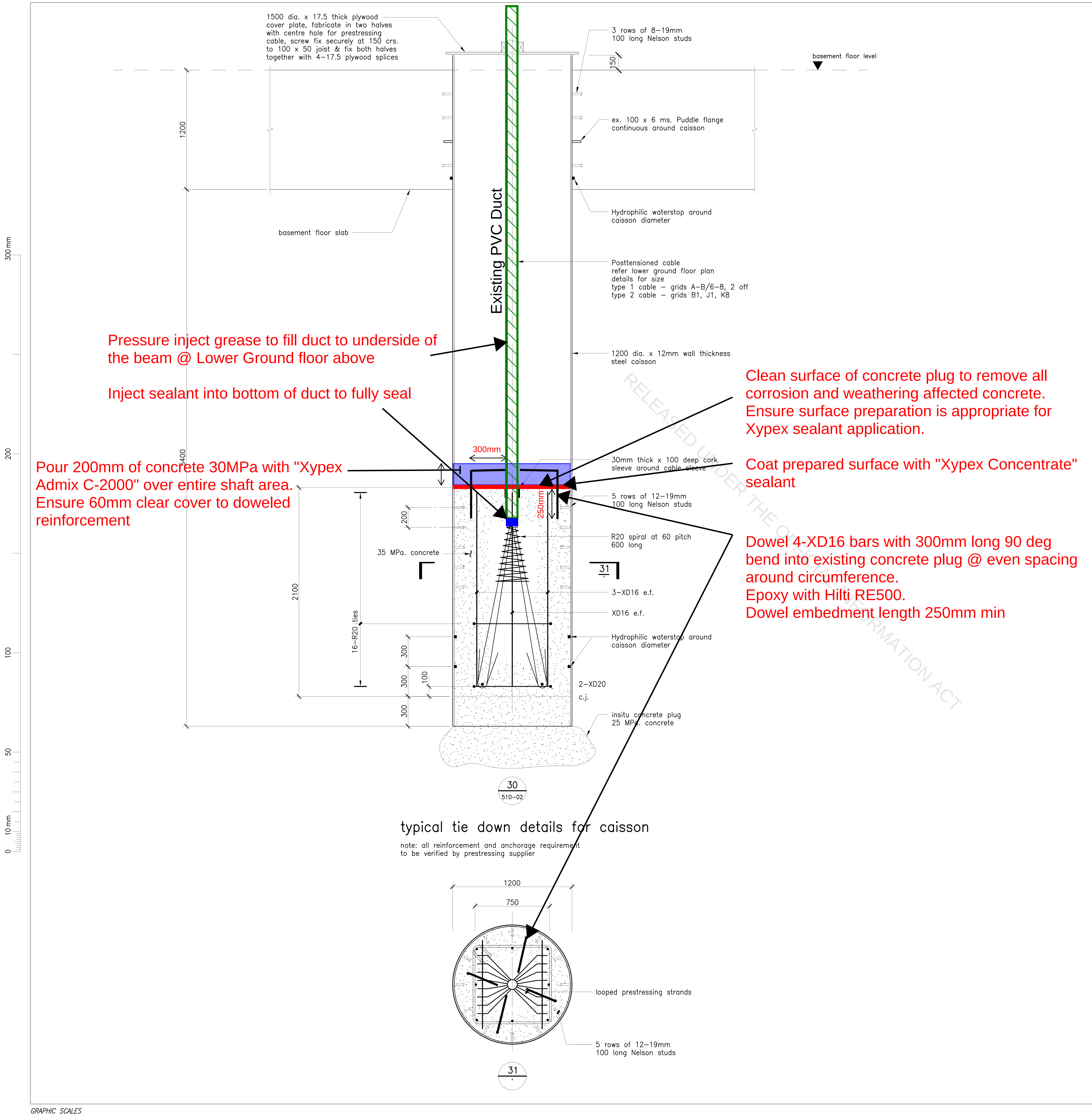
Job Name: CWH Stair Remediation

Title: Details

Job #: 106186.72 Sketch#: SKS-C2b

Date: 20/9/12 Drawn: JDP Rev: 1

Title: Tie Down Caisson waterproofing plug
Job Name: CWH
Job #: 106186.72 SSK#: SKS-C3
Date: 8/2012 Rev: 1



A	Construction Issue		
		CLL	08.11.02
3	Notice to Tenderers		
		DJM	09.09.02
2	Tender Issue		
		DJM	02.09.02
1	Quantity Surveyor Issue		
			12.06.02
	AMENDMENT	APP'D	DATE

	BY	CHECKED	DATE
DESIGN			
DRAWN	C.L.L.		
APPROVED			

HolmesConsultingGroup
STRUCTURAL AND CIVIL ENGINEERS

Canterbury DHB
District Health Board
Te Poari Hauora o Waitaha

Chow:Hill architects
DEVELOPMENT CONSULTANTS
P O Box 23-593, Papatoetoe Auckland
Tel: +64 9 277 8260
Fax: +64 9 277 8261

OPUS
ARCHITECTURE
PO Box 1482 Christchurch, NZ
Tel: 03 363 5400
Fax: 03 365 7858

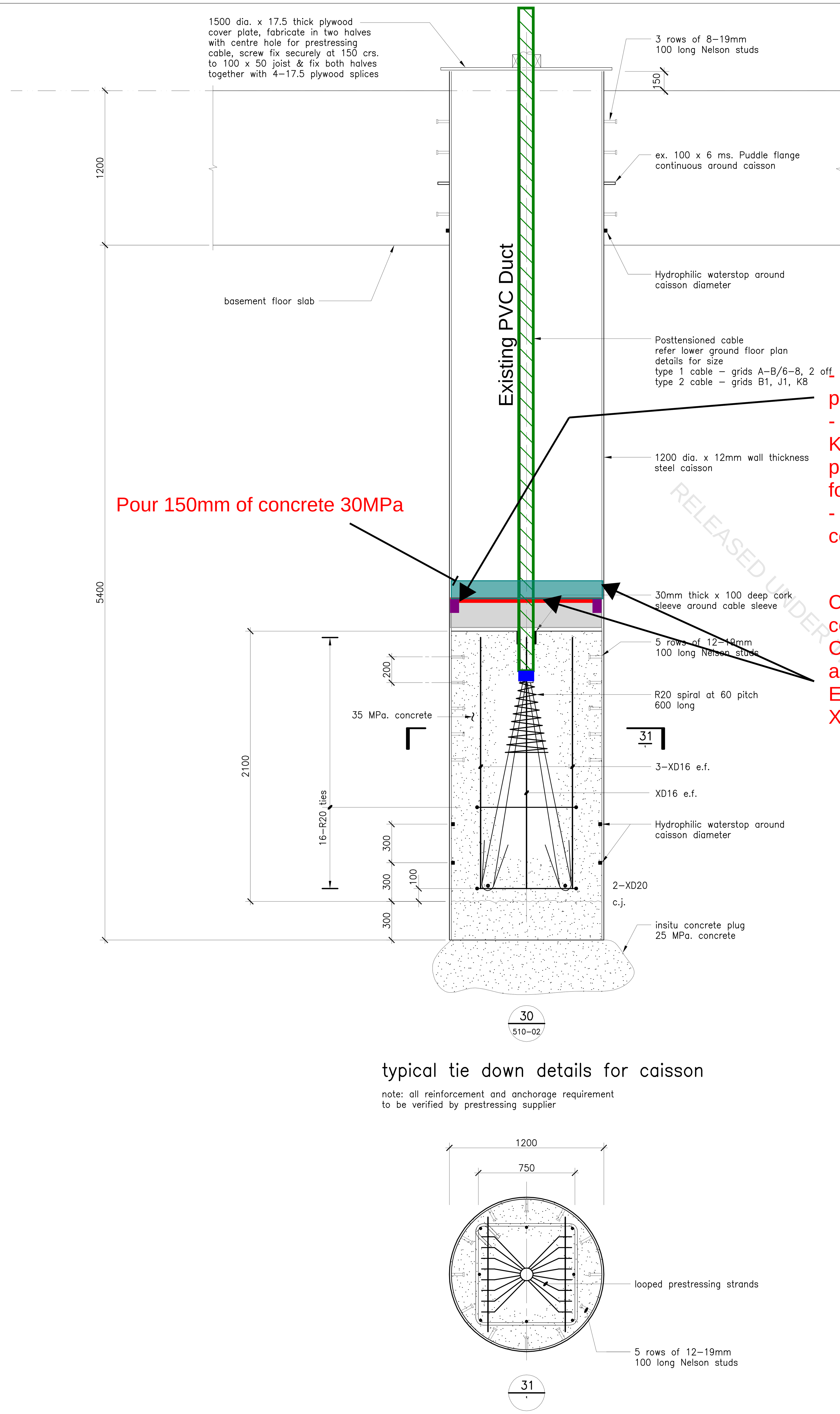
TITLE
CANTERBURY DISTRICT HEALTH BOARD
NEW WOMENS HOSPITAL
and DAY SURGERY UNIT
CHCH. HOSPITAL, RICCARTON AVENUE

FLOOR PLAN
basement floor
details - sheet 8
B

STATUS WORKING DRAWING
SCALE 1 : 20
PLOT DATE 08.11.02
FILE 511-12
CODE
FEATURE IDENTIFIER 30500
REVISION A
SHEET 511-12

ORIGINAL SHEET SIZE A1 [840x594]

CWH Site Report 56
Tie-Down waterproofing
SR59 SKS-001
HCG



- Chip out 40 DP x25mm chase around perimeter of previous plug repair concrete.
- Clean out chase and place 5mm strip of Kuniseal C-31DS waterstop around full perimeter. Butt ends as directed by supplier to form continuous strip
- Cover waterstop with Xypex Patch n Plug combined with Xypex Xycrylic Admix

Clean surface of concrete plug to remove all corrosion and weathering affected concrete.
Clean surface of steel caisson shaft to remove all corrosion.
Ensure surface preparations are appropriate for Xypex and Kuniseal application.

A	Construction Issue	CLL	08.11.02
3	Notice to Tenderers	DJM	09.09.02
2	Tender Issue	DJM	02.09.02
1	Quantity Surveyor Issue		12.06.02
	AMENDMENT	APP'D	DATE

	BY	CHECKED	DATE
DESIGN			
DRAWN	C.L.L.		
APPROVED			

HolmesConsultingGroup
STRUCTURAL AND CIVIL ENGINEERS

Canterbury DHB
District Health Board
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P O Box 23-593, Papatoetoe
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Fax: +64 9 277 8261
DEVELOPMENT CONSULTANTS

OPUS ARCHITECTURE
PO Box 1482
Christchurch, NZ
Tel: 03 363 5400
Fax: 03 365 7858

TITLE
CANTERBURY DISTRICT HEALTH BOARD
NEW WOMENS HOSPITAL
and DAY SURGERY UNIT
CHCH. HOSPITAL, RICCARTON AVENUE

FLOOR PLAN
basement floor
details - sheet 8
B

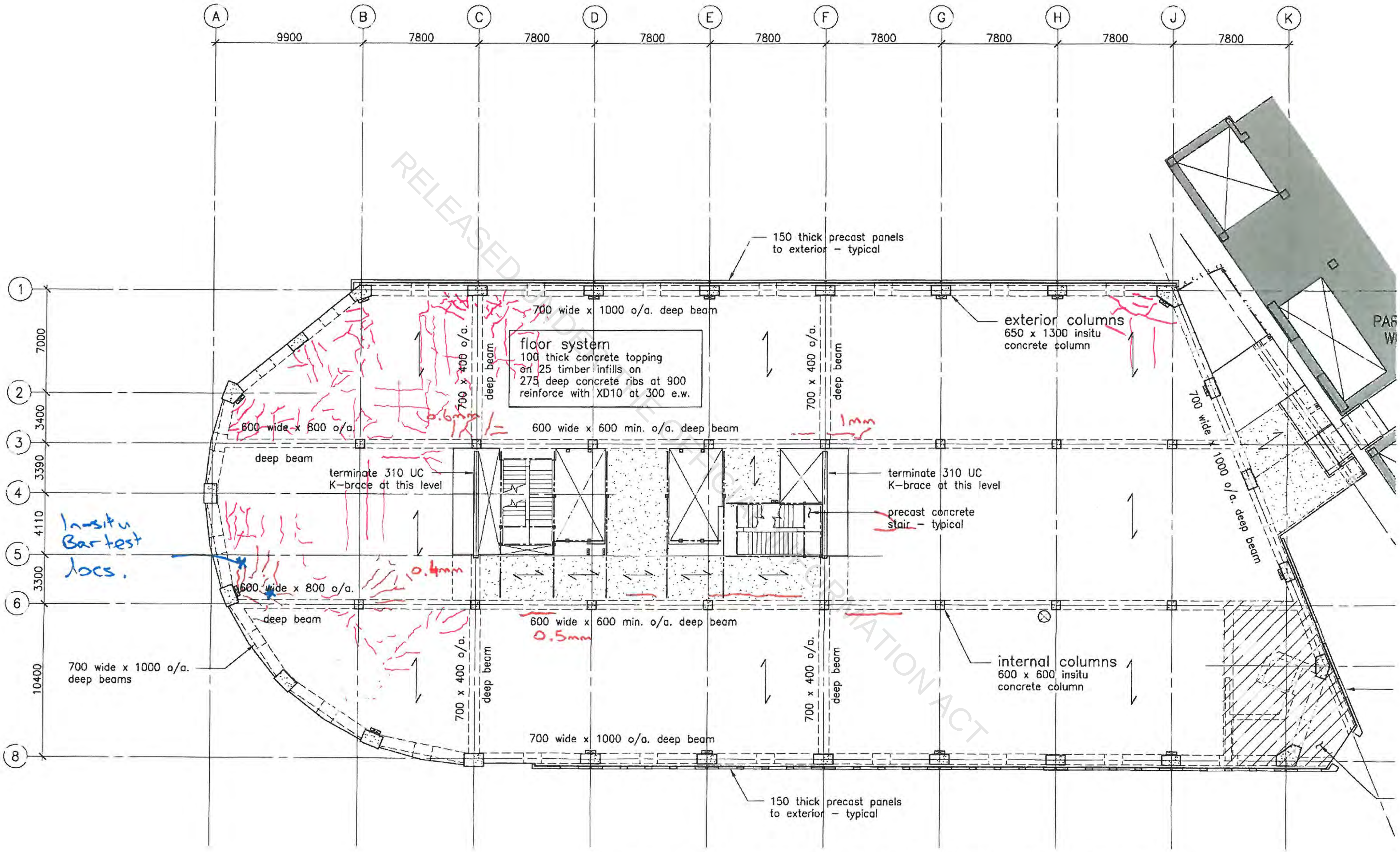
STATUS	WORKING DRAWING		
SCALE	1 : 20	PLOT DATE	08.11.02
FILE	511-12	CODE	
FEATURE IDENTIFIER 30500	REVISION A	SHEET	511-12

GRAPHIC SCALES
0 10 mm 50 100 200 300 mm

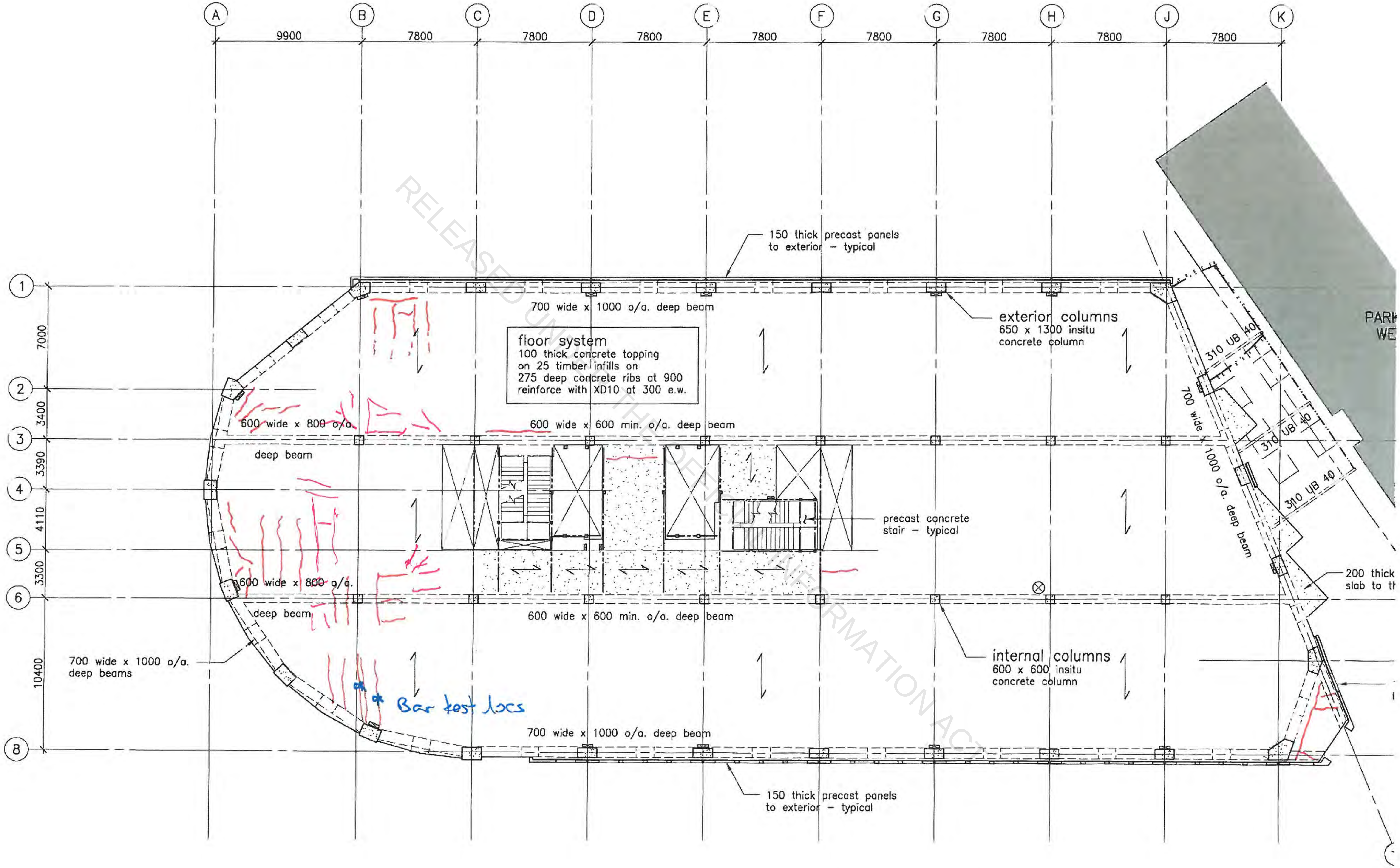
ORIGINAL SHEET SIZE A1 [840x594]

APPENDIX D

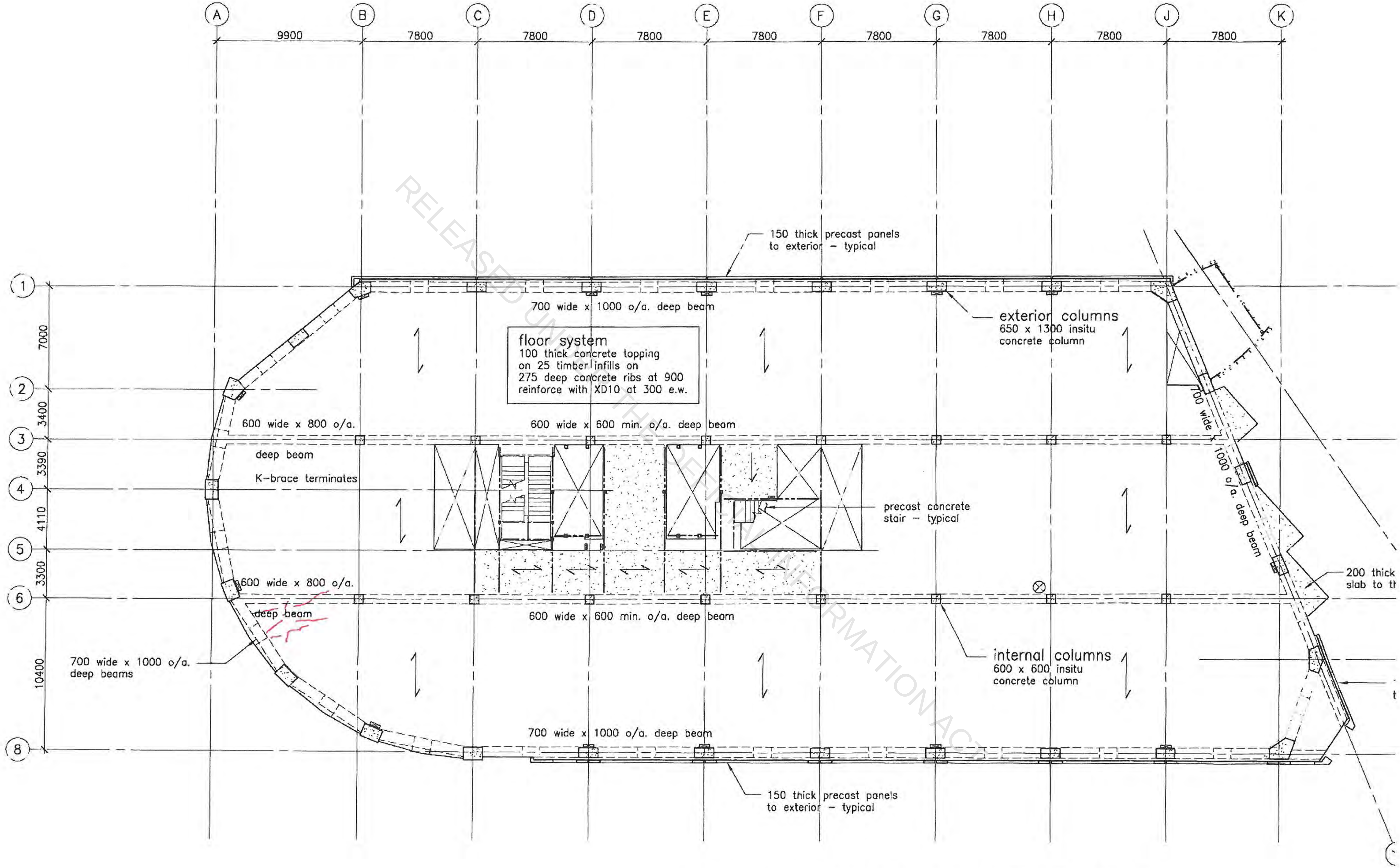
Approximate Crack Maps Levels 3, 4 and 5



THIRD FLOOR PLAN: APPROX CRACK MAP FOR AREAS INSPECTED UP TO 12/6/2013



FOURTH FLOOR PLAN: APPROX CRACK MAP FOR AREAS INSPECTED UP TO 12/6/2013



FIFTH FLOOR PLAN: APPROX CRACK MAP FOR AREAS INSPECTED UP TO 12/6/2012

APPENDIX E

In-situ Slab Reinforcement Testing
report from Holmes Solutions

HOLMESSOLUTIONS

REPORT 108281 (V1.0)

PREPARED FOR HOLMES CONSULTING GROUP

MAY 2012

NON-DESTRUCTIVE TESTING OF THE REINFORCING STEEL IN FLOOR SLABS OF CHRISTCHURCH WOMANS HOSPITAL

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)
MANAGING DIRECTOR

Report Reviewed by:

Andrew Diehl, B.E (Hons)
SENIOR MECHANICAL ENGINEER

REV NO.	DATE	REVISION
V1.0	22/05/12	Issued for client review

UNIT 5, 295 BLENHEIM RD, PO BOX 6718, RICCARTON CHRISTCHURCH, NEW ZEALAND
T+64 3 363 218 F+64 3 379 2169 WWW.HOLMESSOLUTIONS.COM

1.0 EXECUTIVE SUMMARY

Following the recent Christchurch earthquakes a series of cracks were observed in the topping slab of the suspended floor slabs in various locations of the Christchurch Womans Hospital. Concerns were raised about potential damage of the reinforcing steel in areas near the observed cracking. Holmes Solutions were commissioned to investigate the potential damage or degradation that may have resulted in the reinforcing steel of a series of critical concrete elements. To minimise further loss of capacity of the structure, the testing of the reinforcing steel was required to be non-destructive. The results from this project are reported herein.

Holmes Solutions utilised an innovative testing protocol to allow an estimation of the current strain state of the reinforcing steel to be found by measuring the hardness of the steel. The 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 series of multi-dimensional correlations are completed on the obtained results to normalise the hardness factors for the on-site conditions and provide an indication of the steels current strain state. This information is then compared to the laboratory measured stress-strain information of the same steel material. The comparison of results allows an estimation of how much strain capacity the reinforcing steel may have lost due to the previously induced damage from inelastic loading cycles.

The majority of the reinforcing bars investigated in this testing programme showed very limited variations in Leeb hardness. Using the derived correlations, these results suggested that the maximum value of average potential strain in the tested steel had typically not exceeded the strain corresponding with the onset of strain hardening. As such, the steel in these locations was considered to have only lost a small percentage of the available strain capacity.

The results obtained for Test Location CWH4D, located towards the centre of room 4069, indicated that the maximum value of average potential induced strain in the bar was 1.6%, resulting estimated maximum potential loss of strain capacity of 17%.

Test Location CWH4A, located near the entrance door to room 4072, was found to have a maximum value of average potential induced strain equal to 0.9%, resulting in an estimated maximum potential loss of strain capacity of 11%.

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 the suitability of 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 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 were 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 series of multi-dimensional correlation factors. When combined with a series of normalisation techniques we can use the measured Leeb hardness results to provide an indication as to the current strain state of the tested steel sample. The degree of uncertainty in the recorded measurements is decreased with an increased volume of data that is collected.

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.

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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, all in situ hardness testing is completed in direct 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 determined 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]. Due consideration is provided to both Type A and Type B errors in the measurement chain. All reported uncertainties are computed as expanded uncertainties with a coverage factor of 2 (k =2), providing a level of confidence of 95% in the reported results.

This technique has been used extensively by Holmes Solutions in reviewing the performance of structures following the recent earthquakes. On occasion, the findings from the hardness testing have been validated through the use of destructive testing on the tested steel samples. 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.
- Rebound phase, where the impact body leaves the test piece with residual energy, not consumed during the impact phase.

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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. LABORATORY TESTING

A 25 mm diameter deformed reinforcing bar was removed from a concrete wall element in basement of the building, a location that was free from cracks and suspected to have been subjected to low imposed stress. The material properties of the steel sample can be assumed to have been unmodified from previous inelastic strain cycles. The sample was subjected to uniaxial tensile testing to determine the material properties. Leeb hardness testing was completed at various levels of applied strain, both with the load applied and with the load removed from the steel, thereby allowing correlations between imposed steel stress and hardness to be developed for each steel sample. In addition, the derived correlations were benchmarked against previous testing completed on material samples extracted from previously tested buildings with reinforcing steel of the same or similar composition. The obtained stress-strain response from the tested steel sample is shown in Figure *1* below.

The steel sample was subjected to unidirectional cyclic tensile testing rather than cycles of reverse cyclic loading to near equal values of tensile and compressive strain. Experimental investigations has indicated that the correlation between steel Hardness and imposed plastic stress is unaffected by imposed strain history or loading rate [M2]. Furthermore, in the structure, 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 would have been subjected to very 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.

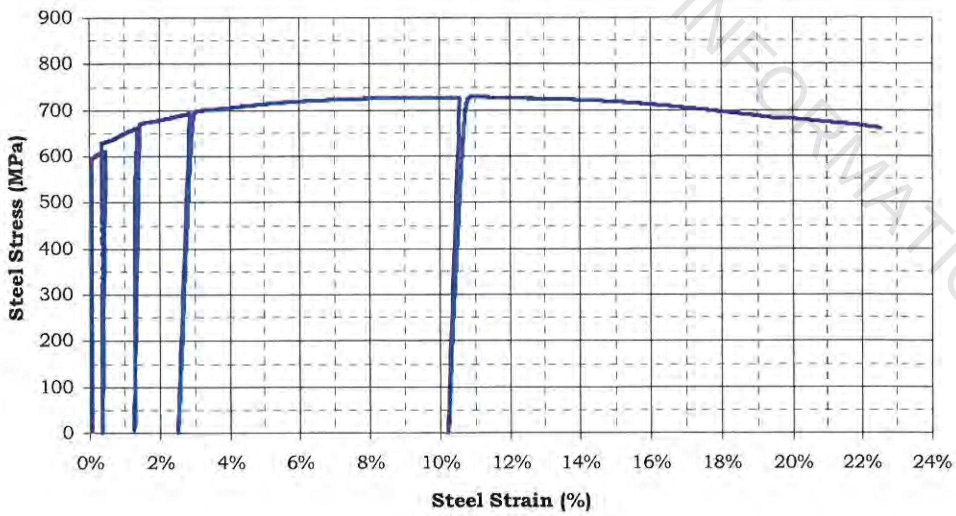

Figure 1 Materials Test Result for the Steel test coupon

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Table 1 Material Properties from Removed Test Sample

Yield Stress, f_y:	598	(MPa)
Strain Hardening Stress, f_{sh}:	601	(MPa)
Ultimate Stress, f_u:	726	(MPa)
Stress Ration, f_u/f_y:	1.21	
Yield Strain, ε_y:	0.2	(%)
Strain Hardening Strain, ε_{sh}:	0.6	(%)
Ultimate Strain, ε_u:	11.5	(%)
Baseline Leeb Hardness:	701	(HLDL)

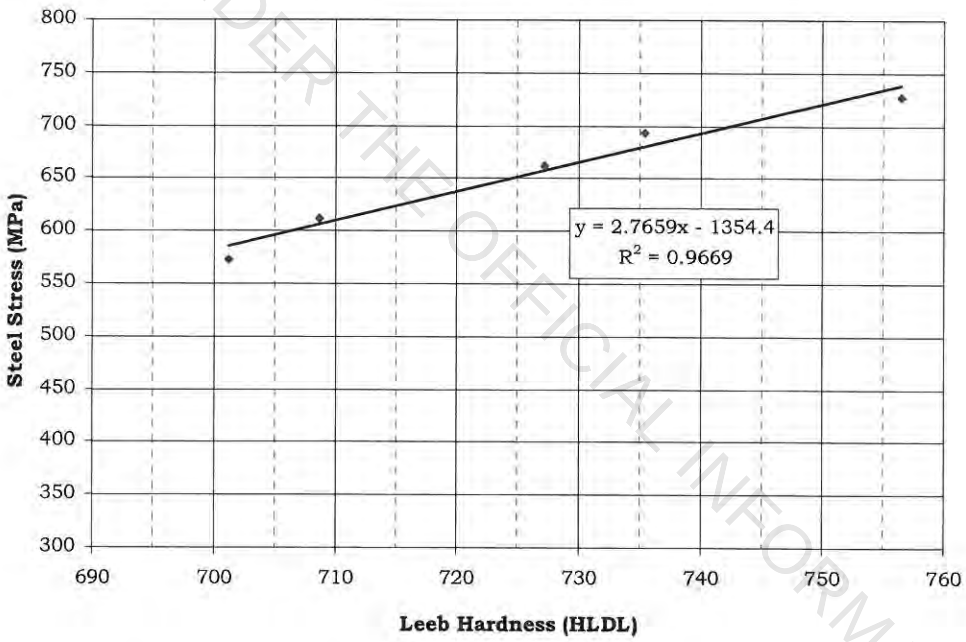

Figure 2 Correlation between Leeb Hardness and Steel Stress

It is our understanding that the steel used in the building complied with the requirements of Grade 500E reinforcement, as detailed in AS/NZS 4671. The recorded yield stress of the material was towards the upper limitation of allowable yield stresses for Grade 500E steel. The stress ratio (f_u/f_y) of the tested steel sample was determined to be 1.21; sitting approximately mid-range of the allowable limits imposed by AS/NZS 4671. This value indicates that the steel has a good likelihood of spreading the zone of yield along the bar. The steel exhibited no significant yield plateau region during the testing.

The baseline Leeb value hardness for the steel was determined to be 701 HLDL. This value will be used as the baseline for all steel analysis completed in this report.

The recorded Leeb hardness for the tested steel sample as a function of the imposed steel stress is presented in Figure 2. A series of 6 individual Leeb hardness test results were completed and the Truncated Mean determined to produce each

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reported value of Leeb hardness. The results indicate a strong correlation across the plastic stress range.

It was noted that the general slope of the Leeb Hardness to Steel Stress plot was relatively low, indicating that the steel showed a limited range of hardness increase with an associated increase in steel stress. This is consistent with similar findings for other buildings with similar material properties. The low variation in hardness reduces the sensitivity of the tested samples and increases the associated uncertainties in the in-situ testing correlations

The reported values of hardness were derived from the steel sample supported in the universal testing machine. Additional hardness tests were completed on the tested steel sample with the bar fully supported in a cement matrix. The results from this testing were used to assist in the normalisation of the in-situ Leeb information, and to provide guidance on the correlations necessary to correct for variation in support conditions.

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 name of the building, the date of the testing, and the individual test location. 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.

Prior to undertaking Leeb hardness testing it is 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 exposed crack, allowing the properties of each steel section to be captured in areas considered to have been subjected to low inelastic demands.

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. In all locations the support conditions for the bar were considered variable or inadequate and additional grout was place around the steel prior to testing. Correlations procedures for the support conditions of each steel location were completed during the post processing of the data.

To assist in the correlations and normalisation of the obtained Leeb hardness data, additional reinforcing bars were exposed for testing remote from any observed cracking. This steel was considered unaffected by any previous inelastic yielding. Testing was competed on this section of reinforcing as a control sample to be used in the normalisation procedures on the tested reinforcing steel.

The results from the Leeb Hardness testing are present 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 key 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.

It should be noted that often the hardness is seen to vary significantly at the ends of the tested region. This variation is typically caused by significant changes in the support conditions of the reinforcing steel in these locations rather than actual variations in the steel properties. Variations in 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. If it is considered the results have been unduly influenced, it may be necessary to remove the steel that crosses the test region to ensure consistent support conditions and to allow better access for testing.

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 (k =2), providing a level of confidence of 95% in the reported results.

The average potential induced strains values are derived directly from the statistical truncated mean of the normalised Leeb hardness values and is reported with a level of confidence of 95%. It should be noted that due to the 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.

Given the correlation between Leeb Hardness and imposed steel stress is primarily applicable for strain hardening region of the steel stress-strain response, it is difficult to determine the value of imposed strain at levels below the strain hardening (ε_{sh}). As such, any values of derived strain below the 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 90% of the peak strains are only reported as such.

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TEST LOCATION: CWH3AB

Test Location CWH3AB was located near the window in room 3089 on the 3rd floor of the building. The test region was chosen to cross a single crack in the floor element. The cover concrete was removed exposing two bars, designated CWH3AB-A and CWH3AB-B respectively. Location CWH3AB-A did not cross the crack surface.

The hardness results for location CWH3AB-A showed little variation in Leeb Hardness results along the length of the test region. The results suggest that the steel had undergone no inelastic deformations.

The hardness results for location CWH3AB-B showed minor variation in Leeb Hardness results. With full consideration for the uncertainty in the measured results, the maximum value of average potential induced strain was determined to be 0.8%, marginally exceeding the onset of strain hardening observed in the laboratory testing completed on the steel sample. It should be noted that this value occurred in the immediate vicinity as it intersecting with CWH3AB-A and as such the increased hardness value may have occurred due to variation in the support conditions.

The level of potential induced strain in the reinforcing bar suggested that the maximum potential loss of strain capacity could be 8% of the peak strain capacity (where peak strain is defined as the strain corresponding with the peak stress of the material).

Figure 3 Test Location CWH3AB

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Table 2 *Test Results for Location CWH3AB-A*

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 (%)
CWH3AB-A-01	15	Normal	710	705	4	0.00	3 - 5%
CWH3AB-A-02	30	Normal	710	705	2	0.00	3 - 5%
CWH3AB-A-03	45	Normal	709	704	3	0.00	3 - 5%
CWH3AB-A-04	60	Normal	706	701	4	0.00	2 - 3%
CWH3AB-A-05	75	Normal	708	703	3	0.00	3 - 3%
CWH3AB-A-06	90	Normal	710	705	4	0.00	3 - 5%
CWH3AB-A-07	105	Normal	708	703	2	0.00	3 - 3%
CWH3AB-A-08	120	Normal	707	702	2	0.00	3 - 3%
CWH3AB-A-09	135	High	713	704	3	0.00	3 - 5%
CWH3AB-A-10	150	High	717	705	3	0.00	3 - 5%
CWH3AB-A-11	165	Normal	712	707	4	0.60	3 - 5%
CWH3AB-A-12	180	Normal	706	701	4	0.00	2 - 3%
CWH3AB-A-13	195	Normal	705	700	2	0.00	3 - 3%
CWH3AB-A-14	210	Normal	705	700	4	0.00	2 - 3%
CWH3AB-A-15	225	Normal	703	698	4	0.00	2 - 3%
CWH3AB-A-16	240	Medium	700	702	4	0.00	3 - 5%
CWH3AB-A-17	255	Normal	706	701	3	0.00	3 - 3%
CWH3AB-A-18	270	Normal	713	708	5	0.60	3 - 5%

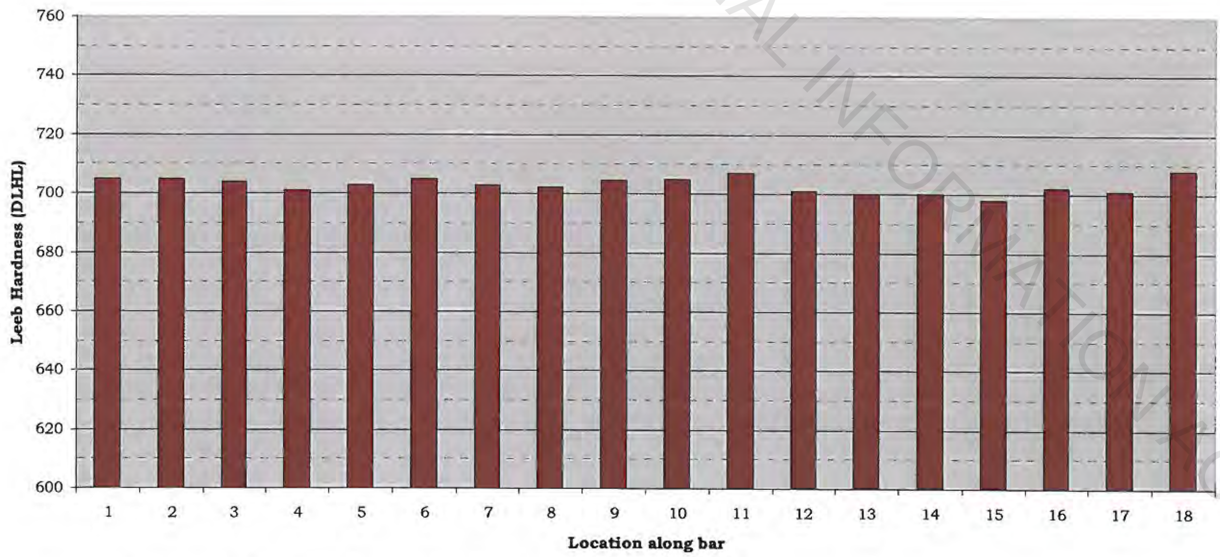

Figure 4 Leeb Hardness plot for Location CWH3AB-A

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Table 3 *Test Results for Location CWH3AB-B*

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 (%)
CWH3AB-B-01	15	Normal	698	703	4	0.00	3 - 5%
CWH3AB-B-02	30	Normal	699	704	4	0.00	3 - 5%
CWH3AB-B-03	45	Normal	699	704	2	0.00	3 - 3%
CWH3AB-B-04	60	Normal	697	702	4	0.00	3 - 3%
CWH3AB-B-05	75	Normal	703	708	4	0.60	3 - 5%
CWH3AB-B-06	90	Normal	704	709	3	0.60	5 - 5%
CWH3AB-B-07	105	Normal	695	700	3	0.00	2 - 3%
CWH3AB-B-08	120	Normal	696	701	5	0.00	2 - 3%
CWH3AB-B-09	135	Normal	691	696	4	0.00	2 - 3%
CWH3AB-B-10	150	Normal	697	702	4	0.00	3 - 5%
CWH3AB-B-11	165	Normal	705	710	3	0.61	5 - 5%
CWH3AB-B-12	180	Normal	705	710	4	0.61	5 - 6%
CWH3AB-B-13	195	Normal	705	710	4	0.61	5 - 6%
CWH3AB-B-14	210	Normal	714	719	3	0.77	6 - 8%
CWH3AB-B-15	225	Normal	707	712	4	0.62	5 - 6%
CWH3AB-B-16	240	Normal	699	704	4	0.00	3 - 5%
CWH3AB-B-17	255	Normal	701	706	3	0.60	3 - 5%
CWH3AB-B-18	270	High	708	706	3	0.60	3 - 5%
CWH3AB-B-19	285	High	707	705	2	0.00	3 - 5%

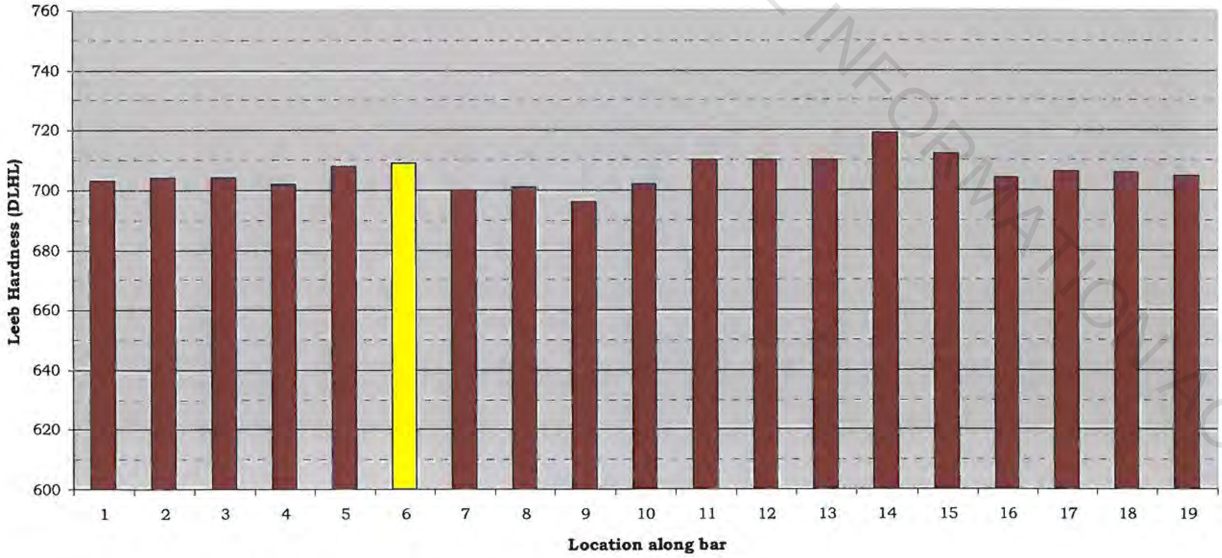

Figure 5 Leeb Hardness plot for Location CWH3AB-B

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TEST LOCATION: CWH3A

Test Location CWH3A was located towards the corridor in room 3089. The test region was chosen to cross a single crack in floor plate. The cover concrete was removed exposing two reinforcing steel bars (12 mm in diameter) in orthogonal directions. The steel bar crossing the noted crack was designated CWH3A-A.

The test results found minor increases in hardness of the steel in the area surrounding the concrete crack location, spreading for a length of up to 150 mm. With full consideration for the uncertainty in the measured results, the maximum value of average potential induced strain was determined to be 0.9% at segment 16. This corresponded to an estimated maximum potential loss of strain capacity of up to 10% of the peak strain capacity.

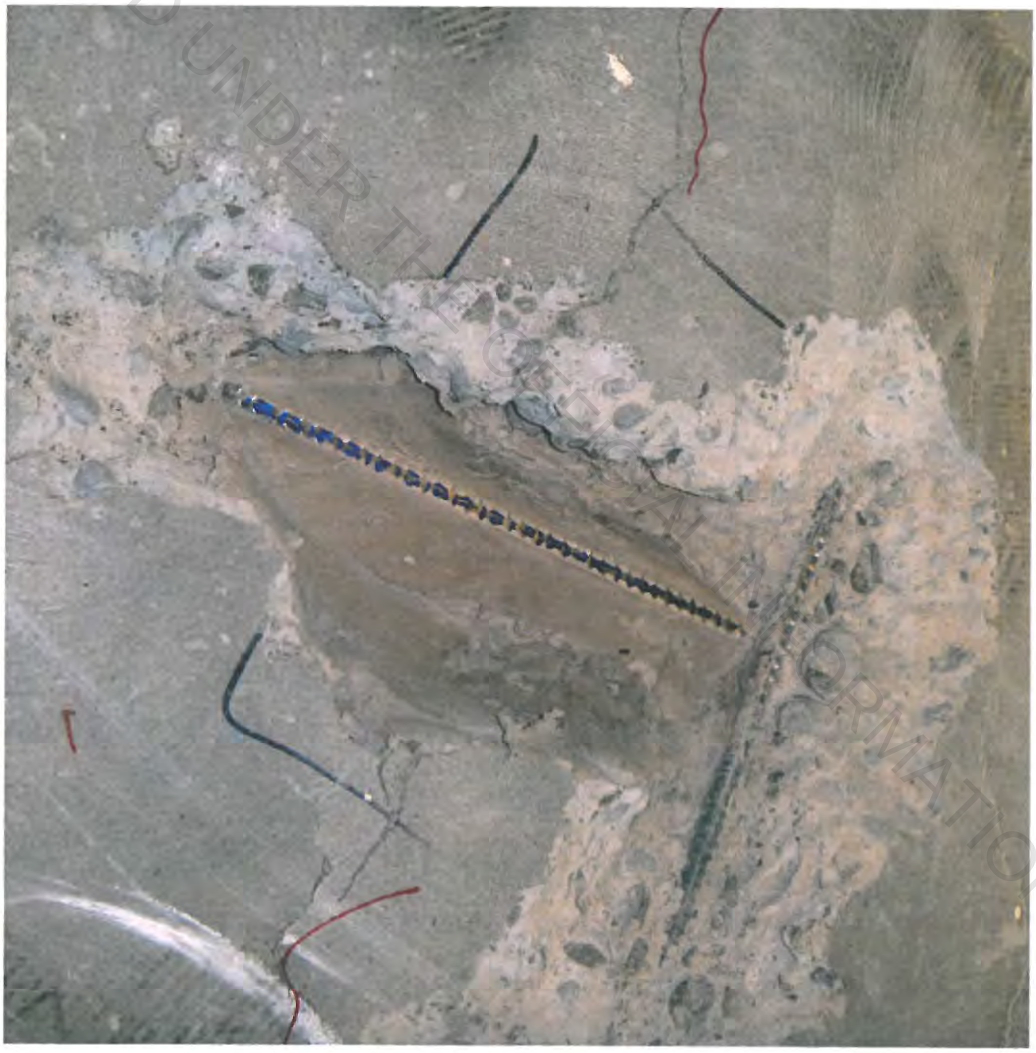

Figure 6 CWH3A-A

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Table 4 *Test Results for Location CWH3A-A*

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 (%)
CWH3A-A-01	15	Normal	662	702	2	0.00	3 - 3%
CWH3A-A-02	30	Normal	662	702	3	0.00	3 - 3%
CWH3A-A-03	45	Normal	662	702	4	0.00	3 - 3%
CWH3A-A-04	60	Normal	661	701	3	0.00	3 - 3%
CWH3A-A-05	75	Normal	663	703	3	0.00	3 - 5%
CWH3A-A-06	90	Normal	664	704	4	0.00	3 - 5%
CWH3A-A-07	105	Normal	667	707	6	0.60	3 - 5%
CWH3A-A-08	120	Normal	664	704	5	0.00	3 - 5%
CWH3A-A-09	135	Normal	675	716	3	0.68	5 - 7%
CWH3A-A-10	150	Normal	673	714	7	0.64	5 - 7%
CWH3A-A-11	165	Normal	669	710	2	0.60	5 - 5%
CWH3A-A-12	180	Normal	673	714	4	0.64	5 - 6%
CWH3A-A-13	195	Normal	671	712	3	0.61	5 - 6%
CWH3A-A-14	210	Normal	679	720	2	0.82	6 - 8%
CWH3A-A-15	225	Normal	667	707	4	0.60	3 - 5%
CWH3A-A-16	240	Normal	680	721	5	0.87	6 - 10%
CWH3A-A-17	255	Normal	667	707	2	0.60	3 - 5%
CWH3A-A-18	270	Normal	657	697	3	0.00	2 - 3%

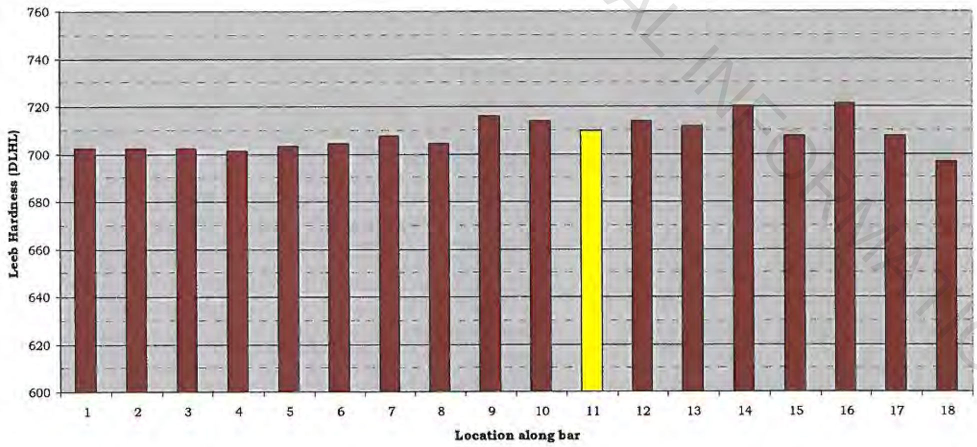

Figure 7 Leeb Hardness plot for Location CWH3A-A

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TEST LOCATION: CWH3B

Test Location CWH3B was located in room 3089, towards the exterior side of the room. The test region was chosen to cross a single crack. Removal of the cover concrete exposed a single 12 mm reinforcing bar crossing the crack and a reinforcing bar of larger diameter running directly underneath the crack location.

The hardness results typically showed very little variation in hardness along the test region, with no noticeable increases in the area of the concrete crack. The results suggest that the steel did not reach the onset of strain hardening in any location and therefore sustained only minor loss of strain capacity as it crossed the crack location.

Figure 8 CWH3B-A

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Table 5 *Test Results for Location CWH3B-A*

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 (%)
CWH3B-A-01	15	Normal	702	702	3	0.00	3 - 3%
CWH3B-A-02	30	Normal	705	705	5	0.00	3 - 5%
CWH3B-A-03	45	Normal	696	696	4	0.00	2 - 3%
CWH3B-A-04	60	Normal	701	701	4	0.00	2 - 3%
CWH3B-A-05	75	Normal	699	699	5	0.00	2 - 3%
CWH3B-A-06	90	Normal	702	702	2	0.00	3 - 3%
CWH3B-A-07	105	Normal	700	700	2	0.00	3 - 3%
CWH3B-A-08	120	Normal	701	701	3	0.00	3 - 3%
CWH3B-A-09	135	Normal	697	697	4	0.00	2 - 3%
CWH3B-A-10	150	Medium	693	700	4	0.00	2 - 3%
CWH3B-A-11	165	Medium	689	703	2	0.00	3 - 3%
CWH3B-A-12	180	Normal	694	694	4	0.00	2 - 3%
CWH3B-A-13	195	Normal	697	697	2	0.00	2 - 3%
CWH3B-A-14	210	Normal	698	698	2	0.00	2 - 3%
CWH3B-A-15	225	Normal	699	699	0	0.00	3 - 3%
CWH3B-A-16	240	Normal	704	704	5	0.00	3 - 5%
CWH3B-A-17	255	Normal	699	699	3	0.00	2 - 3%
CWH3B-A-18	270	Normal	704	704	3	0.00	3 - 5%
CWH3B-A-19	285	Normal	706	706	2	0.60	3 - 5%
CWH3B-A-20	300	Normal	705	705	5	0.00	3 - 5%
CWH3B-A-21	315	High	719	705	2	0.00	3 - 5%
CWH3B-A-22	330	High	713	706	7	0.00	3 - 5%
CWH3B-A-23	345	Normal	704	704	1	0.00	3 - 3%
CWH3B-A-24	360	Normal	702	702	3	0.00	3 - 3%

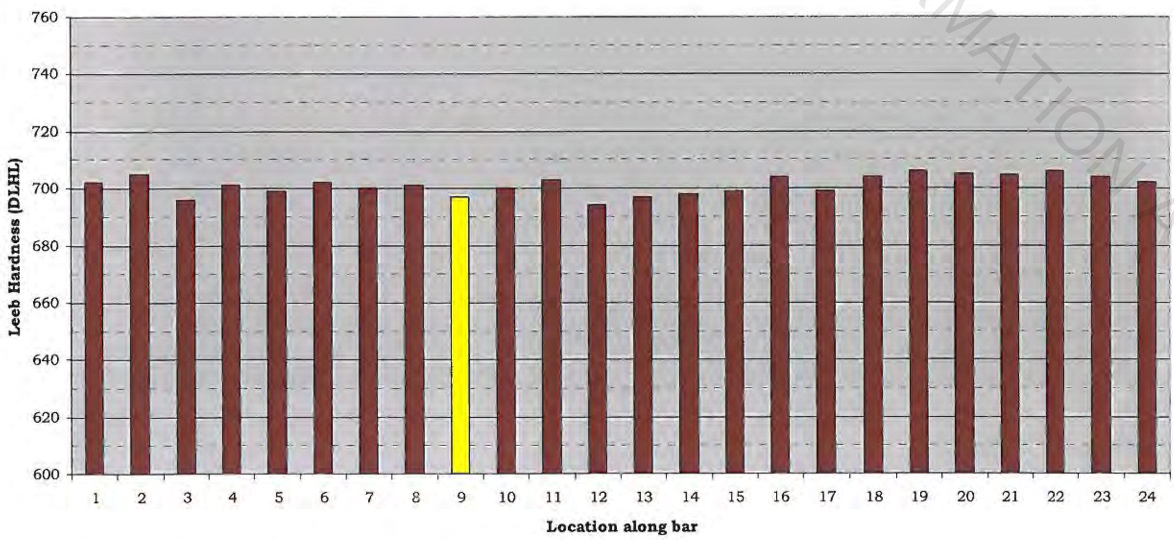

Figure 9 Leeb Hardness plot for Location CWH3B-A

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TEST LOCATION: CWH4A

Test Location CWH4A was located near the entrance door to room 4072 on the forth floor of the building. The test region was chosen to cross a series of two diagonal cracks. The cover concrete was removed exposing two reinforcing bars, labelled CWH4A-A and CWH4A-B respectively.

The hardness results typically showed limited variations along the length of the steel, with minor increases noted at the crack locations. The maximum value of average potential induced strain in sample CWH4A-A was determined to be 0.7%, resulting in an estimated maximum potential loss of strain capacity of 6% of the peak strain capacity.

Test bar CWH4A-B was found to achieve a maximum value of average potential induced strain of 0.9%, resulting in an estimated maximum potential loss of strain capacity of 11% of the peak strain capacity.

