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EXECUTIVE SUMMARY

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 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. 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. Once back-analysis and reinforcement capacity testing work has been completed regarding the floor slab cracks at the west end of the building, strengthening work to the floor diaphragm may be recommended.

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. PRE-EARTHQUAKE BUILDING CONDITION

1.1 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.

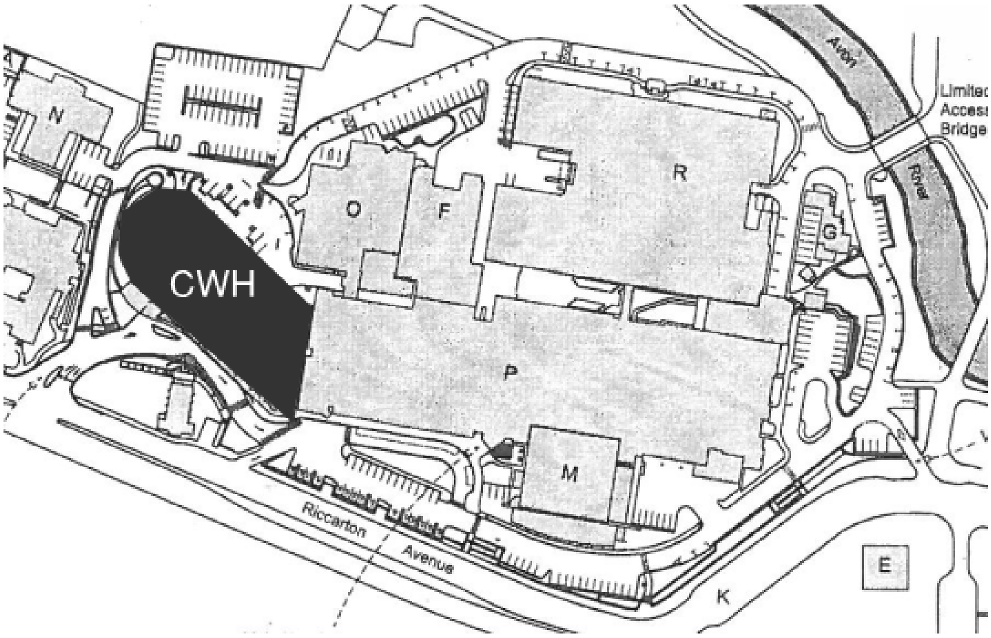

Figure 1-1: Location of Christchurch Women's Hospital

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 1-2: Photo of Christchurch Women's Hospital

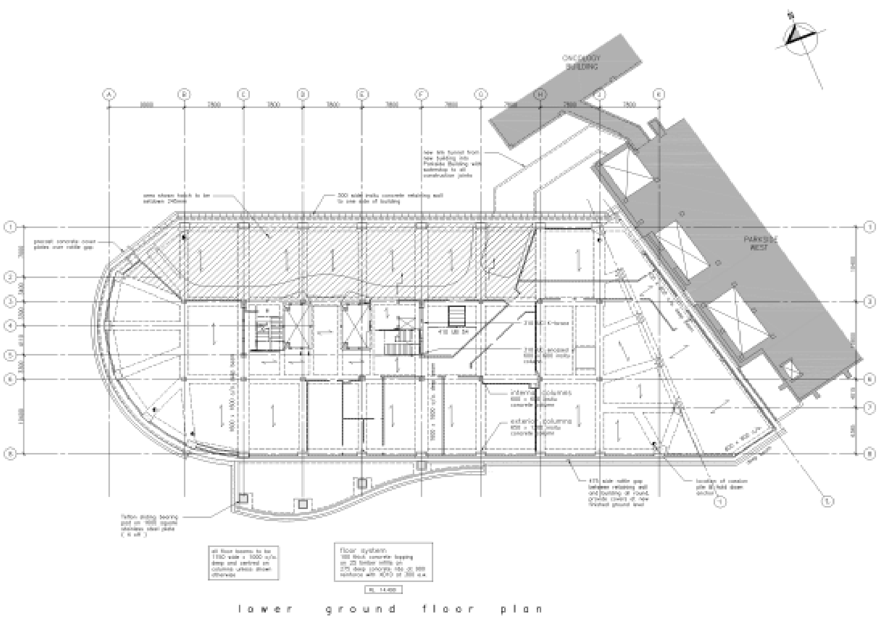

Figure 1-3: Structural plan of CWH

1.2 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.

1.2.1 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 1-1. Figure 1-4 shows the site specific spectrum at damping levels of 30% and 22% which reflects the energy absorption by the isolators at the Design-Basis Earthquake (DBE0 and Maximum Consider Earthquake (MCE) demand levels respectively. This is compared to the NZS1170.5:2004 spectrum at the same levels of damping.

Table 1-1: 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

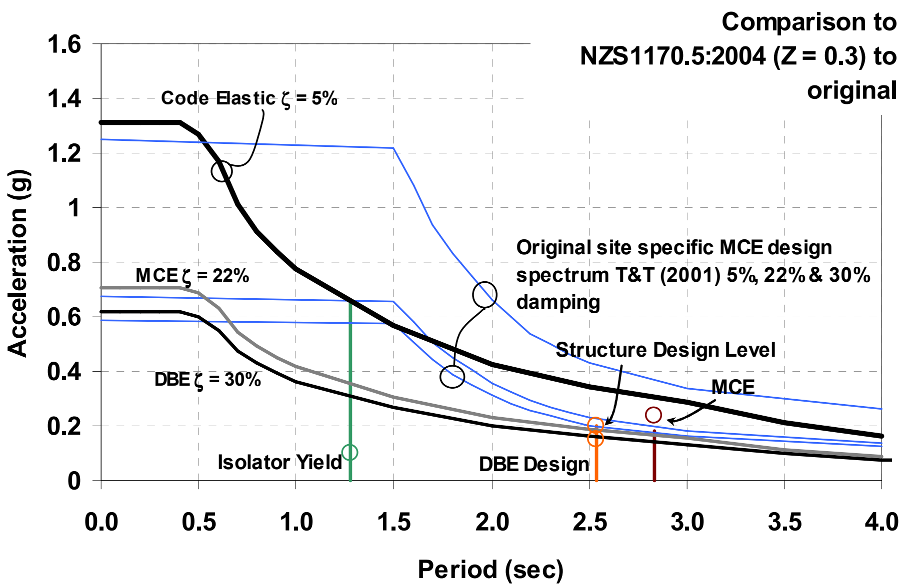

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Figure 1-4: NZS 1170:5:2004 acceleration spectra at damping levels of 5%, 30% and 22% which correspond to Serviceability (SLS), Design Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE) respectively. Circles indicate original design forces for each limit state.

The key points to draw from Figure 1-4 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 yield is 1.28 seconds. Once the isolators have yielded the effective period of the building becomes 2.54 seconds under DBE displacements, and 2.84 seconds under MCE displacements.

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 MCE level demands. The implication of this is that while the structure was designed assuming a design ductility demand of 1.4 (DBE) and 1.8 (MCE), 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.

2. 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 2.1 and 2.2 provided specific comments on probable building response during each of these events.

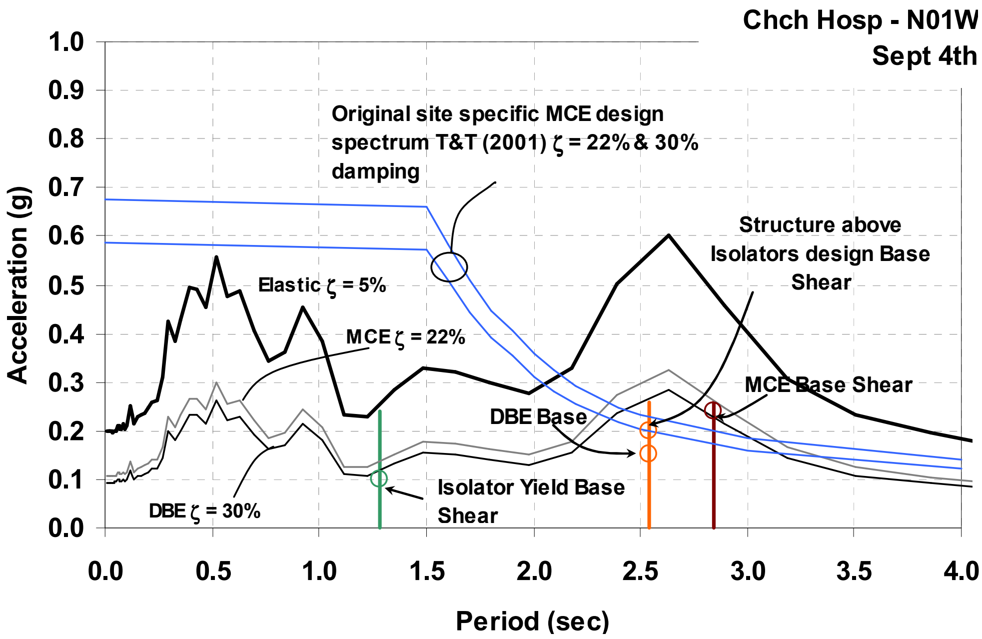
2.1 THE DARFIELD EARTHQUAKE

The Darfield earthquake had stronger ground motion in the north-south direction (N01W), than in the east-west (S89W). Figure 2-1 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 DBE.

At a DBE level the seismic demand shown by the "DBE ζ = 30%" curves ("ζ" represents damping) suggest that the building could have developed the DBE and MCE isolator base-shear levels and the design base-shear demand expected for upper structure. However as noted in Section 1.2.1 the as-designed overstrength has led to a structure that responds in an essentially elastic manner up to MCE 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 (DBE) and 22% damped (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.