Figure 10 CWH4A

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Table 6 *Test Results for Location CWH4A-A*

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 (%)
CWH4A-A-01	15	Medium	663	705	2	0.00	3 - 5%
CWH4A-A-02	30	Medium	678	707	3	0.60	3 - 5%
CWH4A-A-03	45	Medium	683	705	1	0.00	3 - 3%
CWH4A-A-04	60	Normal	688	703	2	0.00	3 - 3%
CWH4A-A-05	75	Normal	695	710	4	0.61	5 - 6%
CWH4A-A-06	90	Normal	692	707	3	0.60	3 - 5%
CWH4A-A-07	105	Normal	692	707	3	0.60	3 - 5%
CWH4A-A-08	120	Normal	693	708	2	0.60	5 - 5%
CWH4A-A-09	135	Normal	696	711	4	0.61	5 - 6%
CWH4A-A-10	150	Normal	696	711	3	0.61	5 - 6%
CWH4A-A-11	165	Normal	696	711	3	0.61	5 - 6%
CWH4A-A-12	180	Normal	694	709	4	0.60	3 - 6%
CWH4A-A-13	195	Normal	686	701	3	0.00	3 - 3%
CWH4A-A-14	210	Normal	684	699	3	0.00	2 - 3%
CWH4A-A-15	225	Normal	687	702	1	0.00	3 - 3%
CWH4A-A-16	240	Normal	692	707	3	0.60	3 - 5%
CWH4A-A-17	255	Normal	688	703	2	0.00	3 - 3%
CWH4A-A-18	270	Normal	687	702	4	0.00	3 - 5%
CWH4A-A-19	285	Normal	692	707	4	0.60	3 - 5%
CWH4A-A-20	300	Normal	694	709	4	0.60	3 - 5%
CWH4A-A-21	315	Normal	700	715	2	0.66	5 - 6%
CWH4A-A-22	330	Normal	694	709	5	0.60	3 - 6%
CWH4A-A-23	345	Normal	695	710	3	0.61	5 - 6%
CWH4A-A-24	360	Normal	700	715	2	0.66	6 - 6%
CWH4A-A-25	375	Normal	697	712	3	0.62	5 - 6%

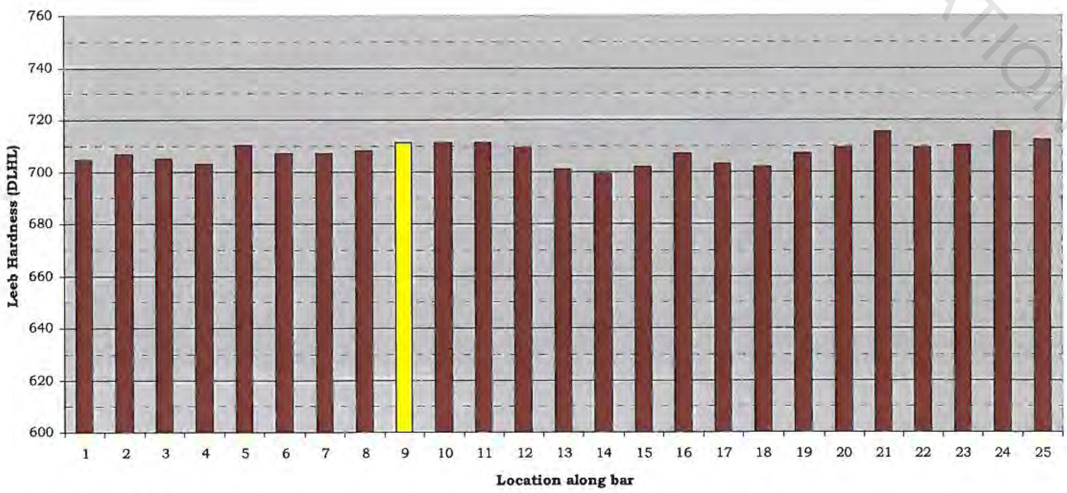

Figure 11 Leeb Hardness plot for Location CWH4-A

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Table 7 *Test Results for Location CWH4A-B*

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 (%)
CWH4A-B-01	15	Normal	681	701	4	0.00	2 - 3%
CWH4A-B-02	30	Normal	691	711	5	0.61	5 - 6%
CWH4A-B-03	45	Normal	693	713	3	0.63	5 - 6%
CWH4A-B-04	60	Normal	688	708	2	0.60	3 - 5%
CWH4A-B-05	75	Normal	687	707	1	0.60	5 - 5%
CWH4A-B-06	90	Normal	692	712	2	0.62	5 - 6%
CWH4A-B-07	105	Normal	690	710	3	0.61	5 - 5%
CWH4A-B-08	120	Normal	689	709	5	0.60	3 - 6%
CWH4A-B-09	135	Normal	693	713	2	0.63	5 - 6%
CWH4A-B-10	150	Normal	696	716	4	0.69	5 - 7%
CWH4A-B-11	165	Normal	692	712	4	0.62	5 - 6%
CWH4A-B-12	180	Normal	696	716	1	0.69	6 - 6%
CWH4A-B-13	195	Normal	701	722	5	0.89	6 - 11%
CWH4A-B-14	210	High	705	718	5	0.75	6 - 9%
CWH4A-B-15	225	High	707	721	4	0.83	6 - 10%
CWH4A-B-16	240	High	704	717	4	0.72	5 - 8%

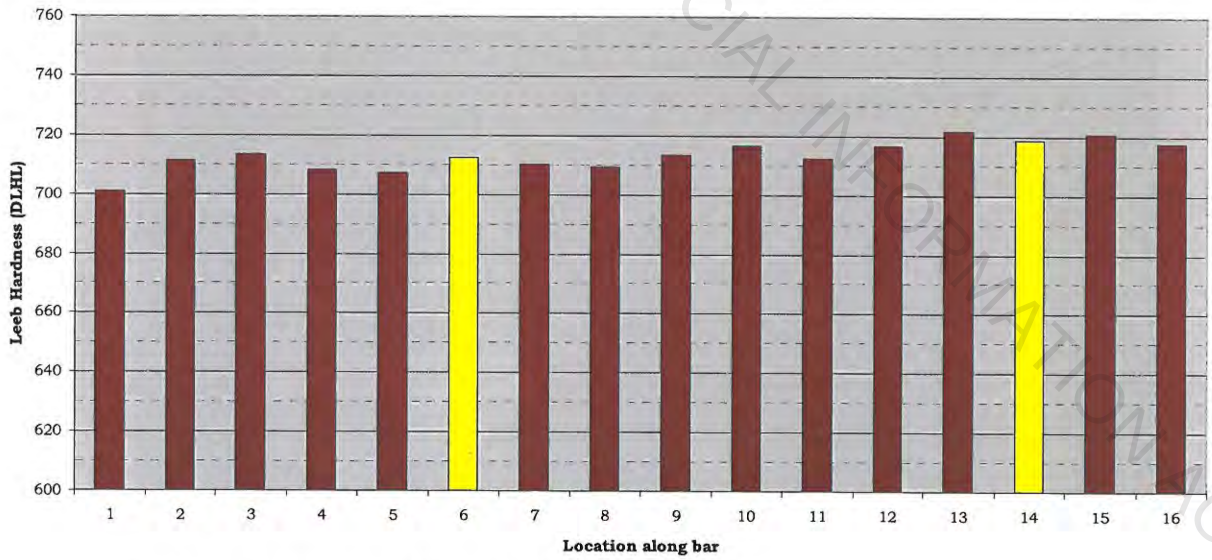

Figure 12 Leeb Hardness plot for Location CWH4A-B

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TEST LOCATION: CWH4B

Test Location CWH4B was located in the far left hand corner of room 4072. The test region was chosen due to a number of cracks in the floor in the general vicinity. The cover concrete was removed exposing a two reinforcing bars, labelled CWH4B-A and CWH4B-B respectively.

The hardness results for both tested bars showed little variation in steel hardness in the area surrounding the crack location. With full consideration for the uncertainty in the measured results, the maximum value of average potential induced strain in CWH4B-A was determined to be 0.8%, whereas the results in CWH4B-B were found to be 0.6%. These results corresponded to an estimated maximum potential loss of strain capacity of less than 9% of the peak strain capacity in both locations.

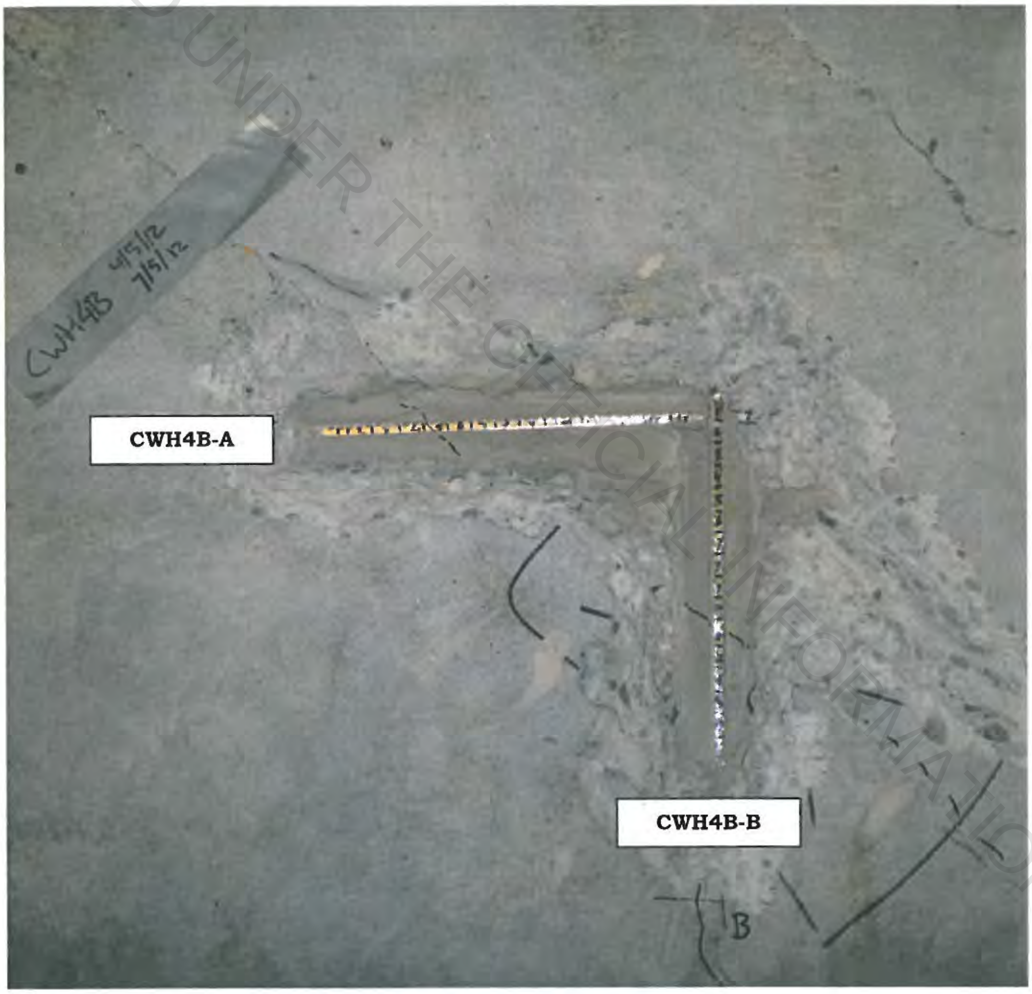

Figure 13 CWH4B

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Table 8 *Test Results for Location CWH4B-A*

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 (%)
CWH4B-A-01	15	Normal	670	700	2	0.00	3 - 3%
CWH4B-A-02	30	Normal	676	706	2	0.60	3 - 5%
CWH4B-A-03	45	Normal	679	709	5	0.60	3 - 6%
CWH4B-A-04	60	Normal	668	698	3	0.00	2 - 3%
CWH4B-A-05	75	Normal	677	707	5	0.60	3 - 5%
CWH4B-A-06	90	Medium	675	712	3	0.62	5 - 6%
CWH4B-A-07	105	Normal	681	711	4	0.61	5 - 6%
CWH4B-A-08	120	Normal	683	714	3	0.63	5 - 6%
CWH4B-A-09	135	Normal	686	717	2	0.69	6 - 7%
CWH4B-A-10	150	Normal	687	718	5	0.72	5 - 8%
CWH4B-A-11	165	Normal	689	720	2	0.80	6 - 8%
CWH4B-A-12	180	Normal	684	715	3	0.65	5 - 6%
CWH4B-A-13	195	Normal	688	719	4	0.76	6 - 8%
CWH4B-A-14	210	Normal	681	711	4	0.61	5 - 6%
CWH4B-A-15	225	Normal	686	717	2	0.69	6 - 7%

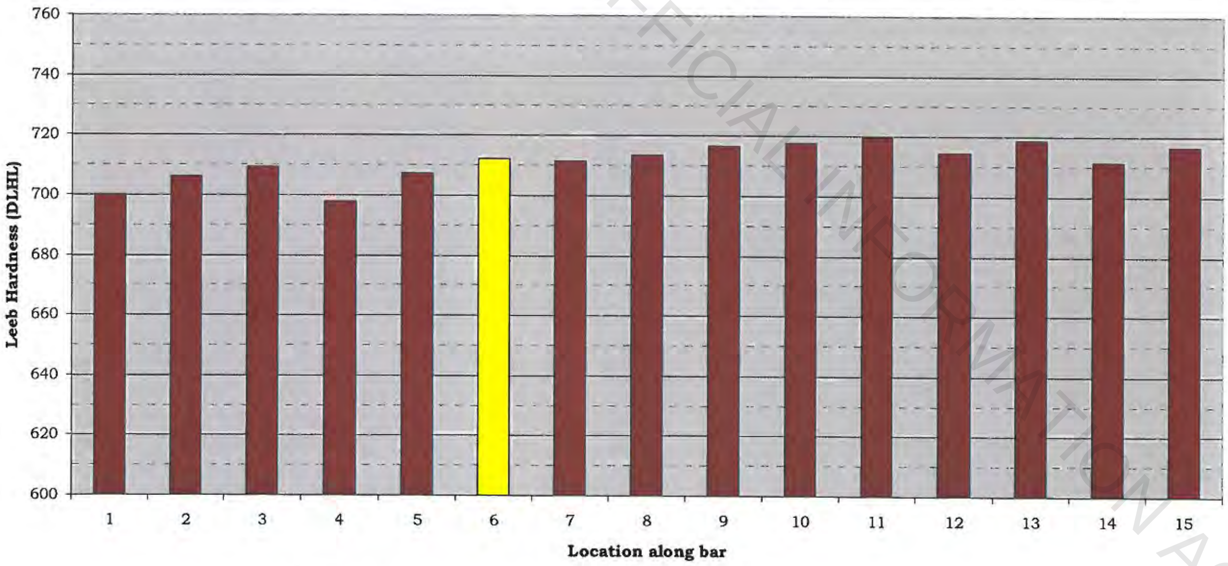

Figure 14 Leeb Hardness plot for Location CWH4B-A

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Table 9 *Test Results for Location CWH4B-B*

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 (%)
CWH4B-B-01	15	Normal	677	697	2	0.00	2 - 3%
CWH4B-B-02	30	Normal	681	701	3	0.00	3 - 3%
CWH4B-B-03	45	Normal	682	702	2	0.00	3 - 3%
CWH4B-B-04	60	Normal	680	700	2	0.00	3 - 3%
CWH4B-B-05	75	Normal	680	700	2	0.00	3 - 3%
CWH4B-B-06	90	Normal	681	701	2	0.00	3 - 3%
CWH4B-B-07	105	Normal	691	711	2	0.61	5 - 5%
CWH4B-B-08	120	Normal	683	703	2	0.00	3 - 3%
CWH4B-B-09	135	Normal	688	708	4	0.60	3 - 5%
CWH4B-B-10	150	Normal	687	707	3	0.60	3 - 5%
CWH4B-B-11	165	Normal	685	705	4	0.00	3 - 5%
CWH4B-B-12	180	Normal	680	700	1	0.00	3 - 3%
CWH4B-B-13	195	Normal	668	688	1	0.00	2 - 2%
CWH4B-B-14	210	Normal	668	688	2	0.00	2 - 2%
CWH4B-B-15	225	Normal	675	695	3	0.00	2 - 2%
CWH4B-B-16	240	Normal	662	681	3	0.00	2 - 2%

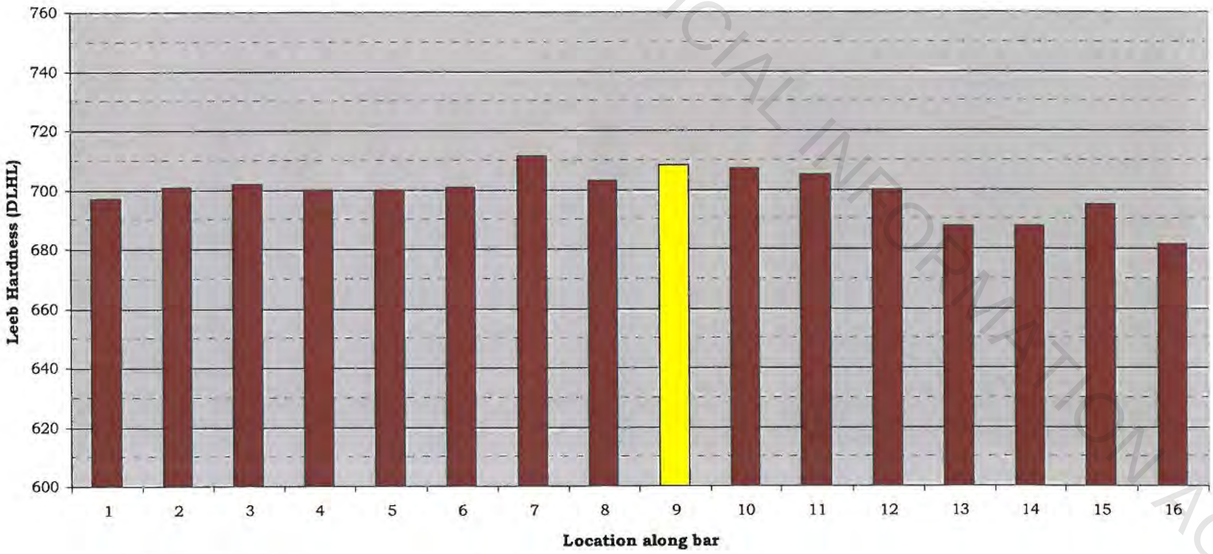

Figure 15 Leeb Hardness plot for Location CWH4B-B

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TEST LOCATION: CWH4C

Test Location CWH4C was located in near the centre consol in room 4072. The test region coincided with a single crack in the floor. Removal of the cover concrete exposed a single 12 mm bar crossing the crack, and a larger diameter steel bar in the perpendicular direction located directly under the concrete crack position.

The hardness results typically showed no significant variations along in steel hardness along the test region. However, the location on the steel that was likely to have sustained the greatest variation in hardness (directly under the crack location) was obscured from testing by the larger diameter steel bar.

In no locations did the estimated potential average strain in the steel bar exceed the value of strain corresponding with the onset of strain hardening and as such the steel was not considered to have loss any significant strain capacity.

Figure 16 CWH4C

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Table 10 *Test Results for Location CWH4C*

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 (%)
CWH4C-01	15	High	724	702	2	0.00	3 - 3%
CWH4C-02	30	Normal	700	700	4	0.00	2 - 3%
CWH4C-03	45	Normal	701	701	4	0.00	2 - 3%
CWH4C-04	60	Normal	699	699	4	0.00	2 - 3%
CWH4C-05	75	Normal	704	704	4	0.00	3 - 5%
CWH4C-06	90	Normal	700	700	4	0.00	2 - 3%
CWH4C-07	105	Normal	696	696	4	0.00	2 - 3%
CWH4C-08	120	Medium	684	698	3	0.00	2 - 3%
CWH4C-09	135	Normal	707	707	3	0.60	3 - 5%
CWH4C-10	150	Normal	706	706	2	0.60	3 - 5%
CWH4C-11	165	Normal	707	707	2	0.60	3 - 5%
CWH4C-12	180	High	710	703	1	0.00	3 - 3%
CWH4C-13	195	High	715	708	1	0.60	5 - 5%
CWH4C-14	210	High	716	709	4	0.60	3 - 5%
CWH4C-15	225	High	715	708	4	0.60	3 - 5%

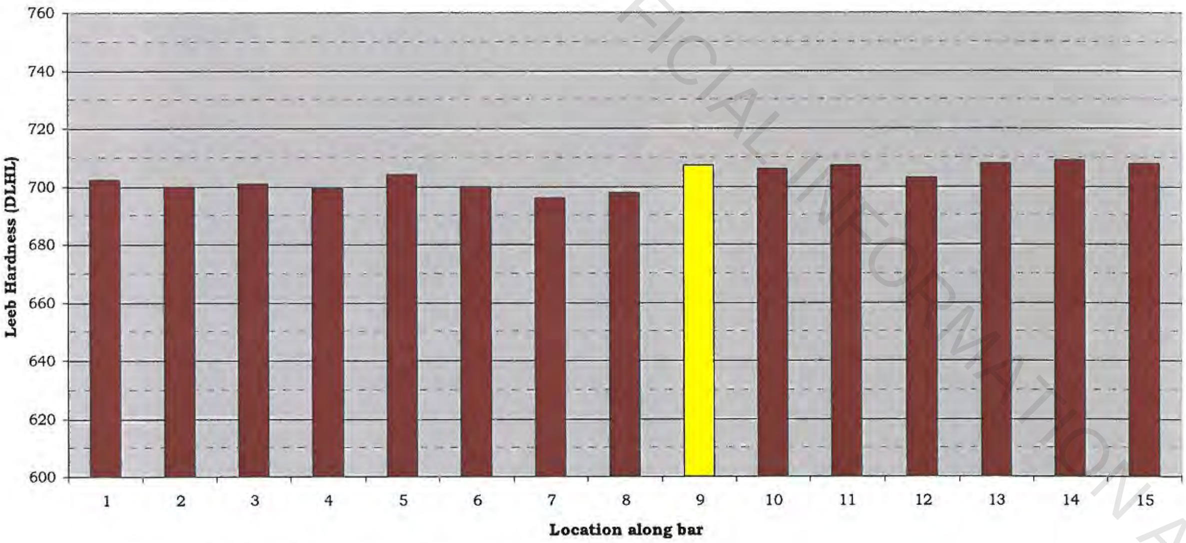

Figure 17 Leeb Hardness plot for Location CWH4C

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TEST LOCATION: CWH4D

Test Location CWH4D was located towards the centre of room 4069. The test region crossed a single concrete crack. Removal of the cover concrete exposed a single 12 mm bar crossing the crack, and a larger diameter steel bar in the perpendicular direction located directly under the concrete crack position.

The hardness results showed an increase in recorded hardness at, and surrounding, the crack location. The results suggest that the steel had undergone inelastic deformations. With full consideration for the uncertainty in the measured results, the maximum value of average potential induced strain was determined to be 1.6% at test segment 18. This corresponded to an estimated maximum potential loss of strain capacity of 17% of the peak strain capacity (where peak strain is defined as the strain corresponding with the peak stress of the material).

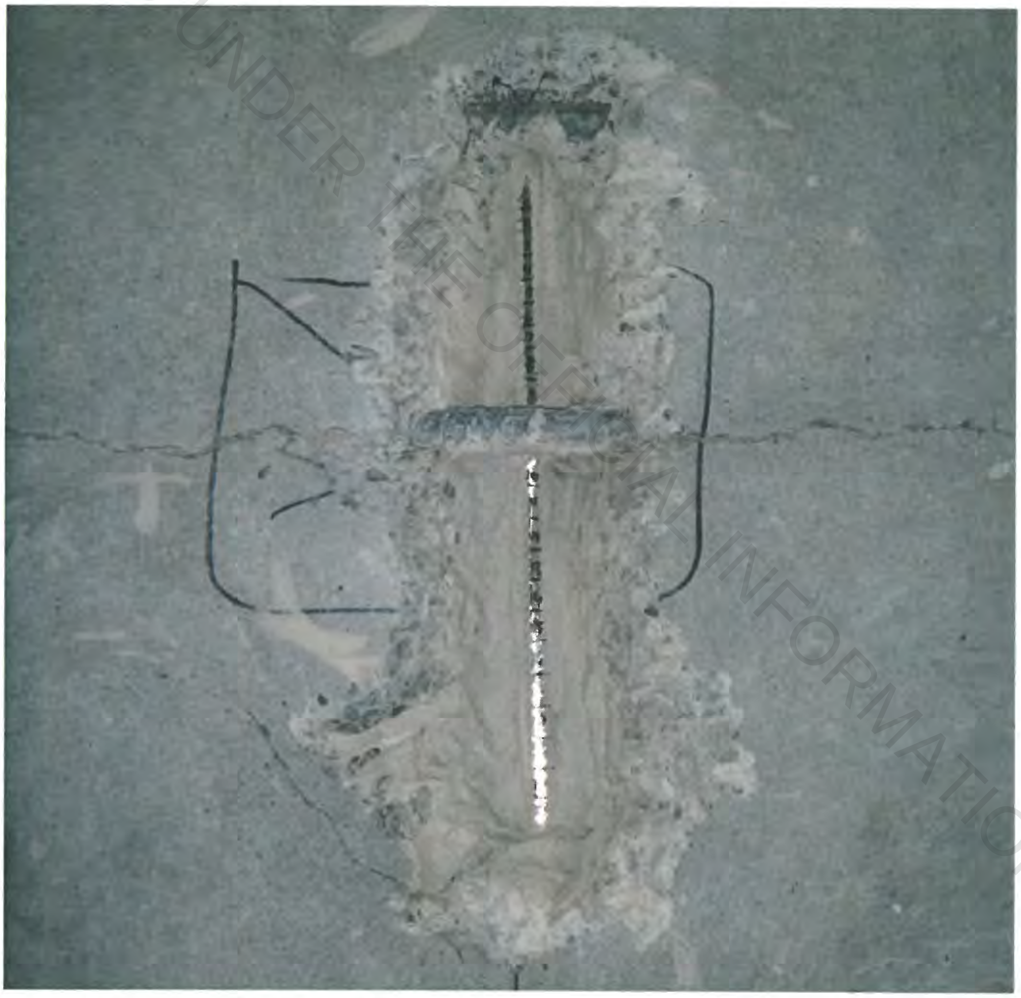

Figure 18 CWH4D-A

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Table 11 *Test Results for Location CWH4D-A*

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 (%)
CWH4D-01	15	High	702	698	4	0.00	2 - 3%
CWH4D-02	30	High	703	699	6	0.00	2 - 3%
CWH4D-03	45	Normal	690	700	2	0.00	3 - 3%
CWH4D-04	60	Normal	689	699	4	0.00	2 - 3%
CWH4D-05	75	Normal	690	700	5	0.00	2 - 3%
CWH4D-06	90	Normal	688	698	3	0.00	2 - 3%
CWH4D-07	105	Normal	693	703	3	0.00	3 - 3%
CWH4D-08	120	Normal	698	708	3	0.60	3 - 5%
CWH4D-09	135	Normal	701	711	2	0.61	5 - 5%
CWH4D-10	150	Normal	702	712	4	0.62	5 - 6%
CWH4D-11	165	Normal	712	722	2	0.93	7 - 9%
CWH4D-12	180	Normal	714	724	1	1.07	9 - 10%
CWH4D-13	195	Normal	712	722	1	0.93	8 - 9%
CWH4D-14	210	Normal	717	727	3	1.34	9 - 15%
CWH4D-15	225	Normal	715	725	4	1.15	8 - 14%
CWH4D-16	240	Normal	710	720	1	0.82	7 - 8%
CWH4D-17	255	Normal	713	723	4	1.00	7 - 12%
CWH4D-18	270	Normal	719	729	2	1.57	11 - 17%
CWH4D-19	285	Normal	716	726	3	1.24	9 - 13%
CWH4D-20	300	Normal	712	722	3	0.93	7 - 10%
CWH4D-21	315	Normal	707	717	4	0.71	5 - 8%
CWH4D-22	330	Medium	669	699	1	0.00	3 - 3%

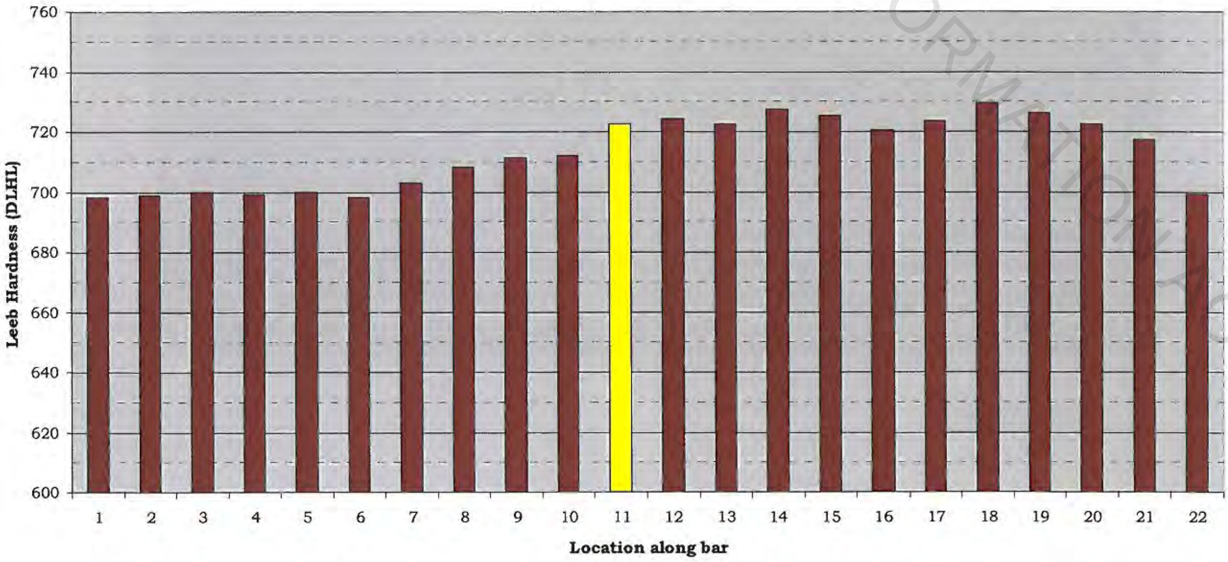

Figure 19 Leeb Hardness plot for Location CWH4D-A

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TEST LOCATION: CWH4E

Test Location CWH4E was located under the bench (near the window) of room 4069. The test region crossed two parallel concrete cracks. Removal of the cover concrete exposed a single 12 mm bar crossing the crack, and two steel bars in the perpendicular direction located directly under each concrete crack position.

The hardness results typically showed an increase in the steel hardness between the two crack locations. With consideration for the uncertainty in the measured results, the maximum value of average potential induced strain was determined to be 0.8%, corresponding to an estimated maximum potential loss of strain capacity of 8% of the peak strain capacity.

Figure 20 CWH4E

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Table 12 *Test Results for Location CWH4E-A*

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 (%)
CWH4E-01	15	Medium	682	701	2	0.00	3 - 3%
CWH4E-02	30	Medium	676	694	4	0.00	2 - 3%
CWH4E-03	45	Medium	677	696	1	0.00	2 - 2%
CWH4E-04	60	Normal	695	700	3	0.00	2 - 3%
CWH4E-05	75	Normal	694	699	4	0.00	2 - 3%
CWH4E-06	90	Medium	693	705	4	0.00	3 - 5%
CWH4E-07	105	Normal	707	712	4	0.62	5 - 6%
CWH4E-08	120	Normal	710	715	2	0.66	5 - 6%
CWH4E-09	135	Normal	714	719	3	0.77	6 - 8%
CWH4E-10	150	Normal	707	712	4	0.62	5 - 6%
CWH4E-11	165	Normal	713	718	2	0.74	6 - 7%
CWH4E-12	180	Normal	709	714	2	0.64	5 - 6%
CWH4E-13	195	Normal	713	718	3	0.74	6 - 7%
CWH4E-14	210	Normal	710	715	3	0.66	5 - 6%
CWH4E-15	225	Normal	707	712	3	0.62	5 - 6%
CWH4E-16	240	Normal	697	702	3	0.00	3 - 3%
CWH4E-17	255	Normal	692	697	4	0.00	2 - 3%
CWH4E-18	270	Normal	698	703	5	0.00	3 - 5%
CWH4E-19	285	Medium	654	698	3	0.00	2 - 3%
CWH4E-20	300	Medium	683	702	1	0.00	3 - 3%

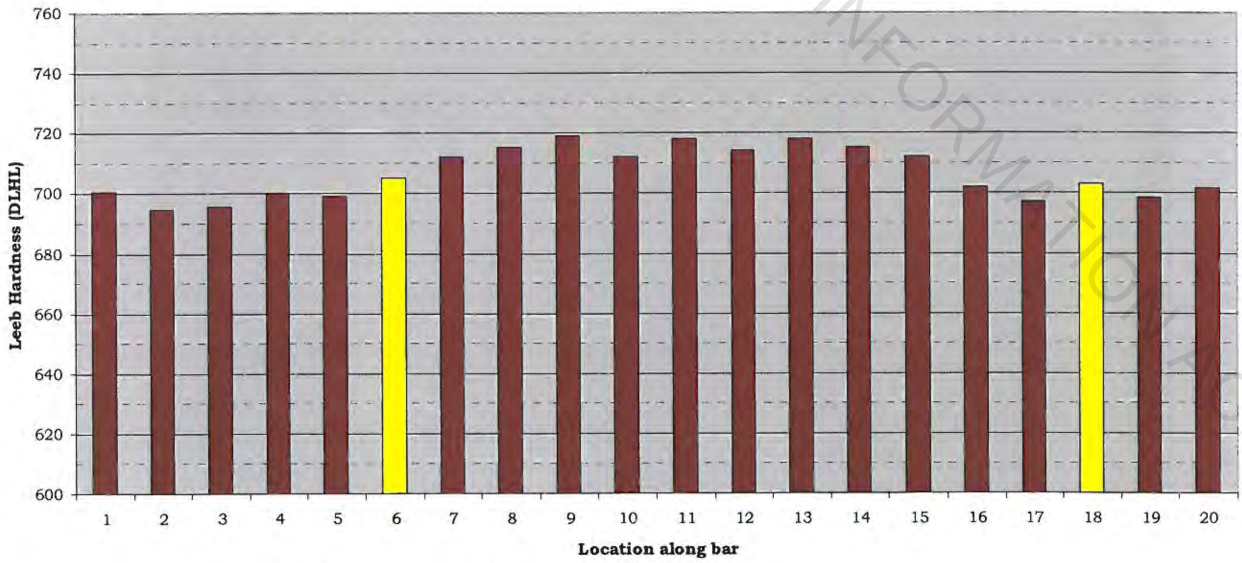

Figure 21 Leeb Hardness plot for Location CWH4E-A

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5.0 REFERENCES

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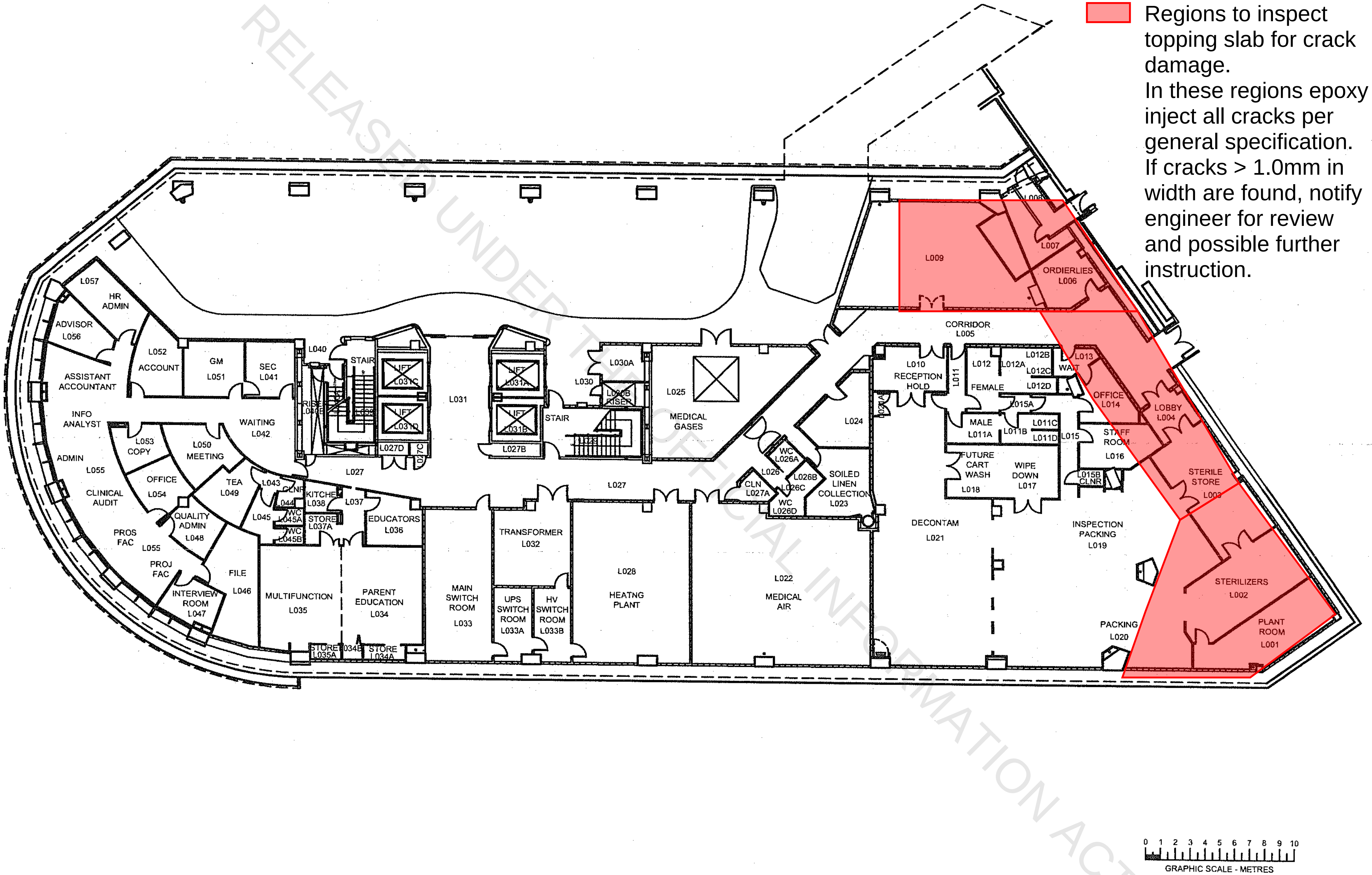
APPENDIX F

HCG Instructions for Precast Rib Inspection and Repair & Topping Slab Inspection and Repair



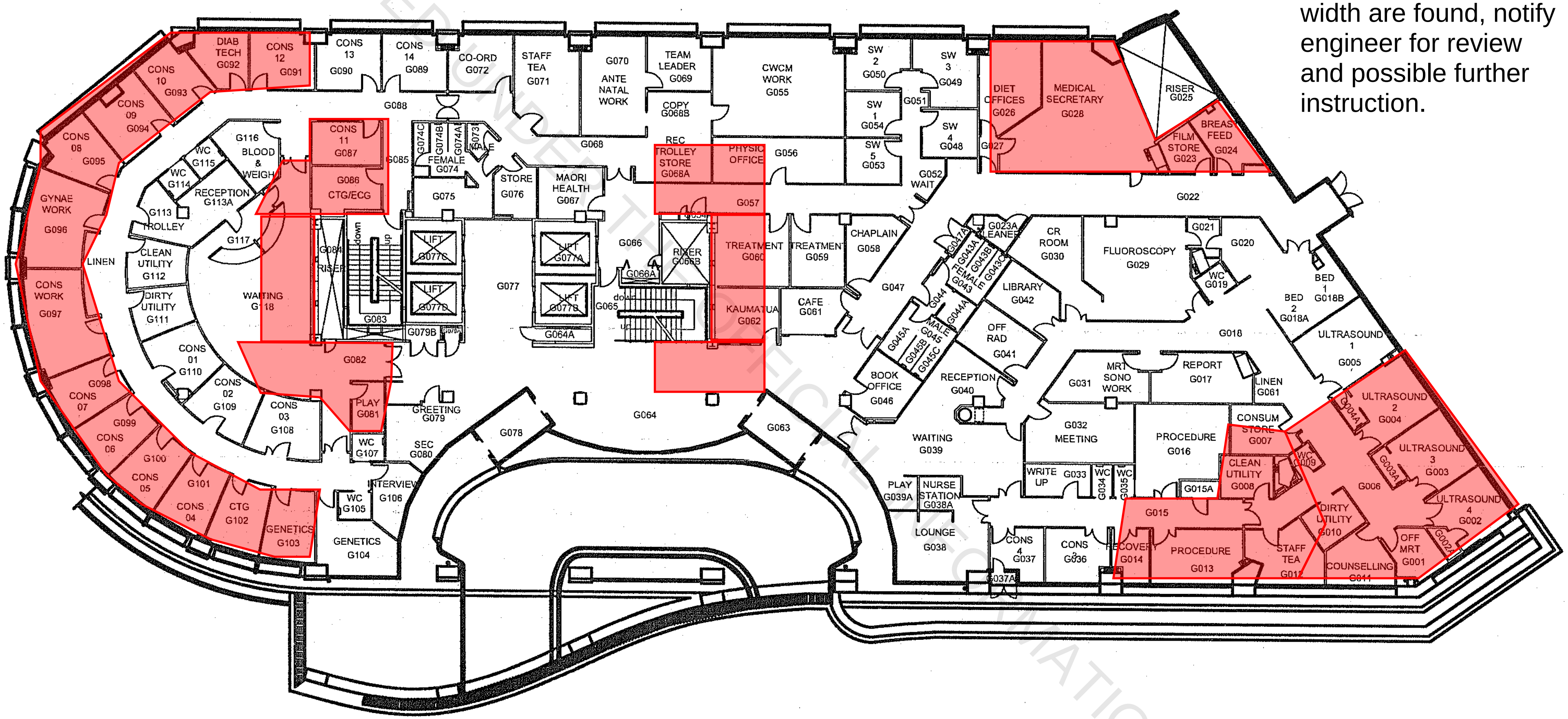
Regions to inspect topping slab for crack damage.
In these regions epoxy inject all cracks per general specification.
If cracks > 1.0mm in width are found, notify engineer for review and possible further instruction.

CHRISTCHURCH HOSPITAL
WOMENS - LOWER GROUND FLOOR
SPACE NUMBERS

Canterbury District Health Board Te Poari Hauora ō Waitaha	DESIGNED: DRAWN: B.Tudehope CHECKED: DATE: 26-10-05	DWG No 01291002 SCALE 1:250 AT A3 REVISED 26-10-05
	XREFED DRAWING/S: 01291001	

Maintenance and Engineering Department - Christchurch Hospital

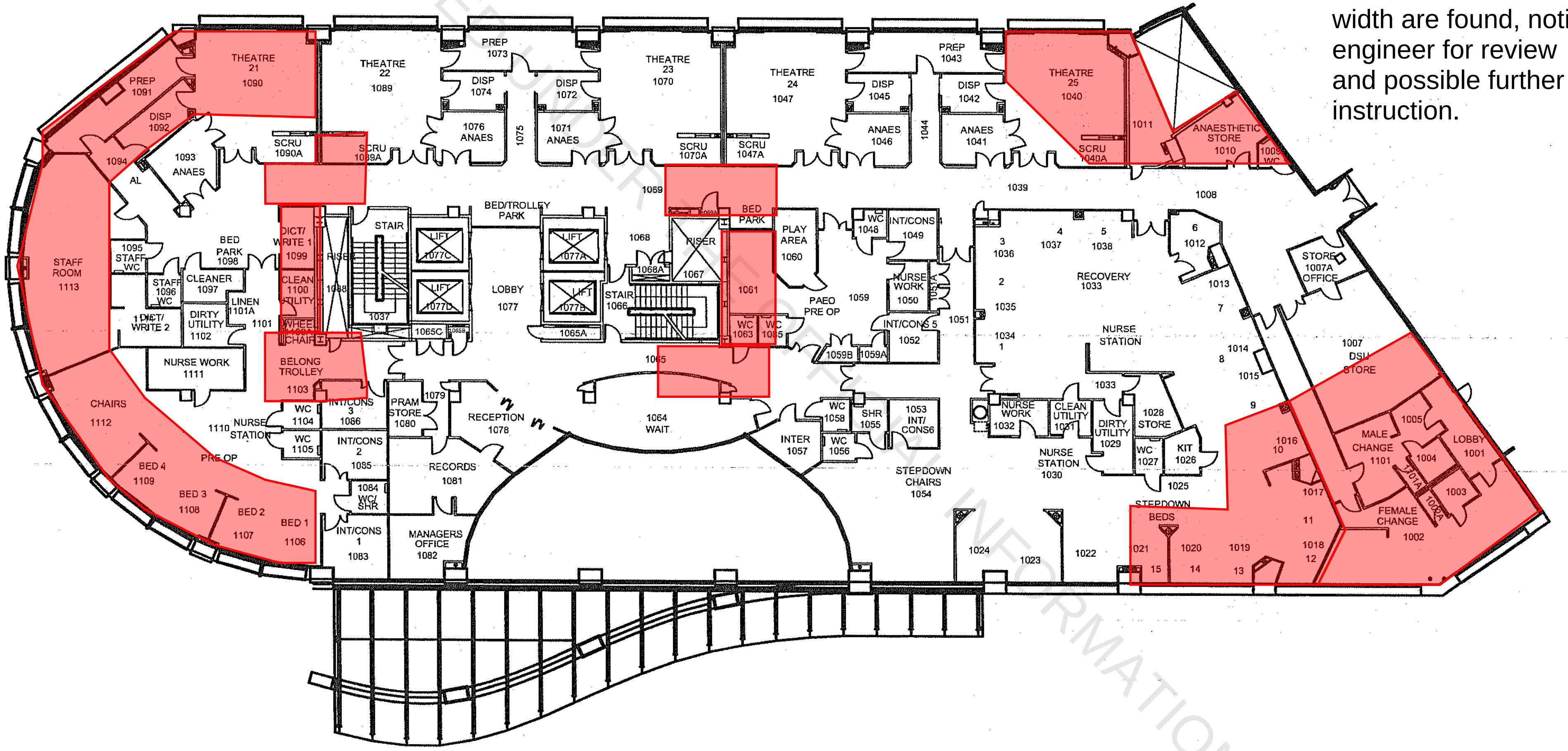
Regions to inspect topping slab for crack damage.
In these regions epoxy inject all cracks per general specification.
If cracks > 1.0mm in width are found, notify engineer for review and possible further instruction.



CHRISTCHURCH HOSPITAL
WOMENS - GROUND FLOOR
SPACE NUMBERS

Canterbury District Health Board Te Poari Hauora ō Waitaha	DESIGNED: DRAWN: D.Tudehope CHECKED: DATE: 26-10-05	DWG No. 01292002 SCALE 1:250 AT A3 REVISED 23-09-09
	XREFED DRAWING/S: 01292001	

Maintenance and Engineering Department - Christchurch Hospital



Regions to inspect topping slab for crack damage.
In these regions epoxy inject all cracks per general specification.
If cracks > 1.0mm in width are found, notify engineer for review and possible further instruction.

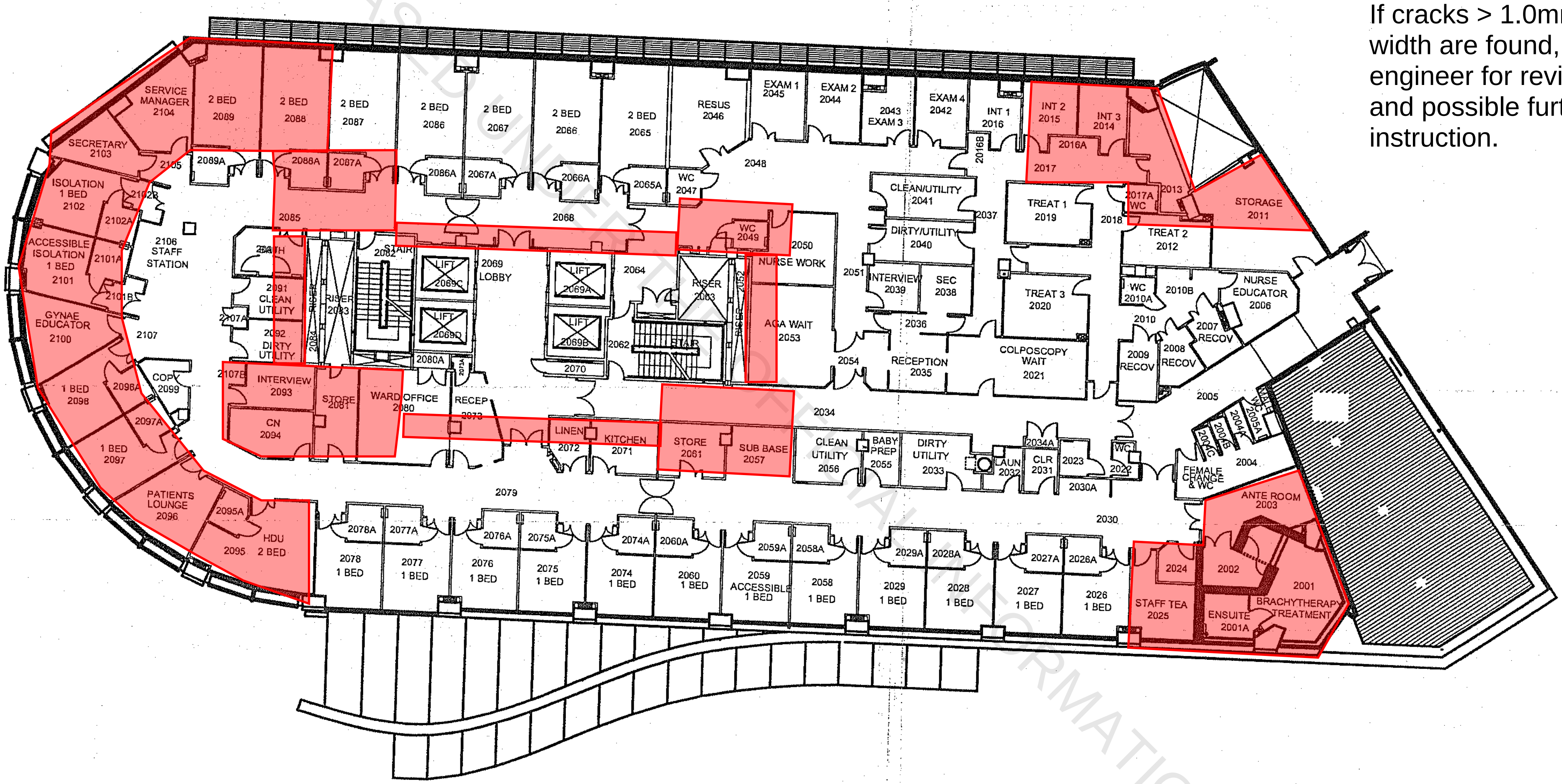
GRAPHIC SCALE - METRES

CHRISTCHURCH HOSPITAL
WOMENS - FIRST FLOOR
SPACE NUMBERS

Canterbury District Health Board Te Poari Hauora ō Waitaha	DESIGNED: DRAWN: B.Tudehope CHECKED: DATE: 27-10-05	DWG No. 01293002 SCALE 1:250 AT A3 REVISED 30-09-08
	XREFED DRAWING/S: 01293001	

Maintenance and Engineering Department - Christchurch Hospital

Regions to inspect topping slab for crack damage.
In these regions epoxy inject all cracks per general specification.
If cracks > 1.0mm in width are found, notify engineer for review and possible further instruction.



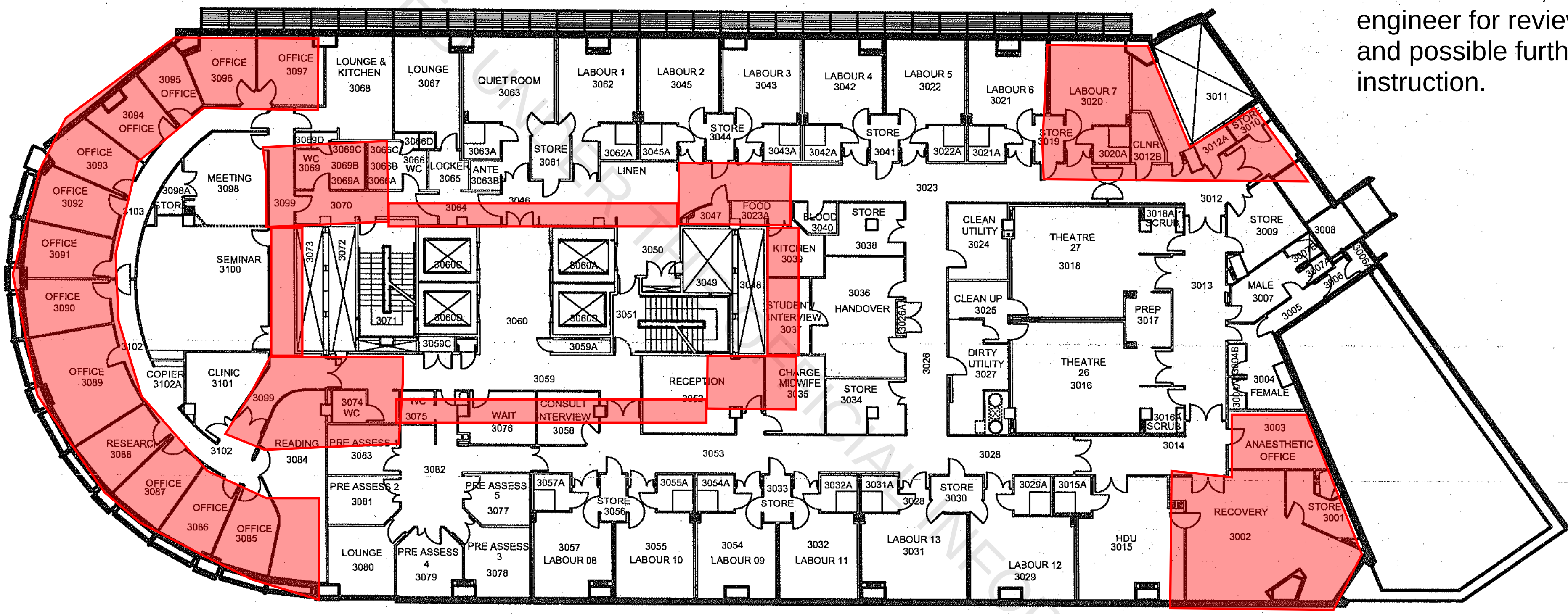
CHRISTCHURCH HOSPITAL
WOMENS - SECOND FLOOR
SPACE NUMBERS

0 1 2 3 4 5 6 7 8 9 10
GRAPHIC SCALE - METRES

Canterbury District Health Board Te Poari Hauora o Waitaha	DESIGNED: DRAWN: B.Tudehope CHECKED: DATE: 09-11-05	DWG No. 01294002 SCALE 1:250 AT A3 REVISED 27-08-08
	XREFED DRAWING/S: 01294001	

Maintenance and Engineering Department - Christchurch Hospital

Regions to inspect topping slab for crack damage.
In these regions epoxy inject all cracks per general specification.
If cracks > 1.0mm in width are found, notify engineer for review and possible further instruction.