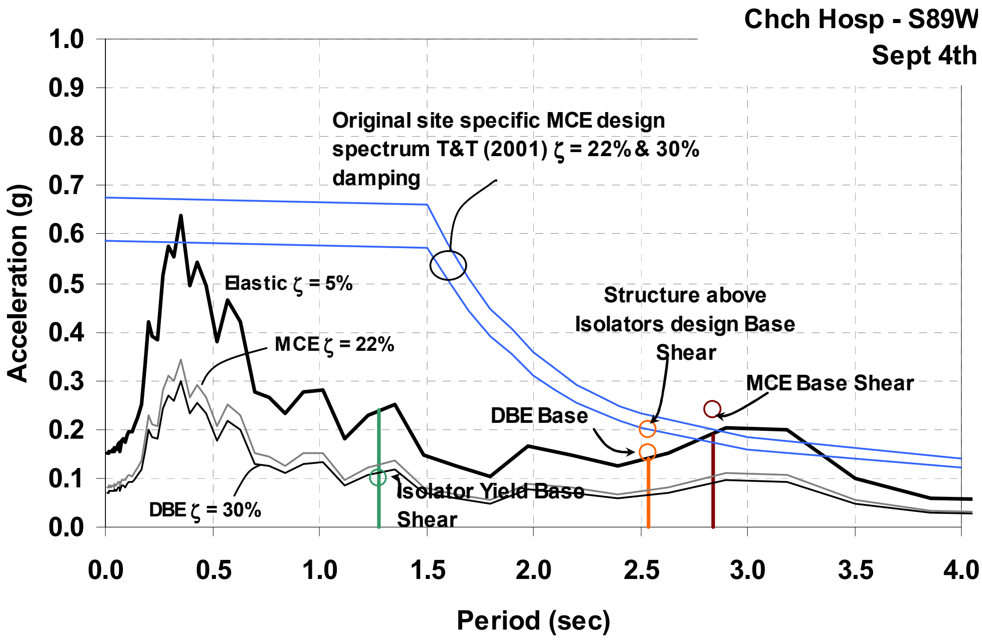


Figure 2-1: N01W and S89W components of the CHHC spectra for the Darfield Earthquake. Shown are the original site specific design spectrum curves for 30% and 22% damping levels corresponding to DBE and MCE earthquake events. The design base shears for each performance level are shown at their respective periods.

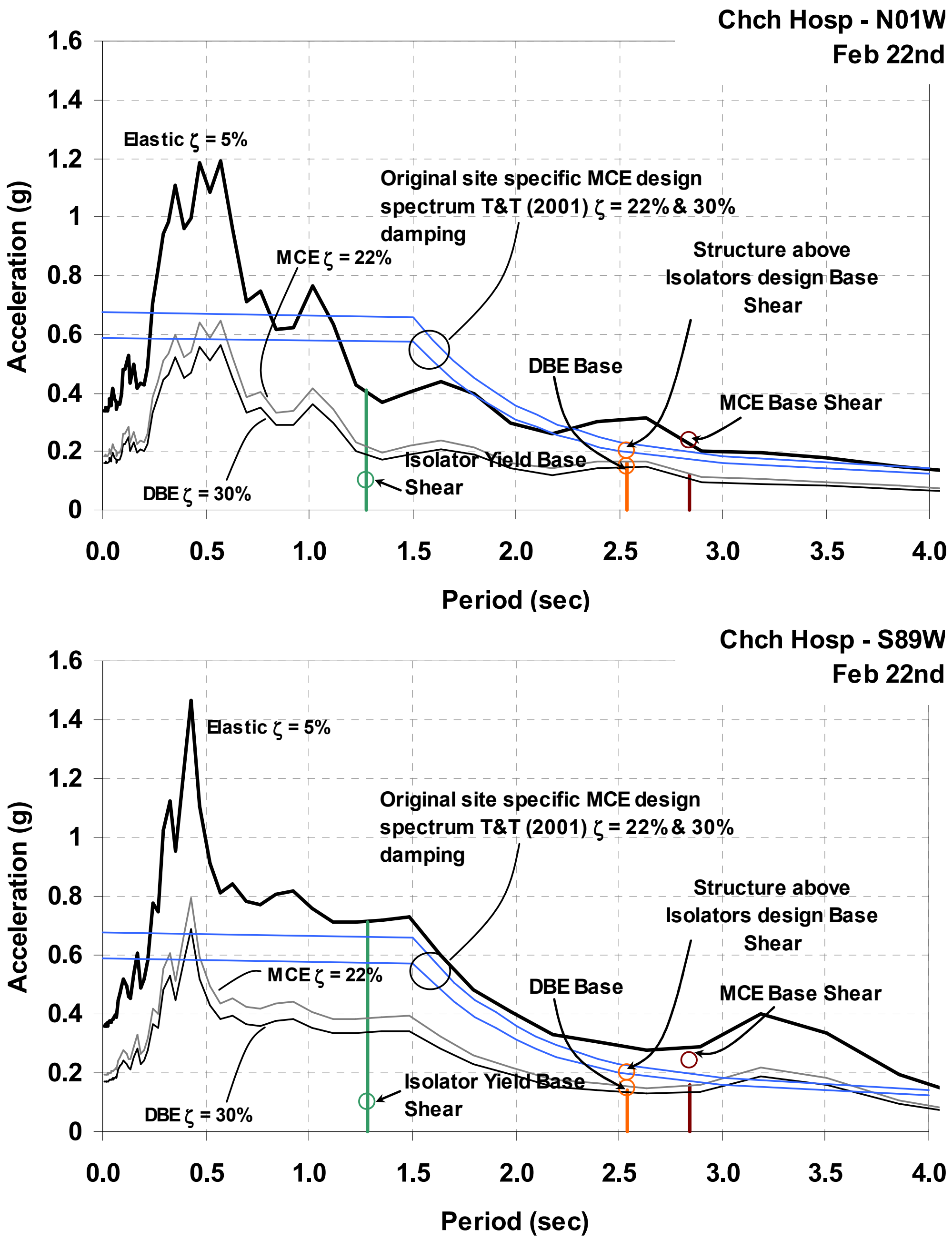


Figure 2-2: N01W and S89W components of the CHHC spectra for the Lyttelton Earthquake. Shown are the original site specific design spectrum curves for 30% and 22% damping levels corresponding to DBE and MCE earthquake events. The design base shears for each performance level are shown at their respective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

2.2 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 2.1 the sequential response of the building can be approximately interpreted from Figure 2-2(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 DBE level base shears, but not MCE level. 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.

2.3 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

Preliminary observations were carried out following the 4th September 2010 and 22th 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.

2.4 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 3, with reference plans describing the location labelling used, included in Appendix B. A full photographic record of the observations is available electronically on request.

2.5 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 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.

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 0.8 mm or more, it is suggested that temporary propping be installed to support the damaged ends of these units until full repairs can be finished.

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. 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.