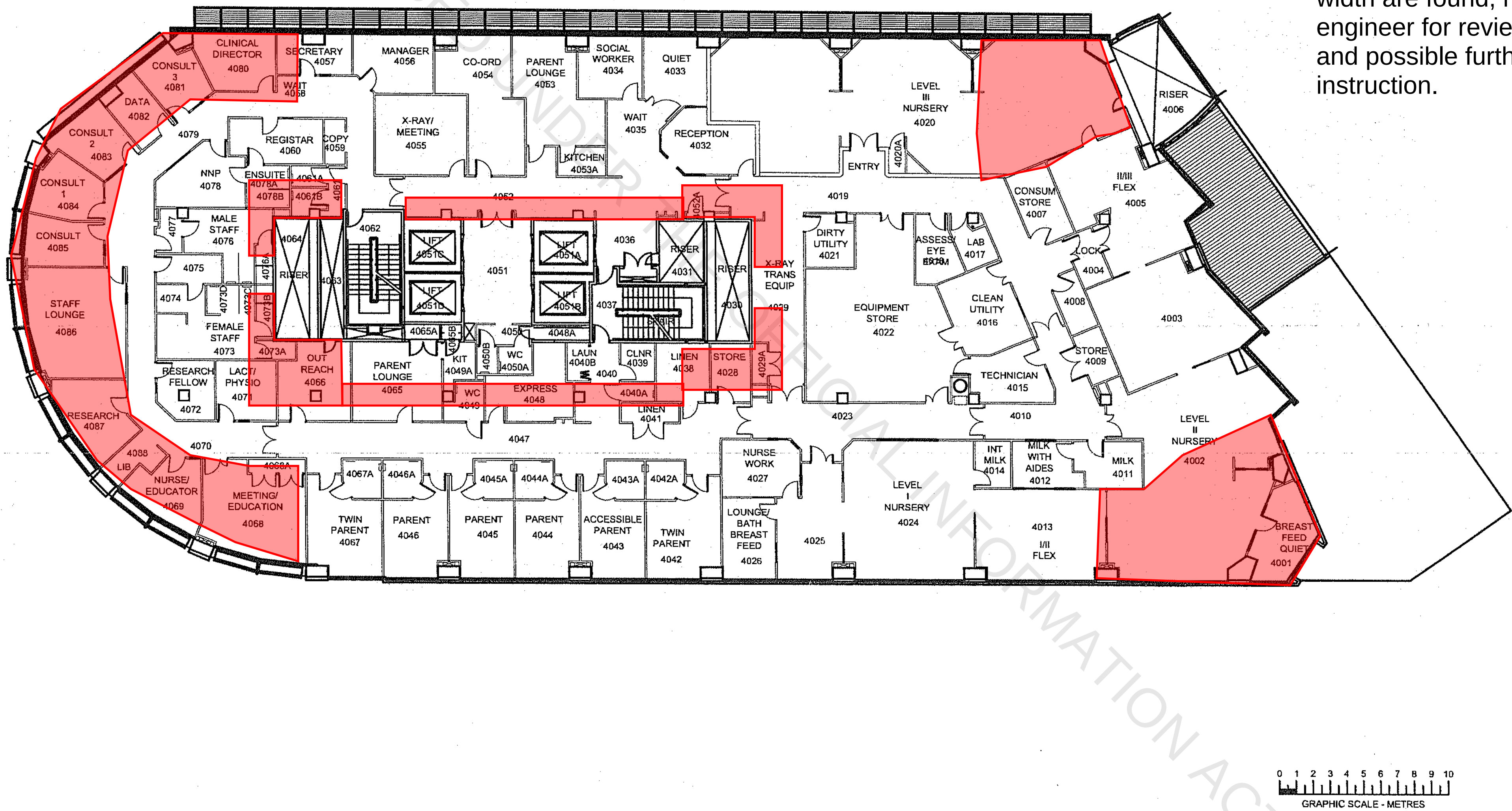
CHRISTCHURCH HOSPITAL
WOMENS - THIRD FLOOR
SPACE NUMBERS

Canterbury District Health Board
Te Poari Hauora ō Waitaha

DESIGNED:
DRAWN: B.Tudehope
CHECKED:
DATE: 16-11-05
XREFED DRAWING/S: 01295001

DWG No. 01295002
SCALE 1:250 AT A3
REVISED

Maintenance and Engineering Department - Christchurch Hospital



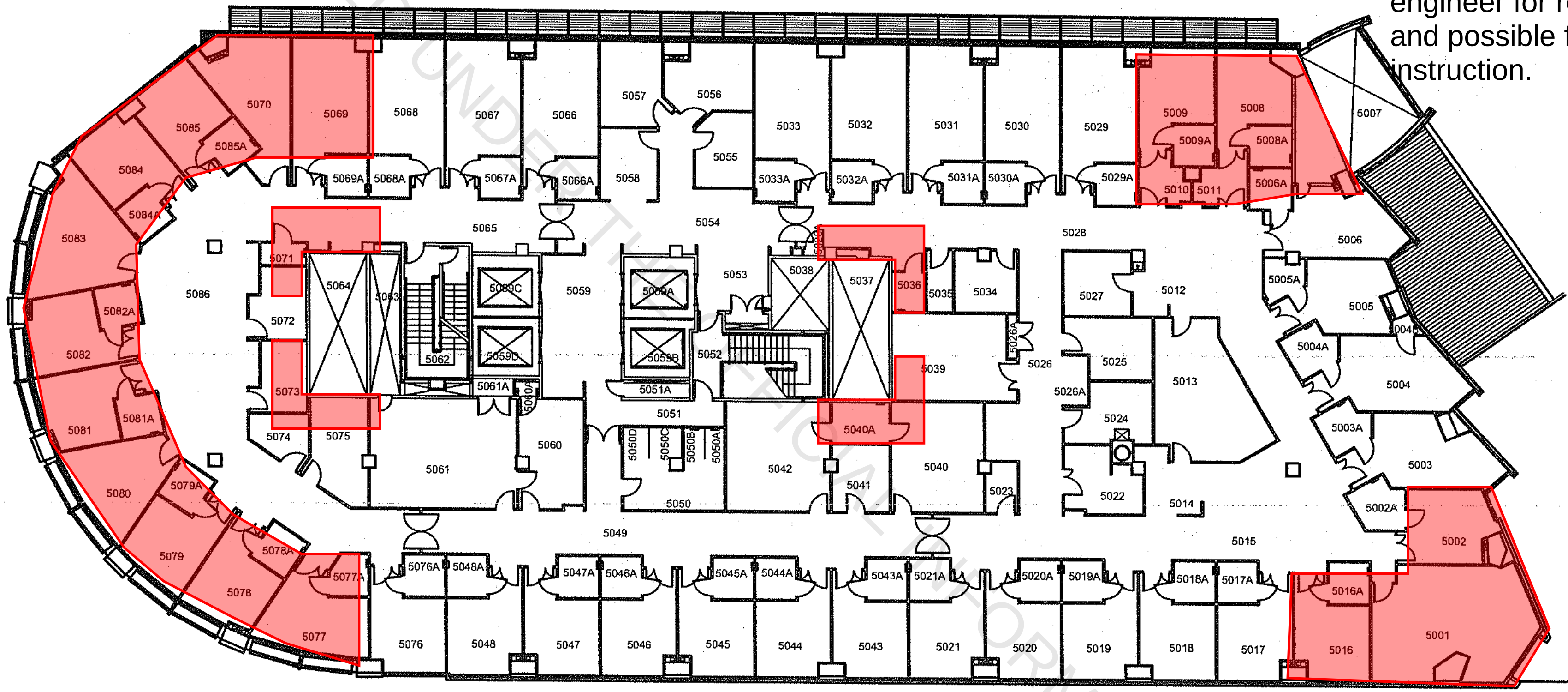
Regions to inspect topping slab for crack damage.
In these regions epoxy inject all cracks per general specification.
If cracks > 1.0mm in width are found, notify engineer for review and possible further instruction.

CHRISTCHURCH HOSPITAL
WOMENS - FOURTH FLOOR
SPACE NUMBERS

Canterbury District Health Board Te Poari Hauora ō Waitaha	DESIGNED: DRAWN: S.Tudehope CHECKED: DATE: 18-11-05	DWG No. 01296002 SCALE 1:250 AT A3 REVISED 18-11-05
	XREFED DRAWING/S: 01296001	

Maintenance and Engineering Department - Christchurch Hospital

Regions to inspect topping slab for crack damage.
In these regions epoxy inject all cracks per general specification.
If cracks > 1.0mm in width are found, notify engineer for review and possible further instruction.



0 1 2 3 4 5 6 7 8 9 10
GRAPHIC SCALE - METRES

CHRISTCHURCH HOSPITAL
WOMENS - FIFTH FLOOR
SPACE NUMBERS

Canterbury District Health Board Te Poari Hauora ō Waitaha	DESIGNED: DRAWN: B.Tudehope CHECKED: DATE: 18-11-05	DWG No 01297002 SCALE 1:250 AT A3 REVISED 18-11-05
	XREFED DRAWING/S: 01297001	

Maintenance and Engineering Department - Christchurch Hospital

HolmesConsultingGroup

Project Name:	CDHB - CHC Womans Hospital
Project No:	106186.72
S.R. No:	40
Date:	6 May 2013
Reviewed By:	JDP

SITE REPORT

STRUCTURAL AND CIVIL ENGINEERS

Holmes Consulting Group LP

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Auckland
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Wellington
Queenstown
San Francisco

Work Reviewed:

Repair of precast floor ribs

Observations & Comments:

Various site inspections have noted cracks in the ends of the precast floor ribs. The following notes provide guidance for further inspection and repairs.

- It is recommended that all ribs be inspected at both ends on all floors, as far as practicable. The reason for this is to identify and record crack locations and estimates of crack width so that these can be reviewed following subsequent earthquakes and additional damage identified. This will also be necessary information for insurance purposes in the event of further earthquake activity.

- Where the ribs are internal (i.e. not exposed to weather or exterior atmospheric conditions) our opinion is that the cracks less than 0.5mm in width do not need to be epoxy injected. Cracks more than 0.5mm should be epoxy injected and noted as such on the record plans for the inspections.

- Where the ribs are external (i.e. the floor areas that extend over the drive-ins entries) we recommend providing an industrial paint coating with sufficient flexibility to span any crack widths, with epoxy injection of cracks more than 0.5mm in width.

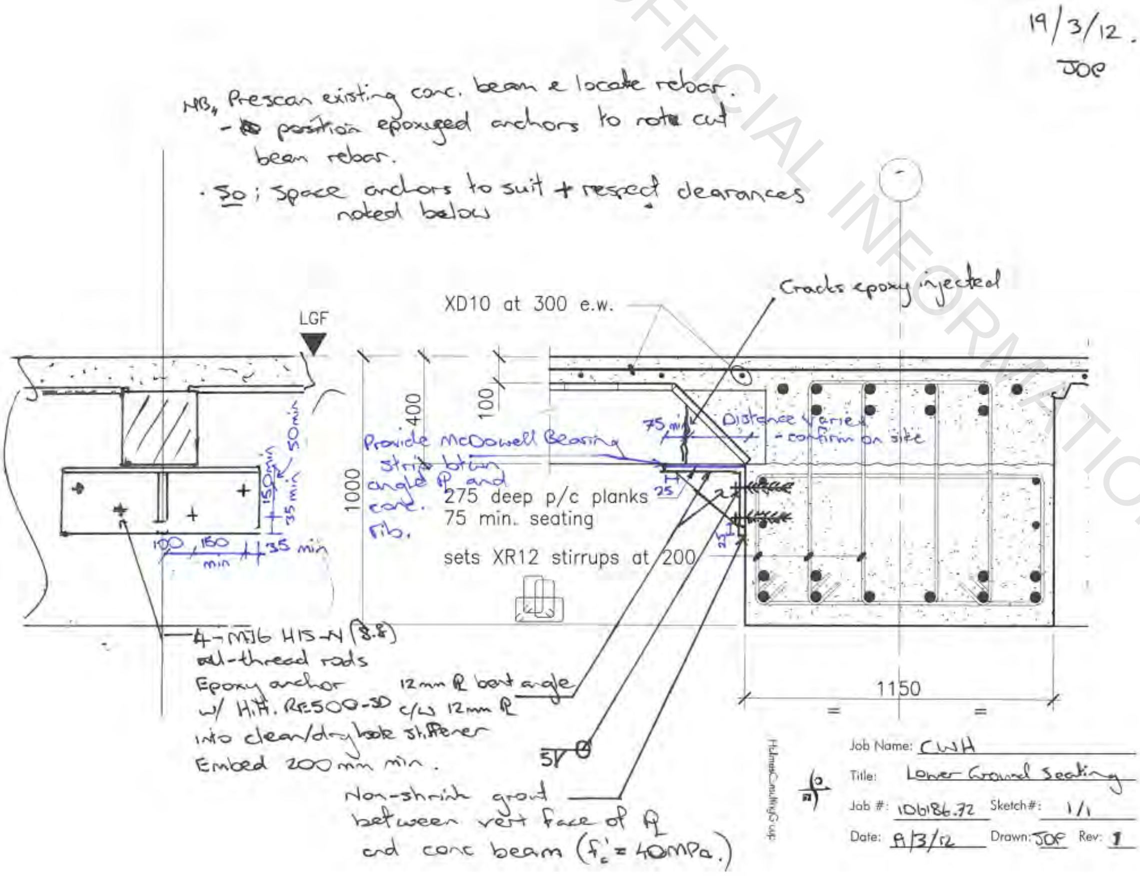
- Where side splitting of the units has occurred (see photo attached), a new seating will be required similar to the detail provided in Site Report 01 (19/3/2012) (see attached sketch). Each instance of this type of damage will need to be reviewed to ensure that the seating extends sufficiently to support the unit.

Report Prepared by:

Didier Pettinga
PROJECT ENGINEER

106186.72SR0605.040.doc

Copies to: Mark Webb, Bruce Goodger, Jenny Fisher, Alastair Cossart


19/3/12.
JOE
NB, Prescan existing conc. beam & locate rebar.
- position epoxyed anchors to not cut beam rebar.
· So; space anchors to suit + respect clearances noted below
Cracks epoxy injected
XD10 at 300 e.w.
LGF
400
100
1000
75 min
Distance varies - confirm on site
Provide McDowell Bearing strip btwn angle PL and conc. fib.
275 deep p/c planks
75 min. seating
25
sets XR12 stirrups at 200
150 min
50 min
35 min
100 / 150 min
35 min
4-M16 HIS-N (8.8) all-thread rods
Epoxy anchor w/ Hilti RE500-SD into clean/dry hole
Embed 200 mm min.
12mm PL bent angle c/w 12mm PL stiffener
5
Non-shrink grout between vert face of PL and conc beam (f'_c = 40MPa.)
1150
Job Name: CWH
Title: Lower Ground Seating
Job #: 10686.72
Sketch#: 1/1
Date: 19/3/12
Drawn: JOE
Rev: 1

HolmesConsultingGroup

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SITE REPORT

Project Name:	CDHB - CHC Women's Hospital
Project No:	106186.72
S.R. No:	46
Date:	10 June 2013
Reviewed By:	JDP

Work Reviewed:

Floor Slab Inspection and Repairs

Observations & Comments:

Following from the floor slab inspection program of the last 12 months, we have prepared the attached plans which provide indications of the key areas for future inspection and repair on each of the main floor for Christchurch Women's Hospital.

These areas have been identified based on how the building responds to earthquake demands as well as the crack patterns and density from observations in the rooms already inspected. We note that at the Lower Ground, 3rd and 4th floors a number of the rooms have been inspected and repaired however we have highlighted these areas in order to provide a consistent outline for future reference. The rooms already dealt with in the current program do not require repeat inspection.

The regions identified are seen as critical for ensuring the floor slab diaphragm has integrity to transfer seismic shear stresses to the exterior moment frames and interior K-braced frames. To this extent we recommend that all cracks in these regions are epoxy injected per the Structural Repair Specification. Other areas may well have cracking present but are unlikely to have the same high stress concentrations; therefore it is our opinion that these unmarked areas do not require specific investigation and repair. It should be noted however that if flooring is lifted in the future and cracks are noted this is an opportunity for repair that could be taken, however it would be possible to work to a larger crack width limit such as 0.5mm.

Note that if cracks larger than 1.0mm are found we recommend that a structural engineer review the crack to determine if further input for repair is required, as such cracks may relate to excessive reinforcement strains.

As the inspection process continues, we recommend that the current approach of crack mapping before surface grinding be carried out to identify new, further damaged existing and existing undamaged cracks.

Similarly QA methods will need to be maintained, in line with recommendations from our current review.

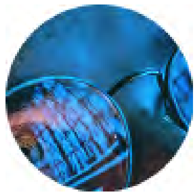
Please do not hesitate to contact the author for any clarification of issues raised in this Consultant Advice.

Copies to:

Report Prepared by:

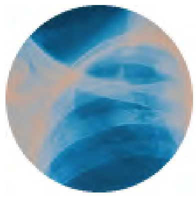
Didier Pettinga
PROJECT ENGINEER

106186.72SR1006.046.doc



DETAILED SEISMIC ASSESSMENT REPORT

STRUCTURAL AND CIVIL ENGINEERS



CHRISTCHURCH HOSPITAL

REPORT 2 – CLINICAL SERVICES

PREPARED FOR

CANTERBURY DISTRICT HEALTH BOARD

106186.09

REPORT REVISION 6 – 04 OCTOBER 2013

HolmesConsultingGroup

CHRISTCHURCH HOSPITAL – DETAILED SEISMIC ASSESSMENT REPORT

REPORT 2 – CLINICAL SERVICES

Prepared For:
CHRISTCHURCH DISTRICT HEALTH BOARD

Date: 04 October 2013
Project No: 106186.09
Revision No: 6

Prepared By:

Jenny Ovens
PROJECT DIRECTOR

Reviewed By:

Stuart Oliver
TECHNICAL DIRECTOR

Updated By:

Matthew Scott
DESIGN ENGINEER

Holmes Consulting Group LP
Christchurch Office

REPORT ISSUE REGISTER

DATE	REV. NO.	REASON FOR ISSUE
24/11/11	1	Interim Report for Review.
16/12/11	2	Updated to include preliminary sketches of strengthening schemes.
2/4/12	3	Updated to include observations following 23 December 2011 earthquake and results from investigations at base of columns supporting the Third floor plant room.
30/7/12	4	Updated to include the results of the levels survey. Introduction added.
12/11/12	5	Includes updated preliminary sketches for replacing lost strength, and sketches showing areas where existing elements require repair (crack repair), or further investigation and possible repair.
04/10/13	6	Updated to include May 2013 and Aug 2013 observations, analysis of the Loading Dock and Store Area, and NLTHA results for the Hydrotherapy Area.

CONTENTS

APPENDICES

TABLES Page

FIGURES Page

Holmes Consulting Group has been engaged by Canterbury District Health Board to complete a full structural review of the Christchurch City 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.

The 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 City Campus and is referred to as required in the specific building reports.

This report covers the structural damage sustained by the Canterbury District Health Board's Clinical Services Block, as a result of the series of Earthquakes that includes the Darfield Earthquake that struck at 4:36am on 4th September 2010, the Lyttelton Earthquake that struck at 12.51 pm on the 22nd of February 2011 and the earthquakes on the 13th June 2011 and 23rd December 2011. The Lyttelton Earthquake has subjected the building to strong ground motions which were possibly equal to full design earthquake load for an IL3 building of this nature. Consequently it is important that a full evaluation is performed.

The information available for the review included: the original structural drawings, the levels survey, the façade damage survey and the geotechnical report.

The Clinical Services Block was designed and constructed in the late 1960's. The original block consists of the four storey Clinical Services Building, constructed over a partial basement containing service tunnels and flanked on the West by the Paediatrics building, on the East by the Gymnasium building, and to the North by the Loading Dock and Store Area, and Hydrotherapy Buildings. Each of the four flanking structures were constructed as single storey attachments sharing common foundations with the Clinical Services Building.

Three floors were added to the gymnasium at the east end of the building in 2000. The Clinical Services Block is separated from the adjacent Central Riverside building by a 100mm seismic gap and is linked to the Parkside buildings by a separated corridor structure.

The structure consists of cast-insitu reinforced concrete waffle slabs, spanning between internal columns and the perimeter walls and frames. The lightweight roof is formed with a grid of steel beams supported by the columns and perimeter walls which extend to the roof level. A plant room slab extends over the central portion of the building. The walls and columns are founded at basement level on a combination of strip footings and isolated pad foundations. Unsealed service trenches run the length of the building above the foundation structure.

The three storeys added to the east were constructed with precast concrete panels, tied together with weldplates. The floors were constructed with prestressed concrete rib and timber infill, supported on steel beams and were tied into the original building by drilled and epoxied starters.

The Loading Dock and Store Area is bounded by the Clinical Services Building, Riverside Central, and Riverside West. The building consists of a canopy formed from a lightweight roof over steel trusses and a concrete slab awning which cantilevers to the West..

The Hydrotherapy Building is bounded by the Clinical Services Building, Riverside Central, and Riverside East. The building is also a single level structure, and has a highly penetrated reinforced concrete monoslope roof supported on reinforced concrete beams and perimeter walls.

Both the Loading Dock and Store Area, and the Hydrotherapy Building Buildings are supported vertically by reinforced concrete walls and laterally restrained in the North-South direction by Clinical Services Building. Above the Lower Ground Floor both the Hydrotherapy Building and Loading Dock are seismically separated from the Riverside Buildings. The reinforced concrete walls are founded at the Lower Ground Floor on a combination of strip footings and isolated pad foundations.

The building is currently designated as an Importance Level 3 structure.

Preliminary and detailed observations have been made of the damage sustained as a result of the earthquake. This report also discusses the building form and likely capacity prior to the earthquakes.

A non-linear time history analysis (NLTHA) has been carried out for the building and the results show that the Clinical Services Building has the capacity to resist approximately 35% of the Design Basis Earthquake (DBE) for an IL3 building. (Note that this is equivalent to a building strength of 45% DBE for an IL2 building and 25% DBE for an IL4 building). The capacity of the building is governed by the roof diaphragm and the face load capacity of the walls above Second Floor and also the floor under the radiation bunker.

The lower level roof and face loaded piers at the east end of building have the capacity to resist 40-45% DBE for an IL3 structure. The longitudinal and transverse walls have the capacity to resist approximately 67% DBE (IL3).

The roof steelwork above the Third Floor plant room has excessive inter-storey drifts and loss of vertical support (i.e. is at collapse) at approximately 55% DBE for an IL3 building. A new building designed to current codes would have a margin of at least 1.5 to 1.8 between Ultimate and Collapse Limit states. If there is to be a margin of 1.8 on collapse, the effective capacity of the pier is 55/1.8 = 30% DBE for an IL3 building. The plant room roof steelwork, however, is a small area of the structure where the consequences of failure are less significant than in the remainder of the building, therefore a margin of 1.5 between Ultimate and Collapse may be justifiable.

One Critical Structural Weaknesses has been identified. The columns below the Third Floor plant room fail and are likely to cause a partial collapse of the Third Floor at 60-70% DBE for an IL3 building. If there is to be a margin of 1.8 on collapse, the effective capacity of the pier is 60-70/1.8 = 33-40% DBE for an IL3 building.

The seismic gap between the Clinical Services Building and Riverside Central and the Parkside Link bridges is 100mm. The analysis indicates that the buildings will start to pound at approximately 40-50% DBE. The pounding will lead to increased accelerations and shears in the buildings and local damage to the structure at the interface between the buildings and to the Parkside Link Bridges.

The stairs constructed from insitu concrete and are completely enclosed within concrete shear walls therefore it is expected that these stairs, although they will be damaged in the Maximum Considered Event (MCE) are unlikely to collapse.

Peak diaphragm accelerations obtained from the NLTHA were used to estimate the capacity of the Hydrotherapy Building structure. The dependable capacity in this area is limited by the connection to the Ground Level floor slab of the Clinical Services Building. The connection is eccentric in height and relies upon shear transfer through the reinforced concrete piers on the north wall of the Clinical Services Building. Capacity of the piers is expected to be exceeded at approximately 50-60% DBE for an IL3 building.

An equivalent static analysis was used to estimate the capacity of the Loading Dock structure. The results indicate that the reinforced block walls supporting the reinforced concrete cantilever canopy, have the capacity to resist approximately 70% DBE loads for an IL3 building.

Following the Lyttelton earthquake moderate cracking was observed to the:

- South wall piers at Second Floor
- North-south central shear wall adjacent to the Riverside lifts at Ground, Lower Ground and basement levels
- Floor slab adjacent to the north-south central shear wall.
- North shear wall.
- Plant room slab upstand in the NW corner, and
- Basement internal shear wall

More minor cracking was observed around openings in shear walls elsewhere. Moderate cracking of the plant room slab occurred. Minor damage to finishes such as seismic gaps and plant room windows was observed with cracking of the external terrazite cladding noted.

The damage observations of the critical areas of the structure were updated following the 23 December 2011 earthquake. A small increase in crack widths in the central north-south shear wall, the radiation bunker and floor around it, the Second Floor south piers and the Ground Floor north piers was observed.

Investigations into the columns supporting the Third Floor plant room exposed 0.3-0.4mm horizontal cracks at the base of the columns when the floor screed was removed. It is likely, from the results of previous testing completed in the Riverside buildings and based on the structural configuration at this level, that strain hardening has occurred in the vertical column reinforcing. The extent of cracking in the north-south central shear wall at basement level, in the south piers at the Second Floor and in the floor slab adjacent to the north-south central shear wall indicates that there has been a loss of strain hardening capacity in these elements also. Testing could be carried out to confirm this if required. Options for replacing the lost strain hardening capacity are presented.

A survey of lower ground floor levels and building verticality was carried out in June 2012. The results of this survey indicate that there has been some settlement of the Clinical Services and Riverside buildings, likely to be as a result of dynamic compaction under the building mass during intense ground shaking. The building has not settled uniformly and in particular the single story portion at the western end of the Clinical Services building has not experienced as much settlement and slopes up towards the western end. Differential settlement of up to 80mm is recorded which is outside the tolerances of NZS 3109 [16].

Some cracking due to this settlement is visible in the basement tunnel walls under the single storey portion; cracks are wider at the base of the wall. Cracking is also expected in the northern and southern basement/foundation walls under the single storey portion. A loss of strain hardening capacity is likely to have occurred in these elements. Repair options are presented.

A review of building damage in May 2013 found no significant increase in damage noted in the previous observations.

In general the structural damage sustained is considered relatively minor and the building's capacity immediately following the earthquake is not considered to have been significantly reduced, however the strain hardening of the reinforcing steel in some elements will have reduced their capacity to withstand repeated cycles of loading.

Following the repairs recommended herein, the lateral load resisting performance of the building should be restored to close to pre-Darfield earthquake capacity.

Preliminary ideas for strengthening schemes are presented. Ideas are presented for mitigating critical structural weakness and increasing the reliable lateral capacity of the building. Strengthening includes adding new concrete walls, concrete spandrels and diaphragm ties. It is recommended that strengthening of the building is undertaken and 67% DBE should be the minimum level considered.

Further items that are required to be reviewed include: plant and water tank restraints, waterproofing and services across seismic gaps.

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

1. INTRODUCTION

Holmes Consulting Group has been engaged by Canterbury District Health Board to complete a full structural review of the Christchurch Hospital Campus following the Lyttelton Earthquake. A series of reports have been completed as part of this. These consist of a base report [2], a number of specific building reports and a repair specification [3]. 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 Clinical Services Block at Christchurch Hospital, 2 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 of June, 2011 and the December Earthquake that struck at 3.18pm on the 23rd of December 2011. The Lyttelton Earthquake, in particular, subjected the building to strong ground motions which significantly exceeded the current code loading demand for buildings of this nature.

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

1.2 LIMITATIONS

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

Our observations have been visual only and limited to representative samples, as described in our record of observations. Our observations have been restricted to structural aspects only. Because all of the structure has not been available for detailed inspection or evaluation, this report is limited to those elements available and engineering judgement as to the likely condition of unseen elements. Waterproofing elements, electrical and mechanical equipment, fire protection and safety systems, service connections, water supplies and sanitary fittings have not been inspected or reviewed, and secondary elements such as windows and fittings have not generally been reviewed.

Our professional services are performed using a degree of care and skill normally exercised, under similar circumstances, by reputable consultants practicing in this field at this time. No other warranty, expressed or implied, is made as to the professional advice presented in this report.

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

2.1 BUILDING FORM

The Clinical Services Block was designed and constructed in the late 1960's. The block is currently designated as an Importance Level 3 building.

The original block consists of the four storey Clinical Services Building, constructed over a partial basement containing service tunnels and flanked on the West by the Paediatrics building, on the East by the Gymnasium building, and to the North by the Loading Dock and Store, and Hydrotherapy Buildings. Each of the four flanking structures were constructed as single storey attachments sharing common foundations with the Clinical Services Building. Three floors were added to the gymnasium at the east end of the building in 2000. The Clinical Services Block is separated from the adjacent Central Riverside building by a 100mm seismic gap and is linked to the Parkside buildings by a separated corridor structure.

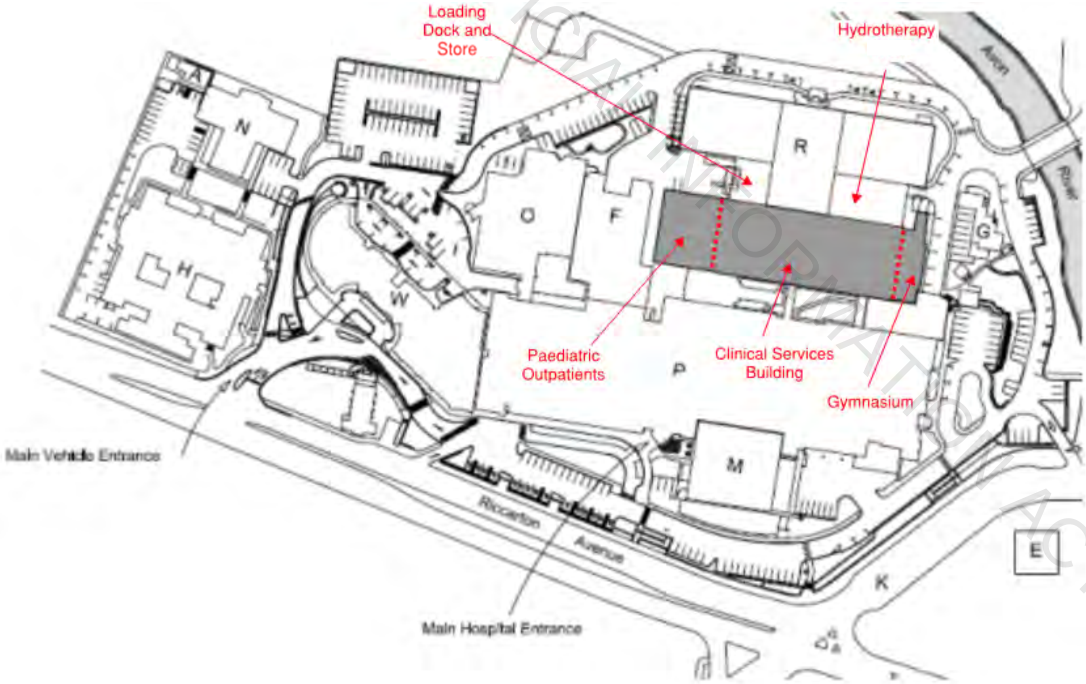


Figure 2-1: Clinical Services Block

The main Clinical Services Building comprises four levels above ground with a partial basement containing service tunnels.

The vertical load resisting structure consists of cast-insitu reinforced concrete waffle slabs, spanning between internal columns and the perimeter walls and frames. The lightweight roof is formed with a grid of steel beams supported by the columns and perimeter walls which extend to the roof level. A plant room slab extends over the central portion of the building.

Lateral forces are resisted by the reinforced concrete walls around the perimeter of the building. These walls are a combination of solid and perforated shear walls. 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 service trenches run the length of the building above the foundation structure.

Figure 2-2: Clinical Services Block – Eastern Elevation

The Paediatric Outpatients wing to the west of the main Clinical Services Building consists of a single storey over a partial basement. The structure is similar to that of the main Clinical Services Building and consists of a cast-in-situ concrete waffle slab floor supported by internal concrete columns and perimeter concrete perforated shear walls.

The three storeys added to the east were constructed with precast concrete panels, tied together with weldplates. The floors were constructed with prestressed concrete rib and timber infill, supported on steel beams and were tied into the original building by drilled and epoxied starters.

The Loading Dock and Store structure to the North-West of the Clinical Services Building consists of a lightweight roof over steel trusses which span north-south between the Clinical Services Building and Riverside West. Additionally the loading dock has a reinforced concrete canopy which cantilevers to the West and is supported by reinforced concrete walls in each direction. The reinforced concrete canopy slab and steel trusses are restrained laterally by the North wall of the Clinical Services Building wall between the Lower Ground Floor and Ground Floor Levels.

The Hydrotherapy Building to the North-East of the Clinical Services Building is a single level structure that consists of a monoslope reinforced concrete slab at roof level over reinforced concrete beams and perimeter walls. The roof is supported laterally by the perimeter concrete walls and is tied into the North Wall of the Clinical Services Building between the Ground Floor and First Floor levels.

2.2 PRE-EARTHQUAKE BUILDING CAPACITY

The building capacity under earthquake actions discussed in this section is compared to the capacity that a similar building would be designed to today. A new building would be designed to 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 [18] 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 [11]. 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 %.

The original Clinical Services Block was designed to predecessor standards of the current NZ Building Code, most likely comprising NZSS 1900:1965 [3] for loadings and concrete. The new extension constructed in 2000 would have been designed to the more recent standards, NZS 4203:1992 [4] (loadings) and NZS 3101:1995 (concrete) [5].

Previous assessments of the Clinical Services Block conducted by Holmes Consulting Group [6] and [7] have found the original 4 storey building to have a capacity of approximately 19% of the Design Basis Earthquake (DBE) loads for an Importance Level 3 building. Critical issues that were identified included: insufficient wall rocking capacity, insufficient wall shear strength (particularly of piers), poor detailing of elements (un-anchored beam stirrups, lack of confining reinforcement, bar laps in critical hinge zones), insufficient seismic separation between buildings.

2.2.1 Non-Linear Time History Analysis (NLTHA)

To gain a better estimate of the buildings capacity a NLTHA has been completed. This analysis gives an improved estimate of the buildings capacity and the response of the structure. The analysis allowed the following issues to be assessed:

- The central north-south shear wall extends to the underside of First Floor only, i.e it does not extend to the Second Floor and the plant room at the Third Floor. This wall is stiffer than the east and west perforated walls therefore significant loads will be transferred out of the east and west walls through the First Floor diaphragm to the central wall.

- Openings have been cut in the spandrels of the original east wall of the building for door openings to allow access to the extensions. These openings have reduced the stiffness and strength of the original east wall.

- The concrete plant room at Third Floor is connected to the north wall and the walls to the radiation bunker only, therefore the lateral loads at this level are partly resisted by 350mm square cantilever columns that are not detailed for ductility.

- The Hydrotherapy Building reinforced concrete roof is restrained laterally by the Clinical Services Building Ground Floor Level floor diaphragm. The connection is eccentric in height, relying on the out-of-plane capacity of the reinforced concrete piers on the north wall of the Clinical Services Building.

- The roof of the main Clinical Services Block is constructed of roofing iron over "Woodtex" panels supported on timber purlins that span between steel rafters supported on the exterior walls and internal columns. The "Woodtex" panels appear to be constructed with the interlocking channel system. The "Woodtex" panels have a limited capacity to act as a diaphragm and to provide support for the walls above the Second Floor under face loads.

Figure 2-3: Roof Structure – "Woodtex" Panels on Timber Purlins

2.2.1.1 Earthquake Load Level

Seismic loads were based on the requirements of NZS1170. For time history analysis, the code specifies a minimum of 3 time histories scaled such that the records envelope the code response spectrum. The appropriate scale factors were determined from the current loadings standard (NZS1170.5:2004) 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:	3
Risk Factor, R:	1.3
Structural Period, T:	<0.4s
Structural Performance Factor, S_p:	1.0

The analysis indicated that the building may be prone to mechanisms involving brittle collapse when pushed beyond the elastic range and therefore a structural performance factor, S_p=1.0 was assumed. Table 2-1 lists the three earthquake records used, together with the scaling factors calculated for the building. Both components of each earthquake used the same scaling factor as the fundamental period for translations direction of the building was below the lower limit of 0.4 seconds set by AS/NZS 1170.5.

Earthquake	R = 1.0
El Centro Array #9 (Imperial Valley, USA) 19 May 1940	1.45
Kalamata (Greece) Earthquake, 13 Sep 1986, Nomapxia	1.41
Llayllay (Chile) Earthquake, 3 March 1985	1.06

Table 2-1 Earthquake Scaling Factors

2.2.1.2 Material Properties

Presumed strength properties used in the building modelling are as follows:

- Concrete waffle slab floors: f'_c =40 MPa
- Concrete columns: f'_c =30 or 40 MPa
- Precast concrete: f'_c =45 MPa
- Shear walls, all other concrete: f'_c =30 MPa
- Concrete reinforcing steel: f_y = 300 MPa
- Structural steel: f_y = 330 MPa
- Plywood panels, 22mm thick: V_u = 11.6 kN/m
- Mineral-fibre diaphragm panels, 40 mm thick: V_u = 8.0 kN/m

Concrete testing carried out on the Riverside and Clinical Services building indicated that the probable strength of the concrete is likely to vary between 30MPa and 60MPa. Where the concrete strength was specified on the original drawings, it was 3000psi (21MPa) and 4000psi (27.5MPa). The probable concrete strength for the analysis was taken as the expected (average) strength f'_c of 1.5 times the specified original concrete strength f'_c. Where no information was available on the original concrete strength, a probable concrete strength of 30MPa was used.

Consideration of concrete reinforcing steel lap splice adequacy was not field evaluated and could change the results if deficient. However, based on the building's vintage, the provided lap lengths are most likely on the order of 32 to 40 bar diameters, 20 bar diameters for columns, which should be satisfactory to develop the forces in this analysis.

Consideration of the foundation vertical compression stiffness and uplift potential has been included by incorporating spring gap elements. Gravity loads from adjacent structures (Riverside Central Building) at common footings have been included as superimposed gravity loads where they occur.

2.2.1.3 Model Assumptions

The geometry of the structure has a number of complexities and the analysis model was constructed in such a way as to reflect these complexities as much as practicable. Aspects of the structure which were explicitly modelled or accounted for in the analysis include:

1. Floor & Roof Diaphragms. The cast-in-place concrete waffle slab floors were assumed to provide effective diaphragms and so a rigid diaphragm was assumed. In the case of floor or roof diaphragms that have a log span-to-depth ratio and where floor stresses need to be calculated directly, panel elements were used for finite stiffness modelling

   For the Clinical Services Building, a finite diaphragm stiffness definition was used at the following locations:
   - 2nd floor level concrete diaphragm (span-to-depth ratio = 2.5:1)
   - Nominally reinforced floor diaphragms at the East Addition (used to capture stress directly)
   - Flat and inclined wooden panellized plywood or mineral fibre roof membranes with steel faming (capture low-stiffness diaphragm behaviour)

2. Foundations. To model rocking and foundation uplift and the associated amplification of seismic structural forces due to period lengthening of this stiff structure (soil-structure-interaction), compression only gap elements have been inserted at the foundation level. The soil compression springs used represent the best opinion of upper bound soil stiffness for a type D underlying soil. No other soil-structure interaction (damping due to sliding, area effects) is considered.

   Since the neigboring Riverside Central Building shares a commoon footing with the Clinical Services Building, the added gravity loads have been incorporated into the Clinical Services Building model as superimposed loads.

3. Mechanical Plant Room Roof Steel Work. At R = 0.8 the structural steel column framing supporting the mechanical roof level has collapsed – lateral story drift at that level is 3 meters, and those runs terminate prematurely. For subsequent runs, the steel framing at the mechanical penthouse is laterally restrained in the model so that a complete suite of time history runs can be achieved to evaluate the capacity of the concrete portions of the structure.

The model geometry is shown in Figure 2-4.

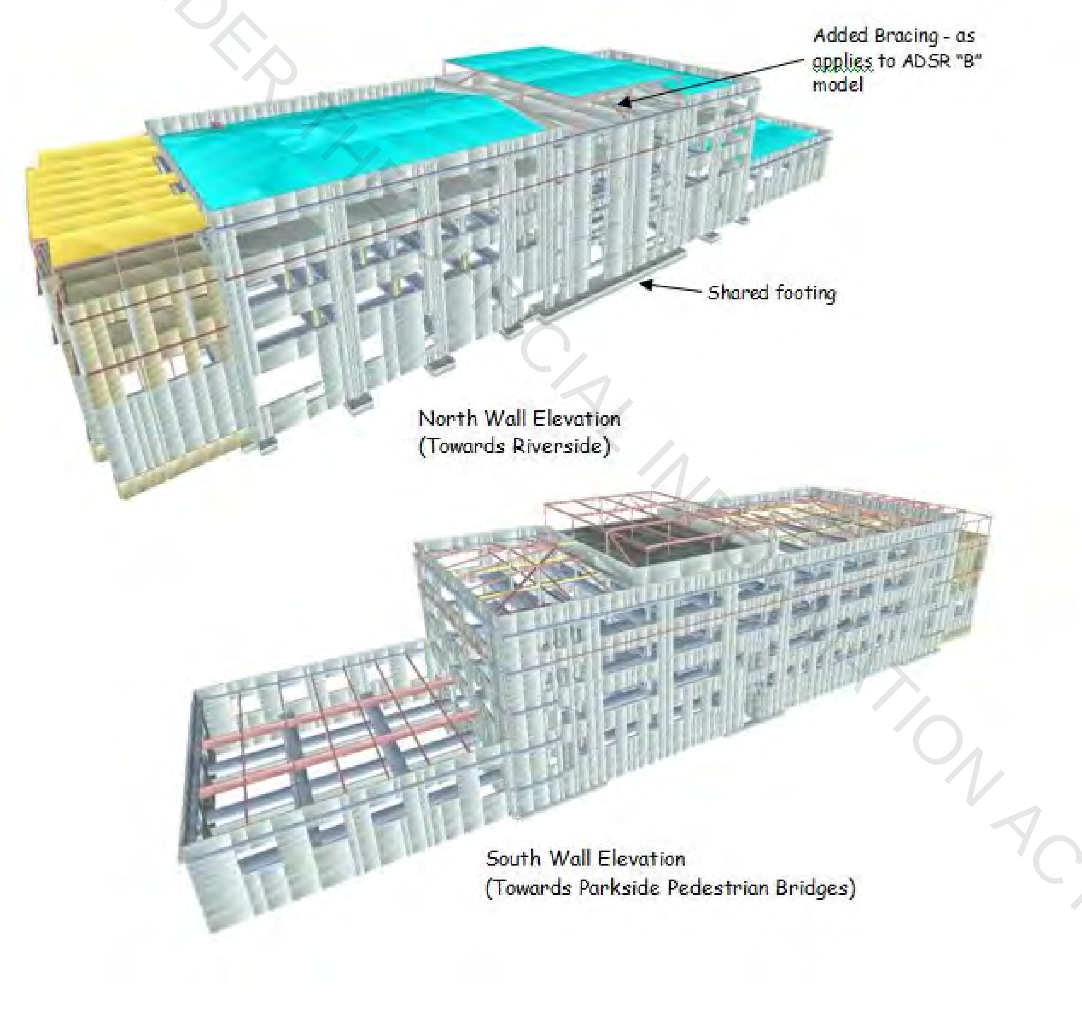


Figure 2-4: Model Geometry

2.2.1.4 Assessment Criteria

The results of the NLTH analysis have been interpreted using ASCE 41-06 [8]. ASCE 41-06 considers three performance limit states, Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP).

Components have been evaluated by categorizing deficiencies as either non-critical (severe damage and impaired function) or critical (potential collapse hazard).

1. Non-critical deficiencies relate to damage which will not form a life safety hazard. These imply severe cracking, impairing function and operability, but not collapse.
2. More severe deficiencies which have a higher probability of leading to partial or total collapse are classified as critical deficiencies.

The following definitions have been used for the assessment:

<u>Critical Deficiencies</u>.

1. Yield of shear reinforcement associated with high superimposed axial stresses (greater than $0.15f'_C$, where f'_C is taken as 1.5 times the analysis values for this check) is a critical deficiency. As discussed previously, the ANSR model defines yield of panel reinforcing as a strain of 0.0045 and so panel elements with an axial stress greater than $2.25f'_c$ are defined as deficient by this criteria.

2. In walls which do not have superimposed axial loads exceeding $0.15f'_C$, a shear strain greater than 1.5% is classified as a critical deficiency. This is the ASCE 41 secondary component shear wall segment CP strain level. Secondary element criteria are used when strength degradation is modelled.

3. A column shear deficiency (insufficient shear reinforcing) is classified as critical if it is associated with a high plastic rotation (greater than the FEMA secondary element limit) or a plastic rotation greater than the primary element shear strain limit. This latter limit of 0.0075 radians generally governs. This latter limit is based on the acceptance criteria which would be used if the column were modelled as a shear panel.

4. Column confinement deficiencies. These are assessed in terms of NZS3101 criteria. For low ductility demands these deficiencies may be remedied by a more detailed assessment using moment curvature calculation. For columns which do not yield under seismic loads insufficient confinement is defined as a non-critical deficiency.

   The critical confinement deficiency for columns with non-code-compliant ties and tie spacings is based on the FEMA 356 column flexure criteria for secondary elements (CP2). The FEMA criteria consider the magnitude of the superimposed axial load.

5. Structural steel framing critical deficiency associated with a story collapse mechanism.

<u>Non-Critical Deficiencies</u>

1. Beam shear deficiencies are classified as non-critical at all levels of plastic rotation. This is on the basis beam failure will not lead to collapse of the structure.

2. A column shear deficiency is classified as non-critical if it is associated with a plastic rotation less than the FEMA secondary element limit or the primary shear strain limit (that is, less than that defining a critical deficiency in item 3 above).

3. In walls which do not have superimposed axial loads exceeding $0.15f'_C$, a shear strain greater than 0.75% but less than 1.5% is classified as a non-critical deficiency. The 1.5% limit is the secondary element criteria.

4. Column confinement deficiencies where columns remain elastic under all seismic loads are classified as a non-critical deficiency. Column confinement deficiencies for yielding columns are assessed in terms of NZS3101 criteria where code-compliant column confining ties are present.

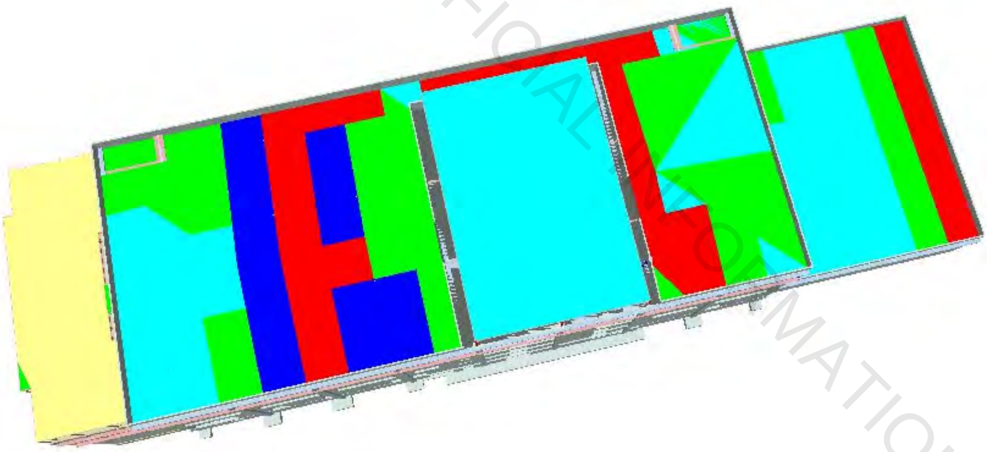
   The confinement of columns with non-code-compliant ties and tie spacings is assessed based on the FEMA 356 column flexure criteria for primary elements (CP).

2.2.2 Non-Linear Time History Analysis (NLTHA) Results

2.2.2.1 IL3 Building Capacity

The results of the NLTH indicate that the Clinical Services Building has the capacity to resist approximately 35% of the new ULS design earthquake for an IL3 building. (Note that this is equivalent to a building strength of 45% DBE for an IL2 building and 25% DBE for an IL4 building).

The capacity of the building is governed by the roof diaphragm and the face load capacity of the walls above Second Floor and also the floor under the radiation bunker.



Local stress > Immediate Occupancy	
Local stress > Life Safety / ULS	
Local stress > Collapse Prevention / CLS	

Figure 2-5: Roof Structure – R=0.7 (50%DBE)

The lower level roof and face loaded piers at the east end of building have the capacity to resist 40-45% DBE for an IL3 building. The longitudinal and transverse walls have the capacity to resist approximately 67% DBE (IL3).

The roof steelwork above the Third Floor plant room has excessive inter-storey drifts and loss of vertical support (i.e. is at collapse) at approximately 55% DBE (Il3). A new building designed to current codes would have a margin of at least 1.5 to 1.8 between Ultimate and Collapse Limit states. If there is to be a margin of 1.8 on collapse, the effective capacity of the pier is 55/1.8 = 30% DBE (IL3). The plant room roof steelwork, however, is a small area of the structure where the consequences of failure are less significant than in the remainder of the building, therefore a margin of 1.5 between Ultimate and Collapse may be justifiable.

The dependable capacity of the reinforced concrete roof over the Hydrotherapy Building is limited by the connection to the Ground Floor Level floor slab of the Clinical Services Building. The connection is eccentric in height and relies upon shear transfer through the reinforced concrete piers on the north wall of the clinical services area. Capacity of the piers is expected to be exceeded at approximately 50-60% DBE for an IL3 building.

2.2.2.2 Critical Structural Weaknesses

The results of the NLTH indicated that there was a critical structural weakness that could lead to collapse or partial collapse of the building. The element has been identified and assessed in accordance with the Engineering Advisory Group Draft Guidelines [9]. The Engineering Advisory Group Draft Guidelines recommend a margin over collapse is used to provide an acceptable risk of collapse. The Guidelines recommend a factor of 2 for qualitative assessments. NLTHA assessments are considered a full detailed assessment and provided a better estimate of capacity than a qualitative assessment. For NLTHA, the Guidelines do not specify a margin of 2, but do require an acceptable margin over collapse limits. As discussed in the Base Report, a factor of 1.8 is used as it is generally accepted that for well detailed new buildings there is a margin of at least 1.5 to 1.8 over the ultimate limit capacity. The following critical structural weakness has been identified:

- The first column below the Third Floor plant room fails at 55% DBE for an IL3 building however this is unlikely to lead to a partial collapse of the Third Floor. Sufficient columns will have failed to cause a partial collapse of the Third Floor at 60-70% DBE for an IL3 building. If there is to be a margin of 1.8 on collapse, the effective capacity of the pier is 60-70/1.8 = 33-40% DBE for an IL3 building.

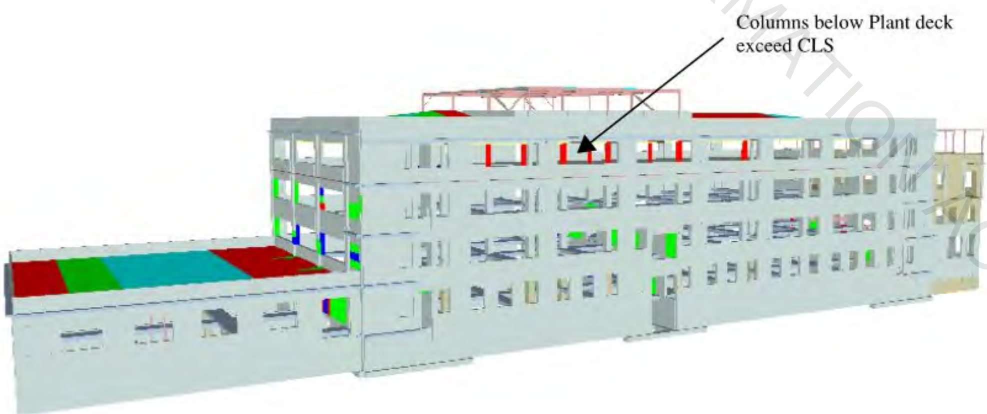


Figure 2-6: Columns Below the Third Floor – R=0.9 (70%DBE)

2.2.2.3 Pounding with Adjacent Structures

The analysis shows that at a code level earthquake the building drifts are large enough to cause pounding with the adjacent buildings. The Clinical Services Building is separated from the Riverside Central Building to the North by a seismic gap of 100mm. The NLTH analysis indicates that the deflection of the Third Floor at 100% of the DBE for an IL3 building is 155mm. It is likely that the buildings will start to pound at approximately 40-50% DBE (IL3). Riverside Central has 9 storeys adjacent to the Clinical Services Building, i.e. it is 4 storeys taller. The pounding will lead to increased accelerations and shears in the buildings and local damage to the structure at the interface between the buildings.

The Clinical Services Building is separated from the Parkside Link Bridges to the South by a seismic gap of 100mm. The NLTH analysis indicates that the deflection of the Second Floor (which is the same height as the link bridges concrete roofs) at 100% DBE earthquake for an IL3 building is 90mm. From the analysis of the Parkside Buildings, it is estimated that the Ultimate Limit State deflection of the upper level of the link bridge is 35mm. It is likely therefore that pounding will occur in the Ultimate Limit State event and this would lead to damage locally to the Clinical Services building and damage to the Parkside link bridge structures themselves.

2.2.2.4 Stairs

The stairs constructed from insitu concrete and are completely enclosed within concrete shear walls therefore it is expected that these stairs, although they will be damaged in the Maximum Considered Event (MCE) are unlikely to collapse.

2.2.3 Equivalent Static Analysis

An equivalent static analysis was used to estimate the capacity of the Loading Dock and Store structure. The results indicate that the reinforced block walls supporting the reinforced concrete cantilever canopy, have the capacity to resist approximately 70% DBE for an IL3 building.

3. POST-EARTHQUAKE BUILDING CONDITION

This section covers the structural damage sustained by the Clinical Services building, as a result of the series of earthquakes that include the Darfield Earthquake that struck at 4:36am on 4th September, 2010, the Lyttelton Earthquake that struck at 12.51 pm on the 22nd of February, 2011 and earthquakes on the 13th June 2011 and 23rd December 2011. The Lyttelton Earthquake subjected the building to strong ground motions 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.

The fundamental period of the Clinical Services Building has been estimated at 0.3-0.4 seconds. Based on the strong motion data downloaded, it appears that the earthquake produced shaking intensities between 75 and 100% DBE for an IL3 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. Rupture of the Alpine Fault 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 [9], [10]
- damage observed after the Darfield Earthquake
- damage observed during an initial walk around after the Lyttelton Earthquake

In conjunction with a review of the structural drawings and previous seismic assessment work associated with this building the following areas were identified for potential damage;

- flexural cracking of the columns/piers
- shear cracking of walls, beams and columns
- damage to hinge zones of columns/piers due to poor detailing
- damage to plant room structure
- pounding at seismic joint to Central Riverside building

Preliminary observations were carried out on 25 February 2011. These identified the following primary areas of damage;

- flexural cracking of piers (top floor in particular)
- shear cracking of shear walls
- diagonal cracking of plant room slab
- finishes damage around seismic joints

In general, the building appears to have behaved in a similar manner to that predicted from our preliminary review, although the level of damage observed to the external punctured shear walls was less than expected.

3.3 DETAILED OBSERVATIONS

The detailed structural observations were completed from 21st to 23th March 2011 with additional items being viewed as the structure was open up to view during April to June 2011. A full record of these observations is attached in Appendix A, with reference plans describing the location labelling used found in Appendix B. A full photographic record of the observations is available electronically on request.

The observations of the central north-south shear wall, plant room, area around the radiation bunker and columns below the plant room were updated following the 23 December 2011 earthquake.

A review of the building damage was conducted in May 2013.

Observations of the northern and southern piers in the lower ground floor of the Paediatric Outpatients Building were undertaken on the 14th August 2013. This area was been identified as having potential for minor damage as a result of settlement in light of the levels survey (Section 3.5).

3.4 SUMMARY OF BUILDING DAMAGE

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

Shear Walls

Diagonal cracking was observed in the central north-south shear wall at all levels and in the shear wall on the west face of the building at Lower Ground level. These walls are the stiffest wall in the north-south direction and would have attracted the majority load. The cracks varied in width between 0.2 and 0.5mm typically and were evenly distributed along the length of the wall.

Whilst no specific testing has been carried out on the reinforcing for strain hardening, based on the testing in the Riverside, the Boiler House chimney and 235 Antigua St buildings indicated that the strain hardening in the reinforcing bars had occurred over 2 bar diameters. This is likely to have been due to the dynamic effect of the concrete bond to the reinforcing bar being very strong under dynamic loading coupled with the reinforcement being engineered to have a very low rate of strain hardening and the strength of the concrete due to its age.

The strain in the reinforcing bar at a 0.4-0.5mm crack if it occurred over 2 bar diameters would therefore be approximately 1.5-2% which would indicate a loss of strain hardening capacity of up to 11-13%. Due to the potential low loss of strain hardening capacity combined with even distribution of cracks the potential loss of strain hardening capacity of the reinforcing bars in the shear walls is not considered significant.

Cracks up to 1mm in width were observed at the north end of the central north-south wall in the basement. The strain in the reinforcing bar at a 1mm crack if it occurred over 2 bar diameters would be approximately 3.1-3.9% which would indicate a loss of strain hardening capacity of up to 20-25%.

The east and west walls of the original building are heavily perforated with original and new openings and cracking up to 0.3mm in width has been observed.

Floor Slabs

Cracks were observed in the floor slabs adjacent the central north-south shear wall. These cracks varied in width up to 1.3mm. These cracks have formed due to the seismic forces being transferred to the central shear wall through the floor diaphragms/slabs. Cracks had also formed in the floor slab at the junction of the original building and the eastern extension and the Third Floor plant room slab.

Second Floor North and South Walls

Cracking at the base of the Second Floor piers was observed on the south face of the building. The roof diaphragm is not strong and the therefore the exterior walls partially cantilever above the Second Floor. The walls have cracked under these face loads. The Third Floor plant room does not have a stiff or strong lateral load resisting system below it, particularly in the north-south direction and therefore would have deflected significantly during the earthquake and thus contributed to the damage visible in the south wall piers. The piers on the north face of the building at the Second Floor are clad and were not able to be viewed at the time of the detailed observations. It is likely that these piers have cracked also and these should be inspected as part of the repair work.

Third Floor Plant Room Upstand Walls

The exterior walls on the west and east faces of the Third Floor plant room where they extend to the roof adjacent to the Riverside Central building have significant cracking a the base of the windows. These cracks are likely to have been caused by face loads on the wall and by forces due to the pounding of the Clinical Services and Riverside Central buildings.

Significant cracking was observed in the beam/wall on the west face of the Third Floor plant room over the radiation bunker and in the bunker walls and the Second Floor slab where it supports the bunker. The Third Floor plant room is not connected to the south wall and the radiation bunker is the only wall below it in the north-south direction. Due to the number and distribution of these walls, Third Floor plant room has a torsional response and the seismic forces and displacements have caused the damage in the structure around the radiation bunker wall.

Second Floor Columns Supporting the Third Floor Plant Room

The Second Floor floor screed adjacent the columns which support the Third Floor plant room was removed on 8th February 2012, exposing a 0.3-0.4mm horizontal crack on the original construction joint at the base of the column. The cracks will have formed due to the forces and deformations during the earthquake on these cantilever columns. It is likely that these cracks were caused by the 22nd February 2011 earthquake and have just been observed in the assessments following the 23rd December 2011 earthquake and additional observations having been made following the December earthquake.

Gravity loads are likely to have caused the crack to close following the earthquake. Therefore, it is difficult to determine if strain hardening has occurred due to the crack width. Based on the results of the testing in Riverside West and the lateral structure below the Third Floor plant room, it is likely that strain hardening of the reinforcing has occurred. This can be confirmed by testing if required.

Basement Service Tunnel

Cracks in the service tunnel in the basement appeared to have opened up during the earthquake and water is currently flowing through some of these cracks. These cracks are up to 3mm in width.

The observations were updated following the 23 December 2011 earthquake. There was a small increase in crack width observed in the diagonal cracks in the central north-south shear wall. New cracks were also observed in the Third Floor plant room upstand walls.

The observations were updated following the May 2013 review of building damage. The cracks in the Third Floor plan room were observed to have been repaired with epoxy injection. Additionally, there was a small increase in crack width observed in one location at the transition between the Clinical Services and Paediatric Outpatients buildings.

3.5 LEVELS SURVEY

A levels survey was carried out by Fox & Associates on 16 June 2011 and the results are summarised in their report dated 28 June 2011 [11].

The results of the verticality survey indicated a consistent minor lean of approximately 1-2mm/m (1:1000 to 1:500) to the south-east. This lean is not considered significant.

A survey of lower ground floor levels was carried out by Fox & Associates on 18-19 June 2012 and the results are presented in their plans dated 20 June 2011 [15].

The results of this survey indicate that there has been some settlement of the Clinical Services and Riverside buildings; mainly to the multi-level portions. The single story portion at the western end of the Clinical Services building has not experienced as much settlement, and slopes upwards towards the western end (that is, the eastern end has been "pulled down" with the multi-level portion of the Clinical Services Building).

There is up to 50mm difference in level between the western and eastern walls of the single storey portion, and up to 80mm differential settlement over the whole building. This is outside the acceptable level tolerances of NZS 3109 [16]. Some vertical cracks are visible in the exterior cladding on the northern and southern walls of the single storey portion. A number of vertical cracks are clearly visible in the basement tunnel walls, which are wider at the base of the wall and up to 3mm in width. Water is exiting through these cracks into the basement tunnel. These cracks are likely to be flexural cracks caused by bending stresses in the walls as they cantilever out of the main portion of the building.

Preliminary settlement analysis using the computer model indicates that if settlement stresses have been relieved to some extent by flexural cracking in basement walls, then additional shear stresses might be imposed on northern and southern piers between Lower Ground Floor level and the single storey roof. Observations were undertaken in August 2013 to view areas identified in the Levels Survey to have been potentially damaged due to the differential settlement. The observations are summarized in Appendix A.

The greater part of this level difference is likely to have occurred as a result of increased vertical actions of the building foundations on the subgrade and dynamic compaction of the subgrade during intense ground shaking (refer also Section 3.6).

3.6 GEOTECHNICAL INVESTIGATION

A geotechnical investigation was carried out by Tonkin & Taylor Ltd in August/ September 2011 and the results are summarised in their report dated September 2011 [12].

The investigation did not specifically address the Clinical Services Building as no significant land damage had been observed around the building and no significant verticality issues had been identified. The investigation specifically addressed the Riverside and Parkside buildings which are to the north and south of the Clinical Services building. From the investigations carried out it can be concluded that the ground conditions for Clinical Services are likely to be similar to that for the Riverside and Parkside buildings, i.e. a non-liquefiable gravel layer present from basement level to 4-5m below basement level with a dense sand layer approximately 2.5m deep below the gravel layer which liquefied during the 22 February 2011 earthquake.

The geotechnical report concluded that for both Parkside and Riverside 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 building foundation during the earthquakes.

The expectations of future land damage have some importance as they relate to the likelihood of further differential settlement and damage to basement walls in the western single storey section (refer section 3.5). The geotechnical report concluded that further damage is only likely should another large earthquake occur [12] and that further settlement is unlikely under smaller more commonly occurring earthquakes. In future large earthquakes, additional settlements of a similar nature may occur.

3.7 FAÇADE SURVEY AND ASSESSMENT

A survey was carried out on the exterior of the building by Goleman and the earthquake damage observed is outlined in their report dated 21 July 2011 [13].

The damage recorded included cracking and spalling of the corners and edges of the terrazite cladding, damage to sealant and damage to flashings.

Investigation into the construction of the exterior walls of the building concluded that it was likely that the precast terrazzo panels were offered up as exterior formwork and the structural walls were poured between these panels and an internal formwork system.

A panel was removed and core samples taken and it was concluded that the terrazzo panels are reinforced with steel mesh and the terrazzo panels are fixed to the concrete shear walls with cast-in “top hat” connectors.

At the test locations there was a strong bond between the terrazzo panel and the concrete shear wall which did not appear to have been weakened by the earthquake.

The core taken at a crack in the terrazzo panel showed that the crack did not extend through into the concrete shear wall. The cracks however should be epoxy injected for durability if the intended life of the building is greater than 5 years.

The spalling and loose sections of terrazzo around the window openings are to be repaired to remove the risk of loose sections of terrazzo dislodging.

The repair work for these items is detailed in Table 3-1.

3.8 MATERIALS TESTING

Testing of the concrete strength in the Riverside and Clinical Services Building has been carried out by Holmes Solutions and the results are outlined in their report dated 15 July 2011 [14]. The testing was carried out 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 of the tests show that there is a large variability in concrete strength throughout the building. The probable strength of the concrete is likely to vary between 30MPa and 60MPa. It is likely that the concrete at the time of construction was batched on site, and it is visually evident that the compaction was not of a consistently high standard, given the incidence of cracking at construction joints. During the testing of the concrete on site, variability of the concrete in terms of colour and quality was very noticeable.

3.9 FURTHER INVESTIGATIONS RECOMMENDED

3.9.1 Investigations Required For Further Assessment

Refer to sketches in the second part of Appendix C which show areas which require further investigation and possible repair. These are areas which have had restricted access for reviews to date, and which are identified as areas at risk of damage based on detailed analysis results for this building and/or experience in analysis and post-earthquake review of similar types of buildings.

3.9.2 Investigations to be Completed During Building Repairs

There are a number of elements identified in Appendix A, the Record of Observations, which are categorised as requiring further investigations. The elements noted are not expected to have a significant impact on the building capacity, stiffness, or durability. These further investigations are required in order to determine the appropriate repair and are not necessary until the time which those repairs are undertaken.

3.10 POST EARTHQUAKE BUILDING CAPACITY

In its damaged state following the earthquakes, we do not consider the Clinical Services Building to have any significant reduction in gravity load resistance.

The cracking to the shear walls observed to date reduces the stiffness of the building. The crack widths and distribution are such that it is likely that strain hardening has occurred to the reinforcing steel in the north-south central shear wall at basement level, the Second Floor columns below the Third Floor plant room, the Second Floor piers on the north and south walls and in the floor slab at First Floor adjacent to the north-south central shear wall. The strain hardening in the reinforcing bars does not reduce the strength of the elements and the building overall but impacts on its ability to withstand repeated cycles of loading.

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 3. Following 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 earthquake.

4. OBSERVED DAMAGE AND REPAIRS REQUIRED

4.1 TYPICAL OBSERVED DAMAGE AND REPAIRS REQUIRED

This section covers the damage noted during our detailed assessment of the building. Note that our observations have been restricted to structural aspects of the building only. Waterproofing elements, electrical and mechanical equipment, fire protection and safety systems, service connections, water supplies and sanitary fittings have not been inspected or reviewed, and secondary elements such as windows and fittings have not generally been reviewed.

Table 4-1 provides a photographic summary of the observed damage and typical repairs required. The table should be read in conjunction with Appendix A – Record of Observations and Appendix B – Location Reference Plans. The Repair Specification referred to in Table 4-1 has been issued separately. Repair sketches, where required, have been included in Appendix C – Repair Sketches. A discussion on the repair work required to replace the lost strain hardening capacity is included in Section 3.1.

In general the aim of the repair work indicated is to restore the structure to its pre-earthquake state as close as practicable. Recommended strengthening to improve the buildings lateral load capacity is outlined in Section 4.

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

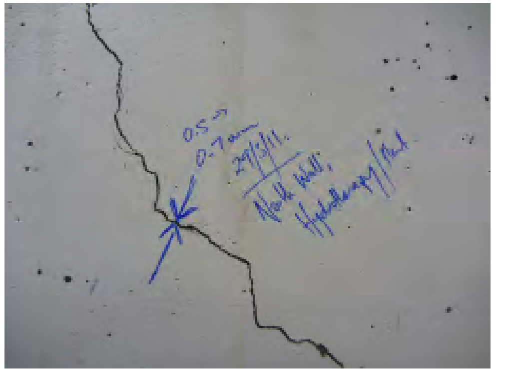
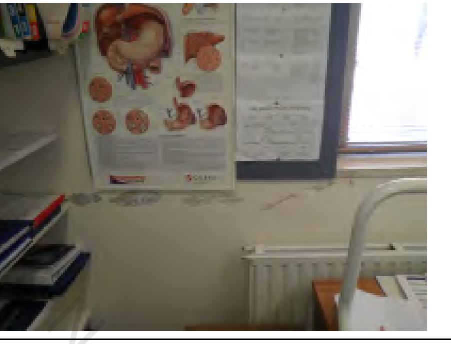
Refer to Section 3.9 and Appendix C for further investigations recommended.

Table 4-1: Repairs Required

Damage	Locations	Recommendation	Example
1. Floor slabs			
1.1. Cracking between 0.2mm and 2mm	PLANT ROOM: Cracking throughout plant room floor up to 0.5mm and at steel column baseplates. Substantial cracking up to 3.5mm in slab upstands/ column baseplates.	Epoxy inject cracks in slab and upstand that are greater than 0.2mm in width where external and 0.3mm in width where internal. Refer to HCG Specification REPAIRS COMPLETE 18/08/11	
1.2. Cracking up to 0.4mm on the floor support the radiation bunker.	SECOND FLOOR: Cracking in floor adjacent to radiation bunker.	Inspect the top and bottom of the floor in the floor bays that contain and surround the radiation bunker and epoxy inject all cracks that are greater than 0.3mm in width. Refer to HCG Specification	No photo
1.3. Cracking up to 1.3mm adjacent to the central north-south shear wall	LOWER GROUND, GROUND AND FIRST FLOOR: Cracking in floor adjacent to the central north-south shear wall	Inspect the floor on both sides of the central north-south shear wall at Lower Ground, Ground and First Floor levels and epoxy fill cracks that are greater than 0.3mm in width. Refer to HCG Specification	

Damage	Locations	Recommendation	Example
1.4. Cracks up to 0.5mm in width at the junction between the original building and the extension to the east.	LOWER GROUND, & GROUND FLOOR: At junction between the original building and the extension to the east.	Inspect the floor joint between the original building and the extension to the east at Lower Ground, Ground, First and Second floor levels and epoxy inject cracks that are greater than 0.3mm in width. Refer to HCG Specification	
1.5. Cracking up to 0.5mm in the floor slab of the east extension	LOWER GROUND FLOOR: Cracking in the floor slab in Room LGE24	Remove vinyl on top of slab to inspect crack location. Epoxy fill cracks that are greater than 0.3mm in width. Refer to HCG Specification	No photo

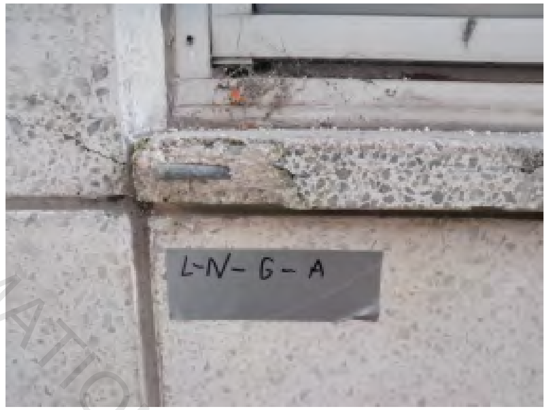
Damage	Locations	Recommendation	Example
2. Shear walls			
2.1. Cracking up to 1mm in the central north-south shear wall.	BASEMENT – LOWER GROUND: Central north-south shear wall.	Inspect the central north-south shear wall at Basement, Lower Ground and Ground and epoxy inject cracks that are greater than 0.3mm in width. Refer to HCG Specification.	
2.2. Cracking to the shear walls on the west face of the building where internal at Lower Ground Level	LOWER GROUND FLOOR: Diagonal cracks in the shear wall at Lower Ground Level and at the base of the piers and spandrels at Ground Level.	Inspect the walls on the west face of the original building at Lower Ground, Ground and First Floor and epoxy inject cracks in the walls that are greater than 0.2mm in width where external and 0.3mm in width where internal. Refer to HCG Specification.	
2.3. Cracking to the walls on the east face of the original building (now internal)	LOWER GROUND FLOOR: Diagonal cracks in the shear walls	Inspect the walls on the east face of the original building at Lower Ground and Ground and epoxy inject cracks in the walls that are greater than 0.2mm in width where external and 0.3mm in width where internal. Refer to HCG Specification.	No photo

Damage	Locations	Recommendation	Example
2.4. Cracking up to 0.7mm in the walls in the hydrotherapy room	LOWER GROUND FLOOR: Cracking up to 0.7mm in hydrotherapy room. Located on the Clinical Services northern shear wall and the northern Hydrotherapy wall shared with Riverside plant room	Inspect the walls to the hydrotherapy room and epoxy inject all cracks that are greater than 0.2mm in width. Refer to HCG specification.	
2.5. Cracking and corroded reinforcement in pier on south face of Second Floor	SECOND FLOOR: South wall pier near column 6, Second Floor	Non-earthquake damage but in a location that requires earthquake repair, therefore needs to be addressed as part of the repair. Break out concrete to expose reinforcing. Repair to be advised when reinforcing has been inspected.	
2.6. Cracking between 0.2mm and 2mm in the north and south exterior walls	SECOND FLOOR, LOWER GROUND, GROUND : South wall piers. North wall piers unable to be viewed due to cladding.	Inspect all piers and spandrels on the north and south faces of the Second Floor and epoxy inject cracks that are greater than 0.2mm in width. Refer to HCG specification	

Damage	Locations	Recommendation	Example
2.7. Cracking up to 1.5mm in exterior walls in the plant room	PLANT ROOM: Pier adjacent windows in northeast and northwest corners of the plant room adjacent to Riverside.	Cut out and replace the concrete upstand with light weight construction. Refer to sketches CS-RC-01 to CS-RC-03. Epoxy inject all remaining cracks that are greater than 0.2mm in width. Refer to HCG specification. REPAIRS COMPLETE 10/07/13	
2.8. Vertical cracking up to 3.5mm in width in the plant room	PLANT ROOM: North end of west wall above radiation storage bunker	Epoxy inject all cracks that are greater than 0.2mm in width. Refer to HCG specification. REPAIRS COMPLETE 10/07/13	

Damage	Locations	Recommendation	Example
2.9. Cracks up to 2mm wide in radiation bunker	SECOND FLOOR Radiation Bunker: Crack up to 2mm wide in what appears to be an original construction joint and diagonal cracks in bunker walls.	Epoxy inject all cracks that are greater than 0.3mm in width. Refer to HCG specification.	
3. Columns			
3.1. Inspection	SECOND FLOOR: Columns supporting the Third Floor plant room. Two columns inspected – both had 0.3-0.4mm horizontal cracks at the base on the original construction joint.	Expose any all columns for inspection below the Third Floor plant room and epoxy inject the cracks.	

Damage	Locations	Recommendation	Example
4. Basement			
4.1. Cracks in north and south walls up to 1mm in width	North and south walls	Inspect all walls and epoxy inject all cracks that are greater than 0.2mm in width. Refer to HCG specification.	
4.2. Cracks in service tunnel walls up to 3mm in width	Service tunnel walls	Inspect all walls and epoxy inject all cracks that are greater than 0.2mm in width. Refer to HCG specification.	
5. Seismic Gaps			
5.1. Damage to cover plates and linings	Seismic gaps to Riverside Central	Make good finishes and cover plates	No photo

Damage	Locations	Recommendation	Example
6. Terrazite cladding	From Goleman Survey		
6.1. Cracking of the Terrazzo cladding	Refer to Goleman Report	Epoxy inject all cracks that are greater than 0.2mm in width. Refer to HCG specification.	
6.2. Spalling of Terrazzo	Refer to Goleman Report	Repair to spalled areas to be specified.	

Damage	Locations	Recommendation	Example
6.3. Damage to flashings	Refer to Goleman Report	Specification by others	

4.2 DAMAGE REMEDIATION

Repair is required to reinstate the capacity of the structure lost due to strain hardening of the reinforcing. Testing of the reinforcing in the Boiler House chimney and in Riverside has shown that the strain hardening has been occurring over a length of bar equivalent to the crack width plus 0.5 to 1 bar diameter on either side of the crack. Strain hardening of reinforcing bars reduces the capacity of the building to resist future earthquakes. Note that when the strain capacity of the bar has been exceeded the bar will break. Fracturing of the bars would lead to a loss of strength in both flexure and shear and reduce the capacity of the building as a whole. Reinstatement of the loss of strain hardening capacity is therefore required.

Based on the testing of the reinforcing bars in Riverside it is likely that strain hardening has occurred in the reinforcing bars in the following locations in the Clinical Services building:

- The central north-south shear wall
- The floor diaphragm adjacent to the central north-south wall
- The Second to Third Floor columns below the Third Floor plant room
- The north and south piers above the Second Floor.

It is likely that some strain hardening has also occurred over cracks in the basement tunnel walls under the western single storey portion of the Clinical Services building, and also to the northern and southern basement walls of the single storey portion below backfill level.

The repair for the strain hardened reinforcing is discussed for each of these areas below. The options are presented in sketch form in Appendix C and are separated into options A and B. Of the options presented below, the Option A scheme sketches generally involve reinstating the capacity of elements by replacement or increasing their size. The option B scheme sketches generally involve the reduction in future strain in the damaged elements (where possible) by the introduction of new elements elsewhere.

4.2.1 Central North-South Shear Wall

At lower ground and ground floor the crack widths indicate that the loss of strain hardening capacity is 10-15% DBE. This level of strain hardening would be considered to be within the levels of accuracy of the testing that can be carried out considering the variability of the parameters effecting strain hardening, therefore no reinstatement of lost capacity is required for these walls.

At basement level, for the reinforcing steel in the 1mm wide crack, the loss of strain hardening capacity is estimated at between 20% DBE and 25% DBE. (Testing can be carried out to confirm this). Reinstatement of the loss of strain hardening capacity is required for the basement wall.

Repair options for the reinforcing that has strain hardened include:

- Cutting out and replacing the reinforcing. This will involve effectively replacing the existing wall.

- The addition of replacement lateral load resisting elements. This can be done by shotcreting new walls on the face of the existing walls. Note that the wall will need to extend from foundation level to the underside of the Ground Level in order to anchor the reinforcing outside the critical areas. The location of the strengthening work to reinstate lost capacity is given in Sketches CS-RC-04 and CS-RC-05 in Appendix C.

4.2.2 Floor Diaphragm Adjacent to the Central North-South Shear Wall

The floor slab adjacent to the shear wall has cracked due to the seismic forces being transferred to the central shear wall through the floor diaphragms/slabs. Cracks up to 1.3mm in width have formed at First Floor, with cracks 0.5mm in width in the Lower Ground floor slab. The central north-south shear wall stops at the underside of First Floor, therefore the First Floor slab acts as a transfer diaphragm and significant shear forces are transferred through the floor diaphragm to the shear wall at this level. Testing of the floor reinforcing in the Diabetes Building showed that there was a loss of strain hardening capacity of up to 20-25% in the reinforcing which crossed then 0.9-1mm wide floor cracks. It is likely therefore that strain hardening has occurred in the reinforcing steel in the floor steel at First Floor. Reinstatement of the loss of strain hardening capacity is required for the First Floor slab for cracks wider than 0.5mm.

Repair options for the reinforcing that has strain hardened include:

- Replacing the reinforcing that has strain hardened which involves breaking out areas of the concrete slab around crack locations to expose the existing reinforcing bars and lapping this with new D16 bars before reinstating the concrete. Where cracks are diagonal to the existing reinforcement the slab steel will need to be replaced in both directions.

- Reducing the future strain in the reinforcing by reducing the forces in the First Floor diaphragm adjacent to the central north-south shear wall. This can be achieved by extending the central north-south shear wall up to the underside of the plant room and thus eliminating the need for the First Floor slab to act as a transfer diaphragm. Extending the north-south shear wall up to the underside of the plant room will mean that the lateral forces from the plant room and Second Floor will transfer directly into the central north-south shear wall rather than transfer through the First Floor diaphragm. Note that the addition of this wall also removes the critical structural weakness of the Second Floor columns that support the plant room as outlined in Section 4.1. The location of this strengthening work is given in Sketches CS-RC-06 and CS-RC-07 in Appendix C.

4.2.3 The Columns below the Third Floor Plant Room

Horizontal cracks have formed at the base of the columns that support the Third Floor plant room. Due to gravity loads causing the crack to close following the earthquake, it is difficult to estimate the strain hardening of the vertical column reinforcing. Based on the results of reinforcing testing in Riverside West and the lateral structure below the plant room, it is likely that a loss of strain hardening capacity of the reinforcing has occurred. Testing can be carried out to confirm this if required. Reinstatement of the loss of strain hardening capacity is required for the columns.

Repair options for the reinforcing that has strain hardened include:

- Cutting out and replacing the reinforcing. This would involve replacing the reinforcing in the columns at both the First and Second Floors.

- Strengthening the columns by increasing their size with a new reinforcing cage outside the existing column, at both the First and Second Floors. Stirrups would be required to be drilled and epoxied through the existing column to provide restraint for the new reinforcing. Note that there would be some loss of utility in the space adjacent to the columns due to their increased size.

- Reducing the future strain in the reinforcing by reducing the loads on the columns. The addition of the shear wall in the north-south direction and extending the plant room floor to be into the south shear wall will reduce the future loads on the columns as they would no longer be required to cantilever above the Second Floor to resist the lateral loads from the plant room. The location of this strengthening work is given in Sketches CS-RC-06 to CS-RC-08 in Appendix C.

4.2.4 The North and South Piers Above Second Floor

Cracks up to 2mm in width have formed at the base of the piers on the south wall of the building at the Second Floor. The roof diaphragm to the Second Floor is flexible and low strength, therefore the north and south walls have cantilevered above the Second Floor to resist the lateral loads from their self-weight and the roof. The cracks at the base of the piers are flexural cracks. It is likely that these cracks have occurred in the north wall at the Second Floor also – this wall has not been viewed to date due to lining on the walls. Based on the results of the strain hardening testing of the reinforcing in the Riverside West Lower Ground shear walls, it is likely that strain hardening has occurred in the vertical reinforcing in the piers. Testing could be carried out to confirm this if required. Reinstatement of the loss of strain hardening capacity is required for the piers.

Repair options for the reinforcing that has strain hardened include:

- Cutting out and replacing the reinforcing. This would involve breaking out and reforming the piers between the Second Floor slab and the underside of the lintel beam above the window.

- Forming new piers on the inside face of the existing piers. These new piers would extend from the First Floor to roof level. It should be noted that this option would reduce the space available for use in the rooms at the First and Second Floors.

- Reducing the future strain in the reinforcing by reducing the loads on the piers. This can be achieved by the addition of roof cross bracing and extending the Third Floor plant room slab to the south wall. The roof cross bracing would transfer the lateral loads from the roof and the tops if the walls to the north-south shear walls and therefore the north and south piers would span between the Second Floor and the roof cross bracing and not be required to cantilever above the Second Floor. The location of this strengthening work is given in Sketch CS-RC-08 in Appendix C.

4.2.5 Basement Walls under Western Single Storey Portion

Vertical cracks up to 3mm in width have formed in the basement tunnel walls as they extend from the main western shear wall of the multi-level portion out to the western end of the single storey portion. Cracks are also likely to have formed in the perimeter basement walls, below the level of backfilling.

The northern and southern basement walls should be exposed to footing level by excavating backfill, and surveyed for cracks. The piers above lower ground level should be surveyed for cracking.

Repair options for the reinforcing that has strain hardened include:

- Cutting out and replacing the reinforcing. This would involve breaking out hit and miss sections of wall in the region of the larger cracks to expose the reinforcing, lapping or welding new reinforcing bars adjacent to existing strain hardened bars and reforming the wall.

- The addition of replacement lateral load resisting elements. This can be done by shotcreting new walls on the face of the existing walls. Note that the existing cracks would need to be epoxy injected to restore proper concrete cover and durability.

5. STRENGTHENING RECOMMENDED

Our assessment suggests that the building's capacity, prior to the earthquake, was likely to be around 35 % DBE and possibly even less than this when taking into account the CERA recommended factor of safety on critical structural weaknesses. If applying for building consent for repairs the Christchurch City Council may require the building to be strengthened to at least 67 % DBE. Irrespective of the council requirements we highly recommend that strengthening of the building is undertaken and 67 % of current code should be the minimum level considered.

There are several issues to consider when deciding what and how to strengthen the building; these are divided into two sections below. The first section describes strengthening required to remove the risk of the critical structural weaknesses that have been identified that govern the building's performance and the second is the strengthening to increase the capacity of the whole structure to a higher percent of the current code.

5.1 STRENGTHENING TO REMOVE CRITICAL STRUCTURAL WEAKNESSES

The NLTHA shows that critical deficiencies (or damage that may lead to partial or total collapse of the structure) may commence at a load level of 60-70 % DBE for an IL3 structure. When this is divided by a factor of safety of 1.8 to provide a margin on collapse this number becomes 33-40 % DBE. Ideally we do not want these Critical Structural Weaknesses to govern the building's performance. Even if the building is not strengthened, the collapse hazard identified in the NLTHA should be mitigated to ensure the capacity of these, divided by 1.8, does not govern the reliable strength.

The elements with critical structural weaknesses are the concrete columns supporting the Third Floor plant room. The critical structural weakness of the concrete columns supporting the Third Floor plant room can be addressed by:

1. Extending the plant room concrete floor slab to the south wall, extending the north-south central shear wall up to the underside of the plant room and detaching the radiation bunker from the mechanical plant room so that it does not take any loads. This scheme has the advantages of:

    - reducing the deflection of the building at the Third Floor and thus the onset of pounding of the building with Riverside Central,

    - reducing the forces being transferred through the diaphragms as the Second and Third Floor forces will transfer directly to the central north-south shear wall and

    - reducing the potential for damage to the walls of and floor supporting the radiation bunker.

2. Extending the plant room concrete floor slab to the south wall and strengthening the columns below the Third Floor plant room from the First Floor to the Third Floor. This scheme is not the preferred scheme however as it does not reduce the deflections of the Third Floor in the north-south direction or reduce the potential for damage to the radiation bunker and lower level diaphragms.

The preliminary scheme for the strengthening work required to remove the critical structural weaknesses as outlined above is given in Sketches CS-CSW-01 to CS-CSW-03 in Appendix D. This strengthening work has also been outlined as an option for replacing the lost strain hardening capacity in the columns below the Third Floor plant room in section 3.1.

5.2 STRENGTHENING TO ACHIEVE 67 % DBE (IL3)

We recommend that strengthening of the building is undertaken and 67 % DBE should be the minimum level considered.

For the Clinical Services Building, the NLTHA model could be used to identify the most efficient strengthening scheme. By inspection likely strengthening required to reach 67 % DBE could include:

- Roof steel cross bracing to the roofs above the Second Floor and above the Ground Floor at the west end of the building.

- Steel cross bracing between the roof and the concrete plant room floor in both the north-south and east-west direction.

- Extending the Third Floor plant room concrete floor slab and connecting it to the south wall (this is part of the strengthening to remove Critical Structural Weaknesses).

- Extending the north-south central shear wall up to the underside of the Third Floor plant room (this is part of the strengthening to remove Critical Structural Weaknesses). The wall will not have to extend for the full width of the building.

- Detaching the radiation bunker from the Third Floor plant room so that it does not take any loads (this is part of the strengthening to remove Critical Structural Weaknesses).

- Strengthen the piers on the south wall of the building at Ground Floor Level by the addition of FRP on the face of the wall or a new concrete overlay.

- New concrete wall/overlay or FRP on the west wall of the original building at Lower Ground, Ground and First Floors.

- New concrete wall/overlay or FRP on the east wall of the original building at the Ground and First Floors.

- New concrete wall/overlay on the central north-south wall of the original building at basement, Ground and Lower Ground Floors (sections of this wall are part of the reinstatement of loss of strain hardening capacity.

- New concrete wall/overlay or FRP on the central portion of the east wall of the extension to the building at Lower Ground Floor.

- A new column at Lower Ground, Ground and First Floors at the north east and north west corners of the building.

- Cut down the west and east walls at the Third Floor plant room level where they extend above the base of the windows.

- Reduce the impact on the structure due to pounding of the building with Riverside Central. This could be achieved by cutting back the structure on the north face of the building to create a larger seismic gap. New floor support structure and walls will be required to be constructed to replace those removed to create the gap.

  Other options of stiffening the structures, tying the structures together or building new structures to tie the buildings together could be assessed with further analysis.

The preliminary scheme for the strengthening work required to increase the capacity of the structure as outlined above is given in Sketches CS-SS-01 to CS-SS-06 in Appendix D. Some of this strengthening work has also been outlined as an option for replacing the lost strain hardening capacity in the columns, floor slab and walls as outlined in Section 3.1.

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. *Model Building By-Law – Basic Design Loads*, NZS1900:1965.

4. *Code of Practice for General Structural Design and Design Loadings for Buildings, NZS4203:1992*, Standards New Zealand, 1992.

5. *Concrete Structures Standard*, NZS3101:1995, Standards New Zealand, 1995.

6. *Christchurch Hospital Seismic Risk Assessment Report*, Holmes Consulting Group, 2000

7. *Seismic Risk Assessment Update - Riverside Buildings*, Holmes Consulting Group, 2006.

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

9. *Clinical Services Block Structural Drawings*, Manson Seward and Stanton, 1967.

10. *Alterations to Clinical Services Building*, Powell Fenwick Consultants, 2000

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

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

13. *Post Earthquake report, Riverside Building* Goleman, 22 July 2011.

14. *Non-Destructive Testing of Reinforcing Steel in Riverside Hospital Building*, CDHB, Holmes Solutions Limited, July 2011

15. *CDHB – CHCH Hospital: Riverside LG Floor Levels; Riverside LG Floor Slope Analysis*, Fox & Associates, 20 June 2012.

16. *Concrete Construction Standard, NZS3109:1997, Standards New Zealand, 1997.*

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

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

APPENDIX A

Record of Observations

APPENDIX A – RECORD OF OBSERVATIONS & REPAIRS - CDHB Clinical Services Building

Inspection date: 21,22 and 23 March 2011, 1 and 8 February 2012, 29 June 2012, 27 May 2013, 14 August 2013

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
3	300	General	Floor Slab	23/3/11 -Up to 0.5mm cracks throughout floor slab and at column baseplates 18/08/11 - Repairs Complete	C	Epoxy inject	
3	300	West & East Walls	Slab Upstand	22/3/11 -Vertical crack up to 3.5mm vertical crack to upstand, NW corner -Diagonal crack up to 1.5mm from corner of window, NE corner. With evidence of displacement causing separation at the window frame. -Vertical crack up to 1mm from penetration above access door to roof space (also cracked window), central East wall -Minor cracking (up to 0.6mm throughout upstand) 1/2/12 Cracking at base of NE and NW mullions may have increased slightly 10/07/2013 Epoxy injection completed on all cracks previously identified.	C	Epoxy inject cracks in upstand. Remove and replace NE mullion. Epoxy inject & provide vertical steel brace to NW mullion.	23/3/11 DSCF0053-64 18/8/11 101_1446, 1447 2012-01-17/21, 838 130710 #001

Refer to Table 4.1 and HCG Specification for repair details

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
3	300	West Wall	Slab Upstand	3/1/12 Review following 23 Dec11 earthquake. No movement observed on cracks that had been epoxied injected. New cracks in upstand wall above the radiation bunker. 10/07/13 Repairs Complete	**C**	Epoxy inject	3/1/12 25-27
3	300	Beam 18-19	Beams	22/3/11 -No visible damage	**N**		22/3/11 DSCF0067-68 23/3/11 P1010059-61
3	300	Columns 18-37	Columns	22/3/11 -No visible damage. Internal columns are clad and could not be inspected below ceiling height	**N**		
2	General	31	Roof Rafter Connections	22/3/11 -No visible damage to cast in connection to spandrels -No visible damage to bolted connection at top of column	**N**		22/3/11 DSCF0051-52/66 23/3/11 P1010062
2	General	Various	Slab	23/3/11 -No visible signs of cracking to underside of slab	**N**		
2	General	Columns 31-40	Column	23/3/11 -No visible signs of damage to top of column or rafter/plantroom beam connection. Internal columns are clad and could not be inspected below ceiling height.	**N**		23/3/11 P1010065

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
2	254	Column 20	Column	1/2/12: Cracking at base identified. Possible crack and delamination of screed. Further investigation called for in SR 2/2/12 8/2/12 Screed had been removed at the base of the column. A crack approximately 0.3-0.4mm wide was observed on the original construction joint at the base of the column.	**Y**	Epoxy inject	8/2/12 8621
2	291	Column 30	Column	22/6/11 -Sample column under plant room over. No apparent damage at top & base of column. 1/2/12: Cracking at base identified. Possible crack and delamination of screed. Further investigation called for in SR 2/2/12 8/2/12 Screed had been removed at the base of the column. A crack approximately 0.3-0.4mm wide was observed on the original construction joint at the base of the column	**Y**	Epoxy inject	22/6/11 101_1129 8/2/12 8620
2	291D	General	Stairs & Stairwells	23/3/11 -Hairline horizontal and vertical cracks at top of door corner	**Y**	Epoxy inject	23/3/11 P1010054-56
2	219, 229, 230	Wall 43-47	North Wall - Spandrels/Piers	Piers and spandrels not able to be viewed due to linings.	**F**		
2	231, 244, 283	Wall 47-49	North Wall - Service Penetrations	23/3/11 -No visible damage around penetrations or to adjacent members	**N**		23/3/11 P1010049-52
2	227, 241, 242, 243, 245, 251, 254	Wall 5-10	South Wall - Spandrels	23/3/11 -Minor horizontal hairline cracks	**N**		

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
2	227, 241, 242, 243, 245, 251, 254	Piers 5-10	South Wall - Piers	22/3/11 -Horizontal cracks up to 1 mm at top and bottom of piers on south face of the building. 1/2/12 - Some of the cracks at the base of the piers have increased in width. Cracks up to 1.4mm in width.	**Y**	Epoxy inject	22/3/11 DSCF046-50
2	242	Pier 6	South Wall - Piers	-Corroded reinforcement is suspected at the base of Pier 6.	**Y**	To be Advised	28/3/11 P1020016-19
2	255, 256, 262, 265, 289	Columns 23, 32, 41	West Wall - Piers/Spandrels	22/3/11 -No visible damage 27/05/13 - 0.1mm vertical crack through spandrel in room 256. 27/05/13 - 0.6 mm diagonal crack through east wall viewed in room 255. Also viewed from stairwell room 291D. 0.2mm diagonal cracking typical in same location, fanning downwards from the 0.6mm crack.	**Y**	Epoxy inject	132705 #066-7 132705 #061-4
2	286	Walls to Room 286/7	Concrete Radiation	28/3/11 -Large (up to 2mm) crack along west wall of box, right hand side of doorway. 22/6/11 -Appears to be a separation at a cold joint SR	**Y**	Epoxy inject	28/3/11 P1020024-5
2	284	Walls of Bunker Room	Concrete Radiation Room, East,North and South Wall	Two diag cracks 0.2mm east wall. Horizontal cracks North & South Walls. 1/2/12 - Fine cracks visible in painted and plastered walls.	**Y**	Epoxy inject	
2	287	By Bunker Room Door	Floor Slab	22/6/11 -Room 287 0.4mm crack migrating away from bunker door. 1/2/12: This repair has been done, but crack is still evident I through the vinyl patch	**Y**	Epoxy inject	13/01/12 P1130028
2	290A	Corridor	Floor slab	1/2/12: Crack evident through vinyl	**F**		

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
2	290A	By door to 287	Floor Slab	13/1/12 - New crease in vinyl indicating that there is a crack in the floor slab below.	**Y**	Epoxy inject	
2	284	Adjacent Bunker Walls	Floor Slab	22/6/11 -0.2mm cracks in adjacent bunker	**Y**	Epoxy inject	
1	132	At North End of Corridor, East Wall	Slabs	218/3/11 -Cracks in vinyl in main corridor, North. Remove vinyl locally to inspect potential cracks in top of slab. 22/6/11 0.8mm to 1.3mm crack identified with some spalling	**Y**	Epoxy inject , Spalling Repair	28/3/11 P1020027-28
1	190		North Wall - Piers	No visible damage to spandrel	**N**		
1	190		North Wall - Piers	No visible damage to pier	**N**		
1	190		North Wall - Corner Piers	No visible damage	**N**		
1	134A	Wall 47-49	North Wall - Service Penetrations	29/3/11 -Penetration has been brickwork in-filled on central side. No visible damage to concrete around penetration	**N**		
1	134A	Wall 47-49	North Wall - Service Penetrations	29/3/11 -Reinforcing bar is exposed beneath penetration and is corroded.G37	**Not EQ Damage**		29/3/11 P1020044-5
1	159,158,158 A		South Wall - Shear Walls	29/3/11 -No visible damage	**N**		
1	165,166		East Wall -Shear Wall	29/3/11 -No visible damage	**N**		
1	154		Columns	29/3/11 -No signs of damage to top of column or capital. Internal columns are clad and could not be inspected below ceiling height.	**N**		

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
G	G66	Column 52	North Wall - Corner Pier	28/3/11 -No visible damage to pier	**N**		
G	G72A, G64, G66	Wall 50-52	North Wall - Spandrels	28/03/2011 -No visible damage to spandrel	**N**		
G	G38A,G38C	Lintel	South Wall - Lintel	28/3/11 -Evidence in vinyl of crack in concrete below.	**F**		28/3/11 P1020005.7
G	G38A,G38C	Pier and spandrel	North Wall- pier & spandrel	1/2/12 - Horizontal crack in pier and vertical crack in spandrel at face of pier	**Y**		
G	G66	Column 41	West Wall - Column	28/3/11 -Remove previously made access in ceiling to inspect top of column	**F**		
G	G57	23-32	West Wall - Spandrel	28/3/11 -Hairline flexural crack <0.2 at Northern end. Middle section not inspected due to cladding	**Y**	Epoxy Inject	
G	G16		East Wall - Pier/Spandrel	28/3/11 -No visible damage Demolition of existing spandrels to accommodate extension highlighted on SK-CS-06. From very limited viewing access, no apparent damage was identified above ground level	**F**		
G	G73	Main Corridor	Floor	6/4/11 -Diag cracks adjacent Room G60	**Y**	Epoxy Inject	
G	G73	Main Corridor	Floor	6/4/11 -Longitudinal crack running adjacent and parallel to shear wall. Identified from soffit below. Crack width un-measured (approx 0.3mm to 0.5mm. Vinyl not lifted for inspection at floor level	**Y**	Epoxy Inject	11/04/06 /024,0205
G	G73	Main corridor	Shear walls	Diagonal cracking 0.3 to 0.6 recored post February 2011. Reinspect 1/2/12: Very little change. One 0.4mm crack was re-measure at 0.5mm	**Y**	Epoxy Inject	
G	G18A	At Junction Existing/Original	Floor	6/6/11 -Crack in floor at joint to extension	**Y**	Epoxy Inject	

Refer to Table 4.1 and HCG Specification for repair details

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
LG	LGE47A		Slab	22/3/11 -Up to 1mm cracks visible (at basement level) to underside of slab adjacent and parallel to internal shear wall. Floor covering to be removed in main corridor at crack locations to inspect top of slab 1mm crack width to be verified	**Y**	Epoxy Inject	22/3/11 DSCF0007/11/12/14
LG	LGE24	At Junction Existing/Original	Slab	0.3mm crack at the entry to the Physio Gym (Joint between existing and extension)	**Y**	Epoxy Inject	
LG	LGE34	Centre of Floor	Slab	Up to 0.5mm crack running N/S	**Y**	Epoxy Inject	
LG	LGE47A	South Wall	Lintel above opening	29/06/12 - Vertical crack in lintel at west side of door opening approximatley 1.5mm in width	**Y**	Epoxy Inject	
LG	LGE47A	Main Corridor East Wall	Internal -Shear wall	21/3/11 -- Diagonal cracks typically up to 0.5mm at Northern end of wall - Diagonal cracks up to 0.4-0.5mm at various locations along wall and at corners of doors Refer crack mapping SK-CS-07. 1/2/12: 9# "fan" cracks at northern end were re-measured at 0.5-0.6mm with some added paint spalling. Crack at south end increased from 0.4/0.5 to 0.6mm.	**Y**	Epoxy Inject	21/3/11 P1010015-24
LG	LGE30, LGE31, LGE32	Wall 43-45	North Shear Wall	29/3/11 -- Diagonal cracks up to 0.5mm at re-entrant corners of doors to wall shared with hydrotherapy pool - Horizontal crack along top of wall/slab junction - Vertical crack up to 0.5mm above doorways in LGE32/hydrotherapy. - Diagonal crack up to 0.5mm in wall, room LGE34	**Y**	Epoxy Inject	29/3/11 P1020029-33/36

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
LG	LGE35B, LGW61, LGW62, LGW66C	Piers 52-47	North -Pier	Up to 0.5mm horizontal crack at sill height	Y	Epoxy Inject	
LG	LGE50A, LGE51A, LGE51C, LGE52A&B	Piers 3-6	South -Pier	Up to 0.5mm cracks in pier at sill height	Y	Epoxy Inject	
LG	LGE24		East -Shear Wall	29/6/11 -Hairline diagonal cracks Existing CS wall between extension: Sections of wall exposed, up to 0.3mm diag cracks. Refer crack mapping SK-CS-06	Y	Epoxy Inject	
LG	LGE29	East Wall	Wall foundation	8/5/12 - Vertical cracks in the foundation wall greater than 5mm in width. Refer SR12 Riverside East 8/5/12	Y	Epoxy Inject	
LG	LGW70A	Wall 32-23	West -Shear Wall	Diagonal cracking up to 0.3mm 27/05/2013 - Same diagonal crack now measured at 0.4mm	Y	Epoxy Inject	29/3/11 P10250040-1
LG	LGW49A	SW	Stairs	29/3/11 -Hairline horizontal and vertical cracks at window corner	Y	Epoxy Inject	02/03/11 #0040
LG	LGW66C	North Wall	Pier / Spandrel	17/08/13 - 0.7mm hoirzontal and vertical cracs to pier at sill level, partially covered by plaster lining. 0.1mm vertical crack in spandrel midway along window. 0.4mm seperation between concrete pier and timber partition forming East wall of room	Y	Epoxy Inject	17/08/13 #043 - 047
LG	LGW66E	North Wall	Pier / Spandrel	17/08/13 - Horizontal vertical and diagonal cracking observed to plaster lining of pier at sill level, 0.1-0.2mm.	F		
LG	LGW66D	North Wall	Pier / Spandrel	17/08/13 - Horizontal crack evident at pier level through lining	F		17/08/13 #048

Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
LG	LGW104	North Wall	Pier / Spandrel	17/08/13 - Horizontal crack evident at pier level through lining, 2 locations within room	F		17/08/13 #049-050
LG	LGW103	North Wall	Pier / Spandrel	17/08/13 - Horizontal crack evident at pier level through lining, 4 locations within room	F		17/08/13 #051-055
LG	LGW101	West Wall	Pier / Spandrel	17/08/13 - Horizontal crack evident at pier level through lining	F		17/08/13 #056
LG	LGW98	West Wall	Pier / Spandrel	17/08/13 - Horizontal crack evident at pier level through lining	F		17/08/13 #057
LG	LGWW80	South Wall	Pier / Spandrel	17/08/13 - Fan of 0.1mm diagonal crack to East wall.	Y	Epoxy Inject	17/08/13 #062-064
LG	LGW79	South Wall	Pier / Spandrel	17/08/13 - Spalling of plaster at sill level, no clear cracking through concrete member.	N		17/08/13 #061
LG	LGW78	South Wall	Pier / Spandrel	17/08/13 - 0.3mm horizontal crack through pier at sill level.	Y	Epoxy Inject	17/08/13 #058-60
LG	LGE35B		Hydrotherapy - North Wall	29/3/11 -Diagonal crack up to 0.7mm at west end visible from adjacent plant room and LGE29. -Vertical crack up to 0.3mm east end of wall	Y	Epoxy Inject	29/3/11 P1020038-39/48-52
B	BS2	North	Internal -Shear Wall	22/3/11 -0.5-1mm cracks at various locations along wall. 1/2/12: No change in crack width	Y	Epoxy Inject	22/3/11 DSCF0004-6
B	BS6	South	Internal - Shear Wall	22/3/11 -Hairline cracks around service penetrations	N		
B	BS3-BS7	Various	Upstand Concrete Retaining Walls	22/3/11 -Up to 3mm cracks at various locations in walls	Y	Epoxy Inject	22/3/11 DSCF0017-23
B	BS7	East column	Gravity Column	27/05/13 - 0.1-0.2mm cracking around the perimeter of column at base on three sides.	Y	Epoxy Inject	27/05/2013 090

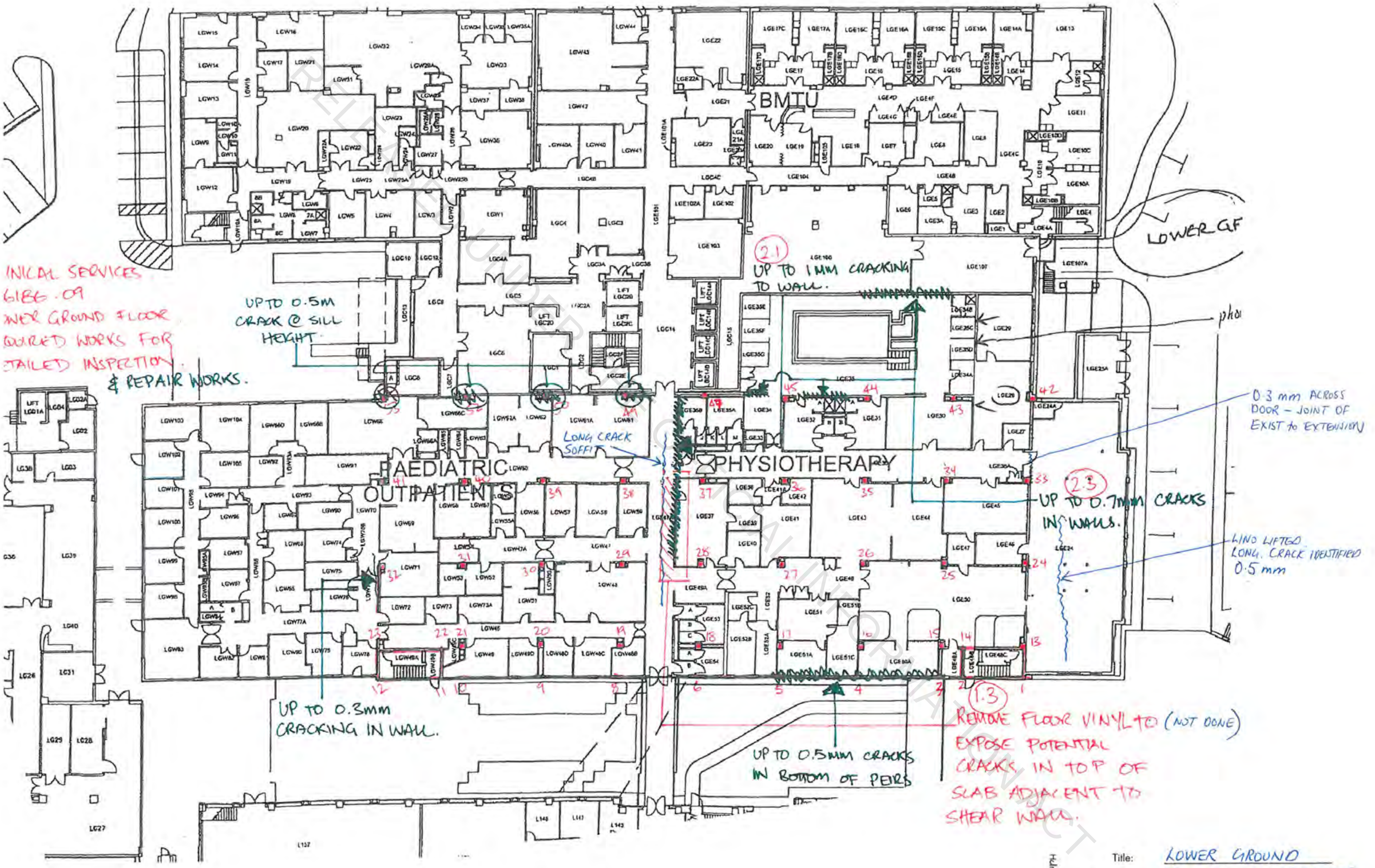
Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
B		General	Retaining Walls	Injection work to stem the ingress of ground water has been progressing as a combined effort with Riverside. This work was suspended in September as to review its effectiveness and monitor the efficiency of the existing pump system			

Refer to Table 4.1 and HCG Specification for repair details

APPENDIX B

Reference Plans

SK-CS-01 to SK-CS-07



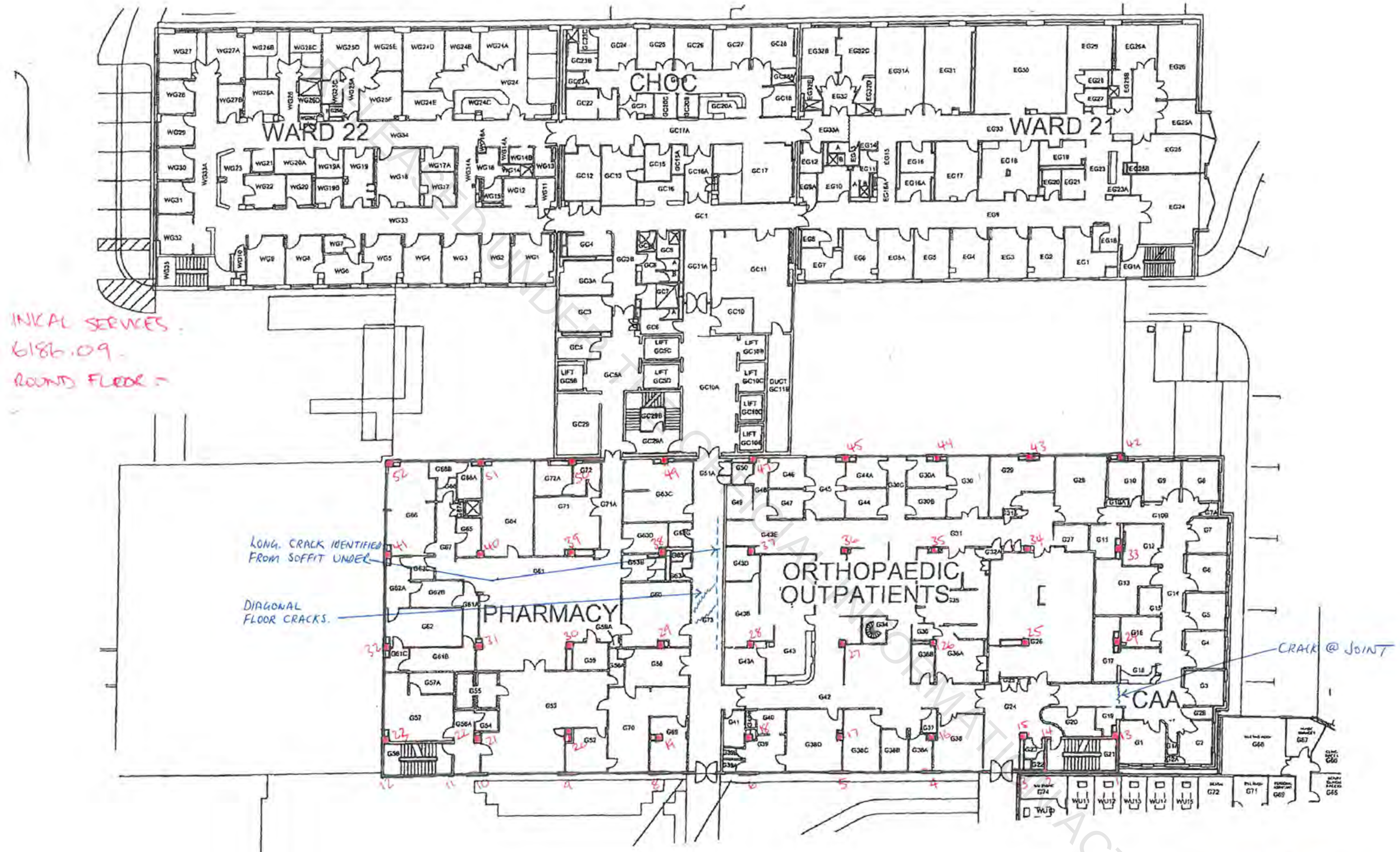
HolmesConsultingGroup

Title: LOWER GROUND

Job Name: CDHB CLINICAL SERVICES

Job #: 106186.09 Rev: 2

Date: OCT 2011 SK-CS-01



HolmesConsultingGroup

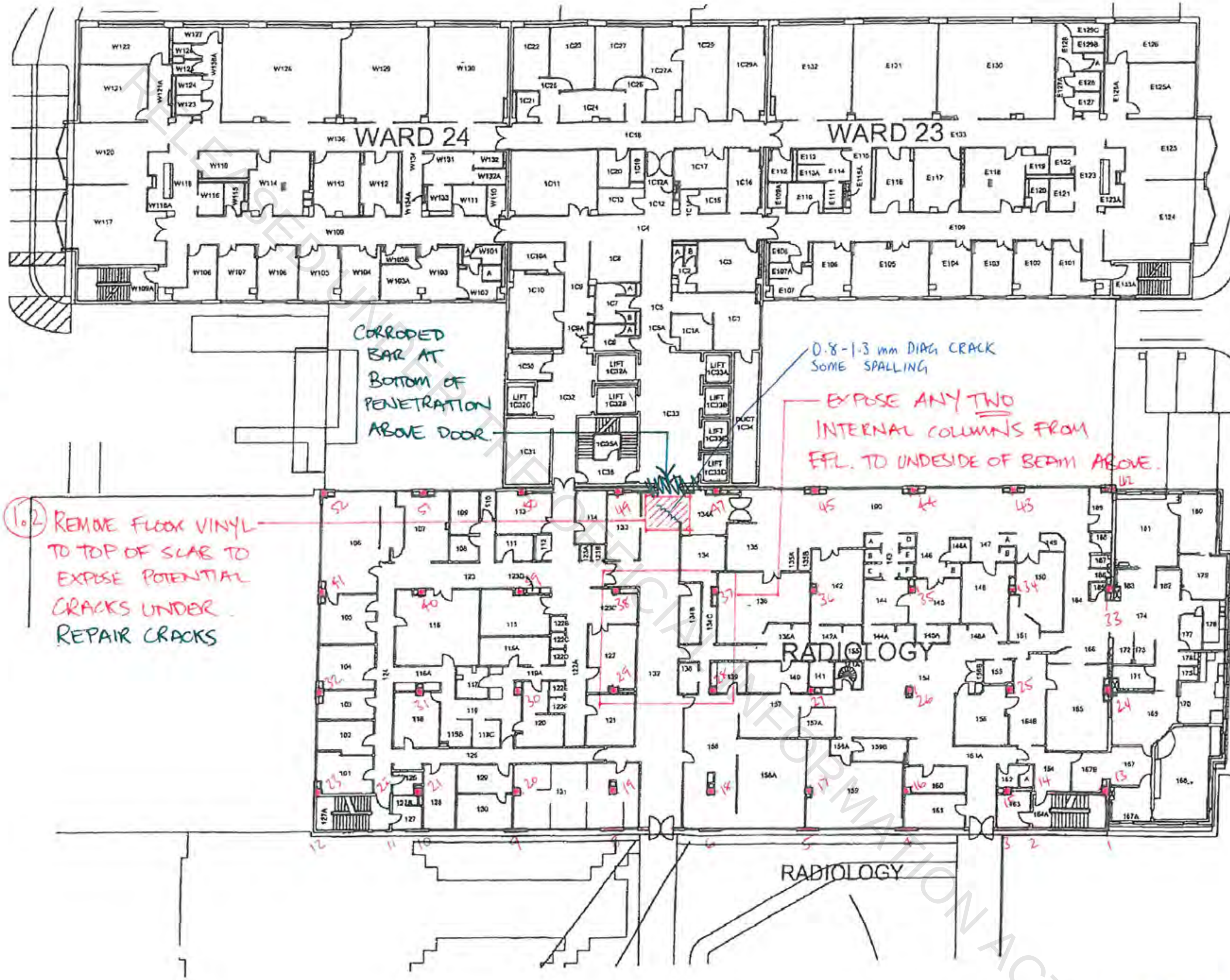
Title: GROUND FLOOR

Job Name: CDHB CLINICAL SERVICES

Job #: 106186.09 Rev: 2

Date: OCT 2011

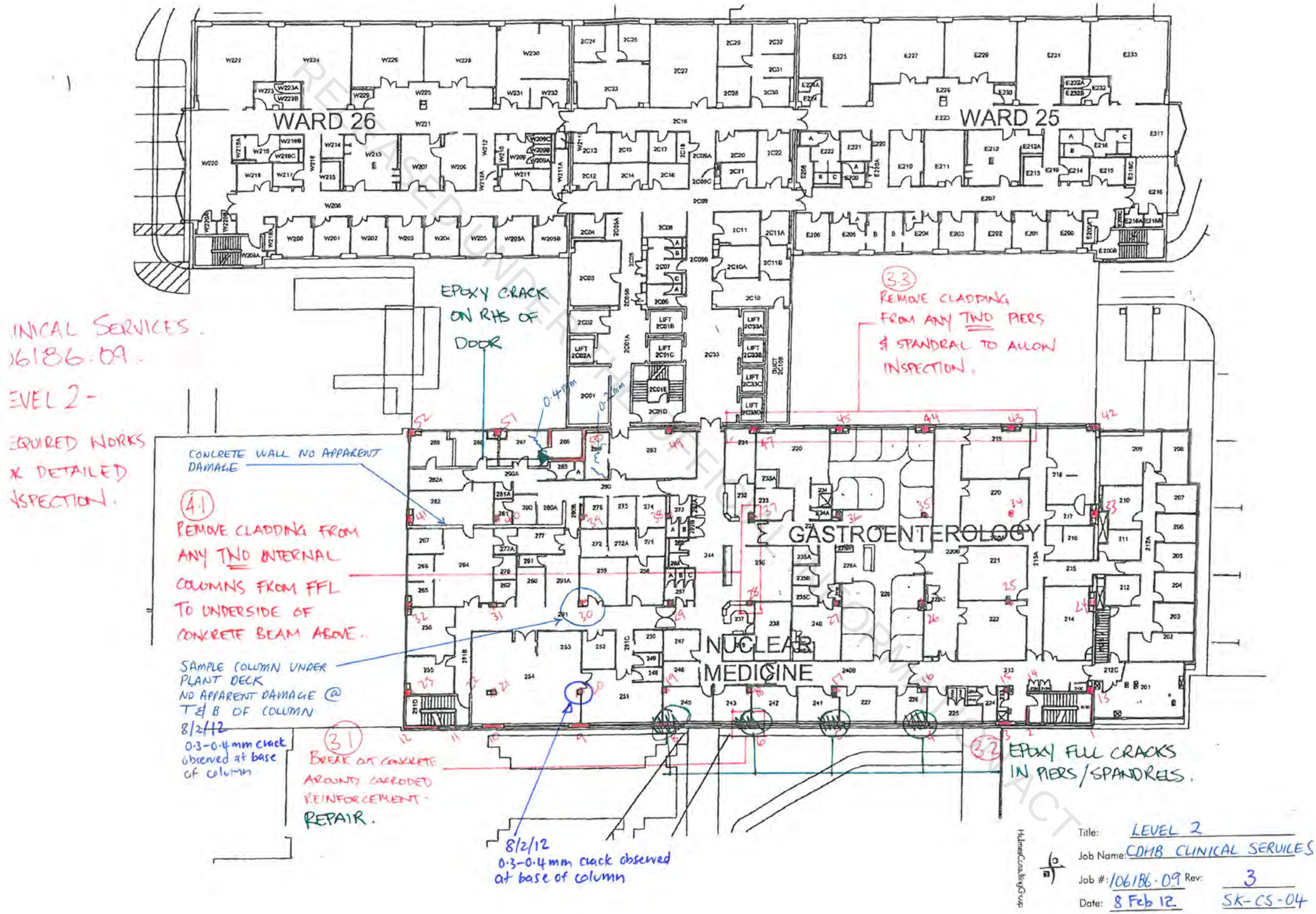
SK-CS-02



INICAL SERVICES.
'6186.09.
EVEL 1.-
EQUIRED WORKS
OR DETAILED
NSPECTION.