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. At this stage representative locations at Level Three and Four have been marked for in-situ testing of the reinforcement to assess the residual steel capacity. A computer model of the Level Four perimeter moment frame has been 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 will provide indications as to whether the cracks have developed solely under earthquake loading or whether they were existing shrinkage cracks that have opened further under the earthquake demands, thus being damage due to the shaking response of the building.

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.

Given the cracking found in the topping slab it is recommended that the Level 3, 4 and 5 precast floor ribs be investigated at multiple locations around the floor plan area. It is possible that these will have developed cracks similar to those found in the basement level, which may also need repair work.

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. It is recommended that further investigation is carried out to confirm if this was existing or if it is a result of the earthquake. Similarly further investigation is needed to identify other locations that have cracked and may need remediation.

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**.** 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.

2.6 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.

2.7 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.

2.8 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.

2.9 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, no in-situ materials testing was carried out.

2.10 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 1.2.1 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.

3. 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, whether earthquake remediation or make-better, 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 3-1 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.

Table 3-1: 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. 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.	Remove carpet/vinyl to inspect the top of the slab along full length of GL 3 & 6, both sides of beam below. Epoxy inject all cracks that are greater than 0.3mm in width. Refer to HCG Specification	
1.3. Cracking up to 0.6mm in topping slab + Inspection	LEVEL 4: Cracking in topping slab parallel to beams on GL 3 & 6. Cracks observed on north and south	Some locations have been epoxy injected already. Where not already remediated remove carpet/vinyl to inspect slab along full length of GL	See above

Damage	Locations	Recommendation	Example
	sides of beams.	3 & 6, both sides of beam below. Epoxy inject all cracks that are greater than 0.3mm in width. Refer to HCG Specification	
1.4. Cracking up to 1.2mm in topping slab + Inspection	LEVEL 4:	Multiple rooms between Grid A and C showing extensive cracks, some linear/parallel and others diagonal and intersecting. Epoxy inject all cracks that are greater than 0.3mm in width. Refer to HCG Specification, however do not inject before reinforcement testing is complete. Further strengthening may be required once all analysis and testing has been completed.	
1.5. Inspection	LEVEL 5:	Room 5080 observed floor cracks similar to L4 and L5. Further inspection of rooms required to confirm full extent of cracks.	

Damage	Locations	Recommendation	Example
1.6. 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.	Remove carpet/vinyl to inspect the slab along full length of GL 3 & 6, both sides of beam below. Epoxy inject all cracks that are greater than 0.3mm in width. Refer to HCG Specification	See above
1.7. 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
1.8. 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	 
1.9. 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
1.10. 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	LEVEL 3, 4 & 5: Possible cracks near supports of precast floor ribs.	Investigation of precast floor ribs supporting L3, L4 and L5 required to confirm if similar cracks near the supports is present (as seen in basement)	No photo
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	

Damage	Locations	Recommendation	Example
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			
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	
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	

4. 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.

4.1 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 preliminary scheme for additional seating angles is provided in sketch SKS-C1 in Appendix C.

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.

The cracking noted in the floor slab of Level Three, Four and Five requires epoxy injection in the short-term either as a remediation for earthquake damage or bettering works if the cracks were pre-existing. Going forward from there it may be necessary to provide a simple retrofit solution to increase the tie capacity of the floor slabs to prevent this type of crack behaviour developing under large earthquakes. Such retrofit may involve carbon-fibre strips being set into the topping slab. The necessity for this approach will be determined from the in-situ reinforcement testing to be carried out, along with analysis of the lateral frame behaviour under the seismic demands.

5. REFERENCES

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

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

3. 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.

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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.

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APPENDIX A

Record of Observations

DRAFT 4/5/2012

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

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

Refer to Table 3.1 and HCG Specification for repair details

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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
L Grnd	L047		Tie Down anchorage	No indication of damage or loss of tensioning	N		20120327: Anchorhead_1
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
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

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Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
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	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

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Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
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
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
L5	5052		Stair mid landing	Crack 0.7-0.8 mm	Y	See full stair repair desc.	11-12-20: 011, 012

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Level	Room Number	Location	Building Element	Observations	Repair Required	Repair	Photo Reference
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
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
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