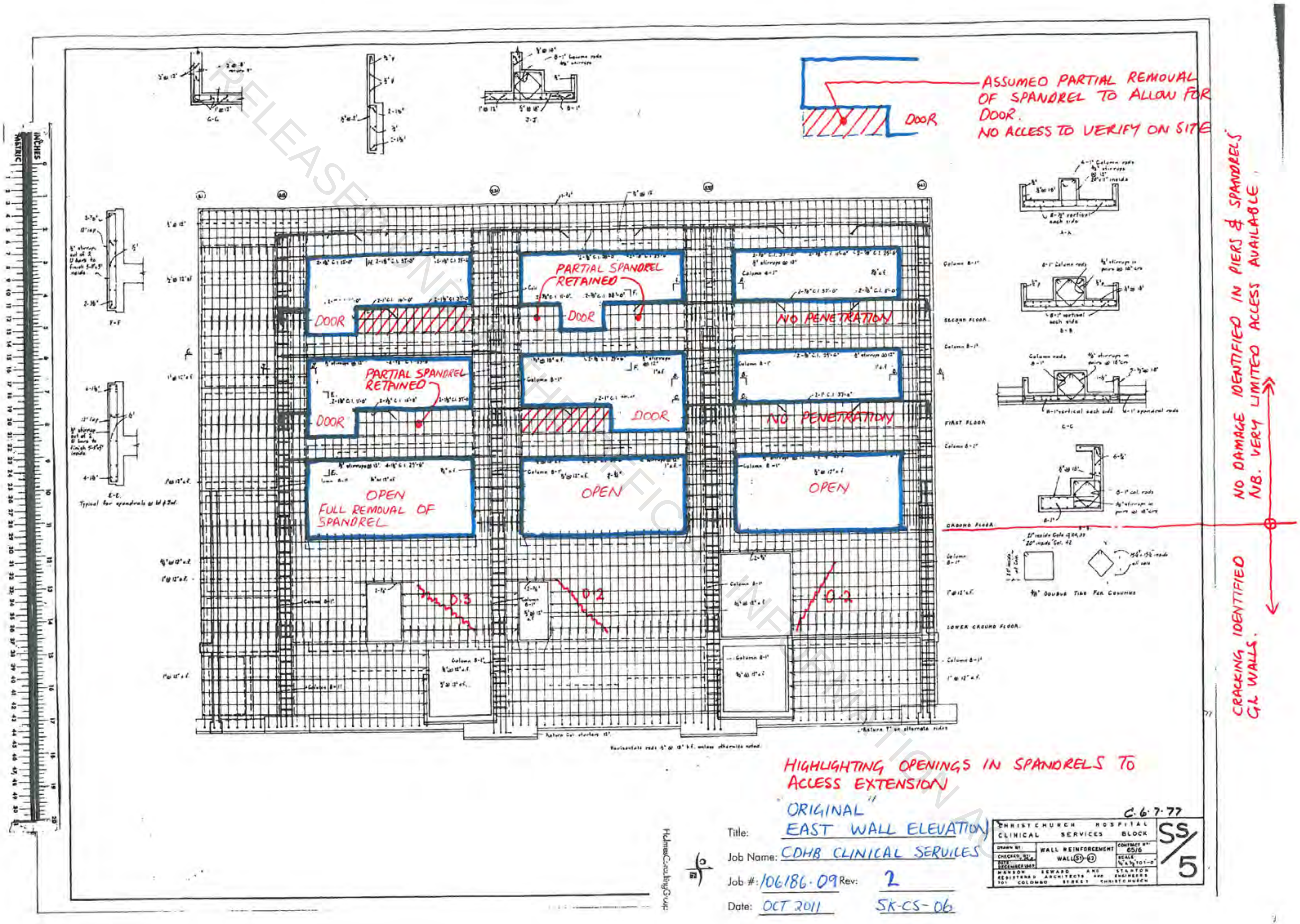
HolmesConsultingGroup

Title: LEVEL 1
Job Name: CDHB CLINICAL SERVILES
Job #: 106186.09 Rev: 2
Date: OCT 2011 SK-CS-03

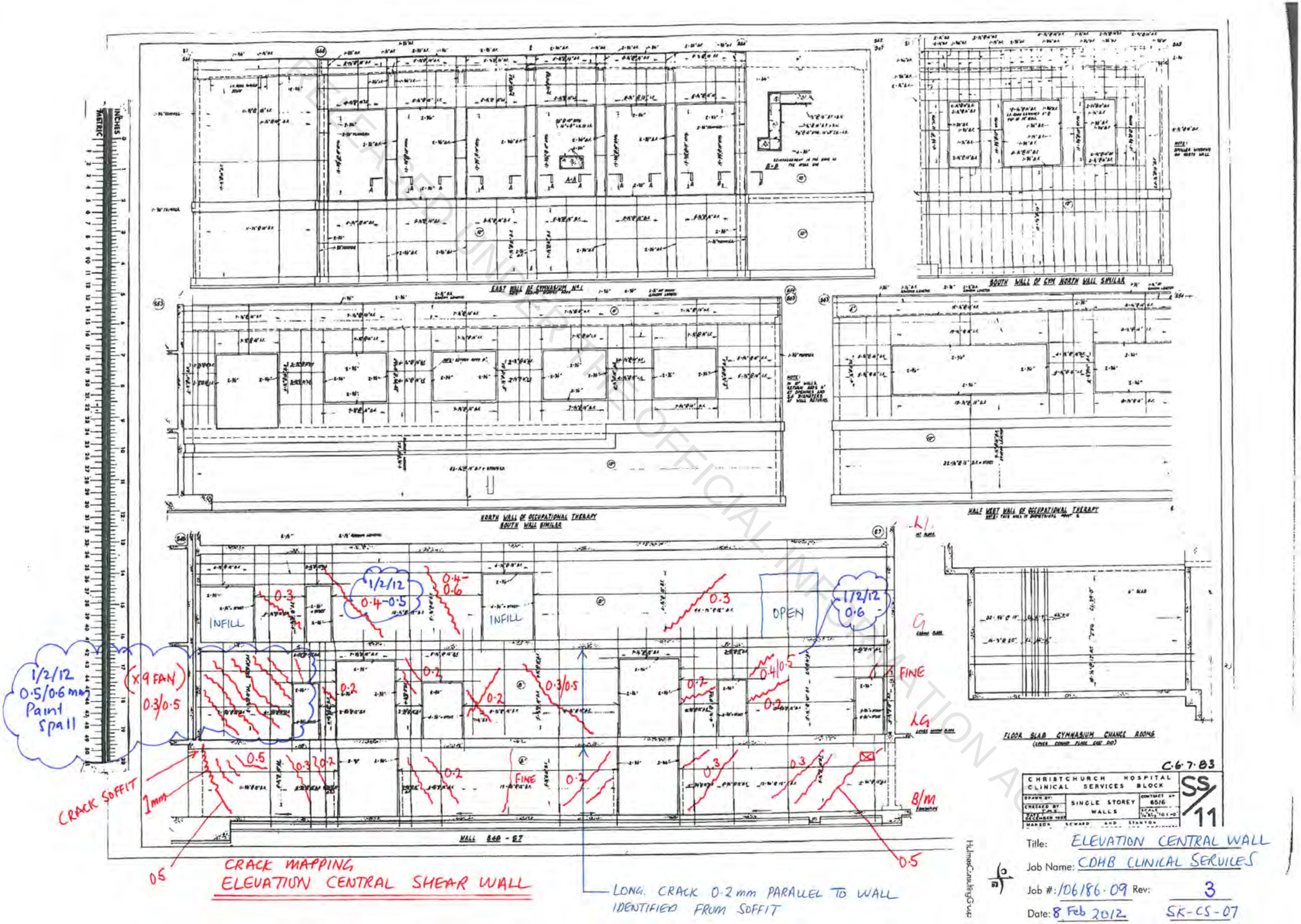

INICAL SERVICES
)6186.09.
EVEL 2 -
EQUIRED WORKS
K DETAILED
NSPECTION.
WARD 26
WARD 25
GASTROENTEROLOGY
NUCLEAR
MEDICINE
EPOXY CRACK
ON RHS OF
DOOR
0.4mm
0.2mm
3.3
REMOVE CLADDING
FROM ANY TWO PIERS
& SPANDRAL TO ALLOW
INSPECTION.
CONCRETE WALL NO APPARENT
DAMAGE
4.1
REMOVE CLADDING FROM
ANY TWO INTERNAL
COLUMNS FROM FFL
TO UNDERSIDE OF
CONCRETE BEAM ABOVE.
SAMPLE COLUMN UNDER
PLANT DECK
NO APPARENT DAMAGE @
T & B OF COLUMN
8/2/12
0.3-0.4mm crack
observed at base
of column
3.1
BREAK OUT CONCRETE
AROUND CORRODED
REINFORCEMENT.
REPAIR.
3.2
EPOXY FILL CRACKS
IN PIERS/SPANDRELS.
8/2/12
0.3-0.4mm crack observed
at base of column
Holmes Consulting Group
Title: LEVEL 2
Job Name: CDHB CLINICAL SERVICES
Job #: 106186.09
Rev: 3
Date: 8 Feb 12
SK-CS-04

Title: LEVEL 3 PLANTROOM
Job Name: CDHB CLINICAL SERVICES
Job #: 06186.09 Rev: 3
Date: 8 Feb 12 SK-CS-05

HolmesConsultingGroup



ASSUMED PARTIAL REMOVAL OF SPANDREL TO ALLOW FOR DOOR.
NO ACCESS TO VERIFY ON SITE
DOOR
PARTIAL SPANDREL RETAINED
DOOR
DOOR
NO PENETRATION
PARTIAL SPANDREL RETAINED
DOOR
DOOR
NO PENETRATION
OPEN
FULL REMOVAL OF SPANDREL
OPEN
OPEN
0.3
0.2
0.2
SECOND FLOOR
FIRST FLOOR
GROUND FLOOR
LOWER GROUND FLOOR
NO DAMAGE IDENTIFIED IN PIERS & SPANDRELS
NB. VERY LIMITED ACCESS AVAILABLE
CRACKING IDENTIFIED GL WALLS.
HIGHLIGHTING OPENINGS IN SPANDRELS TO ACCESS EXTENSION
C. 6. 7. 77
Title: "ORIGINAL" EAST WALL ELEVATION
Job Name: CDHB CLINICAL SERVICES
Job #: 106186.09
Rev: 2
Date: OCT 2011
SK-CS-06
Holmes Consulting Group
CHRISTCHURCH HOSPITAL CLINICAL SERVICES BLOCK
WALL REINFORCEMENT
SS/5


EAST WALL OF GYMNASIUM No 1
SOUTH WALL OF GYM NORTH WALL SIMILAR
NORTH WALL OF OCCUPATIONAL THERAPY
SOUTH WALL SIMILAR
HALF WEST WALL OF OCCUPATIONAL THERAPY
FLOOR SLAB GYMNASIUM CHANGE ROOMS
WALL 56B - 57
INFILL
INFILL
OPEN
1/2/12
0.4-0.5
1/2/12
0.6
0.4-
0.6
0.3
0.3
1/2/12
0.5/0.6 mm
Paint
Spall
(x9 FAN)
0.3/0.5
0.2
0.2
0.2
0.3/0.5
0.2
0.4/0.5
0.2
FINE
0.5
0.3
0.2
0.2
FINE
0.2
0.3
0.3
0.5
CRACK SOFFIT
1mm
0.5
L1
G
LG
B/M
C.6.7.83
CHRISTCHURCH HOSPITAL
CLINICAL SERVICES BLOCK
SINGLE STOREY
WALLS
SS/11
CRACK MAPPING
ELEVATION CENTRAL SHEAR WALL
LONG. CRACK 0.2mm PARALLEL TO WALL
IDENTIFIED FROM SOFFIT
Title: ELEVATION CENTRAL WALL
Job Name: CDHB CLINICAL SERVICES
Job #: 106186.09 Rev: 3
Date: 8 Feb 2012
SK-CS-07

APPENDIX C

Replacing Lost Strength – Scheme A
Replacing Lost Strength – Scheme B

Repair and Further Investigation Sketches

Issued with Rev. 6 report (04/10/13)

APPENDIX C

Replacing Lost Strength – Scheme A
Preliminary Sketches for Pricing
CS-RLSA-01 TO CS-RLSA-06
CS-RLSA-10 TO CS-RLSA-11
CS-RLSA-15 TO CS-RLSA-17
CS-RLSA-20
CS-RLSA-25 TO CS-RLSA-26
CS-RLSA-30

Note: Only elements requiring major repair or additional structural elements are included in this sketch set. This sketch set is to be read in conjunction with Section 3, and the Repair and Further Investigation sketches at the end of Appendix C for areas of the building which require minor repair and epoxy crack injection.

Epoxy inject cracks to extg. wall >0.2mm in width, 100mm new conc. overlay (sprayed or cast in-situ) over cracks > 1.0mm in width.

Where liquid ingress prohibits use of conventional injection techniques, breakout and repair in stages
REFER CS-RLSA-21

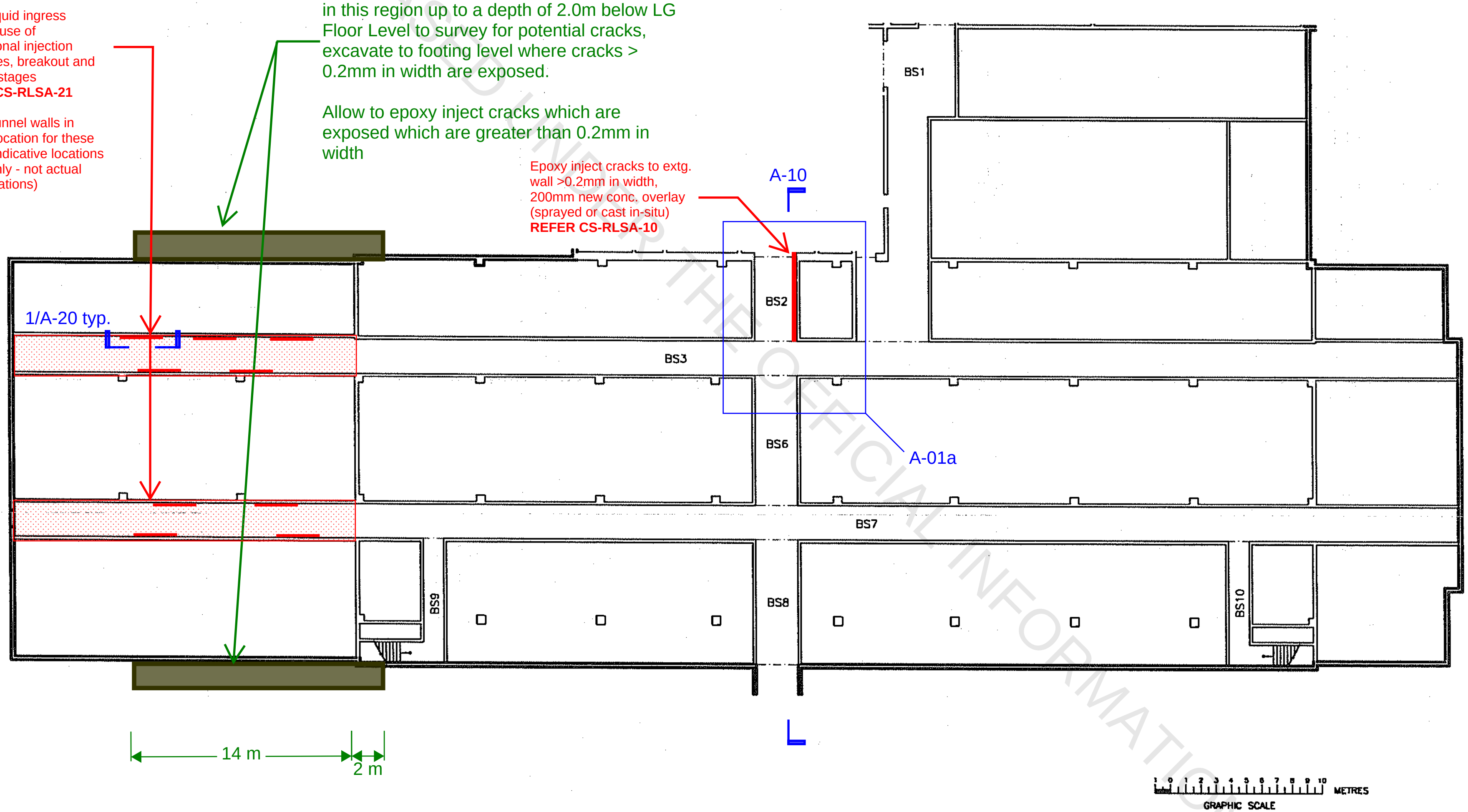
Survey tunnel walls in shaded location for these cracks (indicative locations shown only - not actual crack locations)

Restoring capacity of foundations likely to have experienced settlement damage:

Excavate to expose perimeter basement wall in this region up to a depth of 2.0m below LG Floor Level to survey for potential cracks, excavate to footing level where cracks > 0.2mm in width are exposed.

Allow to epoxy inject cracks which are exposed which are greater than 0.2mm in width

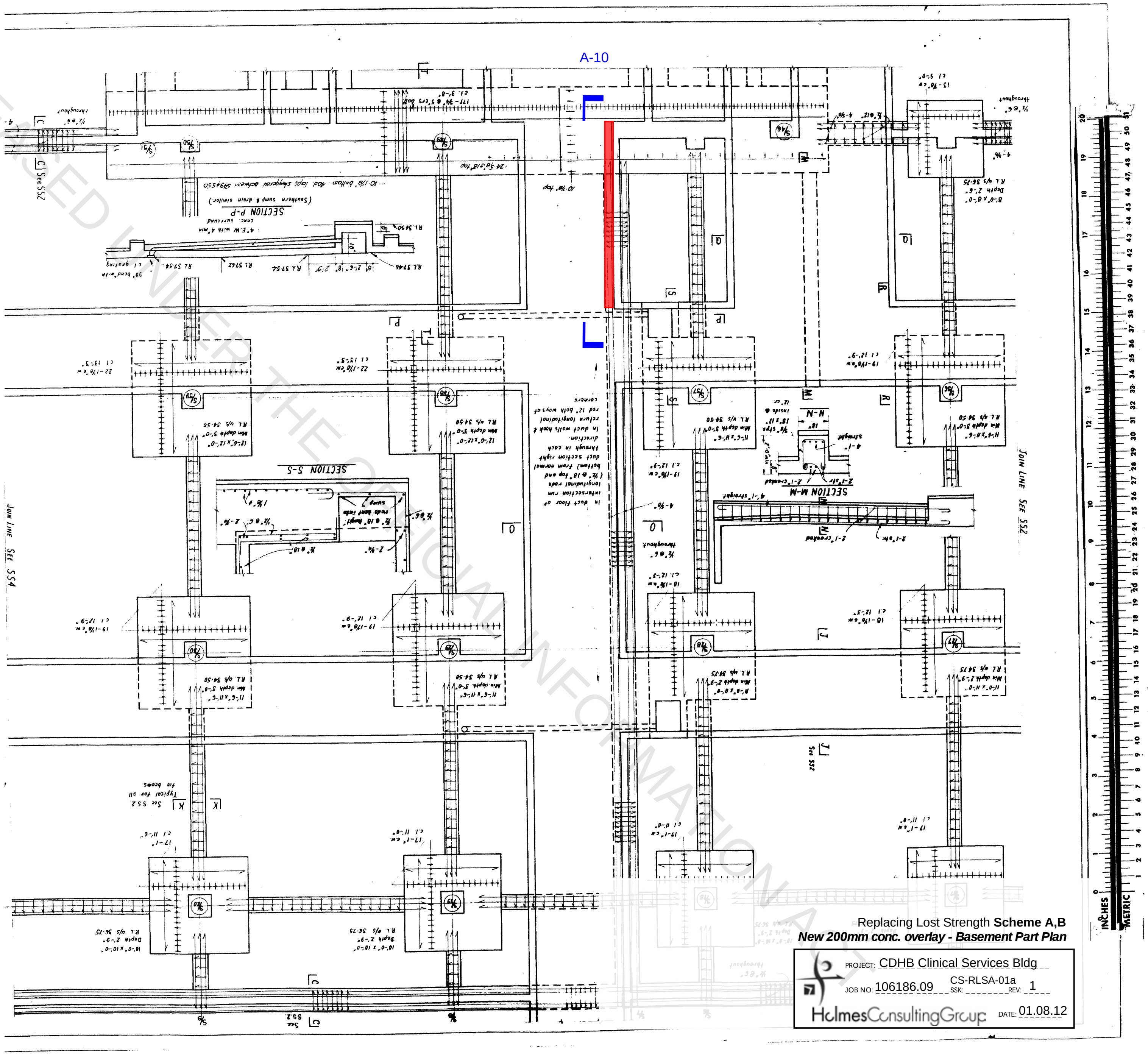
Epoxy inject cracks to extg. wall >0.2mm in width, 200mm new conc. overlay (sprayed or cast in-situ)
REFER CS-RLSA-10



Replacing Lost Strength - **Scheme A, B**
Basement Plan

CHRISTCHURCH HOSPITAL
RIVERSIDE CLINICAL SERVICES - BASEMENT
SPACE NUMBERING

PROJECT: CDHB Clinical Services Bldg
JOB NO: 106186.09 SSK: CS-RLSA-01 REV: 1
HolmesConsultingGroup DATE: 01.08.12



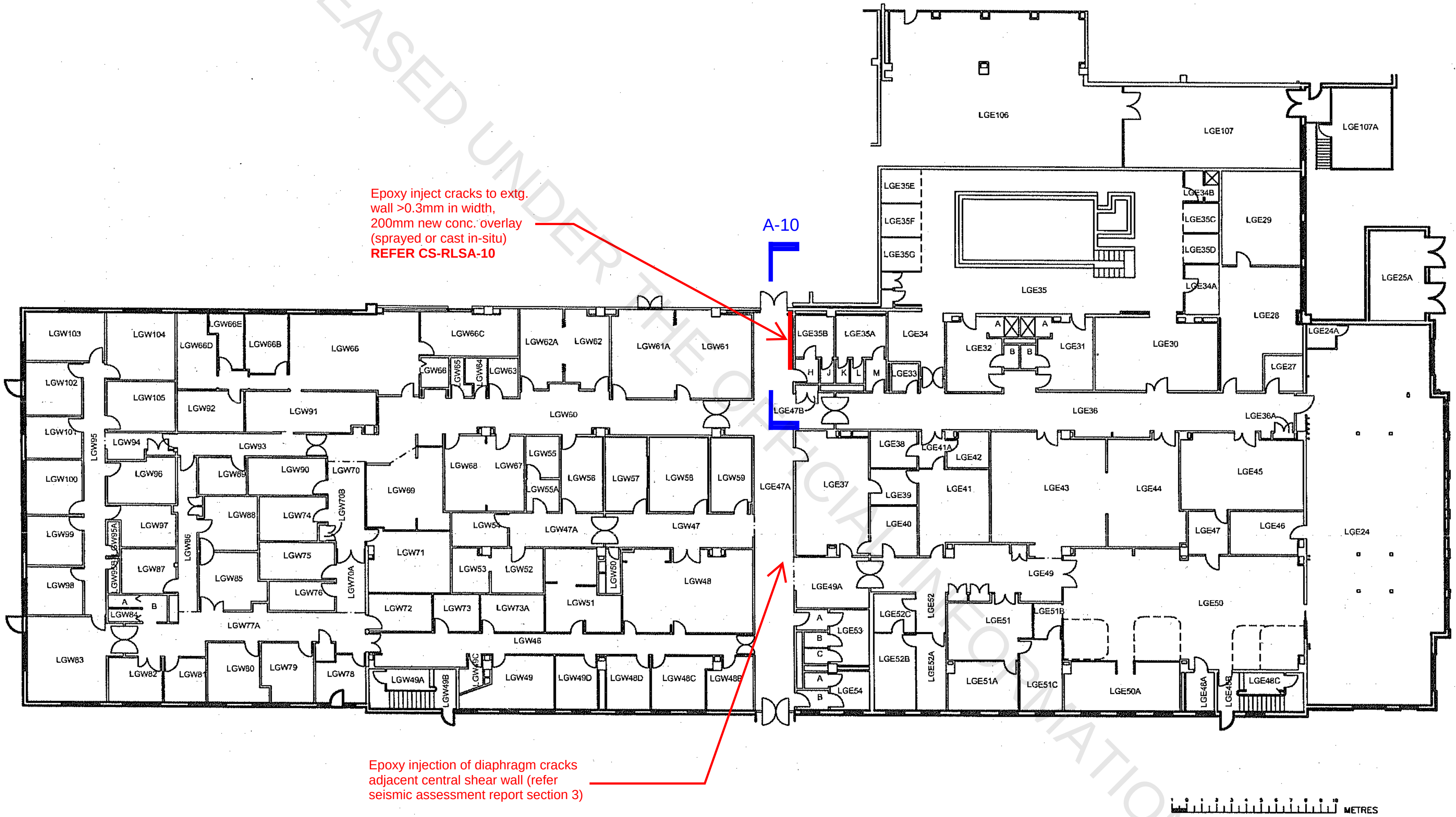
Replacing Lost Strength **Scheme A,B**

New 200mm conc. overlay - Basement Part Plan

PROJECT: CDHB Clinical Services Bldg

JOB NO: 106186.09 SSK: CS-RLSA-01a REV: 1

HolmesConsultingGroup DATE: 01.08.12

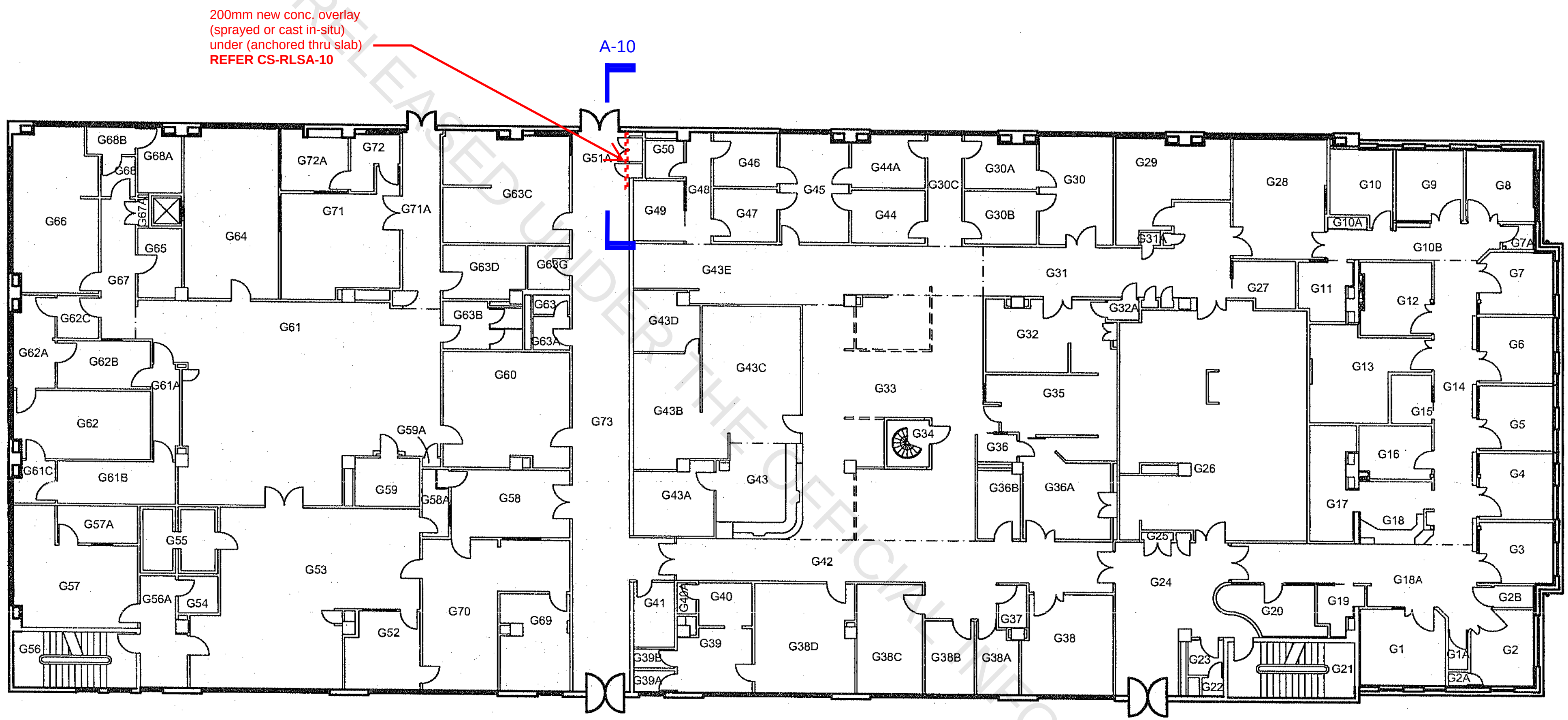


Replacing Lost Strength - **Scheme A,B**

LG Plan

CHRISTCHURCH HOSPITAL
RIVERSIDE CLINICAL SERVICES - LOWER GROUND FLOOR
SPACE NUMBERING

PROJECT: CDHB Clinical Services Bldg
JOB NO: 106186.09 SSK: CS-RLSA-02 REV: 1
HolmesConsultingGroup DATE: 01.08.12



Scheme for Replacing Lost Strength - **Scheme A,B**
Ground Level Plan

CHRISTCHURCH HOSPITAL
RIVERSIDE CLINICAL SERVICES GROUND FLOOR
SPACE NUMBERING

PROJECT: CDHB Clinical Services Bldg
JOB NO: 106186.09 SSK: CS-RLSA-03 REV: 1
HolmesConsultingGroup DATE: 01.08.12

Expose piers to inspect for cracking at base of piers or at floor level.

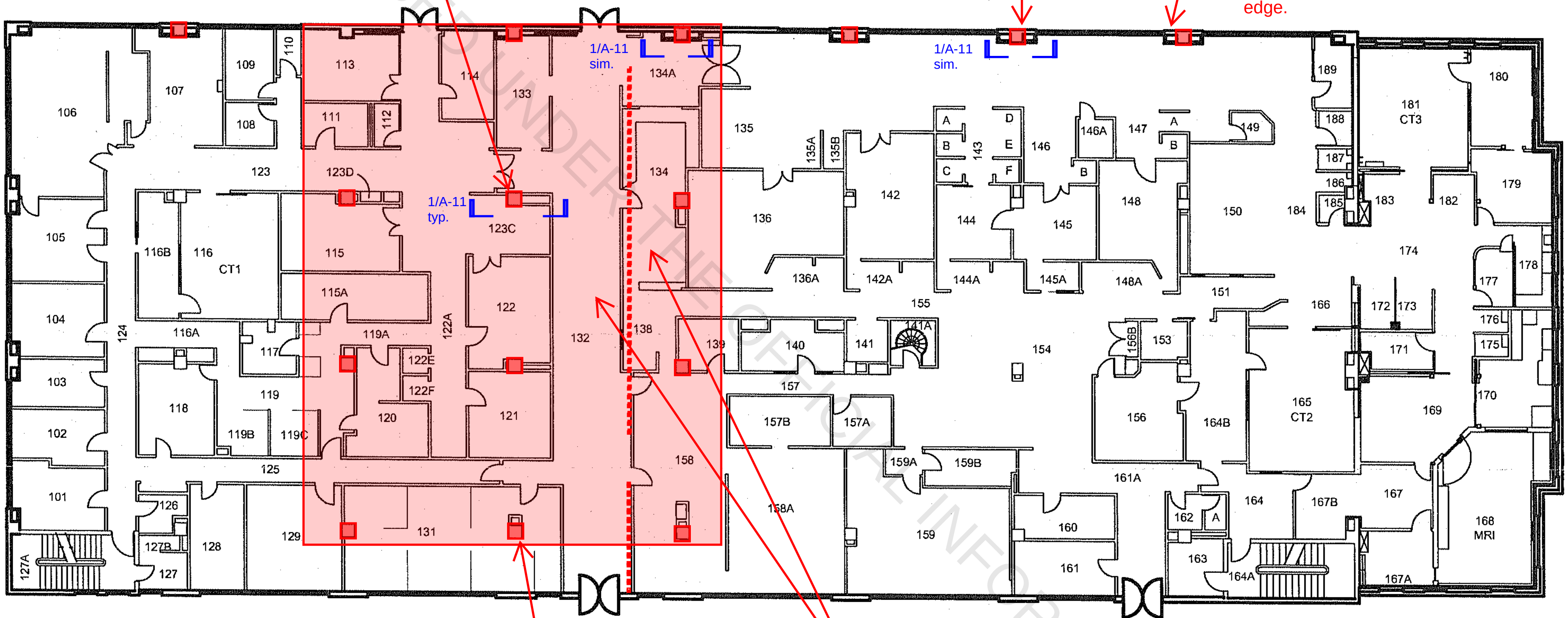
Where cracking present, increase dimension of North wall column pillasters to 800mm sq between L1 and roof beams (up to 4 columns total)

Work to 3 interior faces only

Increase dimension of columns under L3 plant area (outline shaded) to 800mm sq between L1 and L2 (11 columns total)

Note: allow to cut slab to extend services risers where these exist adjacent columns, reconfigure services

Trim cut slab edges with XD16 epoxied into 45mm rebate in slab, extending min. 750mm beyond cut edge.



Central shear wall shown dashed under. Repairs to L1 diaphragm cracking adjacent central shear wall refer seismic assessment report section 3.
REFER CS-RLSA-04a

Note: allow to cut slab to extend services risers where these exist adjacent columns, reconfigure services.

Trim cut slab edges with XD16 epoxied into 45mm rebate in slab, extending min. 750mm beyond cut edge.

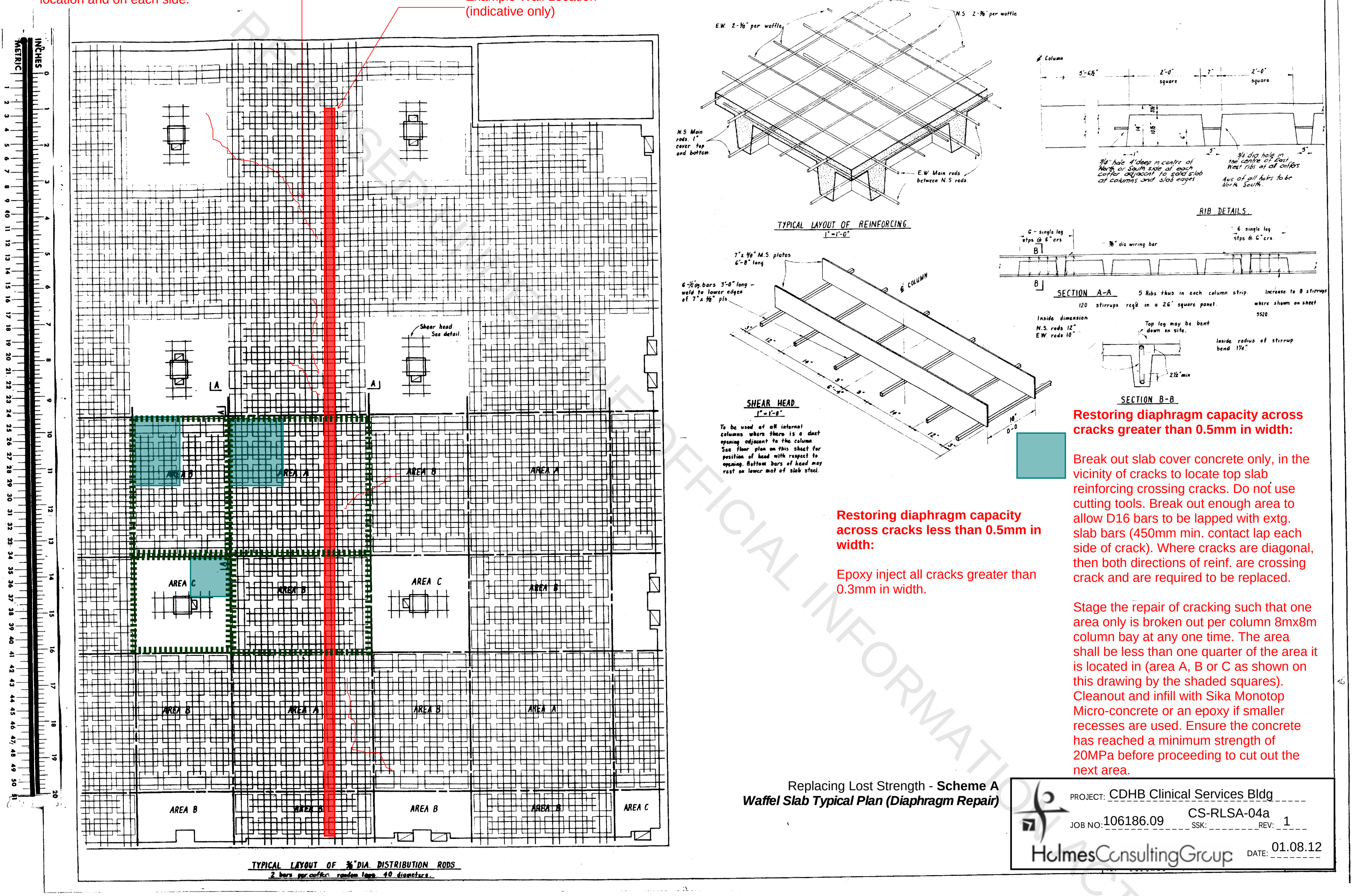
Replacing Lost Strength - **Scheme A**
L1 Plan

CHRISTCHURCH HOSPITAL
RIVERSIDE CLINICAL SERVICES FIRST FLOOR
SPACE NUMBERING

PROJECT: CDHB Clinical Services Bldg
JOB NO: 106186.09 SSK: CS-RLSA-04 REV: 1
HolmesConsultingGroup DATE: 01.08.12

Example diaphragm cracking (actual locations not shown). Actual crack locations to be surveyed on site in the column bay adjacent central shear wall location and on each side.

Example Wall Location (indicative only)



Restoring diaphragm capacity across cracks less than 0.5mm in width:

Epoxy inject all cracks greater than 0.3mm in width.

Restoring diaphragm capacity across cracks greater than 0.5mm in width:

Break out slab cover concrete only, in the vicinity of cracks to locate top slab reinforcing crossing cracks. Do not use cutting tools. Break out enough area to allow D16 bars to be lapped with extg. slab bars (450mm min. contact lap each side of crack). Where cracks are diagonal, then both directions of reinf. are crossing crack and are required to be replaced.

Stage the repair of cracking such that one area only is broken out per column 8mx8m column bay at any one time. The area shall be less than one quarter of the area it is located in (area A, B or C as shown on this drawing by the shaded squares). Cleanout and infill with Sika Monotop Micro-concrete or an epoxy if smaller recesses are used. Ensure the concrete has reached a minimum strength of 20MPa before proceeding to cut out the next area.

Replacing Lost Strength - **Scheme A**

Waffel Slab Typical Plan (Diaphragm Repair)

PROJECT: CDHB Clinical Services Bldg

JOB NO: 106186.09 SSK: CS-RLSA-04a REV: 1

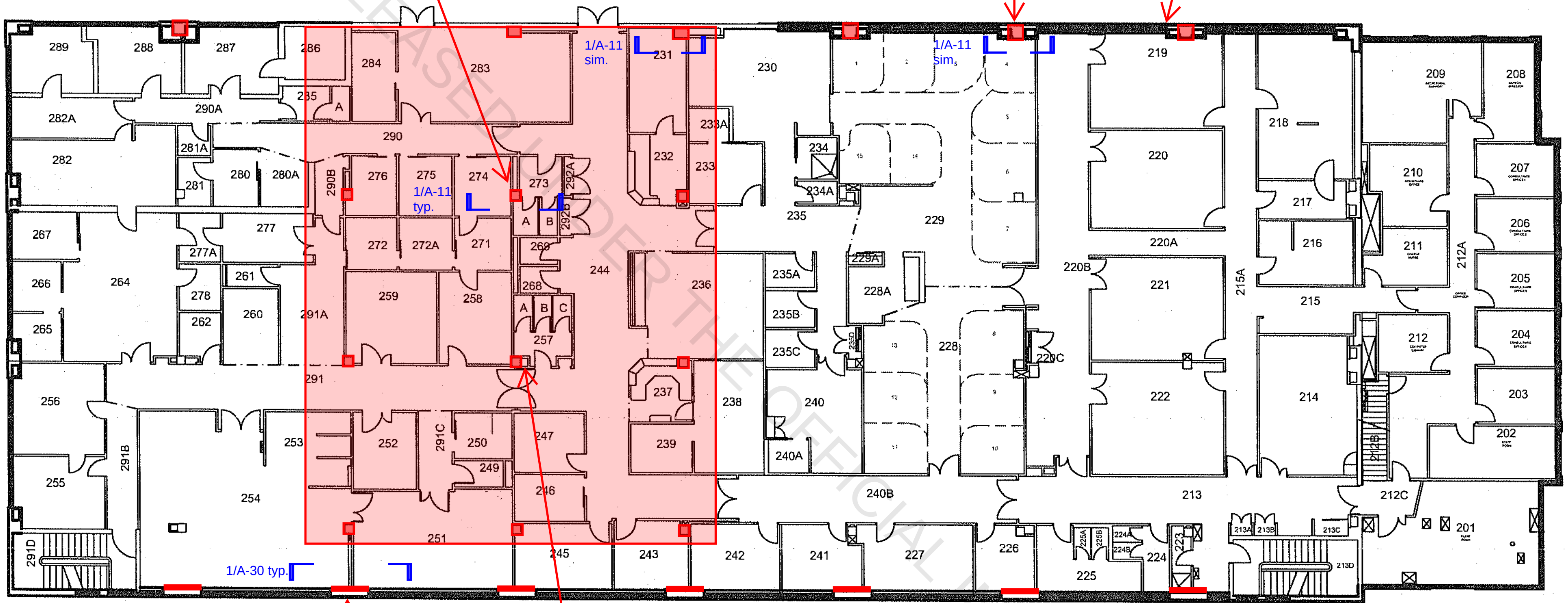
HolmesConsultingGroup DATE: 01.08.12

Increase dimension of columns under L3 plant area (outline shaded) to 600mm sq between L2 and L3 plant (11 columns total)

Expose piers to inspect for cracking at base of piers or at floor level.