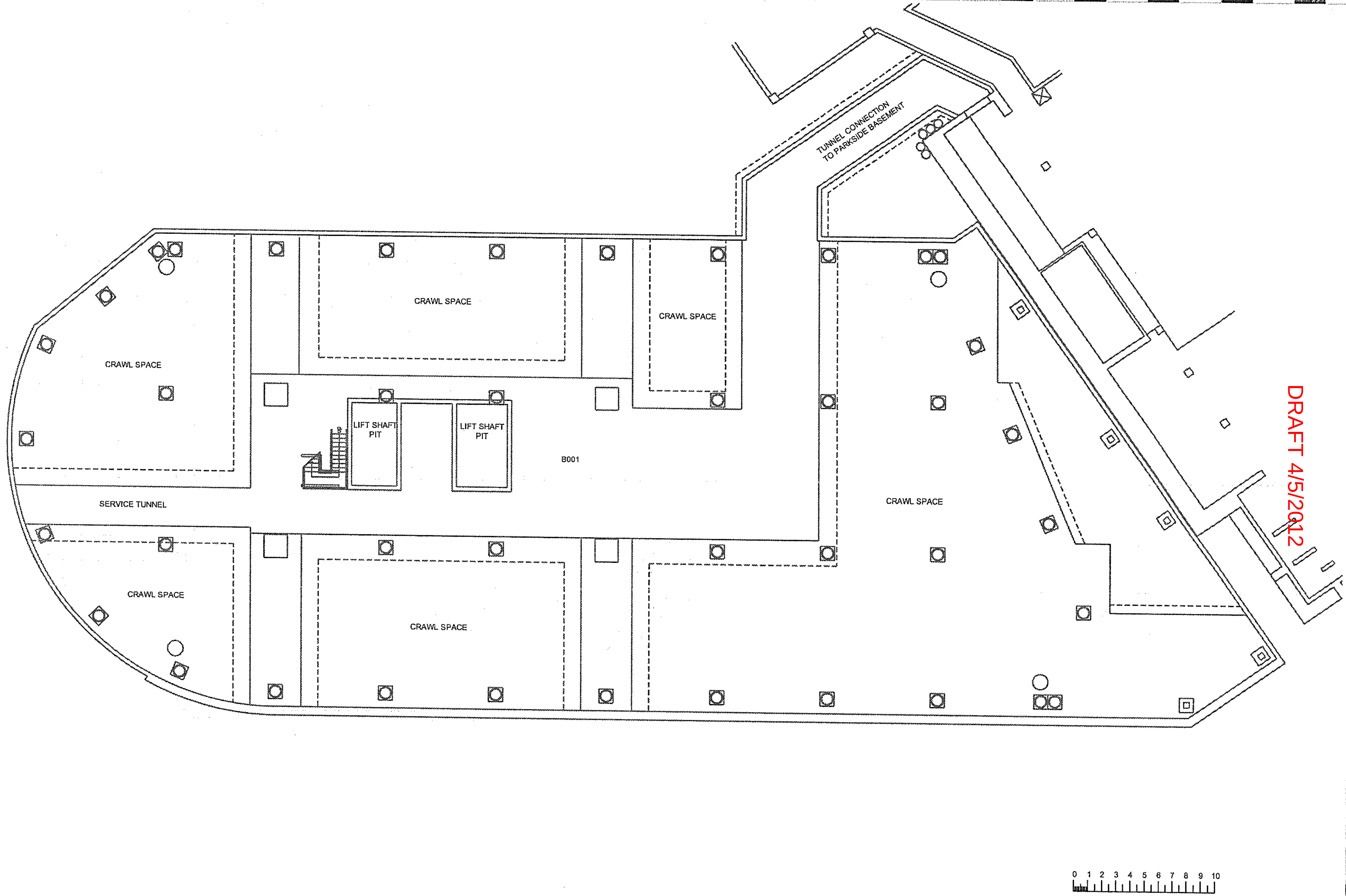
Reference Plans

DRAFT 4/5/2012

Womens

Space Number – Floor Plans

(Print at A3 for 1:200 scale)



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WOMENS - BASEMENT
SPACE NUMBERS

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	XREFED DRAWING/S:	01291006		

Maintenance and Engineering Department - Christchurch Hospital

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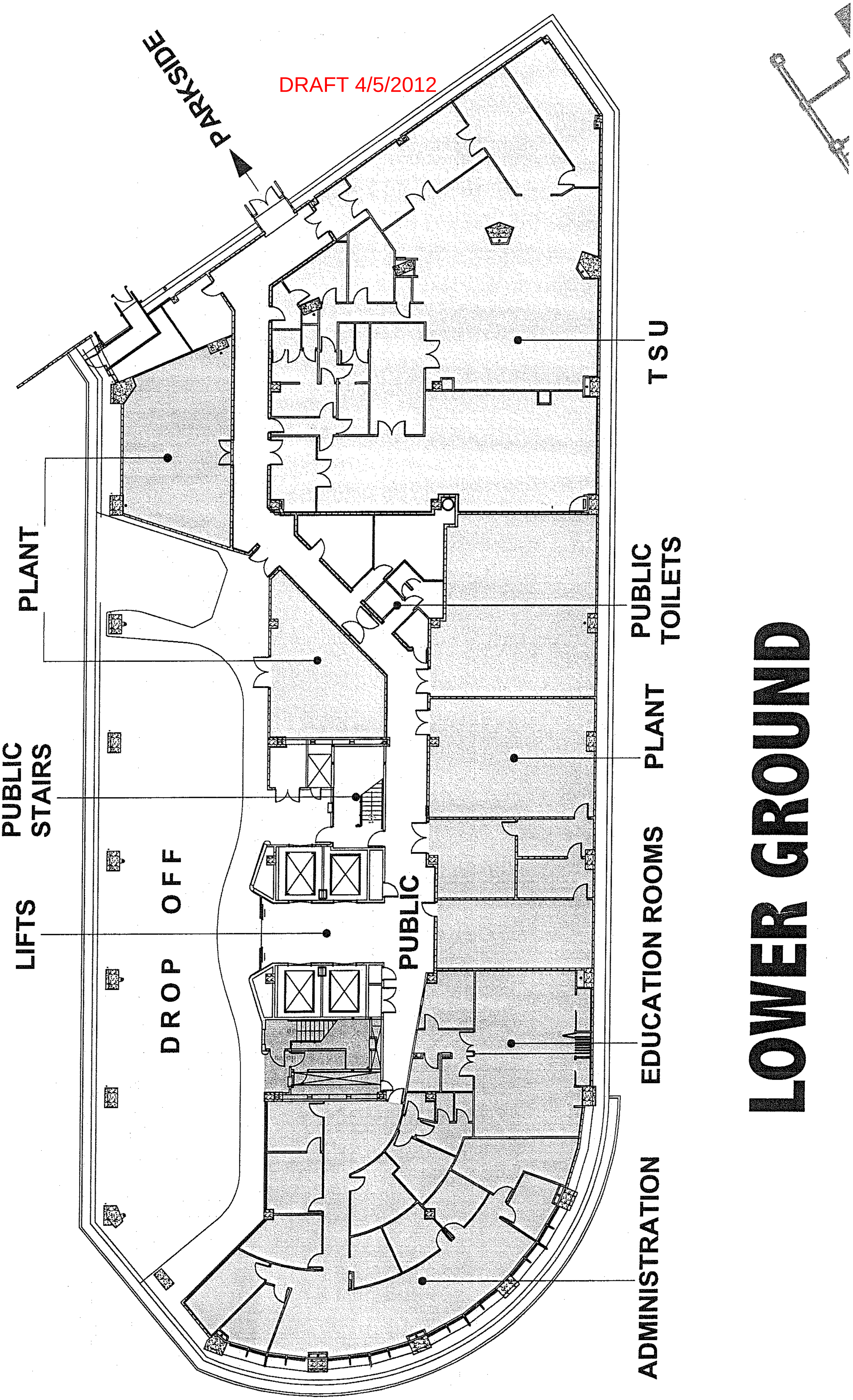

LOWER GROUND

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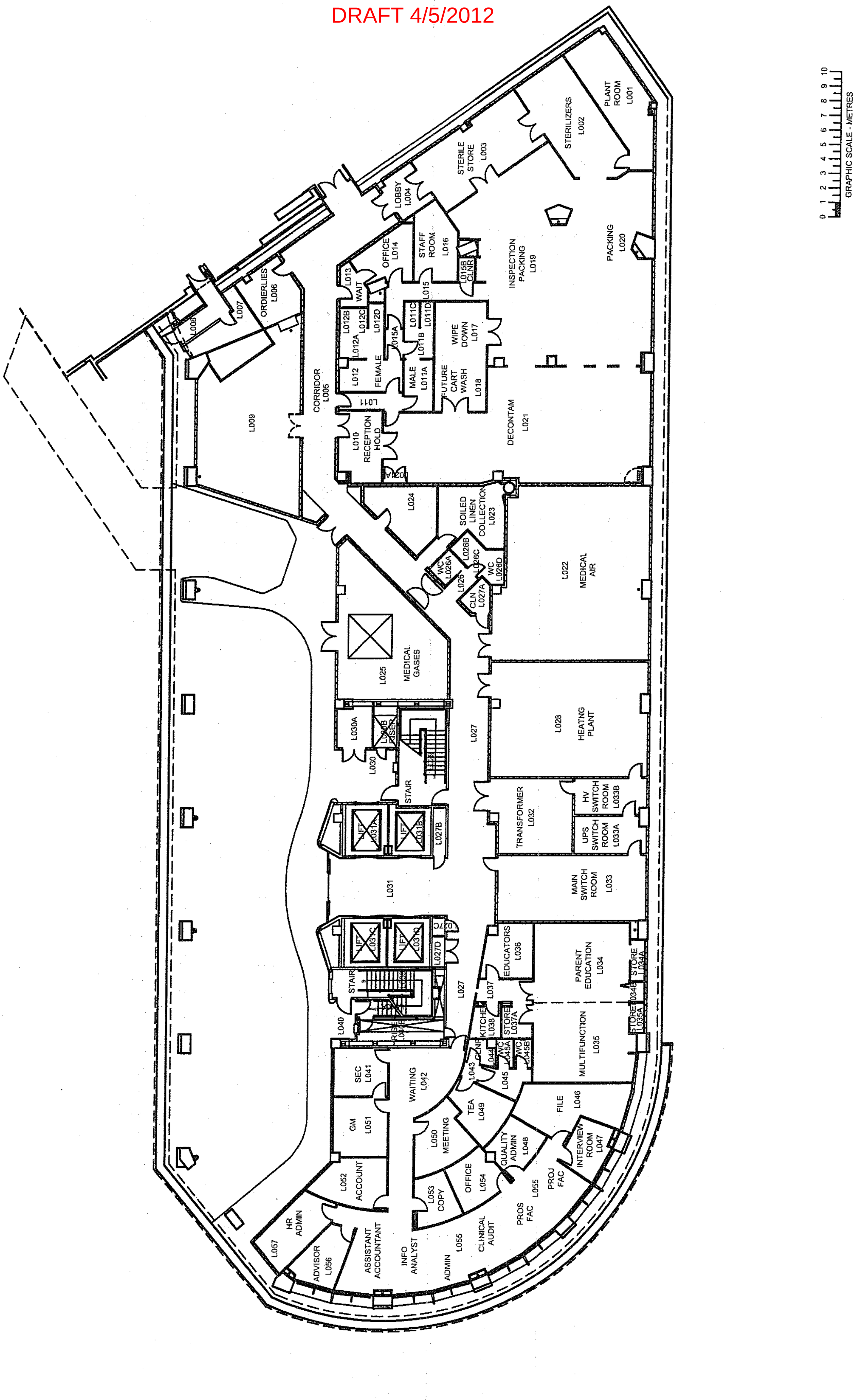