Where cracking present, increase dimension of North wall column pillasters to 800mm sq between L1 and roof beams (up to 4 columns total)

Note: allow to cut slab to extend services risers where these exist adjacent columns, reconfigure services



Note: allow to cut slab to extend services risers where these exist adjacent columns, reconfigure services

New 200mm cast in-situ or sprayed conc. overlay to piers between L2 and roof beams (to interior)

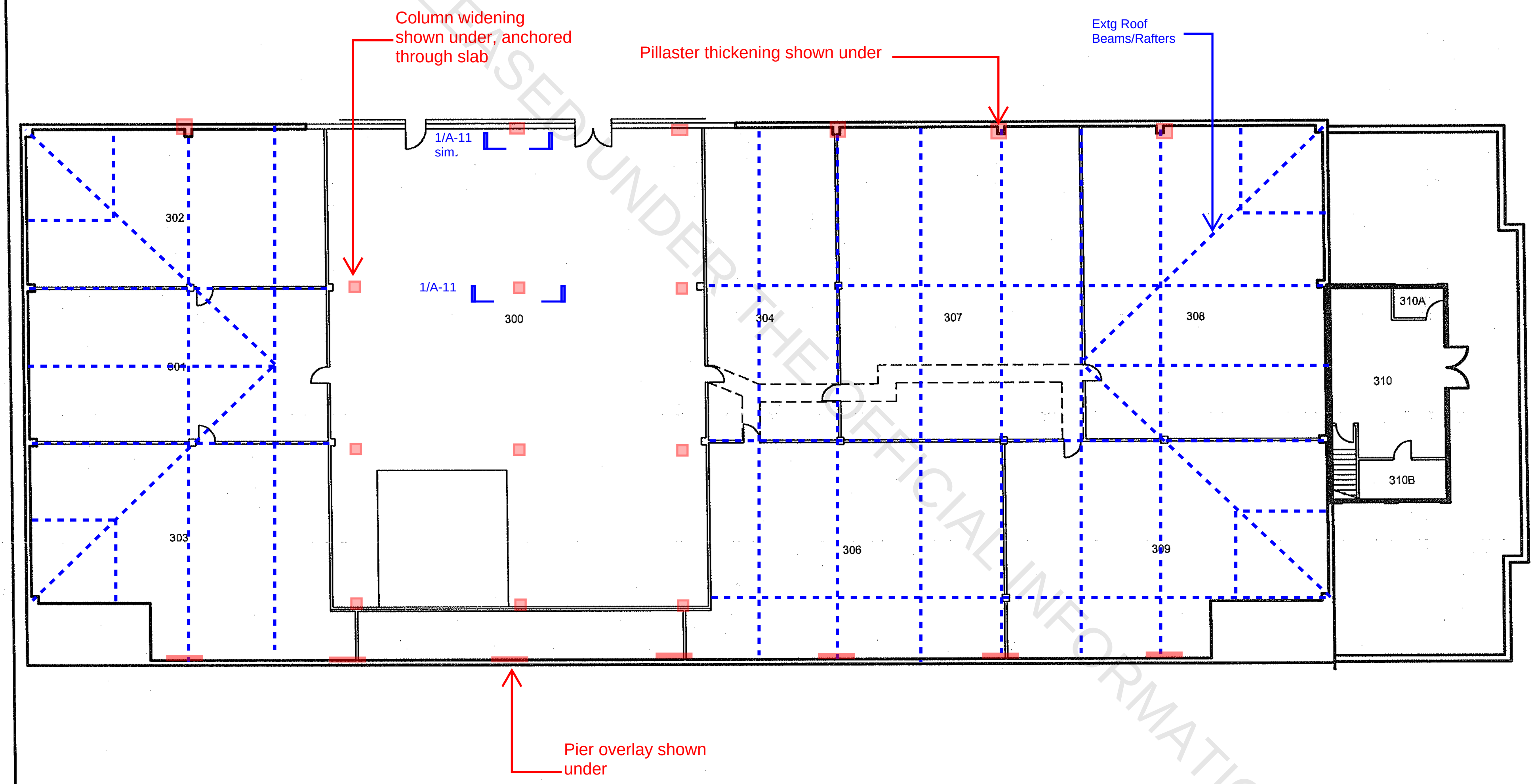
Replacing Lost Strength - **Scheme A**

L2 Plan

CHRISTCHURCH HOSPITAL
RIVERSIDE CLINICAL SERVICES SECOND FLOOR
SPACE NUMBERING

Cant
Distri
Main

PROJECT: CDHB Clinical Services Bldg
JOB NO: 106186.09 SSK: CS-RLSA-05 REV: 1
HolmesConsultingGroup DATE: 01.08.12



Replacing Lost Strength - **Scheme A**

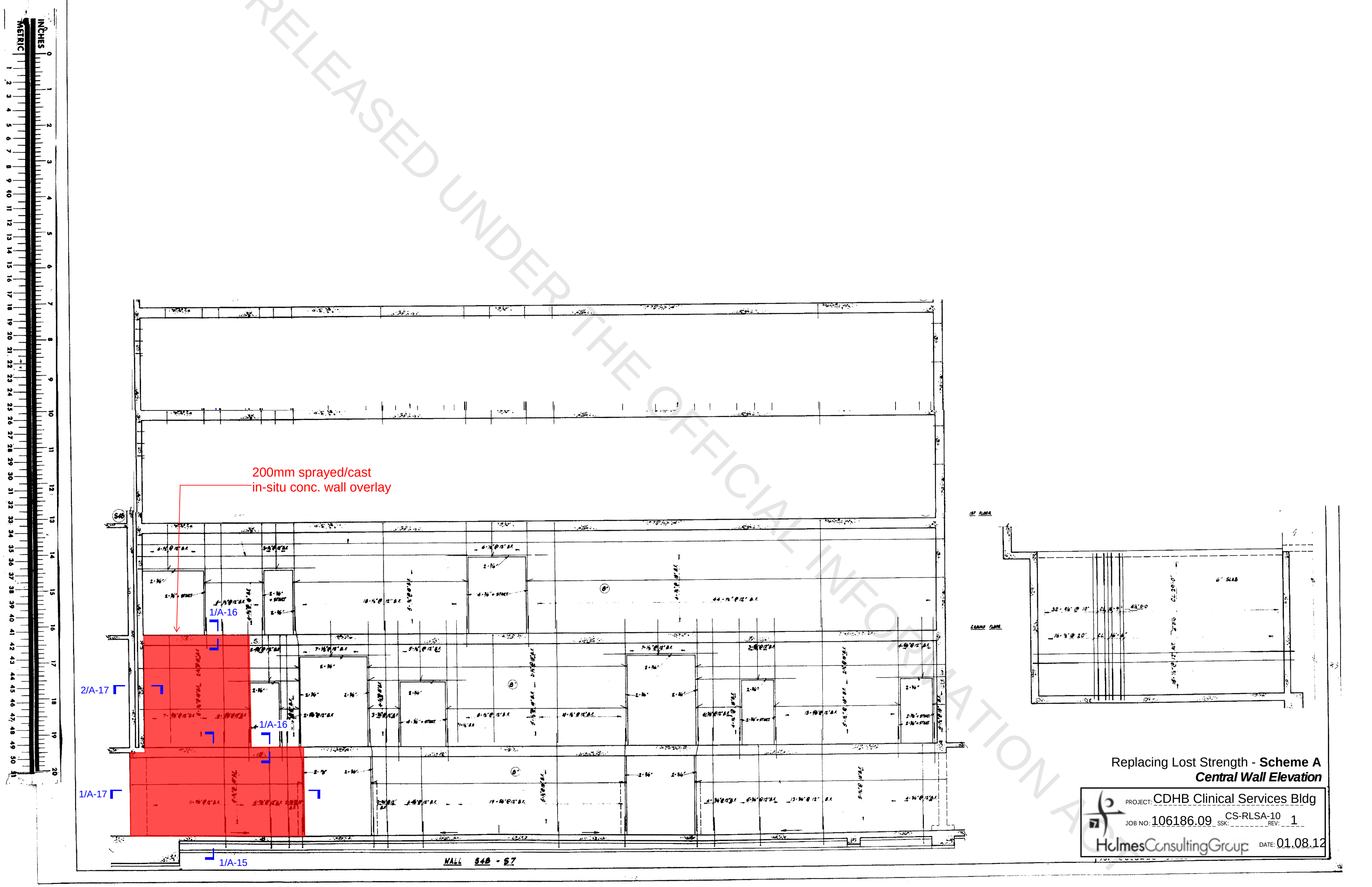
L3 Plant/Main Roof

PROJECT: CDHB Clinical Services Bldg

JOB NO: 106186.09 SSK: CS-RLSA-06 REV: 1

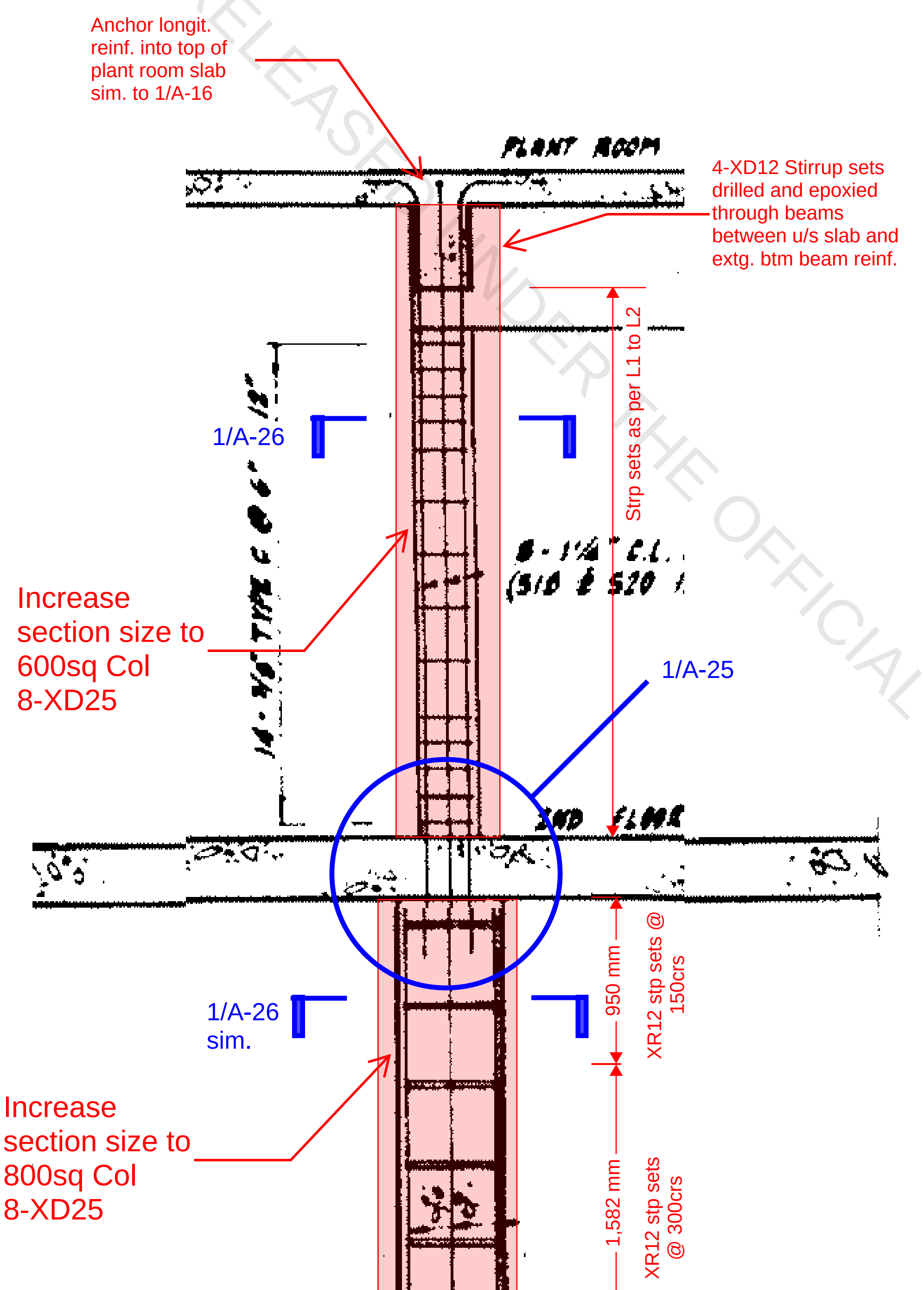
HolmesConsultingGroup DATE: 01.08.12

CHRISTCHURCH HOSPITAL
RIVERSIDE CLINICAL SERVICES THIRD FLOOR
SPACE NUMBERING



Replacing Lost Strength - **Scheme A**
Central Wall Elevation

PROJECT: CDHB Clinical Services Bldg
JOB NO: 106186.09 SSK: CS-RLSA-10 REV: 1
HolmesConsultingGroup DATE: 01.08.12


Anchor longit. reinf. into top of plant room slab sim. to 1/A-16
PLANT ROOM
4-XD12 Stirrup sets drilled and epoxied through beams between u/s slab and extg. btm beam reinf.
Strp sets as per L1 to L2
1/A-26
Increase section size to 600sq Col 8-XD25
1/A-25
2ND FLOOR
950 mm
XR12 stp sets @ 150crs
1/A-26 sim.
Increase section size to 800sq Col 8-XD25
1,582 mm
XR12 stp sets @ 300crs

HolmesConsultingGroup

Project Name: CDHB CSB

Project No: 106186.09

Calcs By: AT

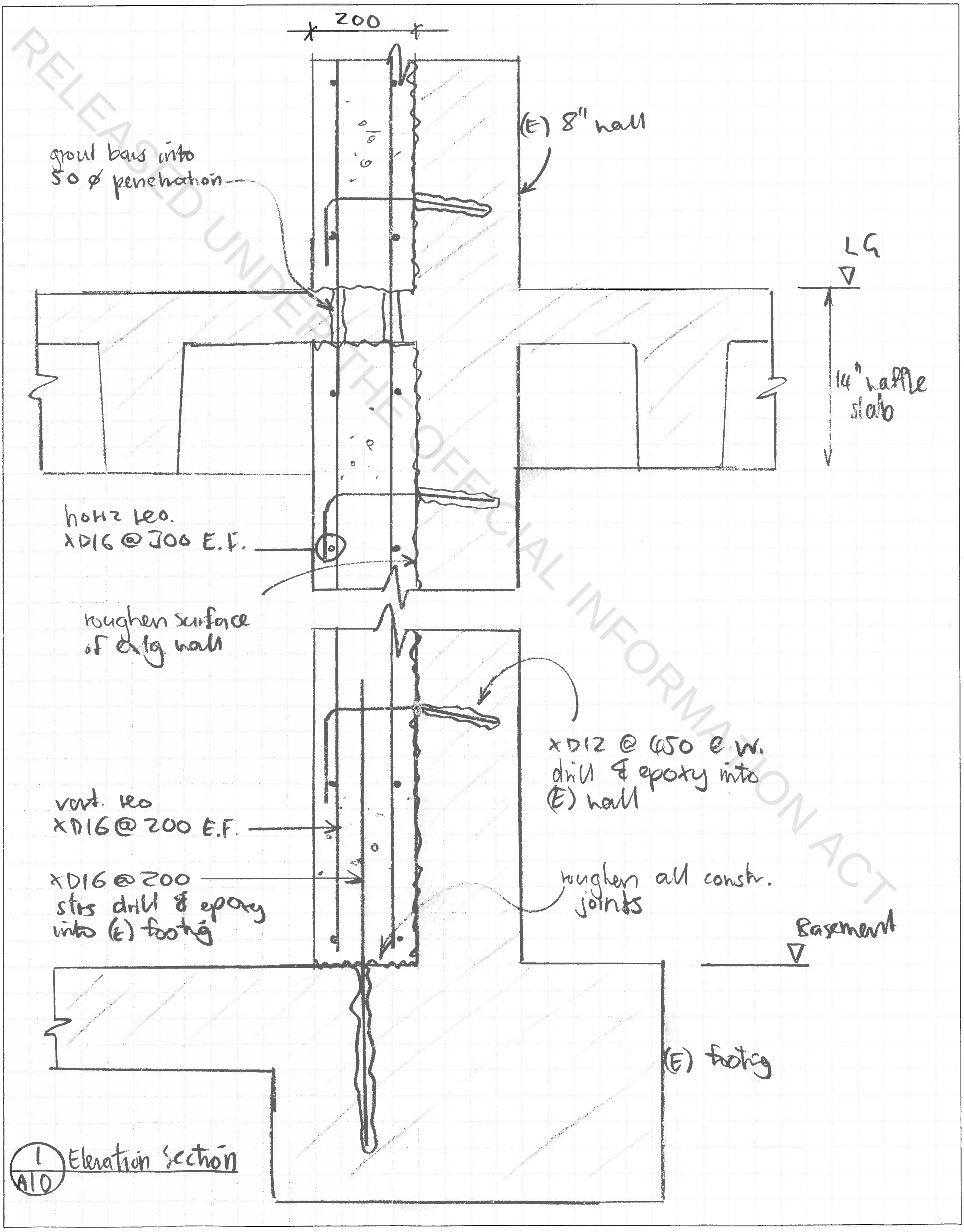
Date: 1/8/12

Page No:

Sketch No: CS-RLSA-15

Revision: 1

CALCS/SKETCHES



HolmesConsultingGroup

Project Name: CDHB CSB

Project No: 106189.09

Calcs By: AT

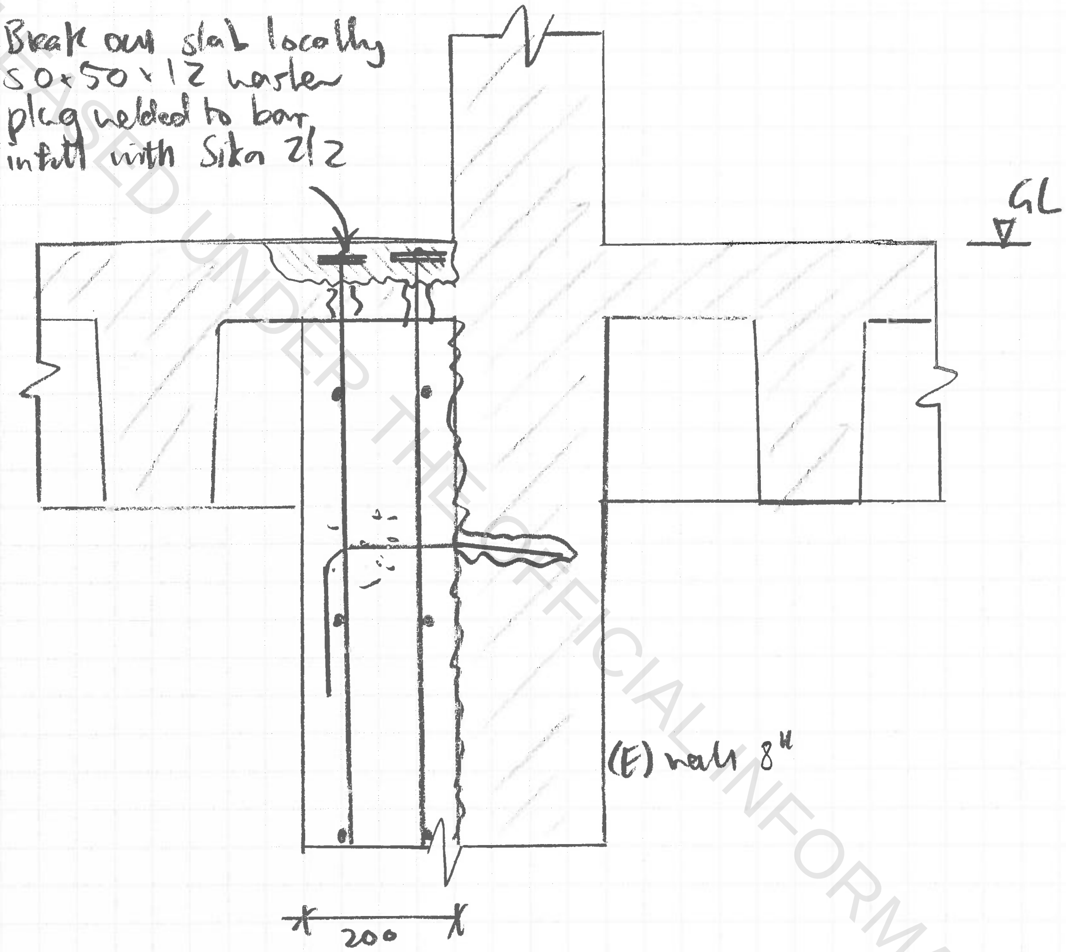
Date: 1/8/12

Sketch No: CS-RLSA-16

Page No:

Revision: 1

CALCS/SKETCHES



1/A10 Elevation Section

HolmesConsultingGroup

Project Name: CDHB CSB

Project No: 106186.09

Calcs By: AT

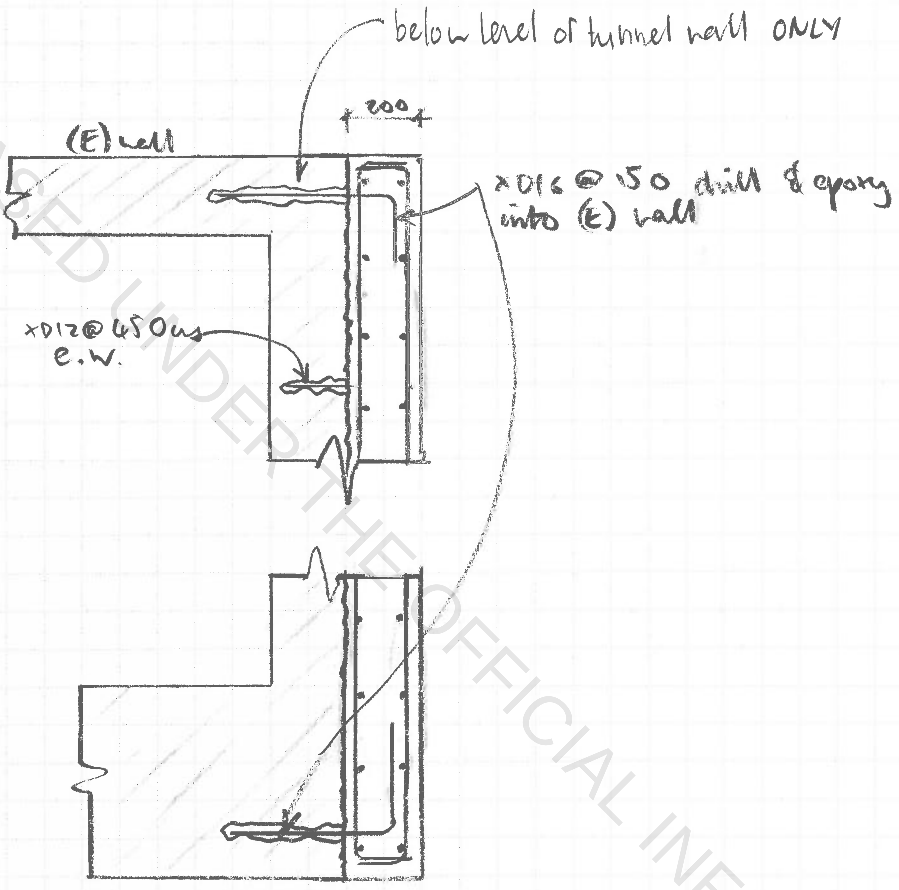
Date: 1/8/12

Sketch No: CS-RLSA-17

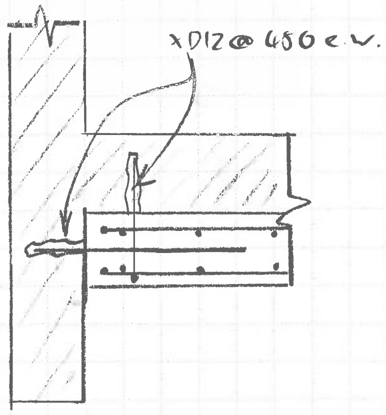
Page No:

Revision: 1

CALCS/SKETCHES



1/A10 Plan Section



2/A10 Plan Section

HolmesConsultingGroup

CALCS/SKETCHES

Project Name: CDHB CSB

Project No: 106186.09

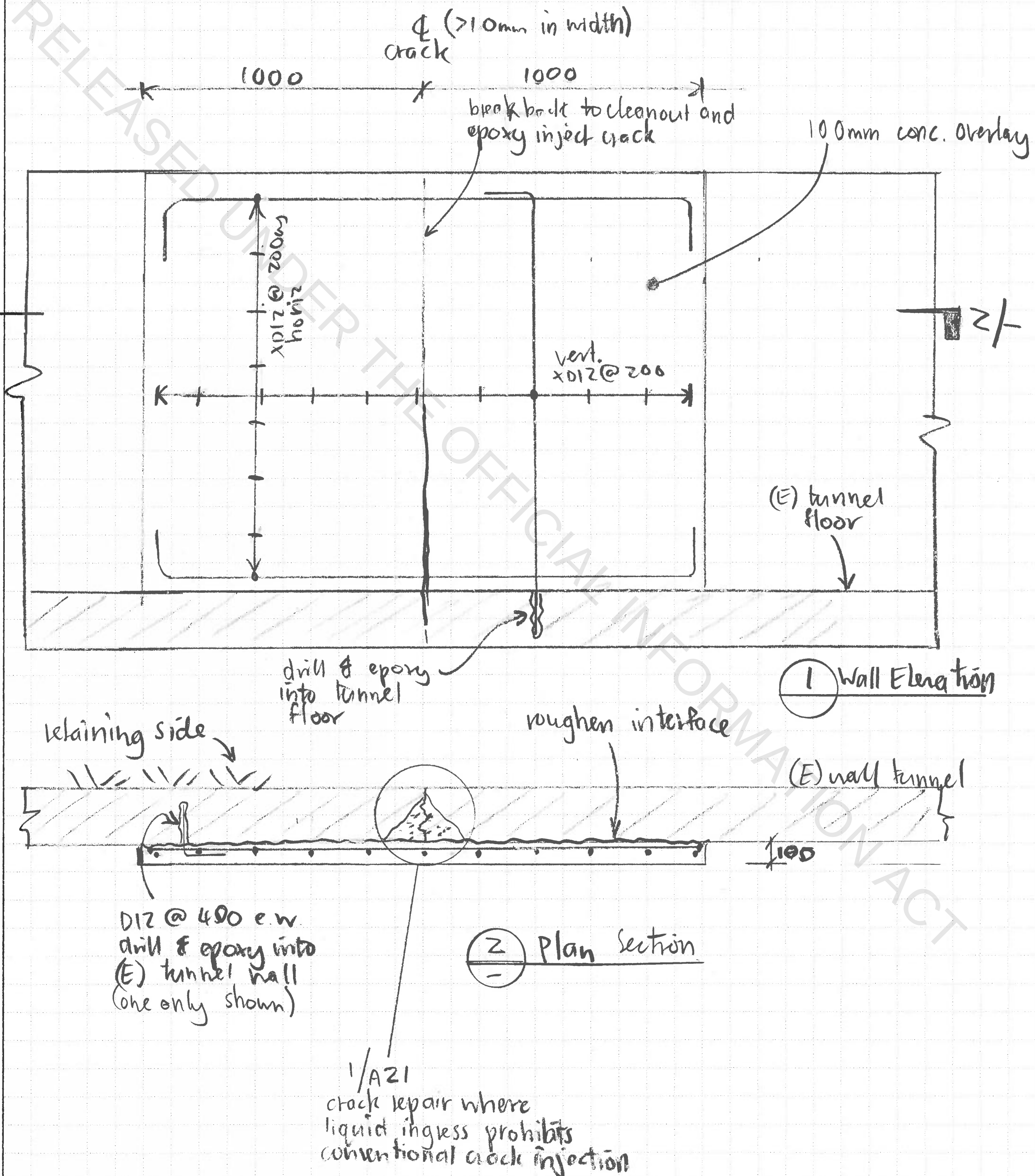
Calcs By: AT

Date: 1/8/12

Page No:

Sketch No: CS-RLSA-20

Revision: 1



HolmesConsultingGroup

Project Name: CPHB CSB

Project No: 106186.09

Calcs By: AT

Date: 1/8/12

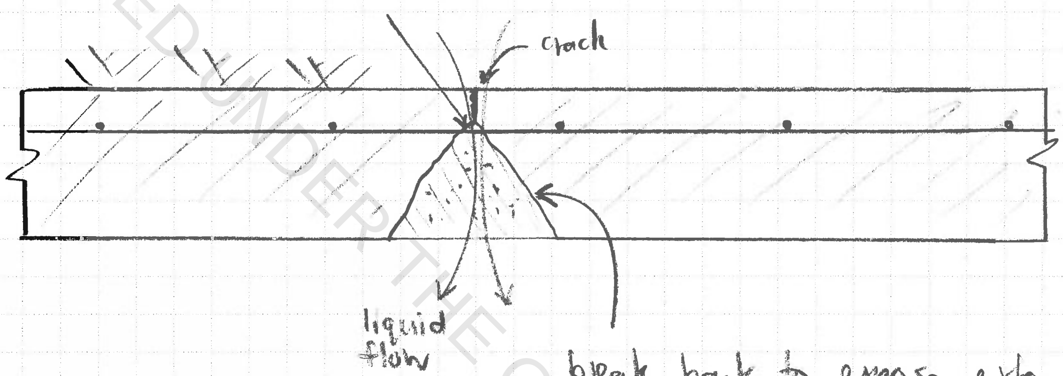
Page No:

Sketch No: CS-RLSA-21

Revision: 1

CALCS/SKETCHES

Seal crack and coat exposed reinf.
with Sika Monotop primer.
Fill back with Monotop flowable micro-concrete



break back to expose extg. reinf. (to just behind reinf.)
Break back only half of the wall height at one time. Fully repair before beginning work on other half of height.

1 Crack repair for cracks > 0.2mm where spoil / liquid flow thru crack prohibits use of conventional crack injection techniques

HolmesConsultingGroup

Project Name: CDHB CSB

Project No: 106186.09

Calcs By: AT

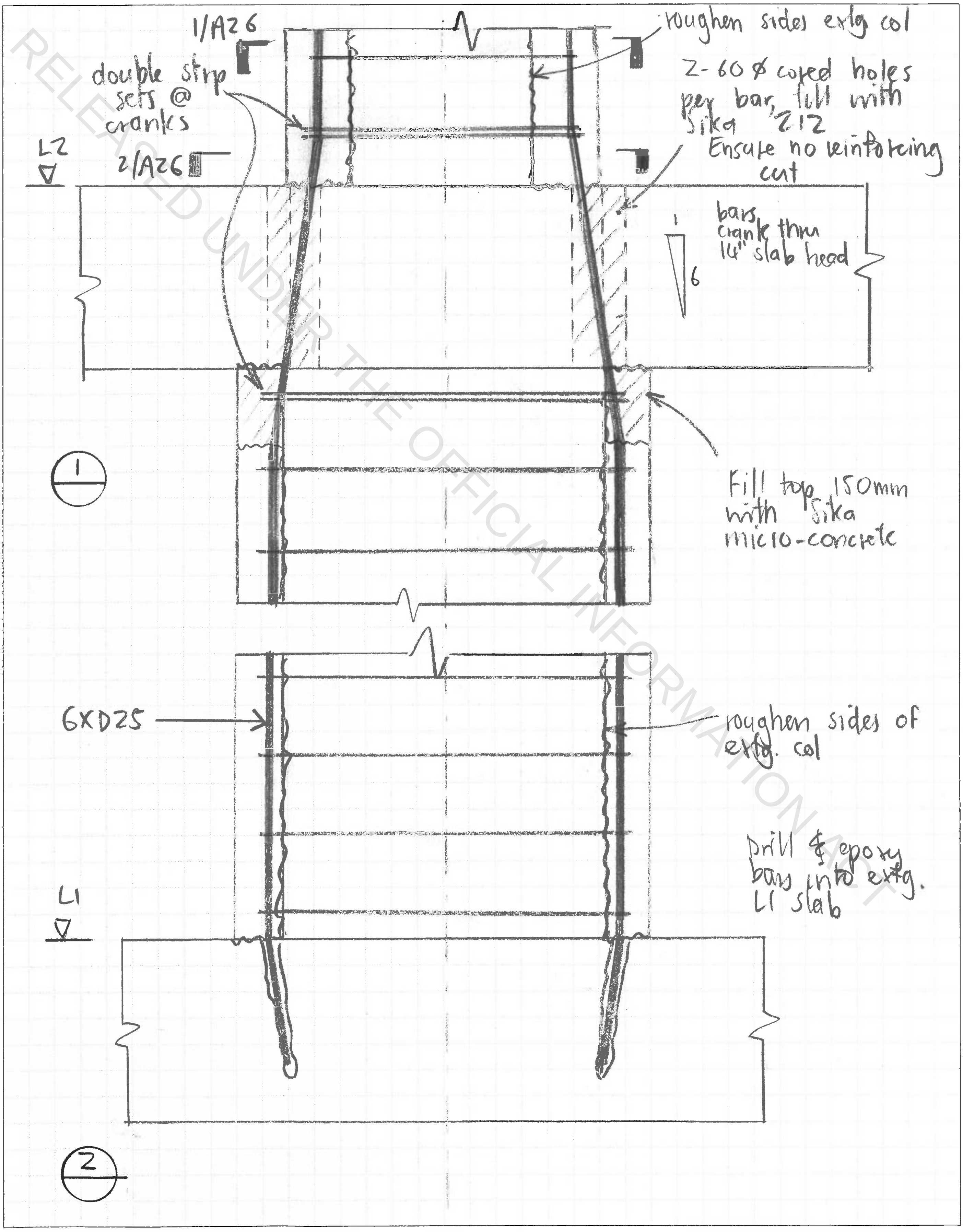
Date: 1/8/12

Page No:

Sketch No: CS-RLSA-25

Revision: 1

CALCS/SKETCHES



HolmesConsultingGroup

Project Name: CDHB CSB

Project No: 106186.09

Calcs By: AT

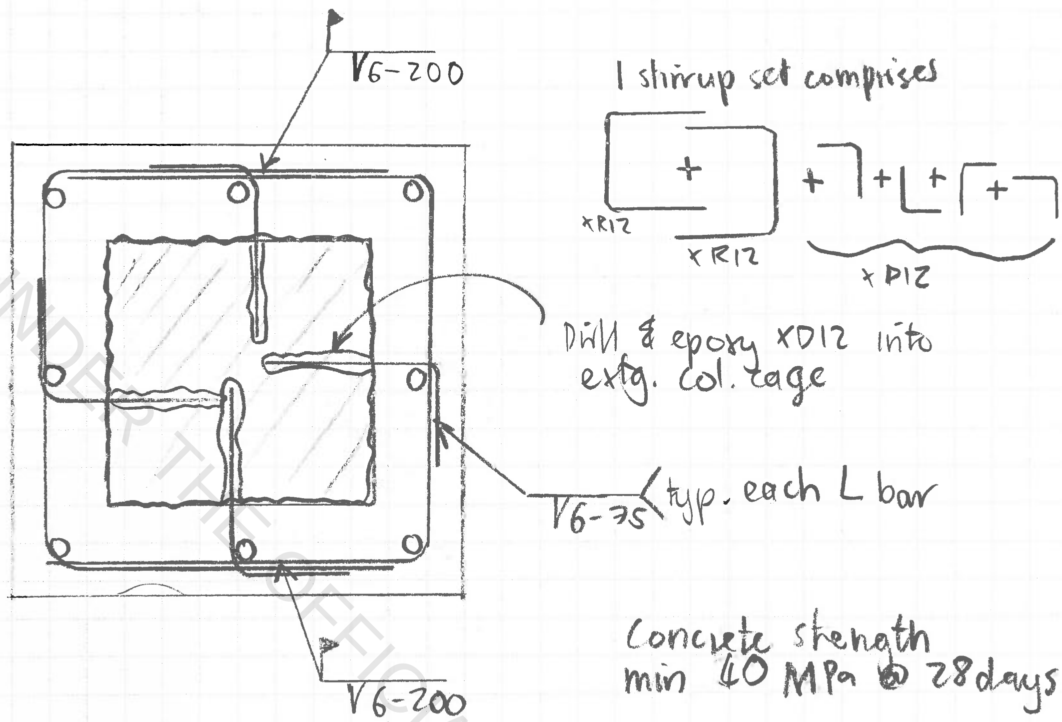
Date: 1/8/12

Page No:

Sketch No: CS-RLSA-26

Revision: 1

CALCS/SKETCHES



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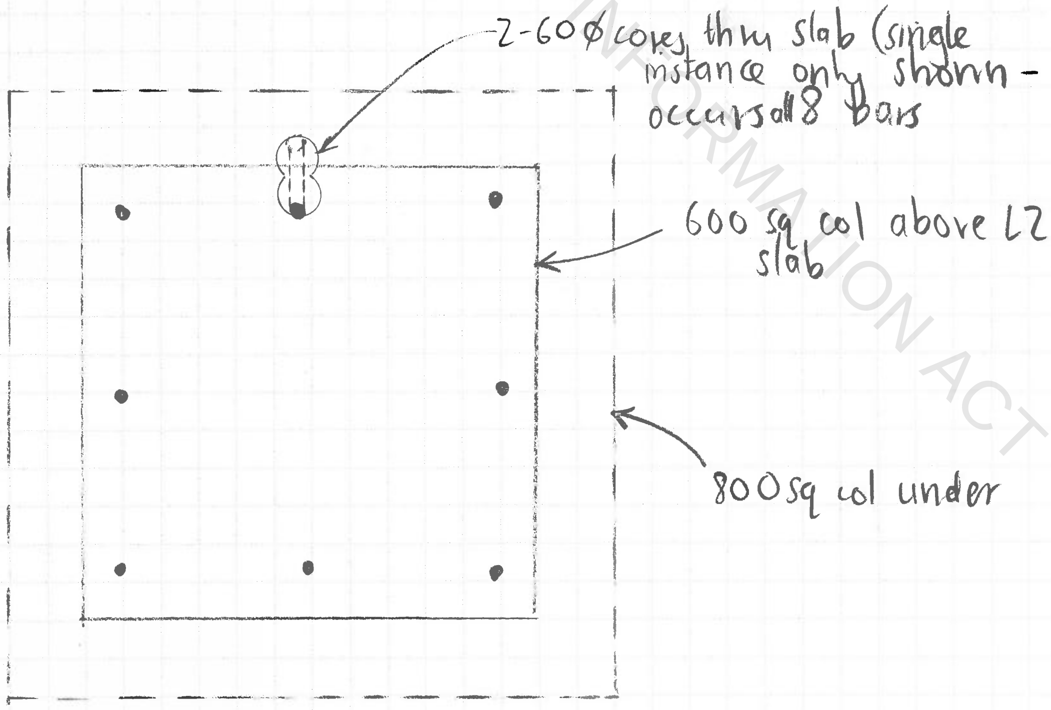

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HolmesConsultingGroup

CALCS/SKETCHES

Project Name: CDHB CSB

Project No: 106186.09

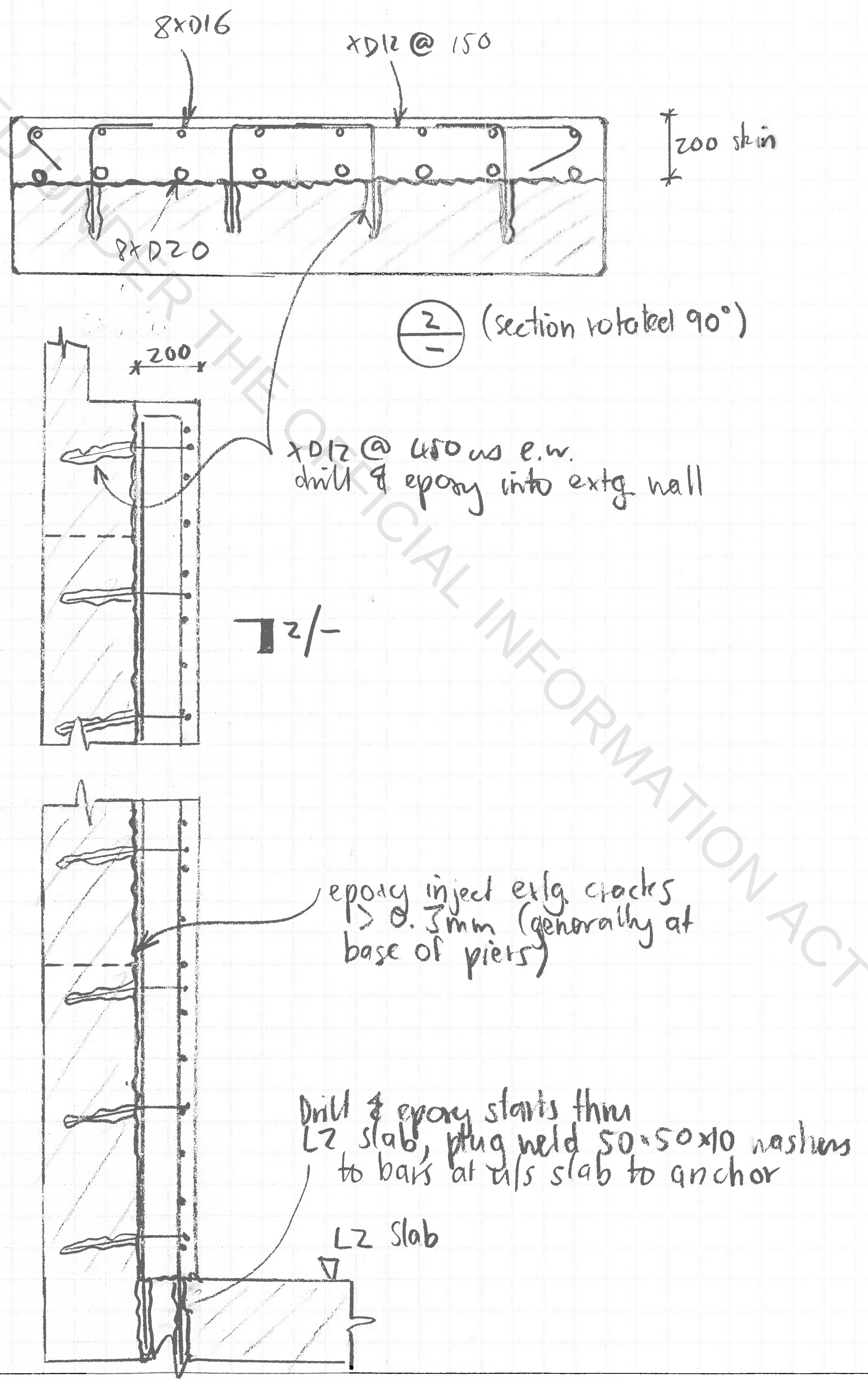
Calcs By: AT

Date: 1/8/12

Page No:

Sketch No: CS-RLSA-30

Revision: 1



APPENDIX C

Replacing Lost Strength – Scheme B
Preliminary Sketches for Pricing
CS-RLSB-01 TO CS-RLSB-06
CS-RLSB-10
CS-RLSB-15
CS-RLSB-20 TO CS-RLSB-21
CS-RLSB-25

Note: Only elements requiring major repair or additional structural elements are included in this sketch set. This sketch set is to be read in conjunction with Section 3, and the Repair and Further Investigation sketches at the end of Appendix C for areas of the building which require minor repair and epoxy crack injection.

Epoxy inject cracks to extg. wall >0.2mm in width, 100mm new conc. overlay (sprayed or cast in-situ) over cracks > 1.0mm in width.

Where liquid ingress prohibits use of conventional injection techniques, breakout and repair in stages
REFER CS-RLSA-21

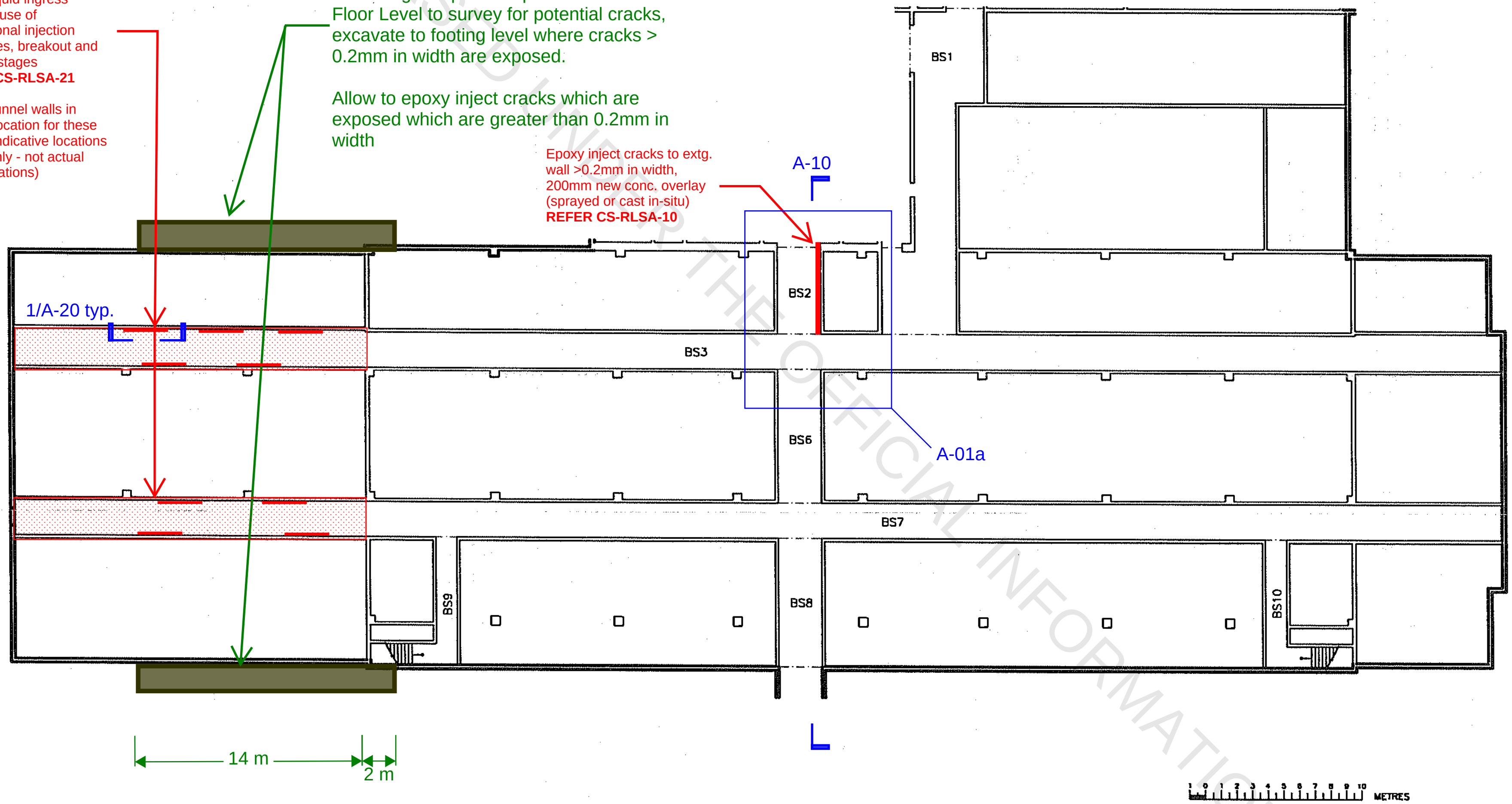
Survey tunnel walls in shaded location for these cracks (indicative locations shown only - not actual crack locations)

Restoring capacity of foundations likely to have experienced settlement damage:

Excavate to expose perimeter basement wall in this region up to a depth of 2.0m below LG Floor Level to survey for potential cracks, excavate to footing level where cracks > 0.2mm in width are exposed.

Allow to epoxy inject cracks which are exposed which are greater than 0.2mm in width

Epoxy inject cracks to extg. wall >0.2mm in width, 200mm new conc. overlay (sprayed or cast in-situ)
REFER CS-RLSA-10

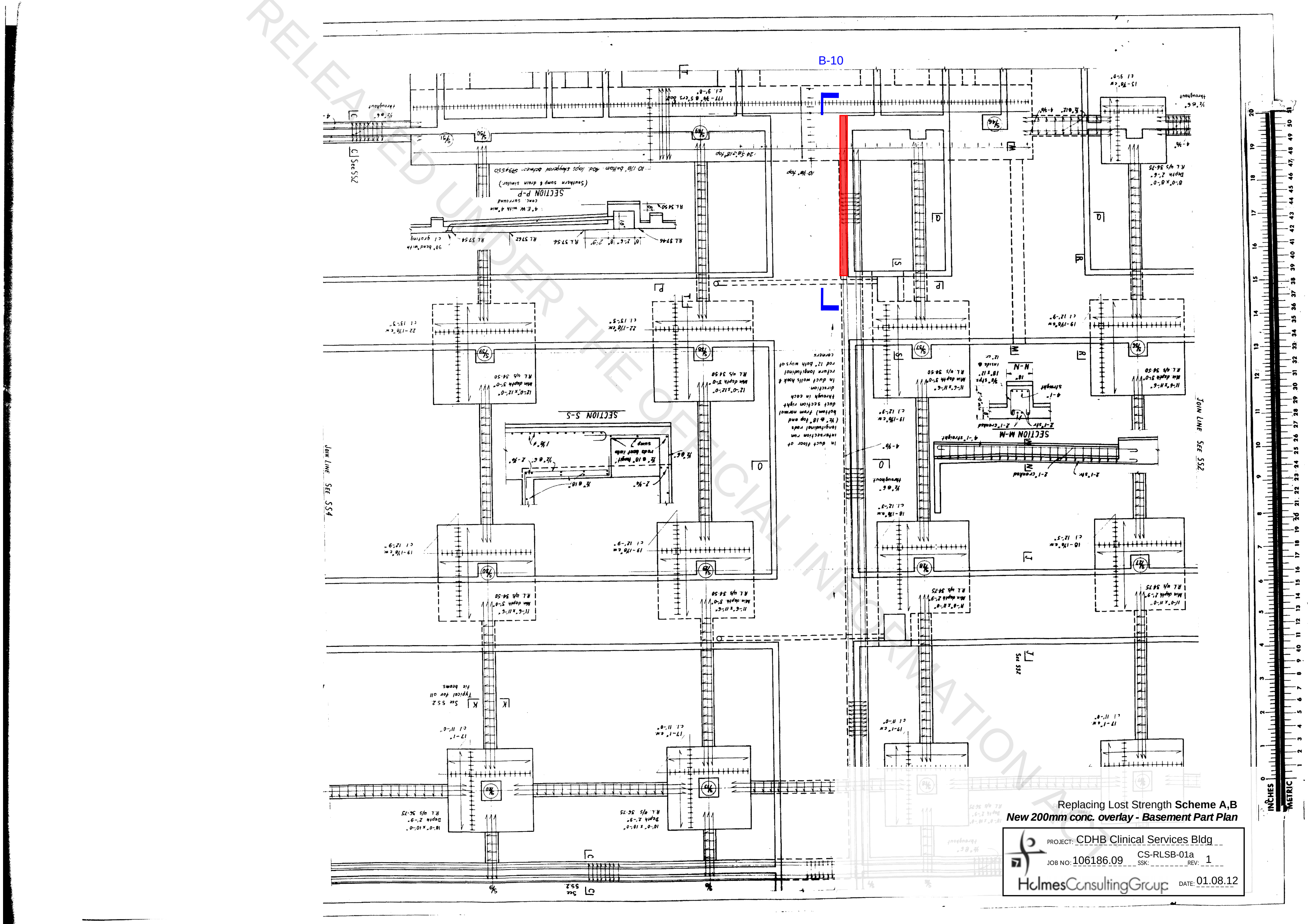


1 0 1 2 3 4 5 6 7 8 9 10 METRES
GRAPHIC SCALE

Replacing Lost Strength - **Scheme A, B**
Basement Plan

CHRISTCHURCH HOSPITAL
RIVERSIDE CLINICAL SERVICES - BASEMENT
SPACE NUMBERING

PROJECT: CDHB Clinical Services Bldg
JOB NO: 106186.09 SSK: CS-RLSB-01 REV: 1
HolmesConsultingGroup DATE: 01.08.12



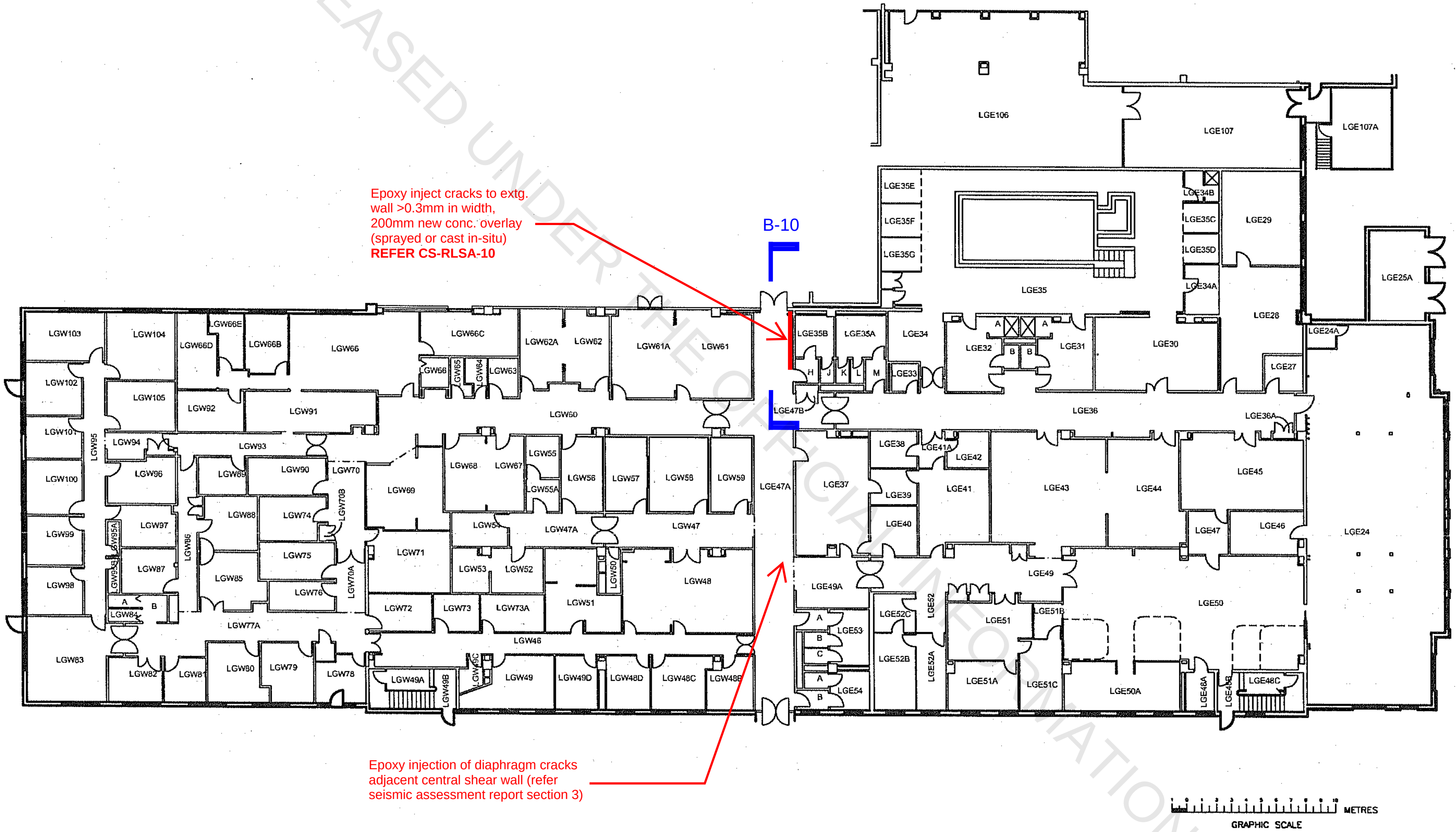
Replacing Lost Strength **Scheme A,B**

New 200mm conc. overlay - Basement Part Plan

PROJECT: CDHB Clinical Services Bldg

JOB NO: 106186.09 SSK: CS-RLSB-01a REV: 1

HolmesConsultingGroup DATE: 01.08.12

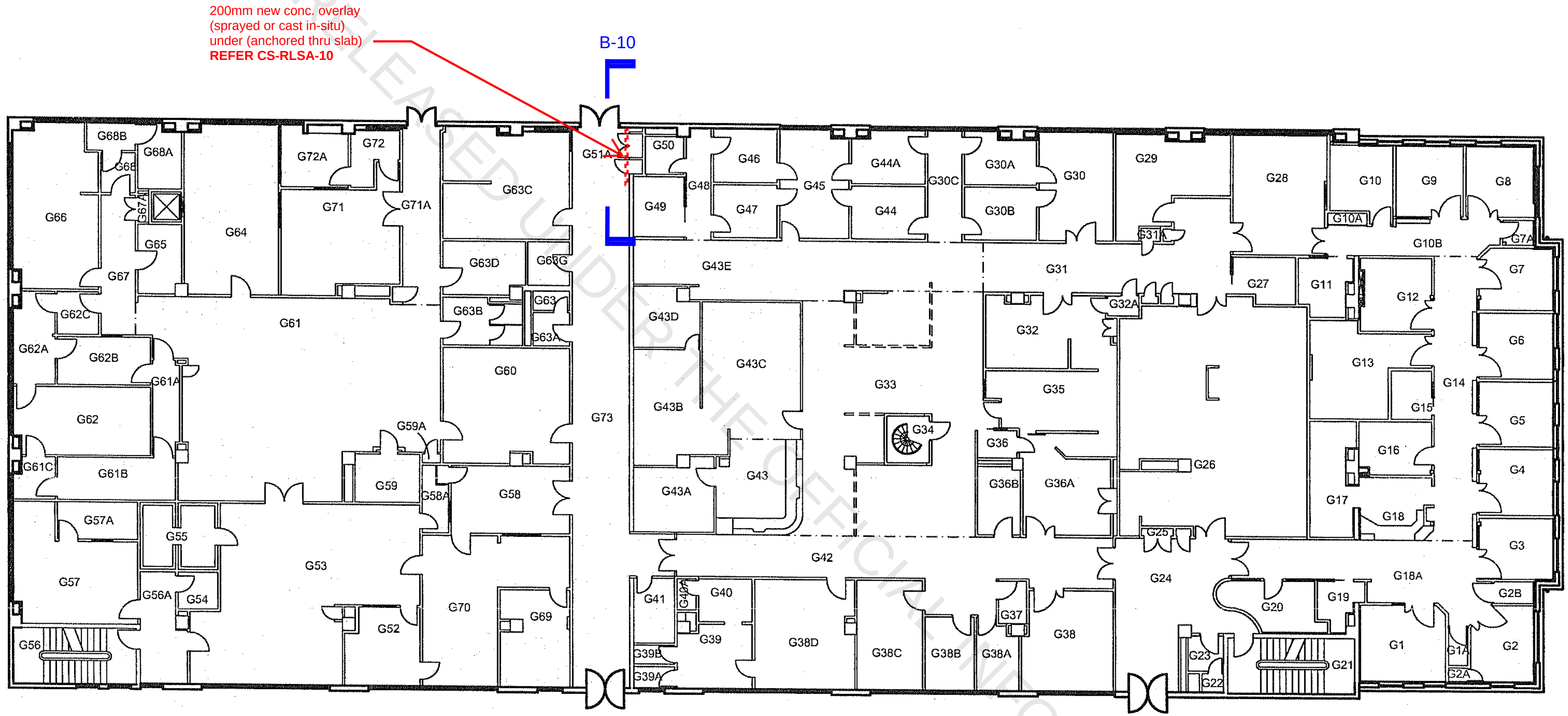


Replacing Lost Strength - **Scheme A,B**

LG Plan

CHRISTCHURCH HOSPITAL
RIVERSIDE CLINICAL SERVICES - LOWER GROUND FLOOR
SPACE NUMBERING

PROJECT: CDHB Clinical Services Bldg
JOB NO: 106186.09 SSK: CS-RLSB-02 REV: 1
HolmesConsultingGroup DATE: 01.08.12

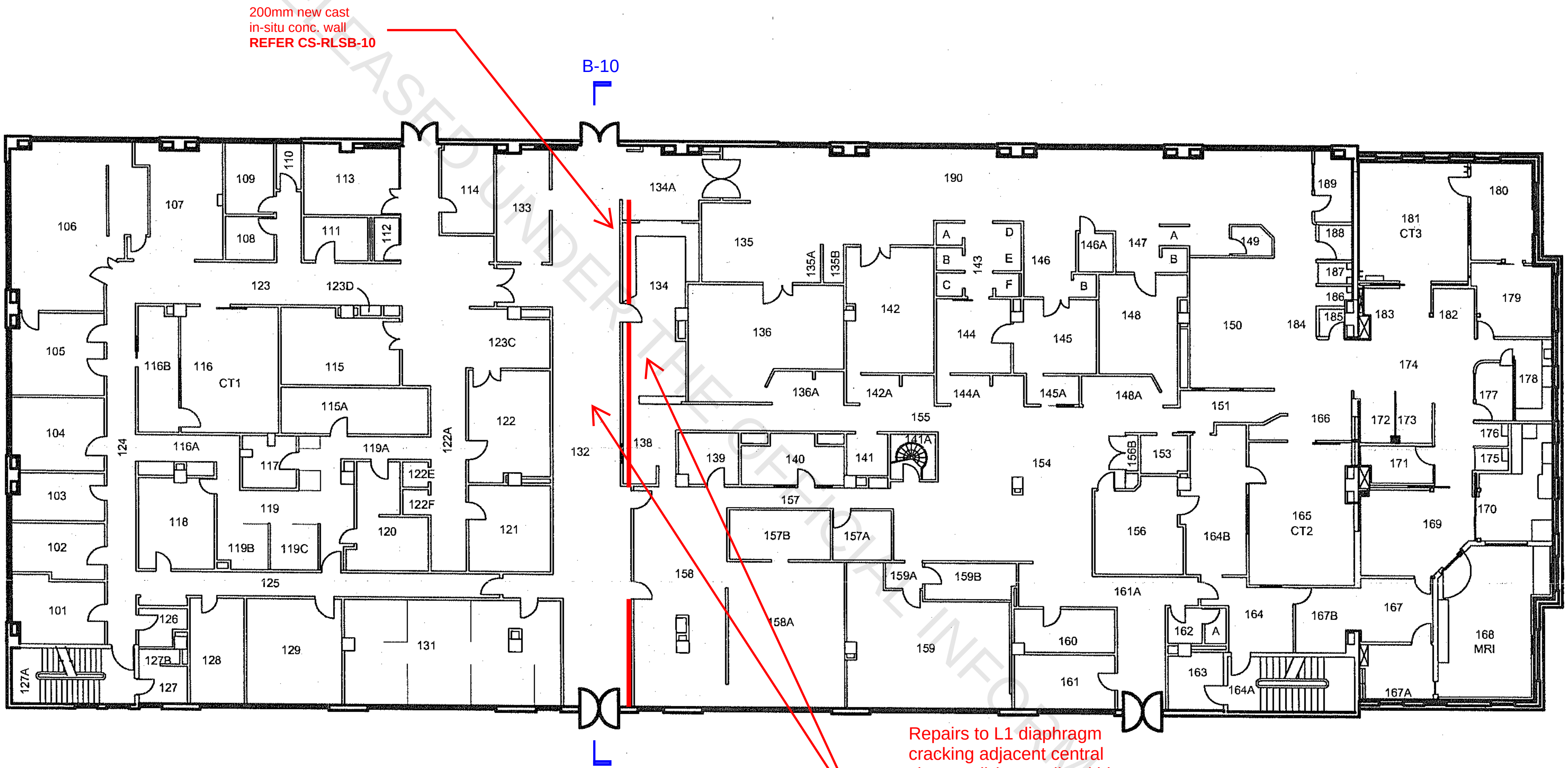


Scheme for Replacing Lost Strength - **Scheme A,B**
Ground Level Plan

CHRISTCHURCH HOSPITAL
RIVERSIDE CLINICAL SERVICES GROUND FLOOR
SPACE NUMBERING

Cante
Distric
Maint

PROJECT: CDHB Clinical Services Bldg
JOB NO: 106186.09 SSK: CS-RLSB-03 REV: 1
HolmesConsultingGroup DATE: 01.08.12

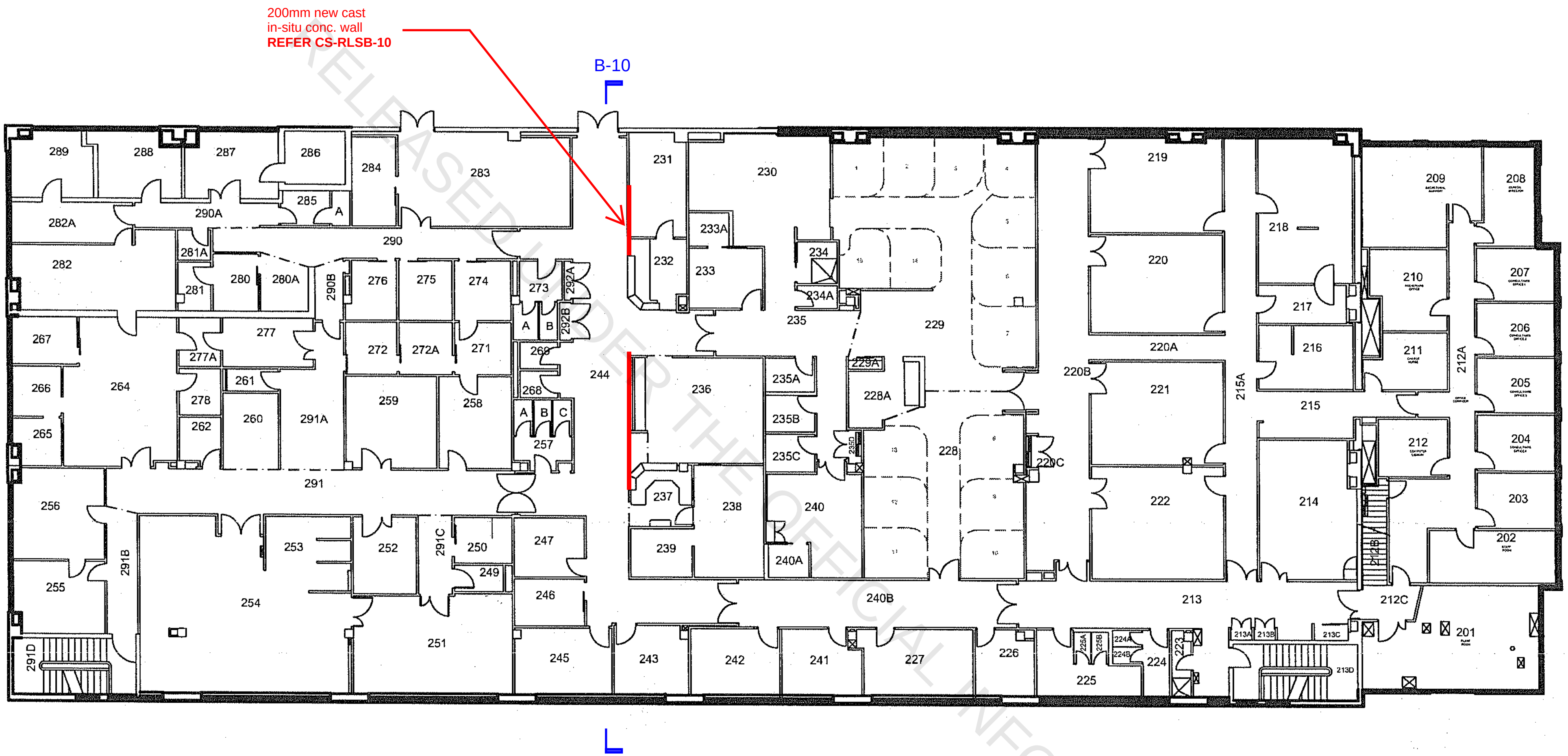


Repairs to L1 diaphragm cracking adjacent central shear wall (generally within the column bay on either side of the wall). Epoxy inject all cracks > 0.3mm width. Refer seismic assessment report section 3.