CHRISTCHURCH HOSPITAL
WOMENS - LOWER GROUND FLOOR
SPACE NUMBERS

Canterbury District Health Board
Te Poari Hauora ō Waitaha

Maintenance and Engineering Department - Christchurch Hospital

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CHECKED:	
DATE:	26-10-05
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DWG No	01291002
SCALE	1:250 AT A3
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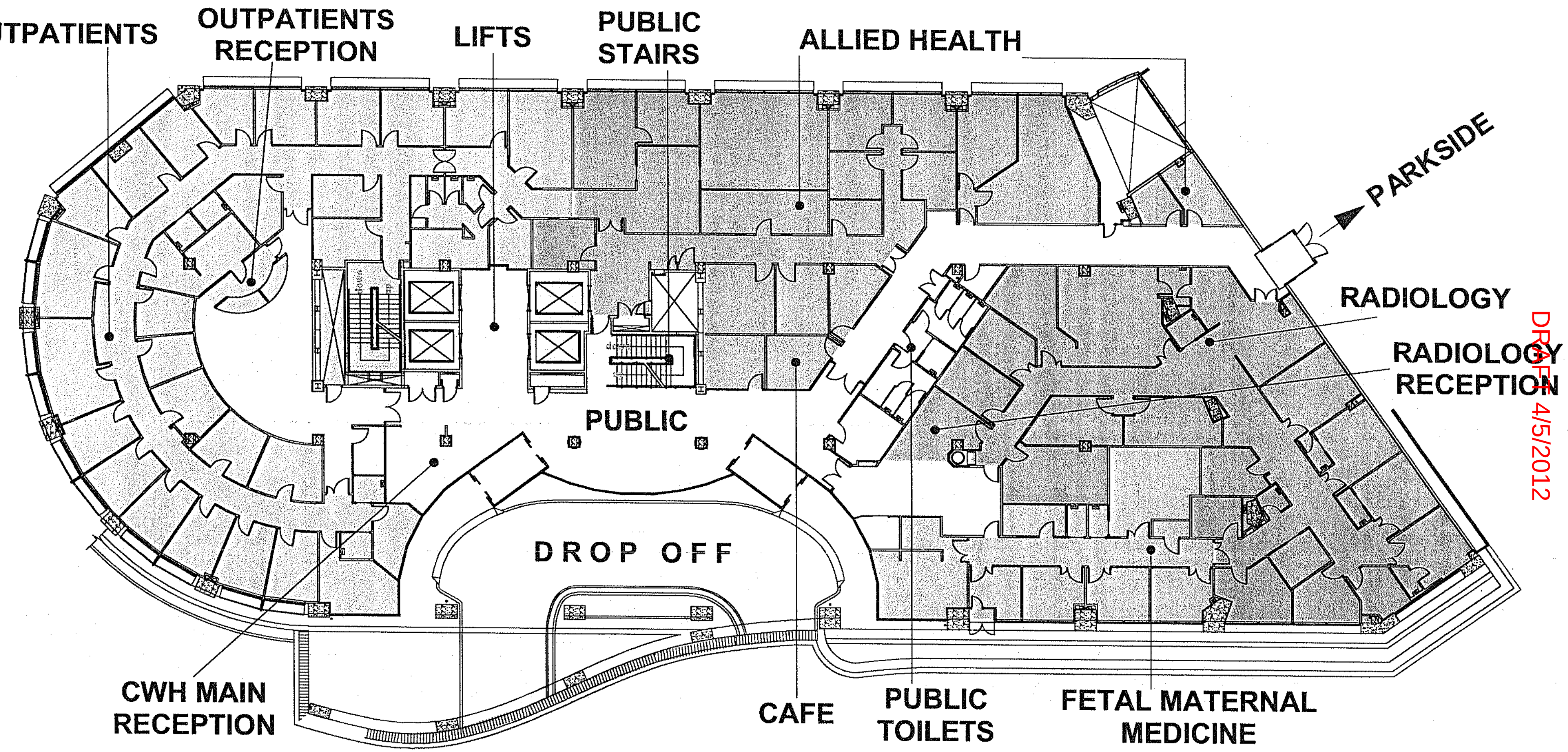
DRAFT 4/5/2012



GROUND FLOOR

OUTPATIENTS & RADIOLOGY

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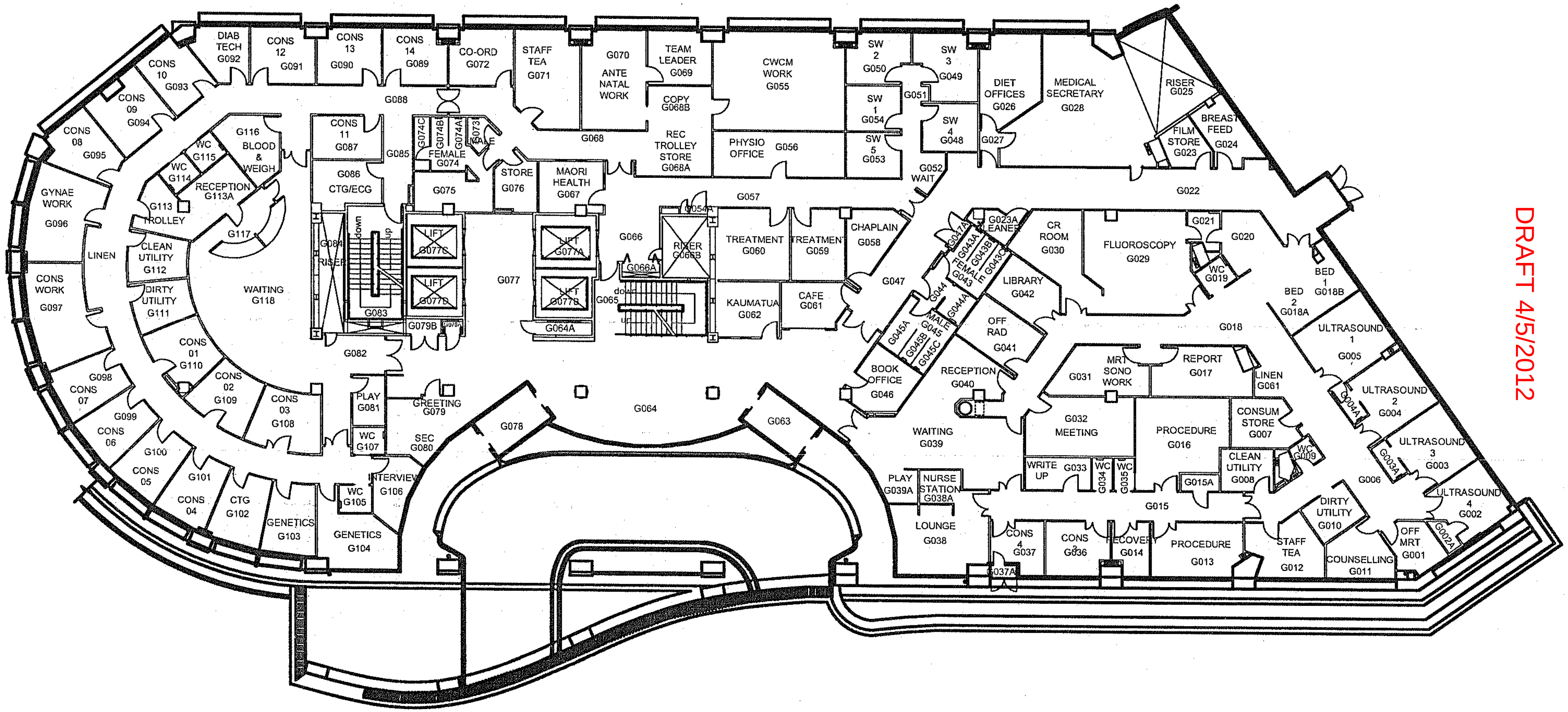