Replacing Lost Strength - **Scheme B**
L1 Plan

CHRISTCHURCH HOSPITAL
RIVERSIDE CLINICAL SERVICES FIRST FLOOR
SPACE NUMBERING

PROJECT: CDHB Clinical Services Bldg
JOB NO: 106186.09 SSK: CS-RLSB-04 REV: 1
HolmesConsultingGroup DATE: 01.08.12



Replacing Lost Strength - **Scheme B**

L2 Plan

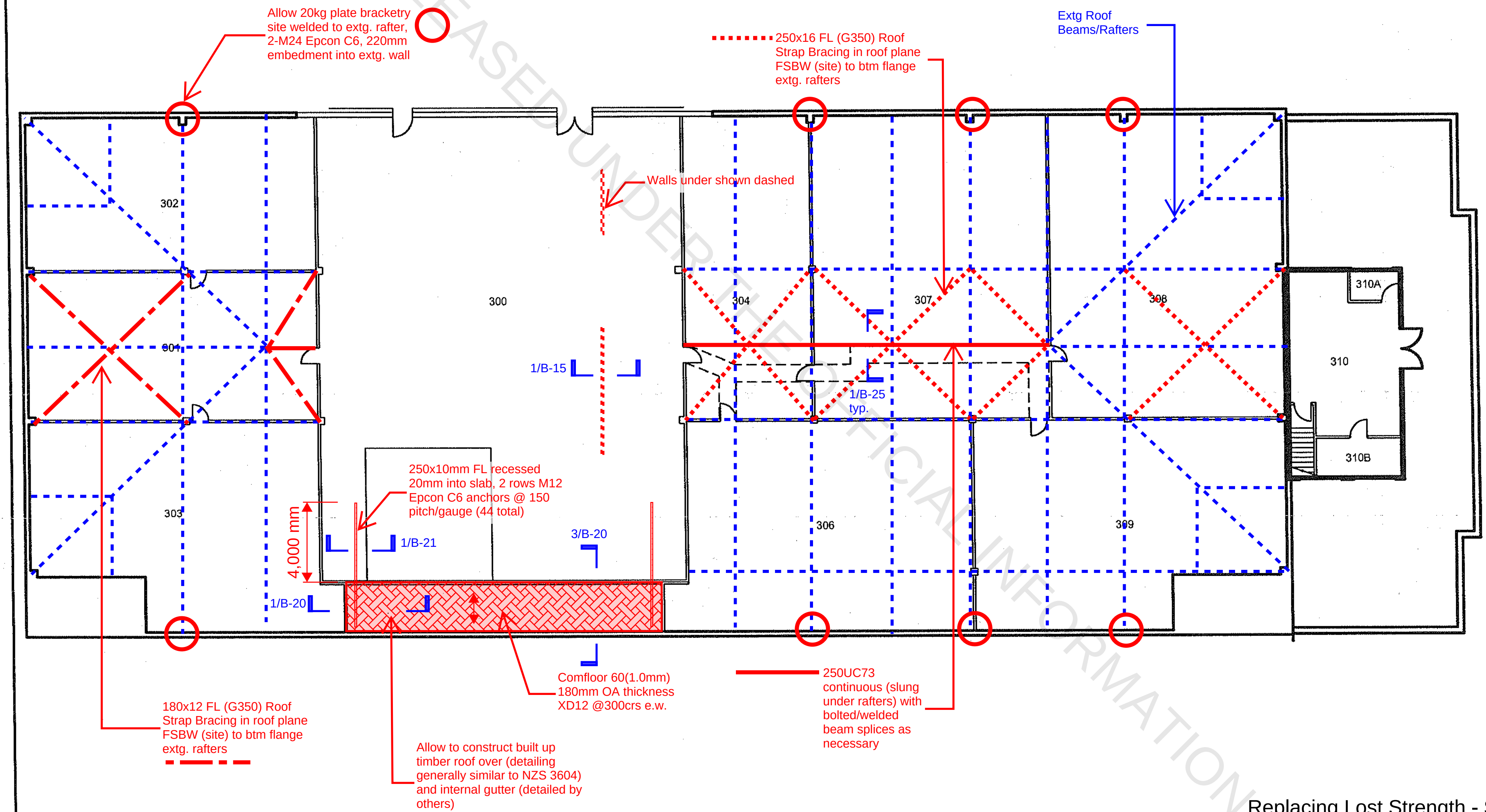
CHRISTCHURCH HOSPITAL
RIVERSIDE CLINICAL SERVICES SECOND FLOOR
SPACE NUMBERING

Cant
Distri
Main

PROJECT: CDHB Clinical Services Bldg

JOB NO: 106186.09 SSK: CS-RLSB-05 REV: 1

HolmesConsultingGroup DATE: 01.08.12



Replacing Lost Strength - **Scheme B**

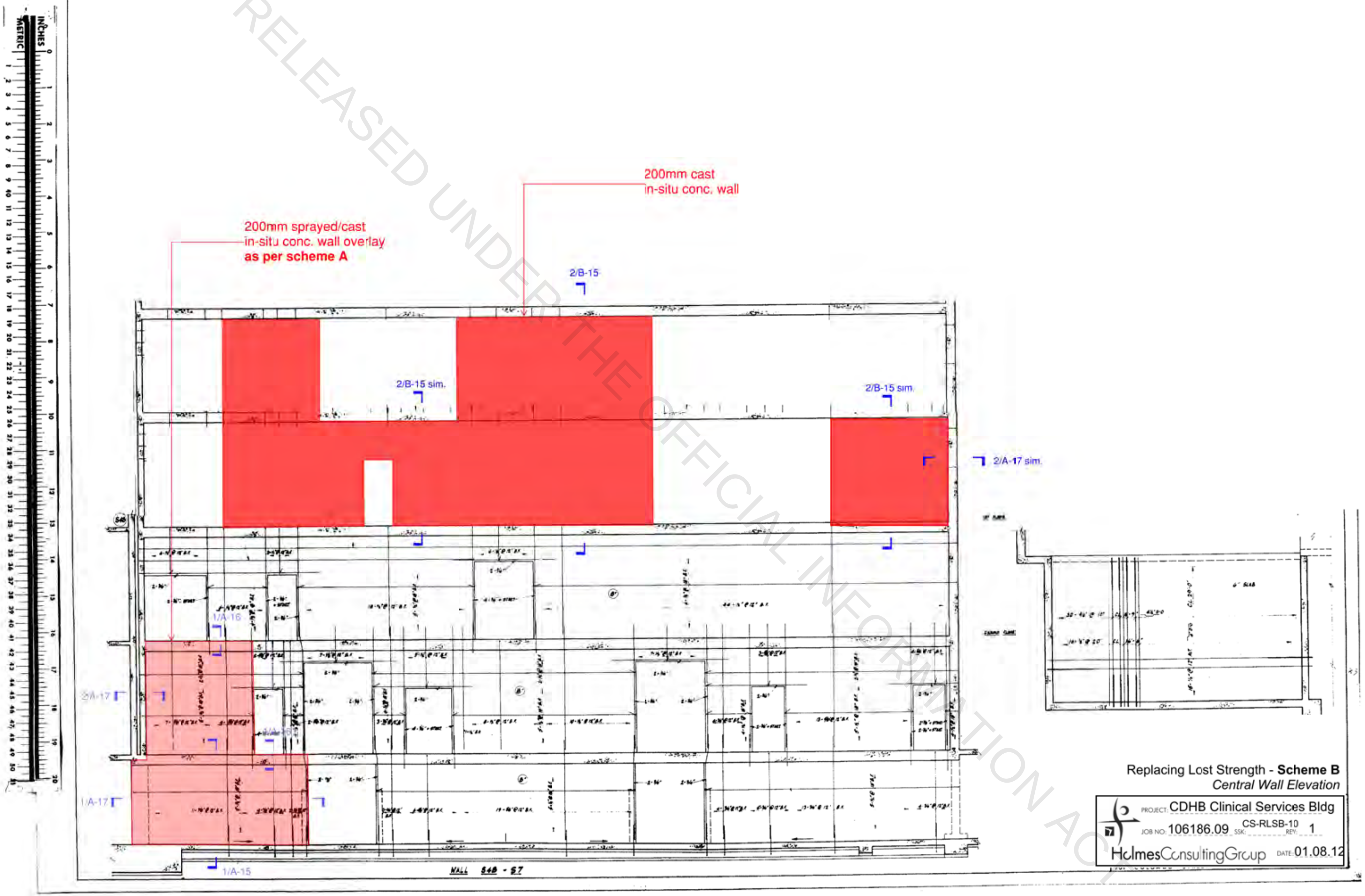
L3 Plant/Main Roof

CHRISTCHURCH HOSPITAL
RIVERSIDE CLINICAL SERVICES THIRD FLOOR
SPACE NUMBERING

PROJECT: CDHB Clinical Services Bldg

JOB NO: 106186.09 SSK: CS-RLSB-06 REV: 1

HolmesConsultingGroup DATE: 01.08.12



Replacing Lost Strength - **Scheme B**
Central Wall Elevation

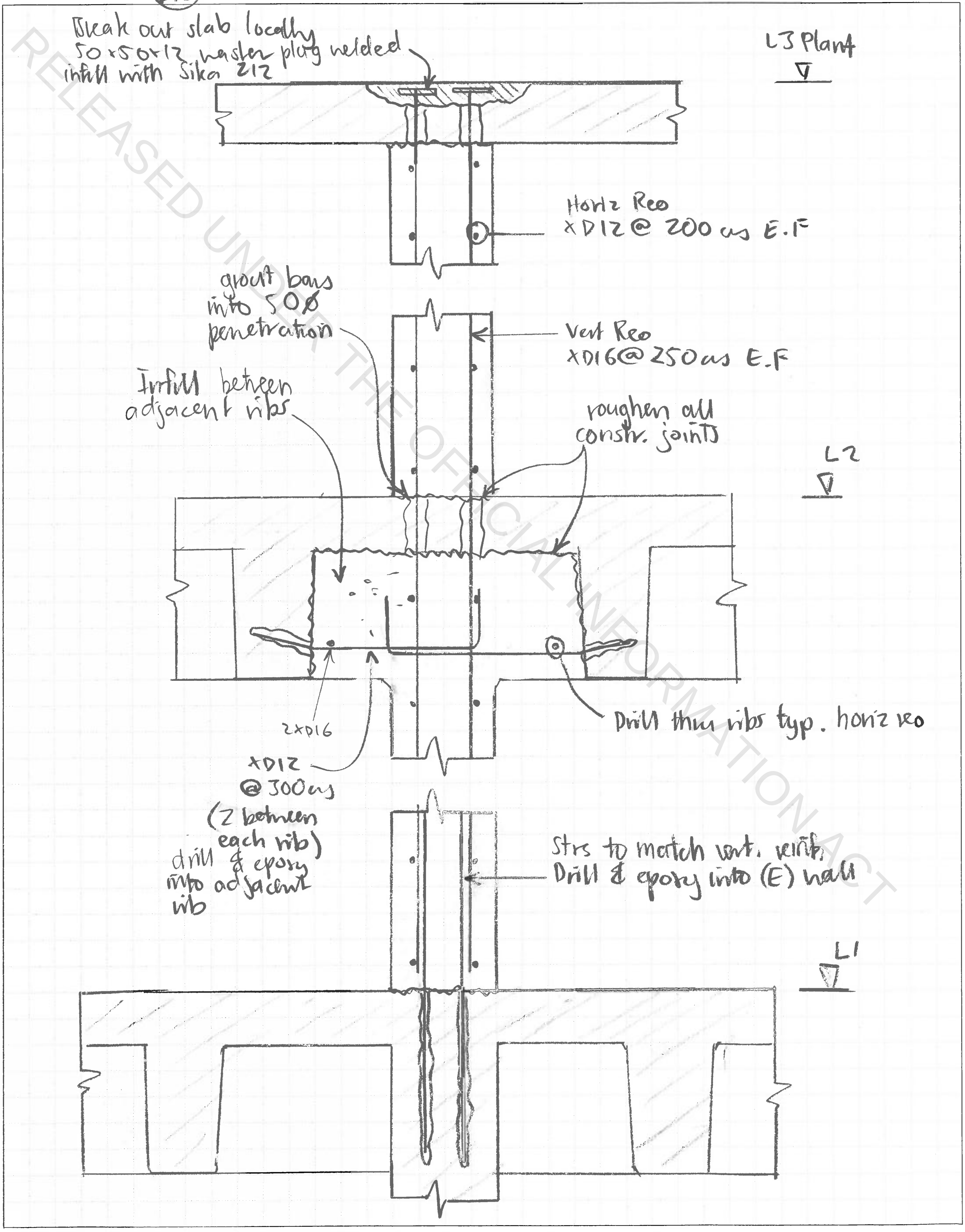
PROJECT: CDHB Clinical Services Bldg
JOB NO: 106186.09 SSK: CS-RLSB-10 REV: 1
HolmesConsultingGroup DATE: 01.08.12

HolmesConsultingGroup

Project Name: CDHB CSB
Project No: 106186.09
Calcs By: AT
Date: 1/8/12
Sketch No: CS-RLSB-15
Page No:
Revision: 1

CALCS/SKETCHES

1/B10 Elevation Section



HolmesConsultingGroup

Project Name: CDHB CSB

Project No: 106186.09

Calcs By: AT

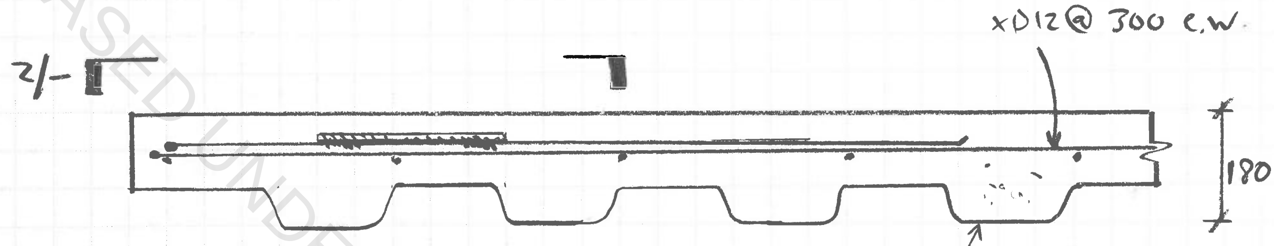
Date: 1/8/12

Page No:

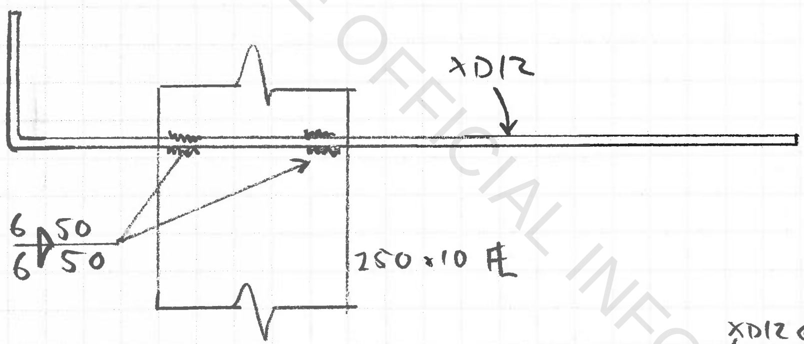
Sketch No: CS-RLSB-20

Revision: 1

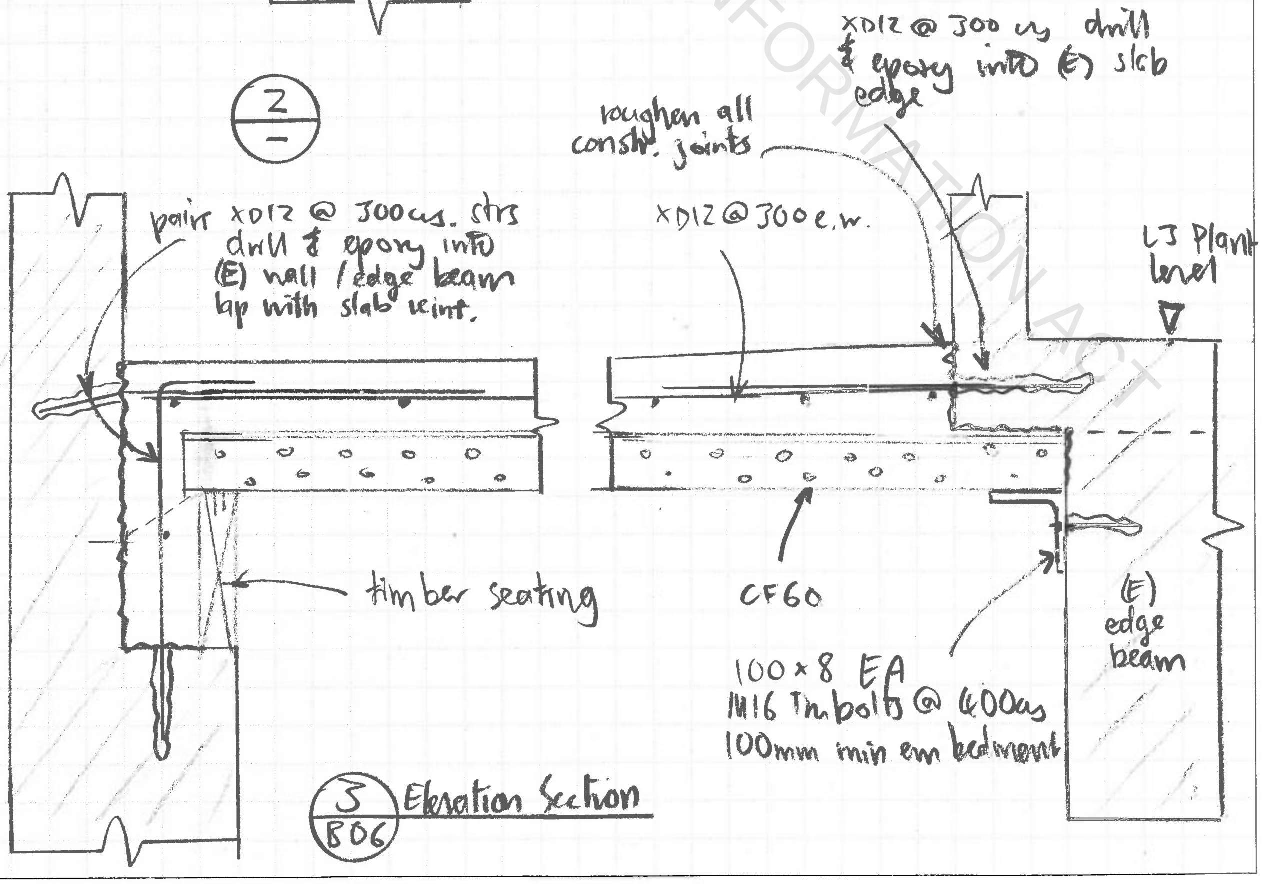
CALCS/SKETCHES



1 / B06 Elevation/Section



2 / –



3 / B06 Elevation Section

HolmesConsultingGroup

Project Name: CDHB - CSB

Project No: 106185.09

Calcs By: AT

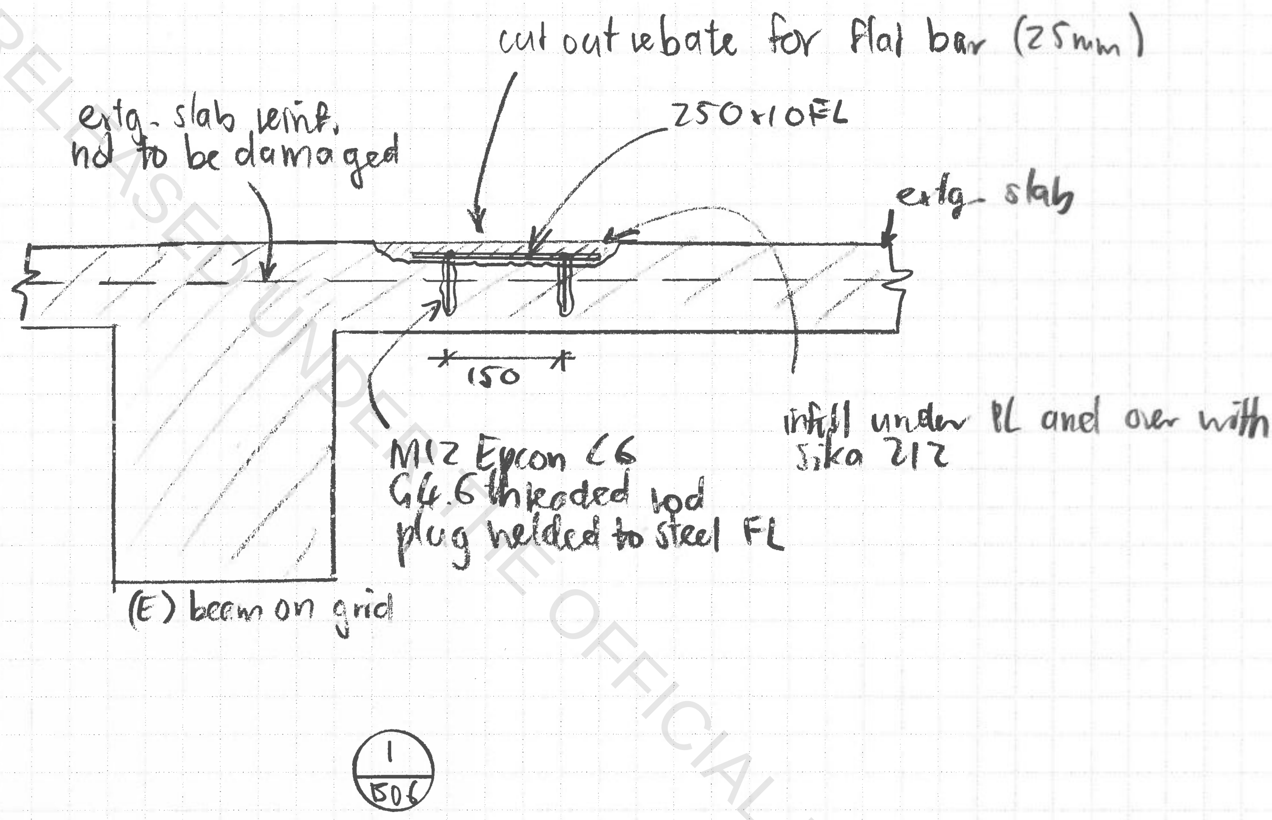
Date: 1/8/12

Page No:

Sketch No: CS-RCSB-21

Revision: 1

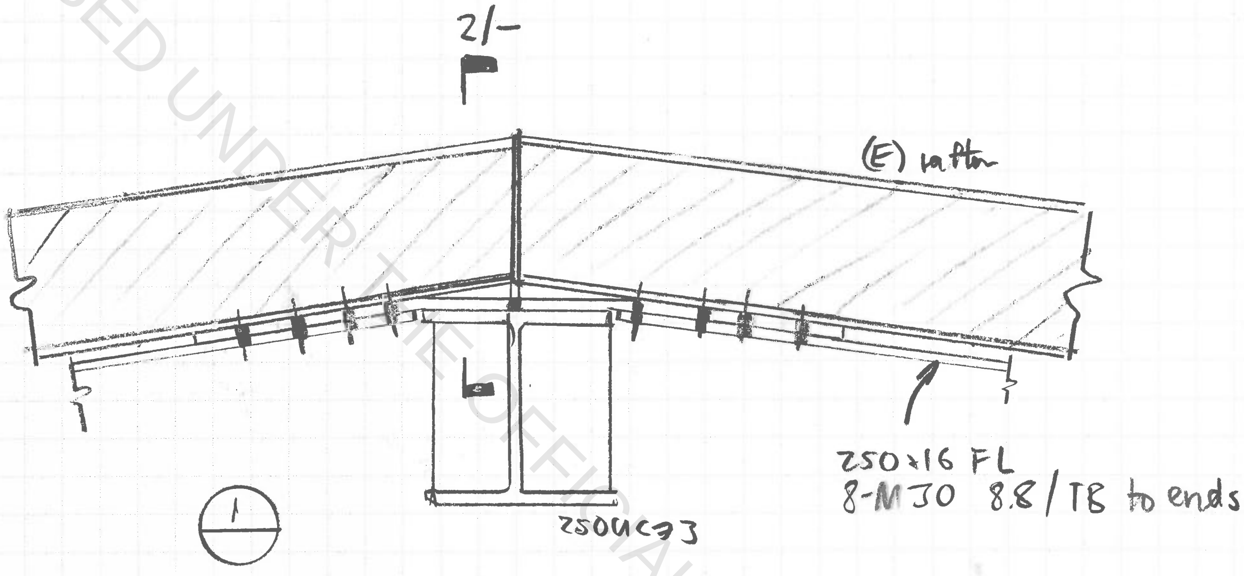
CALCS/SKETCHES

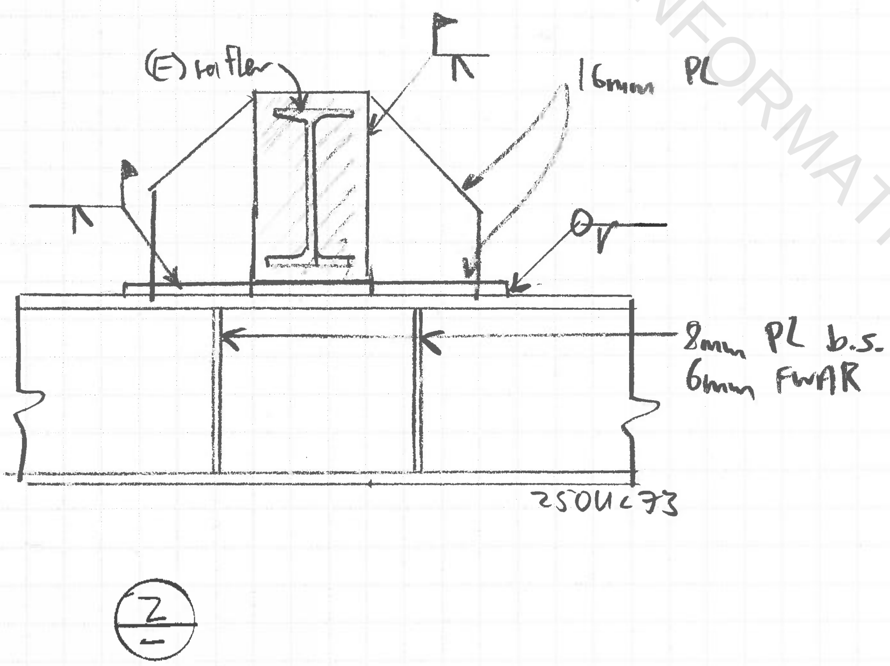


HolmesConsultingGroup

Project Name: CDHB CSB
Project No: 10618609
Calcs By: AT
Date: 1/8/12
Page No:
Sketch No: CS-RLSB-25
Revision: 1

CALCS/SKETCHES





APPENDIX C

Repair and Further Investigation Sketches
CS-RFI-01 TO CS-RFI-06
CS-RFI-10 TO CS-RFI-14

Legend:

Structural Walls - cracks identified during detailed inspection - requiring injection

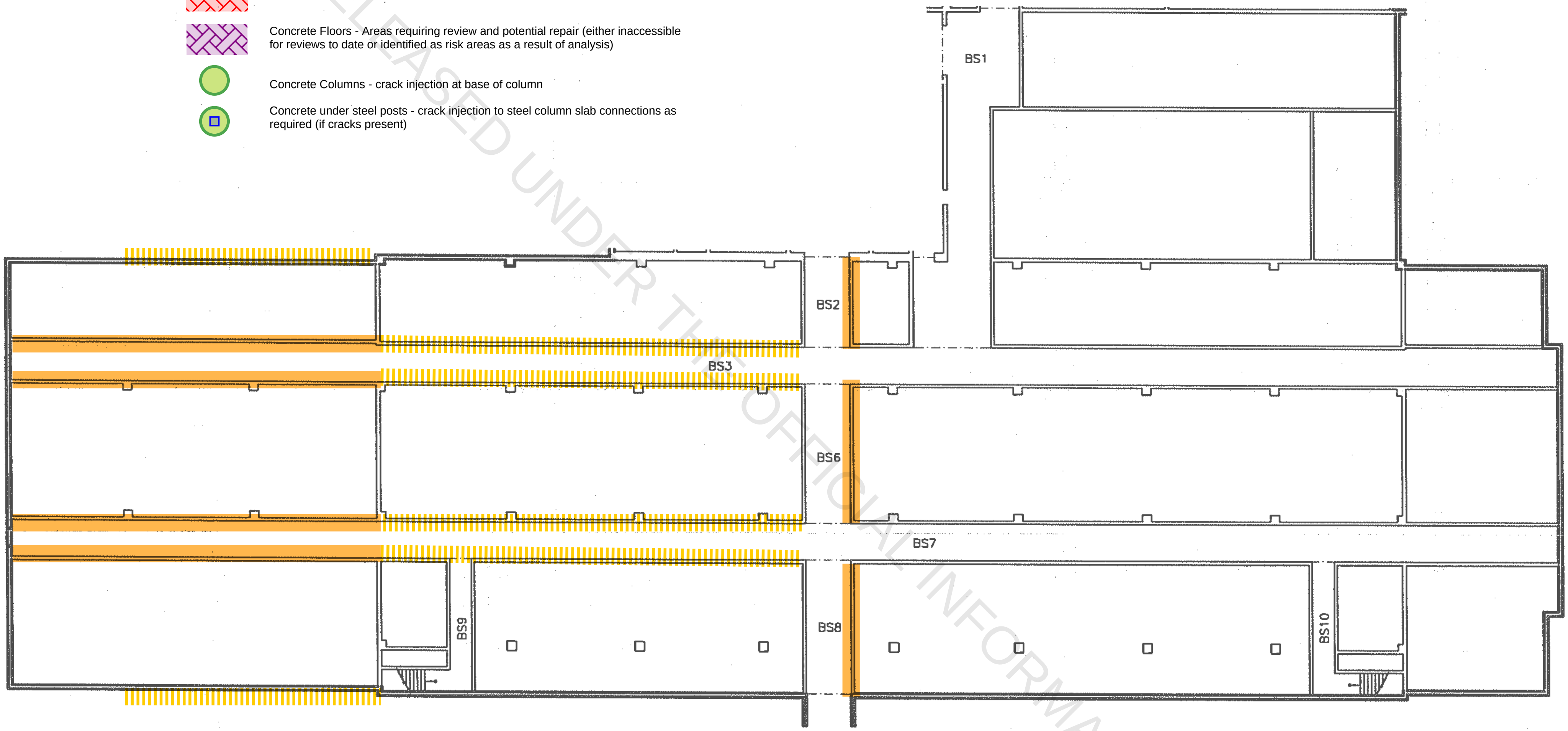
Structural Walls - Areas requiring review and potential repair (either inaccessible for reviews to date or identified as risk areas as a result of analysis)

Concrete Floors - cracks identified during detailed inspection - requiring injection

Concrete Floors - Areas requiring review and potential repair (either inaccessible for reviews to date or identified as risk areas as a result of analysis)

Concrete Columns - crack injection at base of column

Concrete under steel posts - crack injection to steel column slab connections as required (if cracks present)



Approximate Scale 1:250 @ A3

CDHB CSB
Repair and Further Investigation Required
General Layouts
Basement Plan

PROJECT: CDHB CSB - Repair / Investigation
JOB NO: 106186.09 SSK: CS-RFI-01 REV: 1
DATE: 20.09.12
HolmesConsultingGroup

Legend:

Structural Walls - cracks identified during detailed inspection - requiring injection

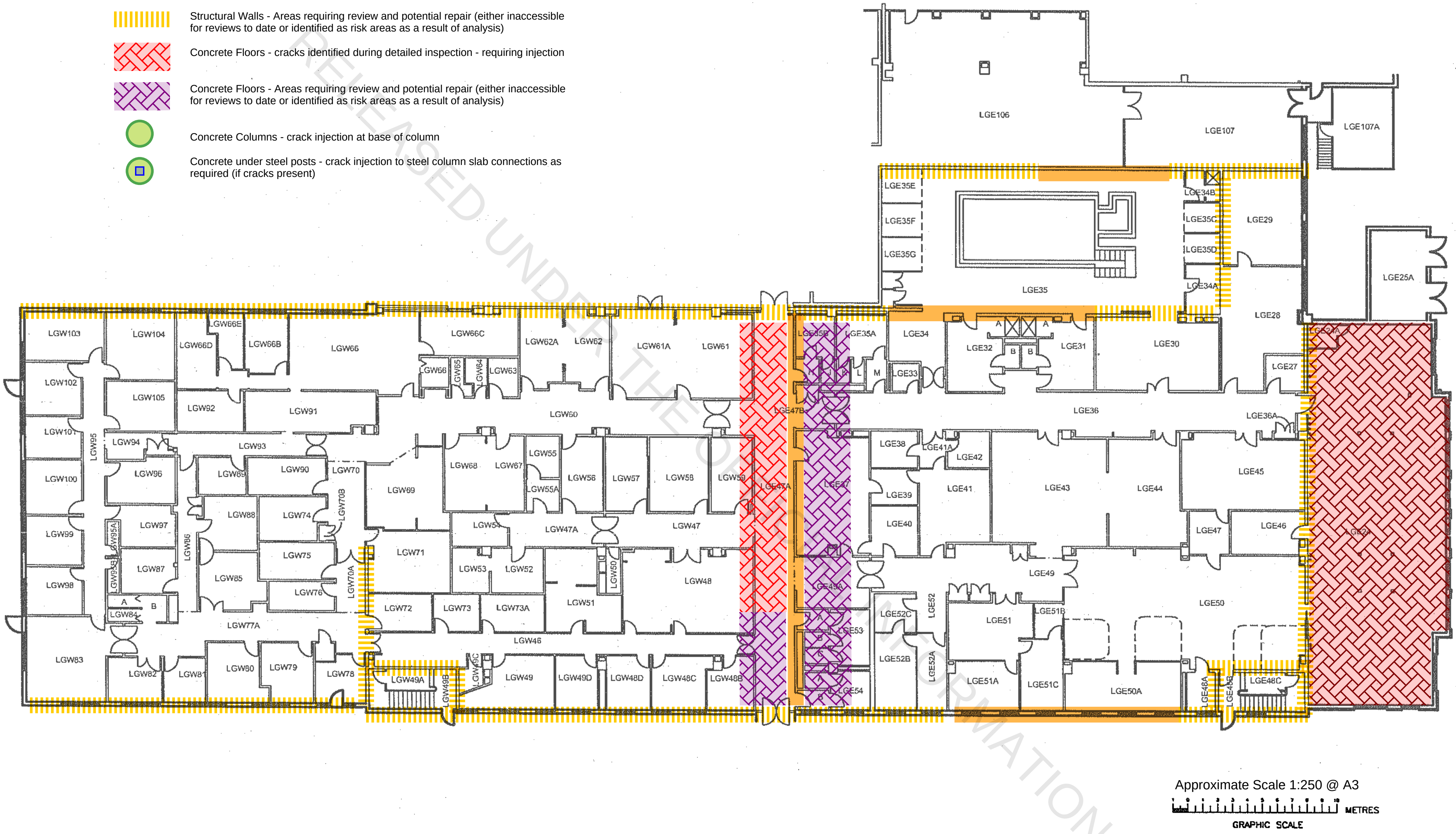
Structural Walls - Areas requiring review and potential repair (either inaccessible for reviews to date or identified as risk areas as a result of analysis)

Concrete Floors - cracks identified during detailed inspection - requiring injection

Concrete Floors - Areas requiring review and potential repair (either inaccessible for reviews to date or identified as risk areas as a result of analysis)

Concrete Columns - crack injection at base of column

Concrete under steel posts - crack injection to steel column slab connections as required (if cracks present)



Approximate Scale 1:250 @ A3

GRAPHIC SCALE METRES

CDHB CSB

Repair and Further Investigation Required

General Layouts

Lower Ground Floor Plan

PROJECT: CDHB CSB - Repair / Investigation

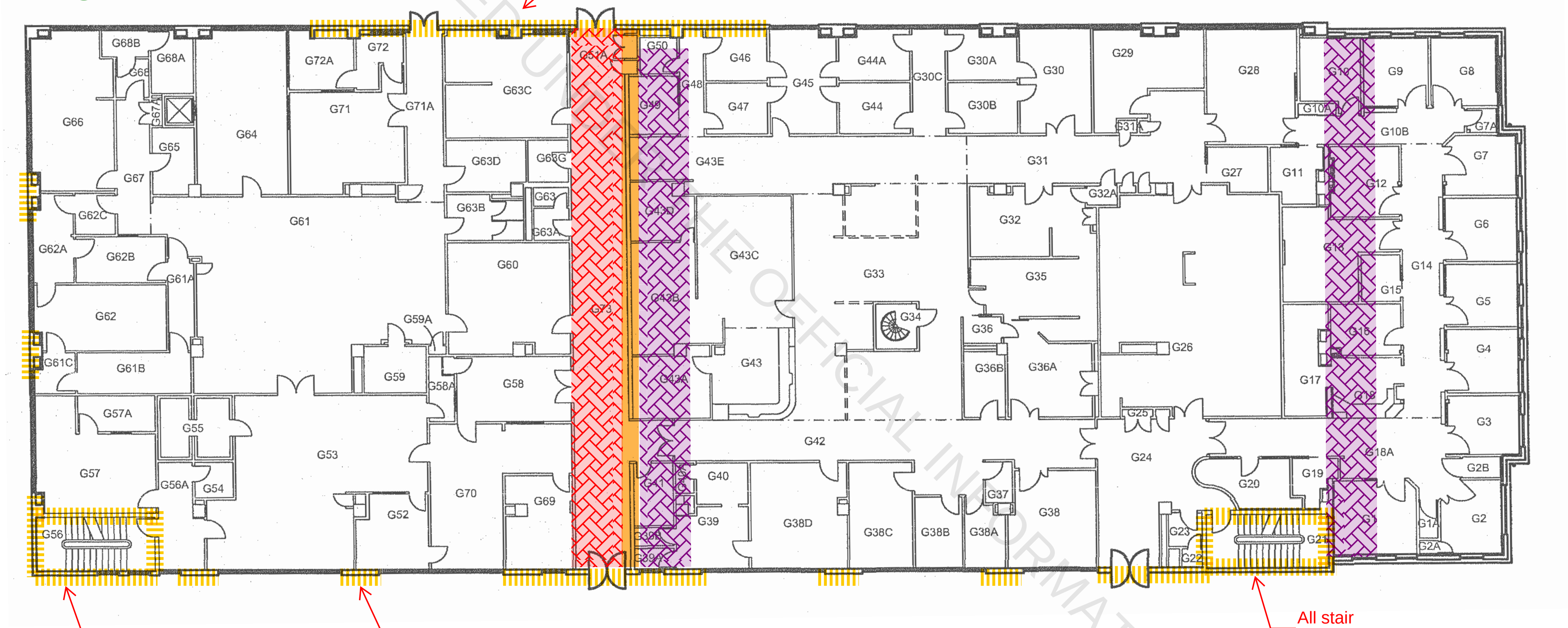
JOB NO: 106186.09 SSK: CS-RFI-02 REV: 1

HolmesConsultingGroup DATE: 20.09.12

Legend:

- Structural Walls - cracks identified during detailed inspection - requiring injection
- Structural Walls - Areas requiring review and potential repair (either inaccessible for reviews to date or identified as risk areas as a result of analysis)
- Concrete Floors - cracks identified during detailed inspection - requiring injection
- Concrete Floors - Areas requiring review and potential repair (either inaccessible for reviews to date or identified as risk areas as a result of analysis)
- Concrete Columns - crack injection at base of column
- Concrete under steel posts - crack injection to steel column slab connections as required (if cracks present)

North shear wall along interface



All stair walls

Piers between window opening (potential shear cracking)

Window opening corners over, (potential vertical cracking/spalling)

All stair walls

Approximate Scale 1:200 @ A3

10 m

CDHB CSB

Repair and Further Investigation Required

General Layouts

Ground Level Plan

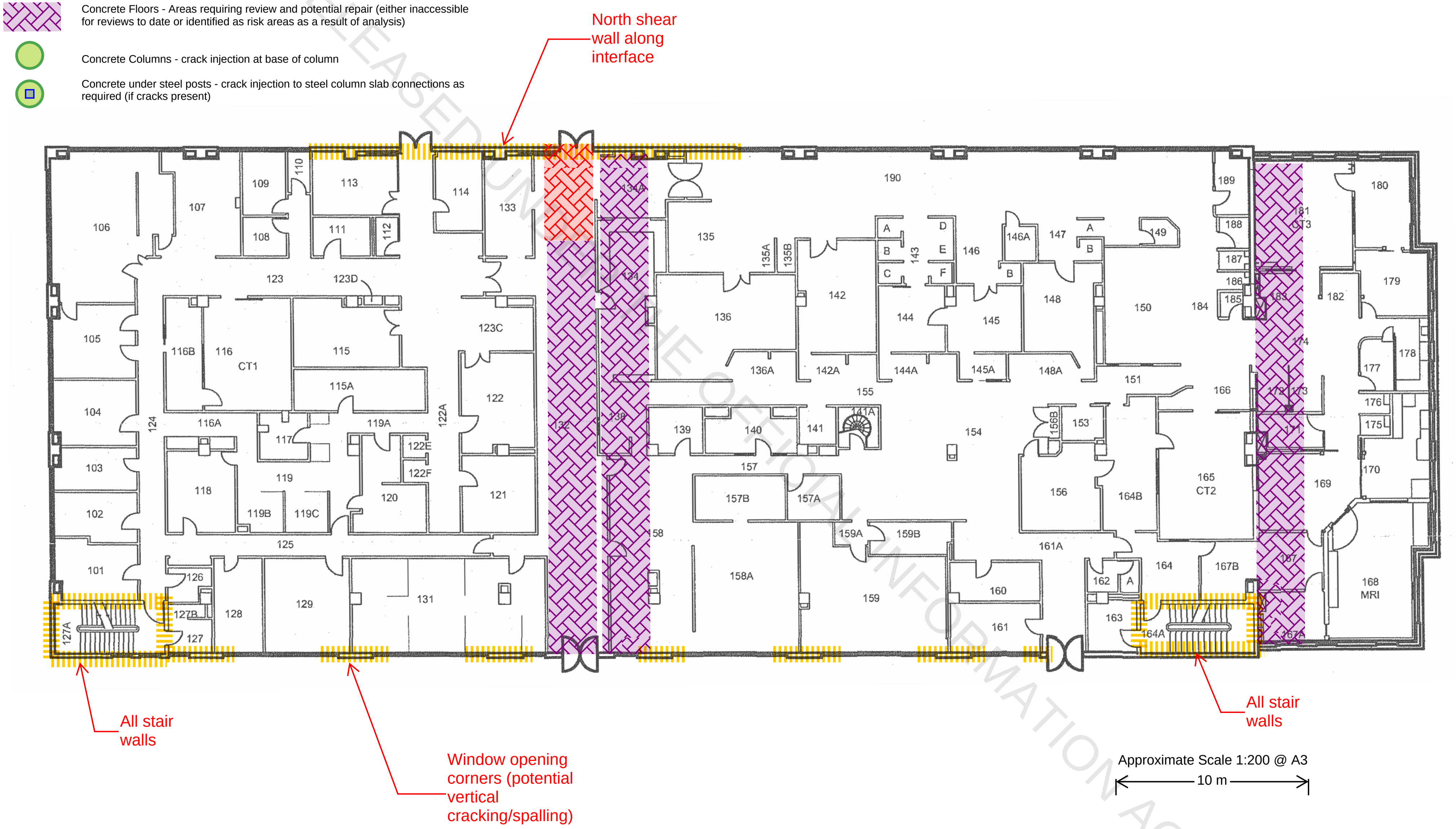
PROJECT: CDHB CSB - Repair / Investigation

JOB NO: 106186.09 SSK: CS-RFI-03 REV: 1

HolmesConsultingGroup DATE: 20.09.12

Legend:

- Structural Walls - cracks identified during detailed inspection - requiring injection
- Structural Walls - Areas requiring review and potential repair (either inaccessible for reviews to date or identified as risk areas as a result of analysis)
- Concrete Floors - cracks identified during detailed inspection - requiring injection
- Concrete Floors - Areas requiring review and potential repair (either inaccessible for reviews to date or identified as risk areas as a result of analysis)
- Concrete Columns - crack injection at base of column
- Concrete under steel posts - crack injection to steel column slab connections as required (if cracks present)



CDHB CSB
Repair and Further Investigation Required
General Layouts
Level 1 Plan

PROJECT: CDHB CSB - Repair / Investigation
JOB NO: 106186.09 SSK: CS-RFI-04 REV: 1
HolmesConsultingGroup DATE: 20.09.12

Legend:

- Structural Walls - cracks identified during detailed inspection - requiring injection
- Structural Walls - Areas requiring review and potential repair (either inaccessible for reviews to date or identified as risk areas as a result of analysis)
- Concrete Floors - cracks identified during detailed inspection - requiring injection
- Concrete Floors - Areas requiring review and potential repair (either inaccessible for reviews to date or identified as risk areas as a result of analysis)
- Concrete Columns - crack injection at base of column
- Concrete under steel posts - crack injection to steel column slab connections as required (if cracks present)

CDHB CSB
Repair and Further Investigation Required
General Layouts
Level 2 Plan

PROJECT: CDHB CSB - Repair / Investigation
JOB NO: 106186.09 SSK: CS-RFI-05 REV: 1
HolmesConsultingGroup DATE: 20.09.12

Legend:

Structural Walls - cracks identified during detailed inspection - requiring injection

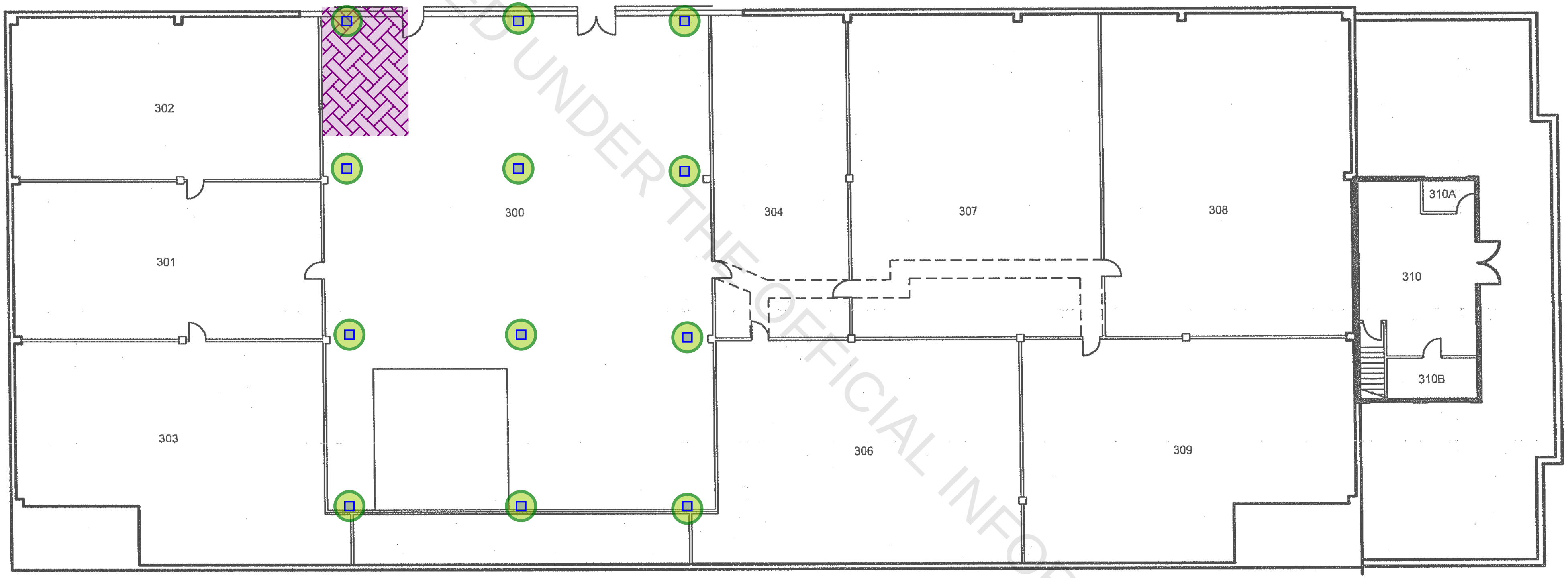
Structural Walls - Areas requiring review and potential repair (either inaccessible for reviews to date or identified as risk areas as a result of analysis)

Concrete Floors - cracks identified during detailed inspection - requiring injection

Concrete Floors - Areas requiring review and potential repair (either inaccessible for reviews to date or identified as risk areas as a result of analysis)

Concrete Columns - crack injection at base of column

Concrete under steel posts - crack injection to steel column slab connections as required (if cracks present)



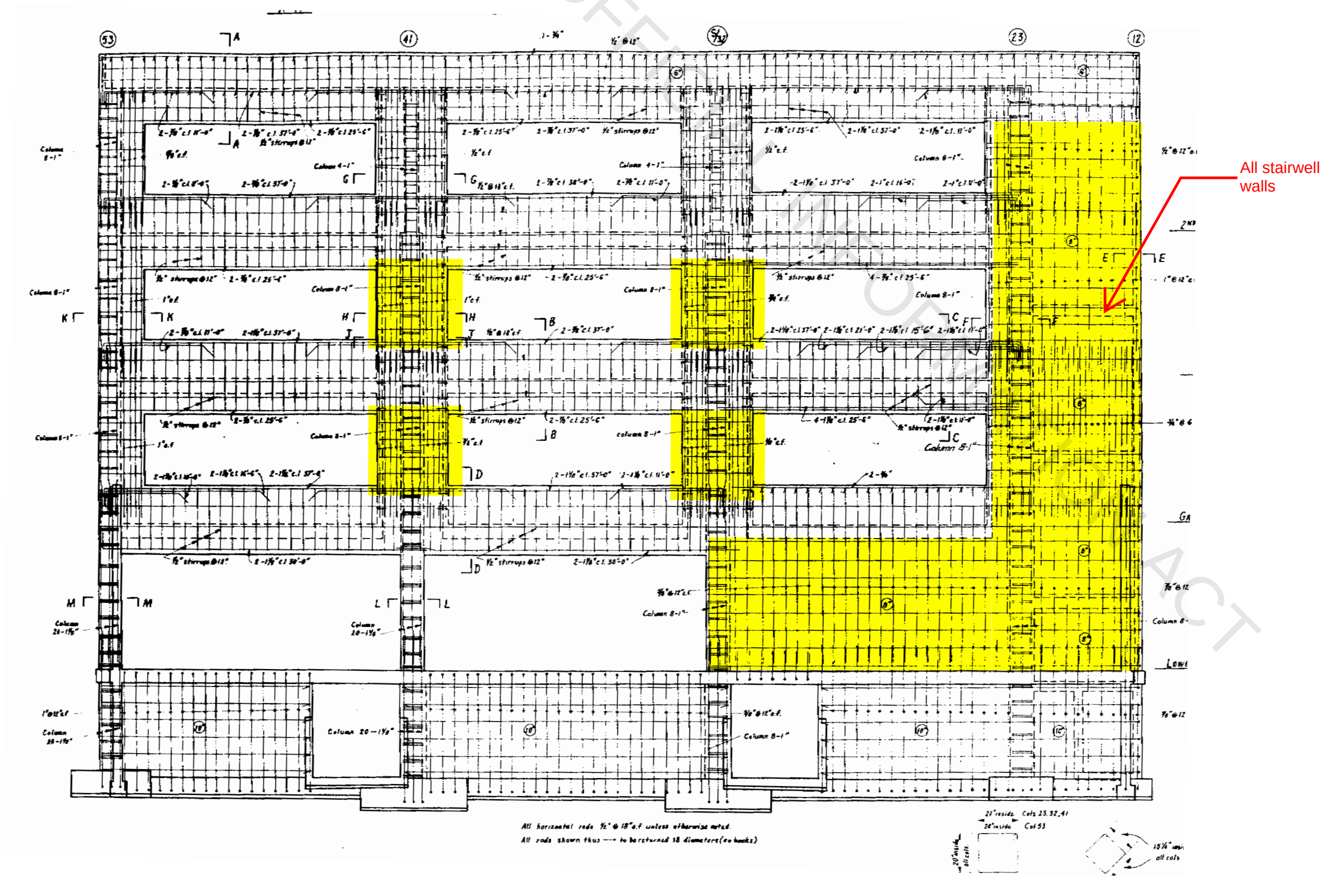
Approximate Scale 1:200 @ A3

10 m

CDHB CSB
Repair and Further Investigation Required
General Layouts
L3 Plant Floor Plan

PROJECT: CDHB CSB - Repair / Investigation
JOB NO: 106186.09 SSK: CS-RFI-06 REV: 1
HolmesConsultingGroup DATE: 20.09.12

Eastern Shear Wall (original east wall)



Western Shear Wall

CDHB CSB

Repair and Further Investigation Required

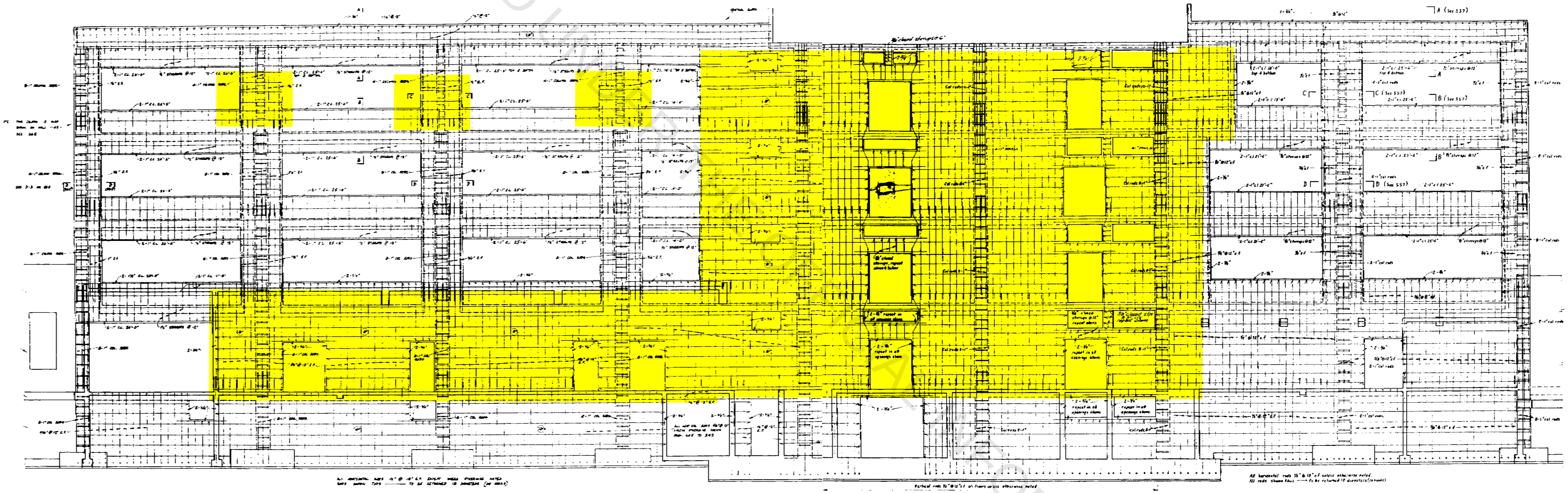
General Layouts

East/West Wall Elevations

PROJECT: CDHB CSB - Repair / Investigation

JOB NO: 106186.09 SSK: CS-RFI-10 REV: 1

HolmesConsultingGroup DATE: 20.09.12



North Shear Wall (view from outside)

CDHB CSB
Repair and Further Investigation Required
General Layouts
North Wall Elevations

PROJECT:	CDHB CSB - Repair / Investigation
JOB NO:	106186.09
SSK:	CS-RFI-11
REV:	1
DATE:	20.09.12

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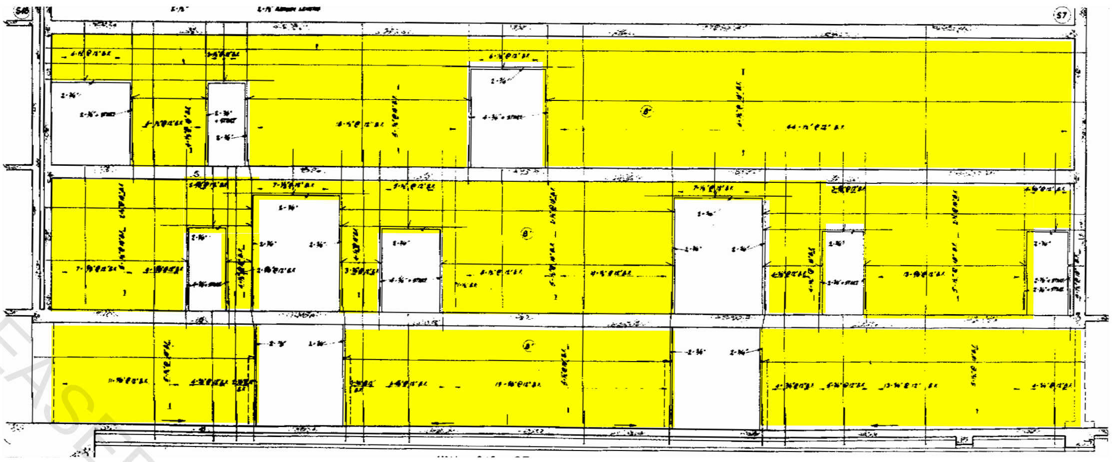
South Shear Wall (view from outside)

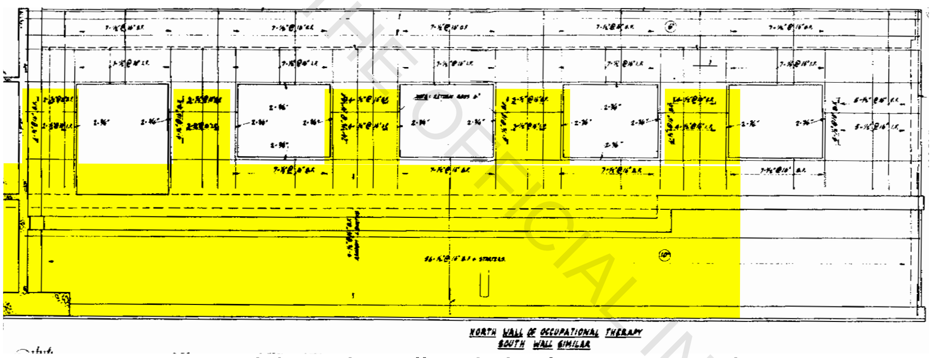
CDHB CSB
Repair and Further Investigation Required
General Layouts
South Wall Elevations

PROJECT:	CDHB CSB - Repair / Investigation
JOB NO:	106186.09
SSK:	CS-RFI-12
REV:	1
DATE:	20.09.12

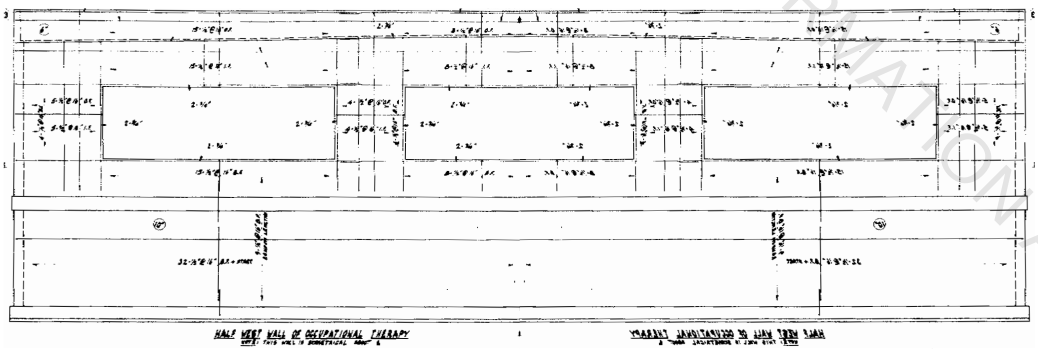
HolmesConsultingGroup



Central shear wall



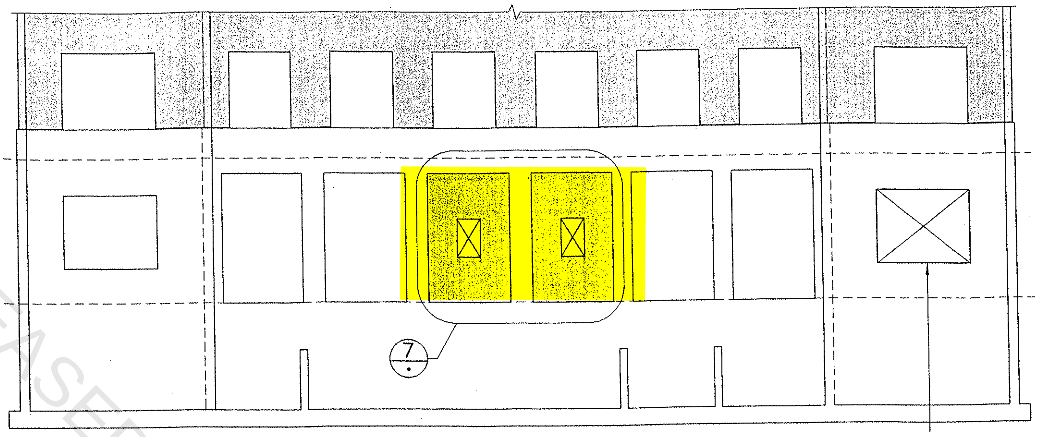
North/South walls of single storey portion



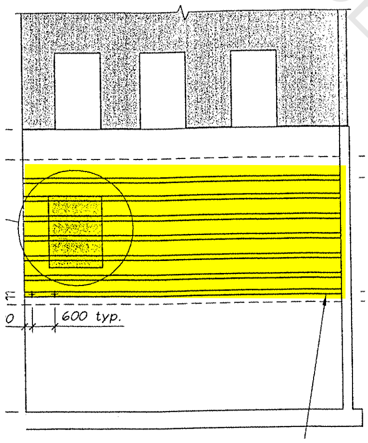
West wall of single storey portion

CDHB CSB
Repair and Further Investigation Required
General Layouts
Single Storey/Central Wall Elevations

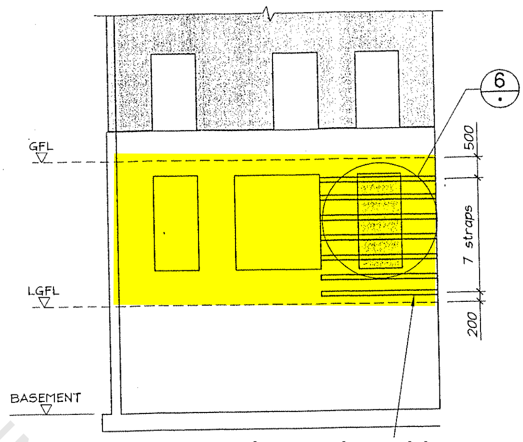
PROJECT: CDHB CSB - Repair / Investigation
JOB NO: 106186.09 SSK: CS-RFI-13 REV: 1
HolmesConsultingGroup DATE: 20.09.12



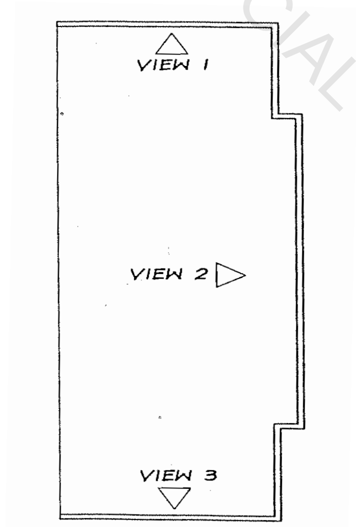
View 2 (Far east wall)



View 1 (north)



View 3 (south)



Plan view of Eastern Add-on

CDHB CSB
Repair and Further Investigation Required
General Layouts
Eastern Add-on Wall Part Elevations

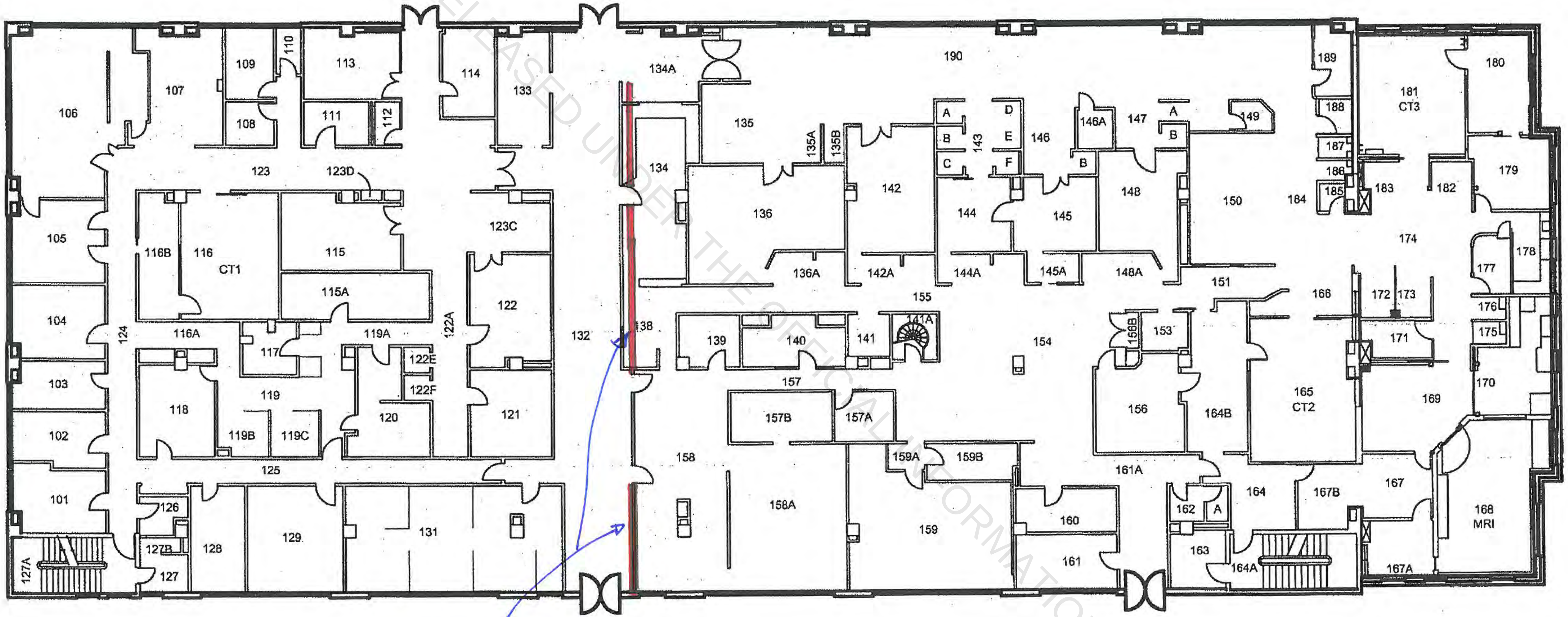
PROJECT:	CDHB CSB - Repair / Investigation
JOB NO:	106186.09
SSK:	CS-RFI-14
REV:	1
DATE:	20.09.12

HolmesConsultingGroup

APPENDIX D

Preliminary Strengthening Sketches

CS-CSW-01 to CS-CSW-03
CS-SS-01 to CS-SS-06



New 200mm concrete walls

HolmesConsultingGroup

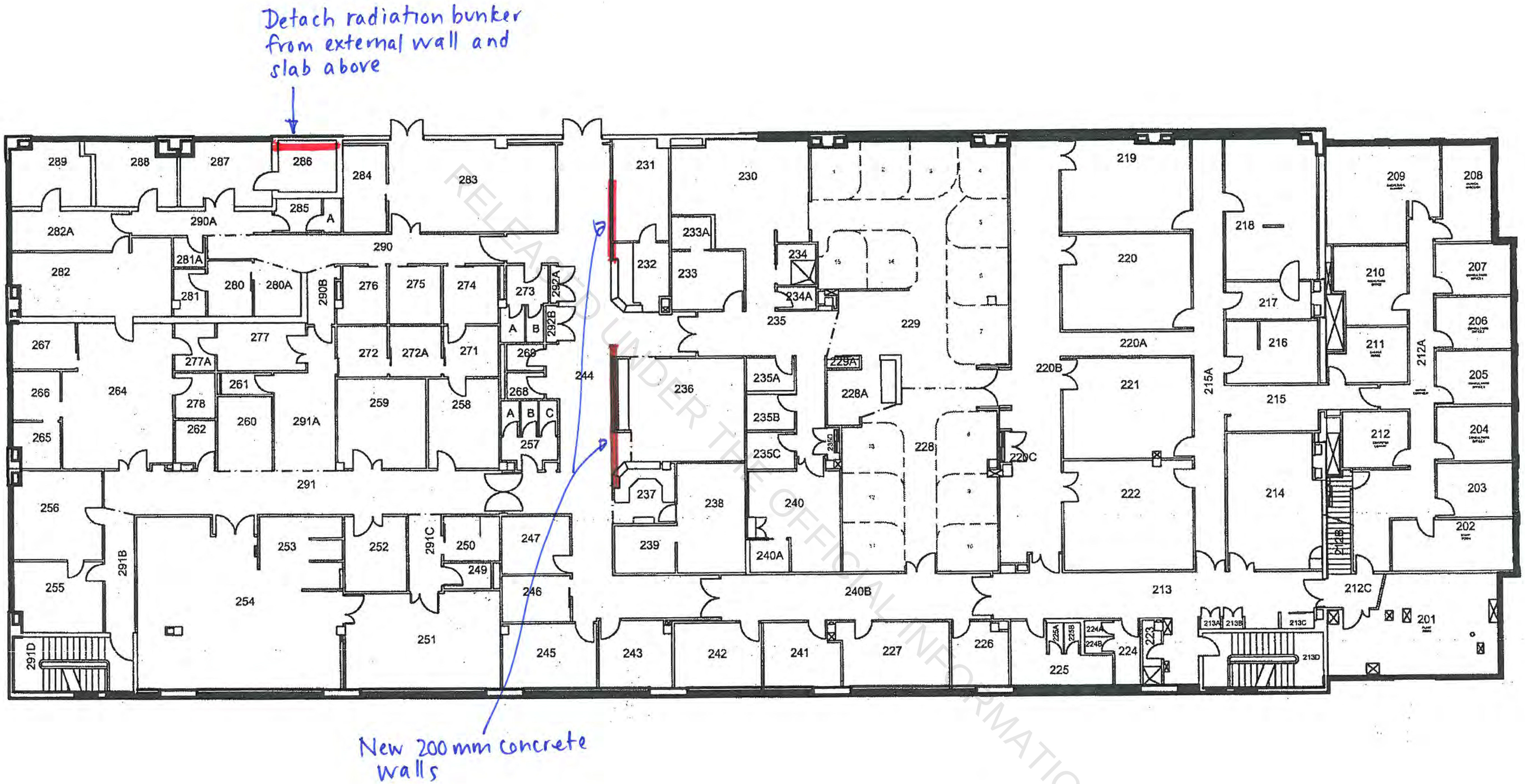
Title: Preliminary Scheme to Remove Critical Structural Weaknesses - First Floor

Job Name: CDHB - Clinical Services Building

Job #: 106186.09 SSK#: CS-CSW-01

Date: 16/12/11 Rev: B

CHRISTCHURCH HOSPITAL
RIVERSIDE CLINICAL SERVICES FIRST FLOOR
SPACE NUMBERING



CHRISTCHURCH HOSPITAL
RIVERSIDE CLINICAL SERVICES SECOND FLOOR
SPACE NUMBERING

Cant
Distri
Mai

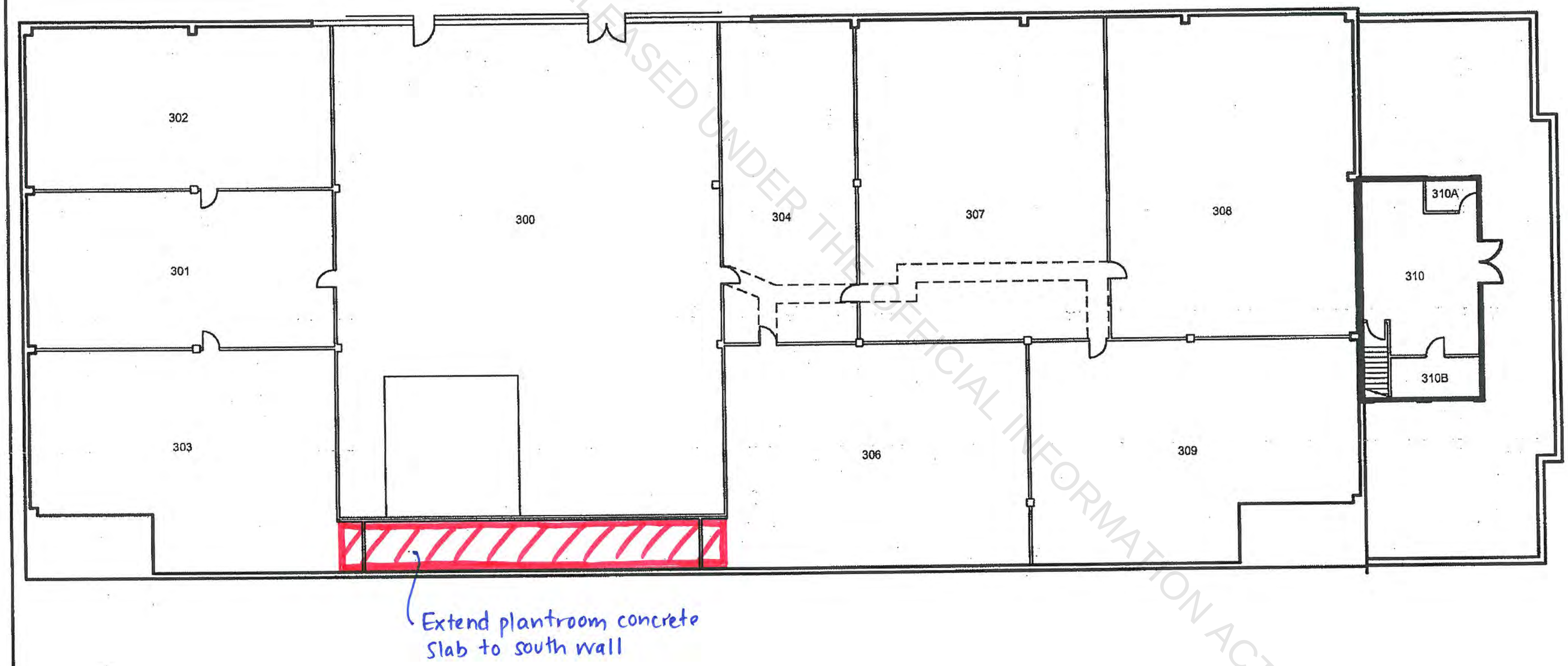
HolmesConsultingGroup

Title: Preliminary Scheme to Remove Critical Structural Weaknesses - Second Floor

Job Name: CDHB Clinical Services Building

Job #: 106186.09 SSK#: CS-CSW-02

Date: 16/12/11 Rev: B



CHRISTCHURCH HOSPITAL
RIVERSIDE CLINICAL SERVICES THIRD FLOOR
SPACE NUMBERING

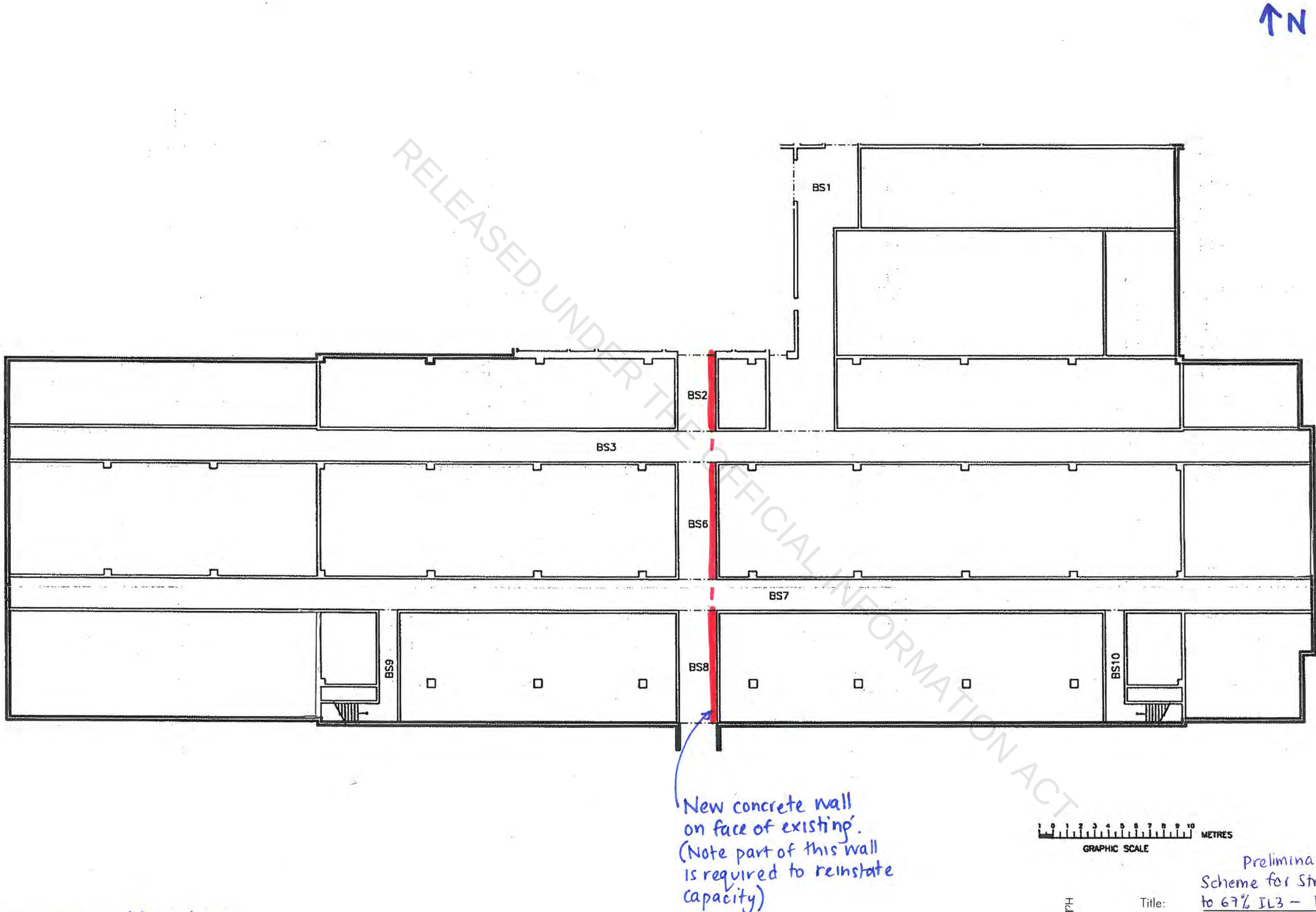
HolmesConsultingGroup

Title: Preliminary Scheme to Remove Critical Structural Weaknesses – Third Floor.

Job Name: CDHB Clinical Services Building

Job #: 106186-09 SSK#: CS-CSW-03

Date: 12/12/11 Rev: B



Note: Work required to reduce the impact of pounding with Riverside Central not shown.

CHRISTCHURCH HOSPITAL
RIVERSIDE CLINICAL SERVICES - BASEMENT
SPACE NUMBERING

HolmesConsultingGroup

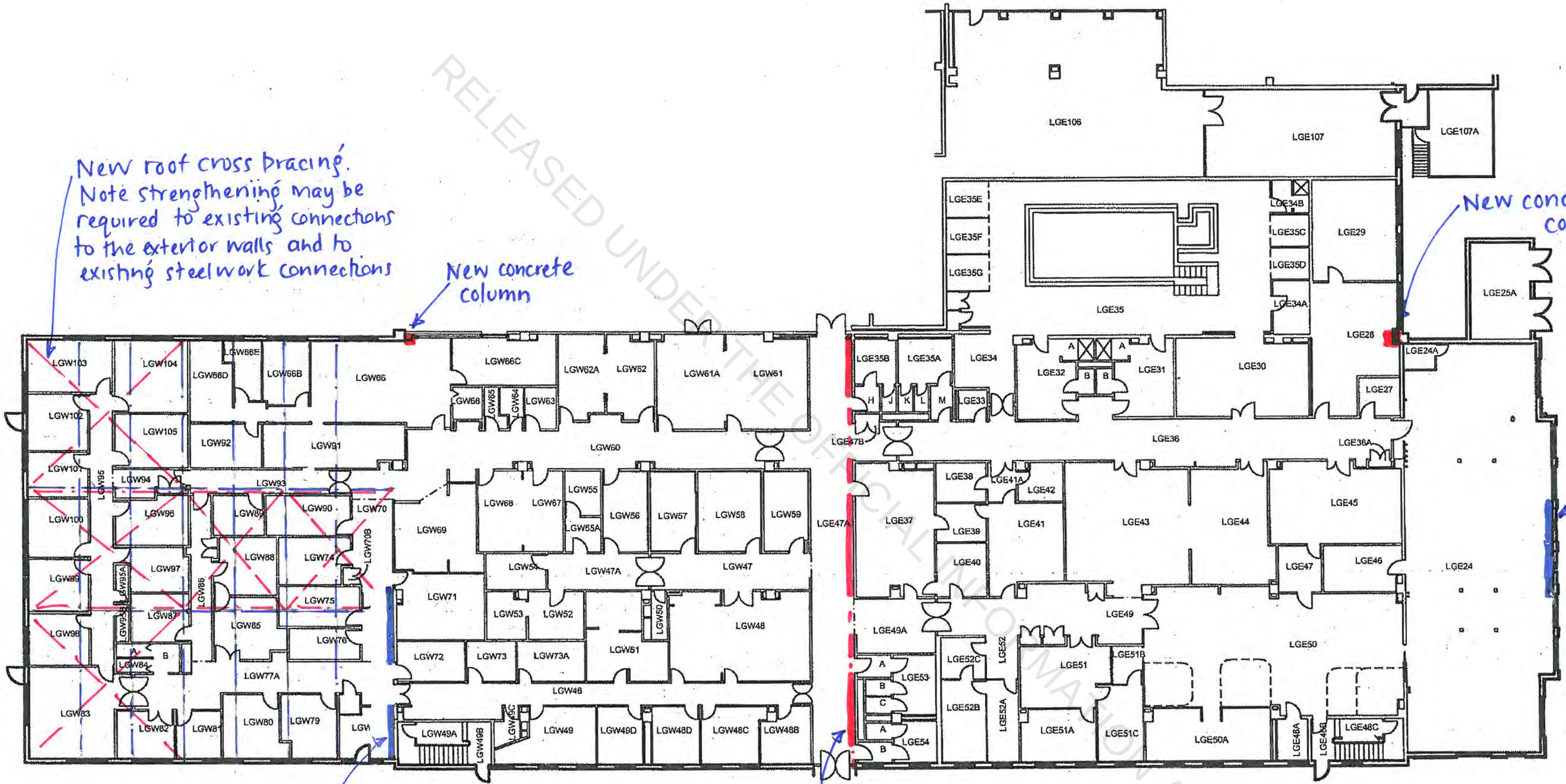
Title: Preliminary Scheme for Strengthening to 67% IL3 - Basement
Job Name: CDHB Clinical Services Building
Job #: 106186.09 SSK#: CS-SS-01
Date: 12/12/11 Rev: B

N

New roof cross bracing.
Note strengthening may be required to existing connections to the exterior walls and to existing steelwork connections

New concrete column

New concrete column.



FRP on inside face of wall

FRP on face of wall full height. Concrete skin if strengthening above is concrete.

New concrete wall on face of existing (Note part of this wall is required to reinstate capacity)
New wall on corridor face of existing due to access to construct new wall at basement level

Note: Work required to reduce the impact of pounding with Riverside Central not shown.

METRES
GRAPHIC SCALE

HolmesConsultingGroup

Title: Preliminary Scheme for Strengthening to 67% IL3 - Lower Ground Floor

Job Name: CDHB Clinical Services Building

Job #: 106186.09 SSK#: CS-SS-02

Date: 16/12/11 Rev: B

CHRISTCHURCH HOSPITAL
RIVERSIDE CLINICAL SERVICES - LOWER GROUND FLOOR
SPACE NUMBERING

↑N

New concrete column

New concrete column

Strengthen West wall as for South wall except stair wall strengthening to be full height not just between the windows.

New concrete wall on face of existing. On corridor face due to access to construct new wall at basement level.

Strengthen the piers between the windows
Can be : FRP on inside face
FRP on outside face (requires removal of existing cladding panels)
Concrete on outside face (in place of existing cladding panel).
All solutions are on the face of the wall only. Not required to return at the ends of the piers

FRP on face of wall

Note : Work required to reduce the impact of pounding with Riverside Central not shown.

CHRISTCHURCH HOSPITAL
RIVERSIDE CLINICAL SERVICES GROUND FLOOR
SPACE NUMBERING

Cante
Distric
Main

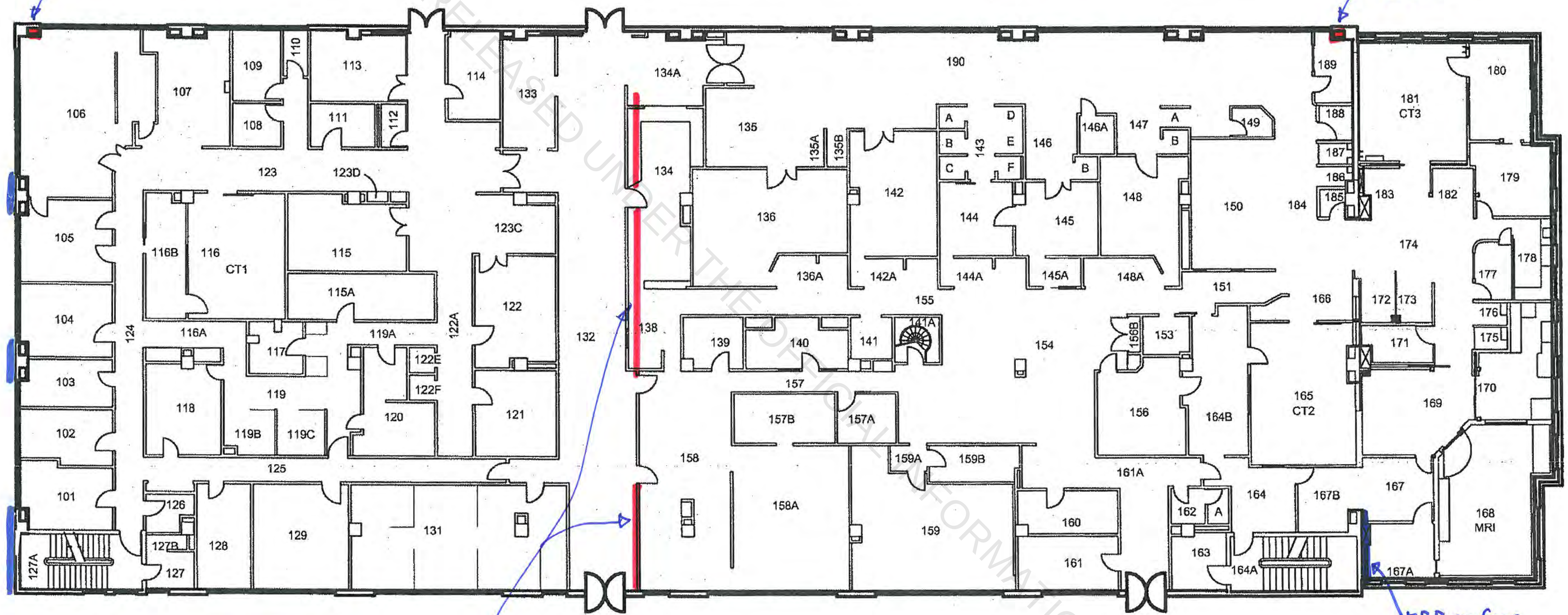
HolmesConsultingGroup

Title: Preliminary Scheme for Strengthening to 67% JL3 — Ground Floor
Job Name: CDHB Clinical Services Building
Job #: 106186.09 SSK#: CS-SS-03
Date: 16/12/11 Rev: B

N

New concrete column

New concrete column



Strengthen the stair wall full height and the piers between windows.
Strengthening can be:
FRP on inside face
FRP on outside face (requires removal of existing cladding panels)
Concrete on outside face (in place of existing cladding panels)

New 200 mm concrete walls (Part of CSW strengthening)

FRP on face of wall

Note: Work required to reduce the impact of pounding with Riverside Central not shown.

CHRISTCHURCH HOSPITAL
RIVERSIDE CLINICAL SERVICES FIRST FLOOR
SPACE NUMBERING

HolmesConsultingGroup

Title: Preliminary Scheme for Strengthening to 67% IL3 - First Floor

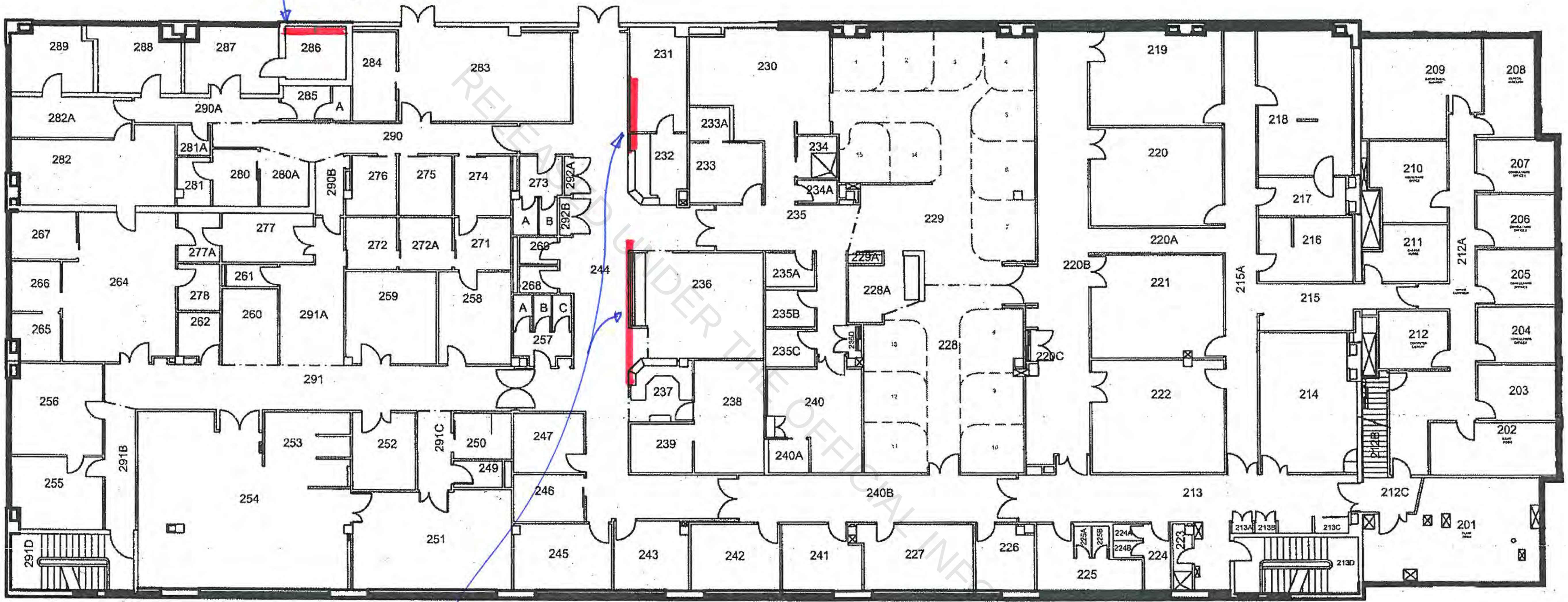
Job Name: CDHB Clinical Services Building

Job #: 106186.09 SSK#: CS-SS-04

Date: 16/12/11 Rev: B

↑N

Detach radiation bunker from external wall and slab above. (Part of CSW strengthening)



New 200mm concrete walls (Part of CSW strengthening)

CHRISTCHURCH HOSPITAL
RIVERSIDE CLINICAL SERVICES SECOND FLOOR
SPACE NUMBERING

Cant
Distri
Main

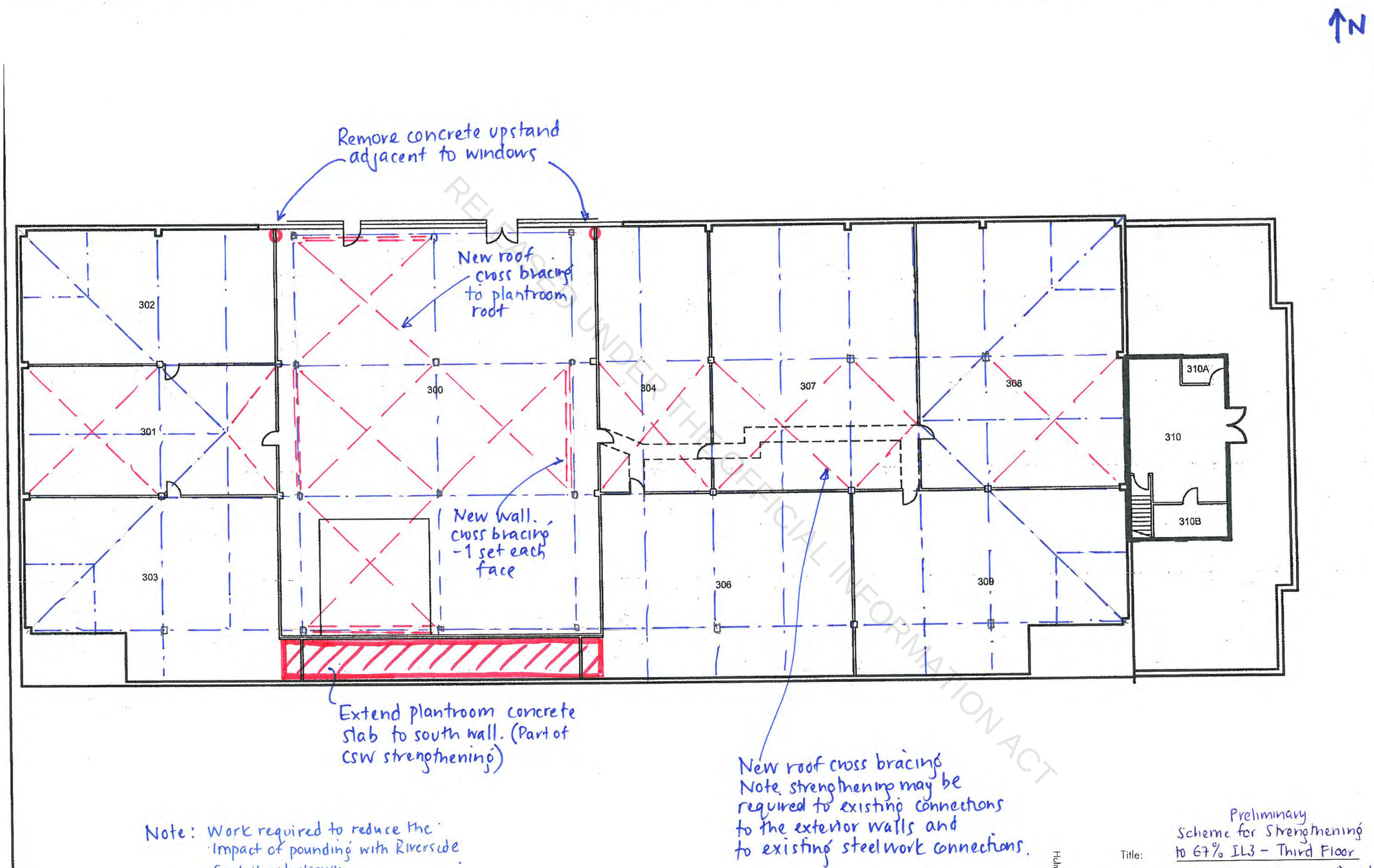
HolmesConsultingGroup

Title: Preliminary Scheme for Strengthening to 67% IL3 - Second Floor

Job Name: CDHB Clinical Services Building

Job #: 106186.09 SSK#: CS-SS-05

Date: 16/12/11 Rev: B



Note: Work required to reduce the impact of pounding with Riverside Central not shown

CHRISTCHURCH HOSPITAL
RIVERSIDE CLINICAL SERVICES THIRD FLOOR
SPACE NUMBERING

HolmesConsultingGroup

Title: Preliminary Scheme for Strengthening to 67% IL3 - Third Floor

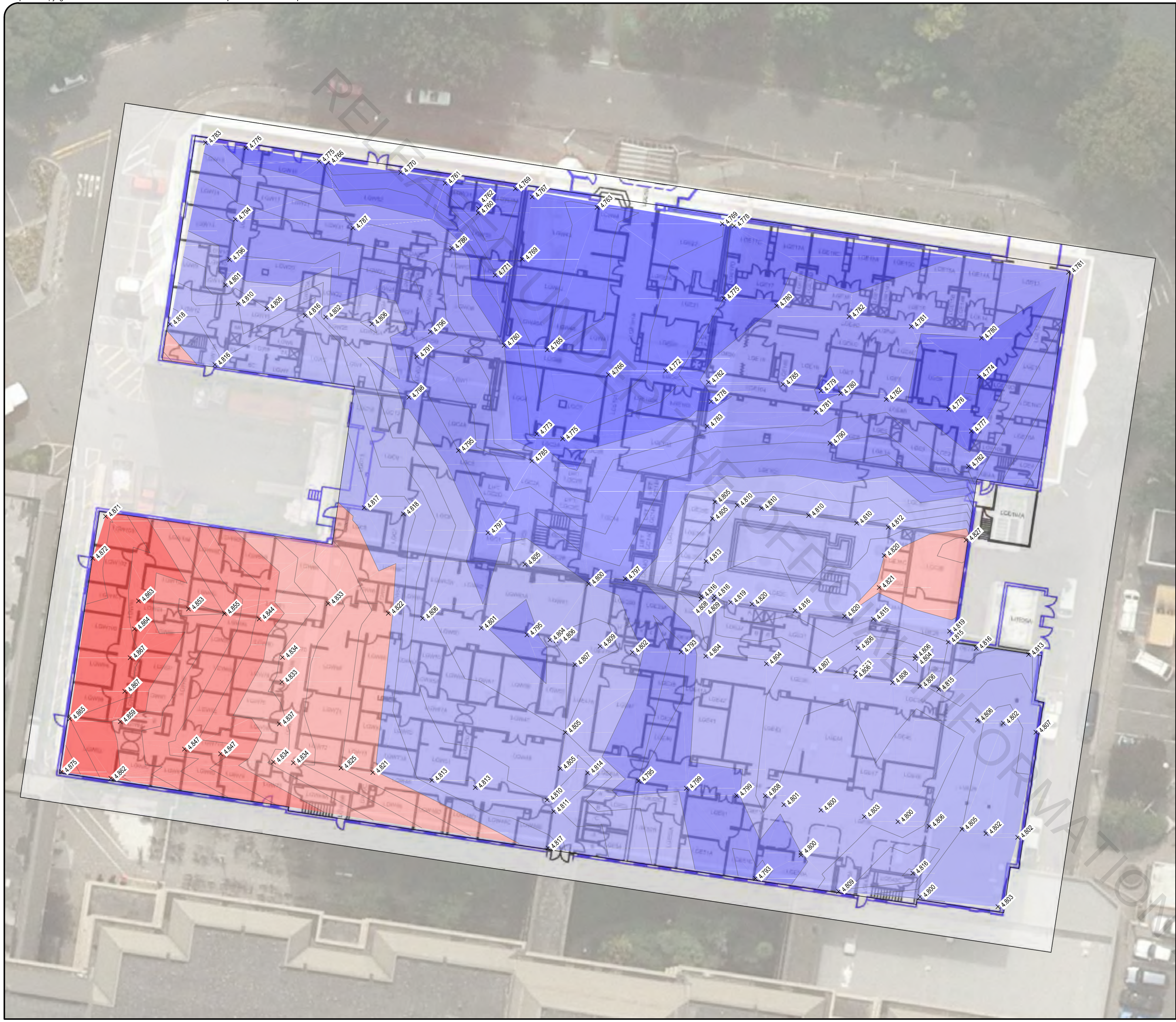
Job Name: CDHB Clinical Services Building

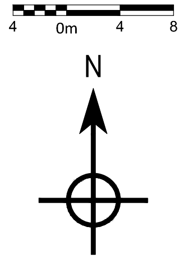
Job #: 106186.09 SSK#: CS-SS-06

Date: 16/12/11 Rev: B

APPENDIX E

Fox and Associates Levels Survey





Surface Legend			
#	Min. Depth (m)	Max. Depth (m)	Colour
1	4.760	4.780	
2	4.780	4.800	
3	4.800	4.820	
4	4.820	4.840	
5	4.840	4.860	
6	4.860	4.880	

NOTES:

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

SPATIAL DATA:

- Levels observed using Trimble DiNi Level with an expected accuracy of 1ppm (1mm per km) and an estimated accuracy of +/-2mm relative to benchmark nails.
- Horizontal position of levels is approximate only and relates to internal structures shown on building floor plans. Plans rotated and scaled into main drawing as cross-referenced underlays.
- Contour and slope analysis surfaces are derivative products based on the limited floor level positions observed, and provided only to enable further investigation of potential building issues and should not be used in isolation to draw conclusions on the state of the buildings surveyed.

FLOOR COVERINGS:

- Levels are reduced to slab level by applying the following vertical reductions based on field measurements:

Linoleum	0.002m
Carpet	0.006m
Tile	0.010m
Concrete slab	0.000m

DATE OF SURVEY:

- Field survey carried out on 18-19 June 2012.

COORDINATE SYSTEM:

- Mount Pleasant 2000

LEVEL DATUM:

- Origin of levels: Nail 20B RL = 4.671m, Fox and Associates CDHB Survey Control. NW corner of Riverside building, in footpath.
- New Zealand Vertical Datum 2009.

CONTOUR INTERVAL:

- Major Contour interval 0.010m
- Minor Contour interval: 0.005m

SERVICES DATA:

- Building plans have been sourced from CDHB plan 01021262 30/10/2008. Accuracy of these records cannot be guaranteed and these have been shown only for the purposes of plotting context.

CERTIFICATION:

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

____

Michael Martin - Registered Professional Surveyor

FOX AND ASSOCIATES
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0800 FOX SURVEY
P.O.Box 20-074
CHRISTCHURCH

CDHB - CHCH Hospital

Riverside LG Floor Levels

Address: 2 Riccarton Ave

Consent No: ----

Project Phase: ----

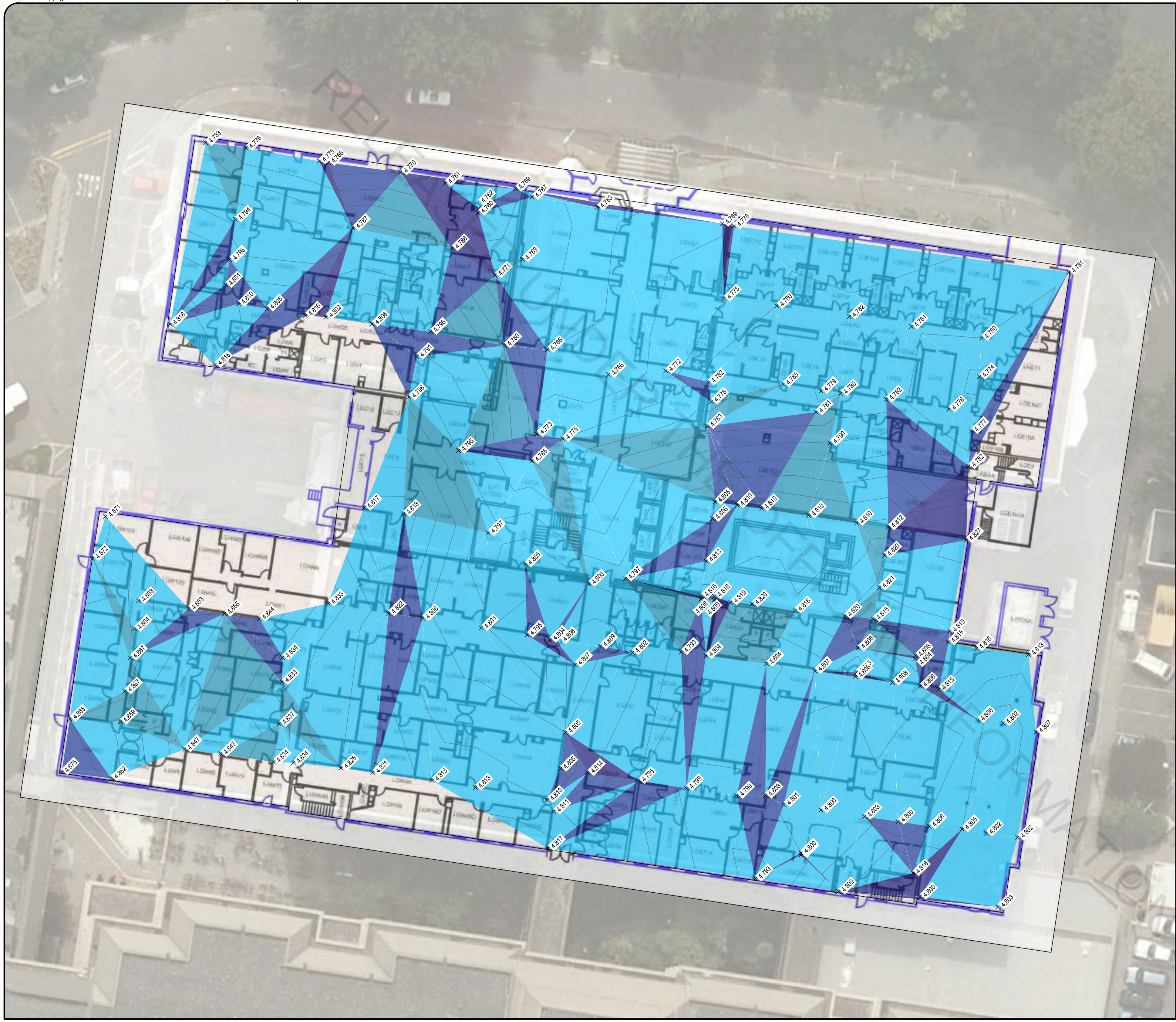
No.	Revision	Appd	Date
G			
F			
E			
D			
C	----	----	----
B	----	----	----
A	----	----	----

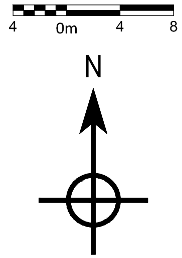
Scale (A1)	1:200
Reduced (A3)	1:400
Designed	----
Drawn	DHP
Checked	MJM
Date	20/06/2012

Job No. 2434C/6

Sheet No. 1 of 2

Rev. ----





Slopes Analysis

Number	Minimum Slope	Maximum Slope	Colour
1	0.01%	0.25%	
2	0.25%	0.30%	
3	0.30%	0.50%	
4	0.50%	2.84%	

FOX AND ASSOCIATES
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P.O.Box 20-074
CHRISTCHURCH

CDHB - CHCH Hospital

Riverside LG Floor Slope Analysis

Address: 2 Riccarton Ave
Consent No: ----
Project Phase: ----

No.	Revision	Appd	Date
G			
F			
E			
D			
C	----	----	----
B	----	----	----
A	----	----	----

Scale (A1)	1:200	Job No. 2434C/6
Reduced (A3)	1:400	
Designed	----	Sheet No. 2 of 2
Drawn	DHP	
Checked	MJM	Rev. ----
Date	20/06/2012	