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GRAPHIC SCALE - METRES

CHRISTCHURCH HOSPITAL
WOMENS - GROUND FLOOR
SPACE NUMBERS

Canterbury
District Health Board
Te Poari Hauora ō Waitaha

DESIGNED:	
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CHECKED:	
DATE:	26-10-05
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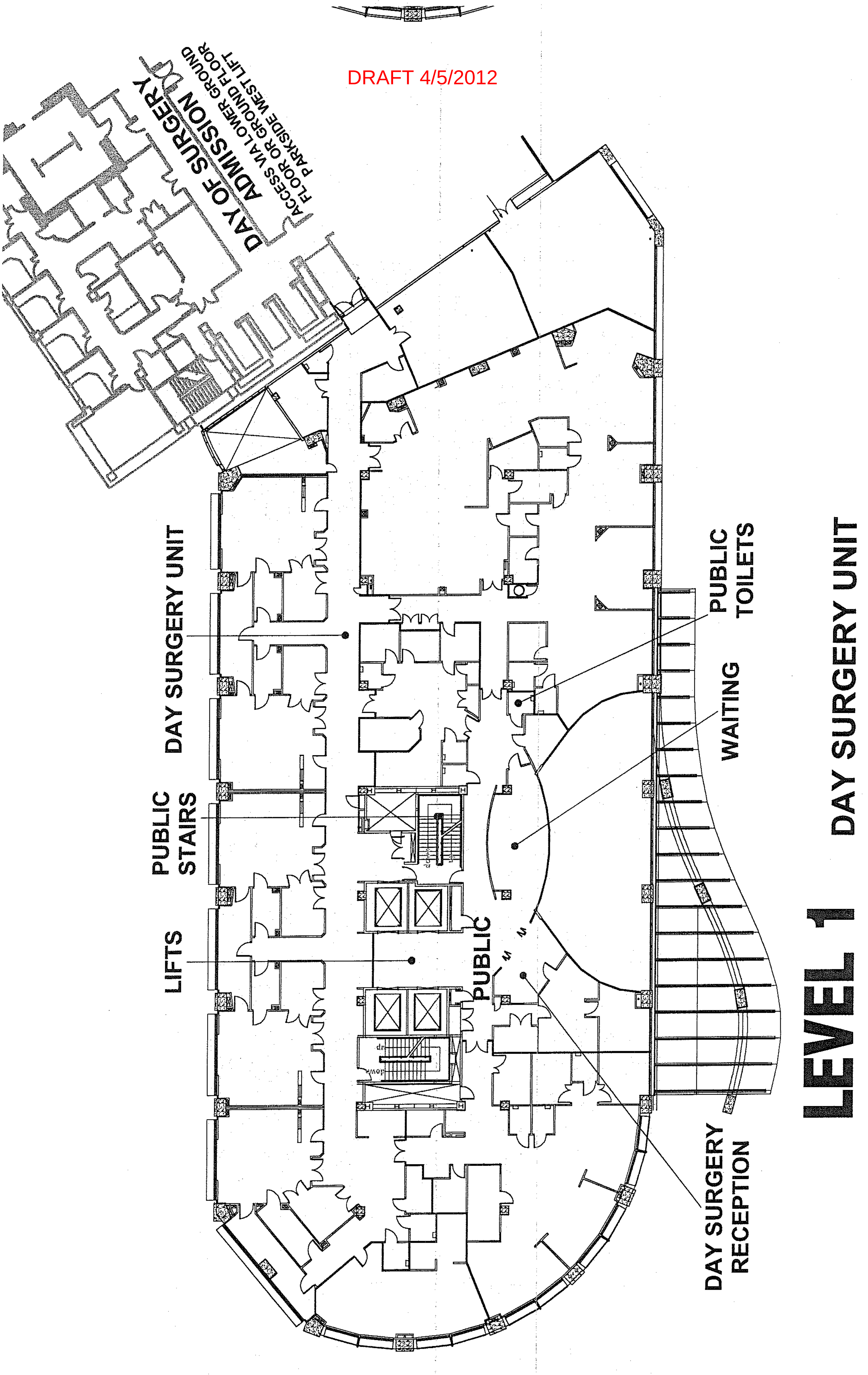

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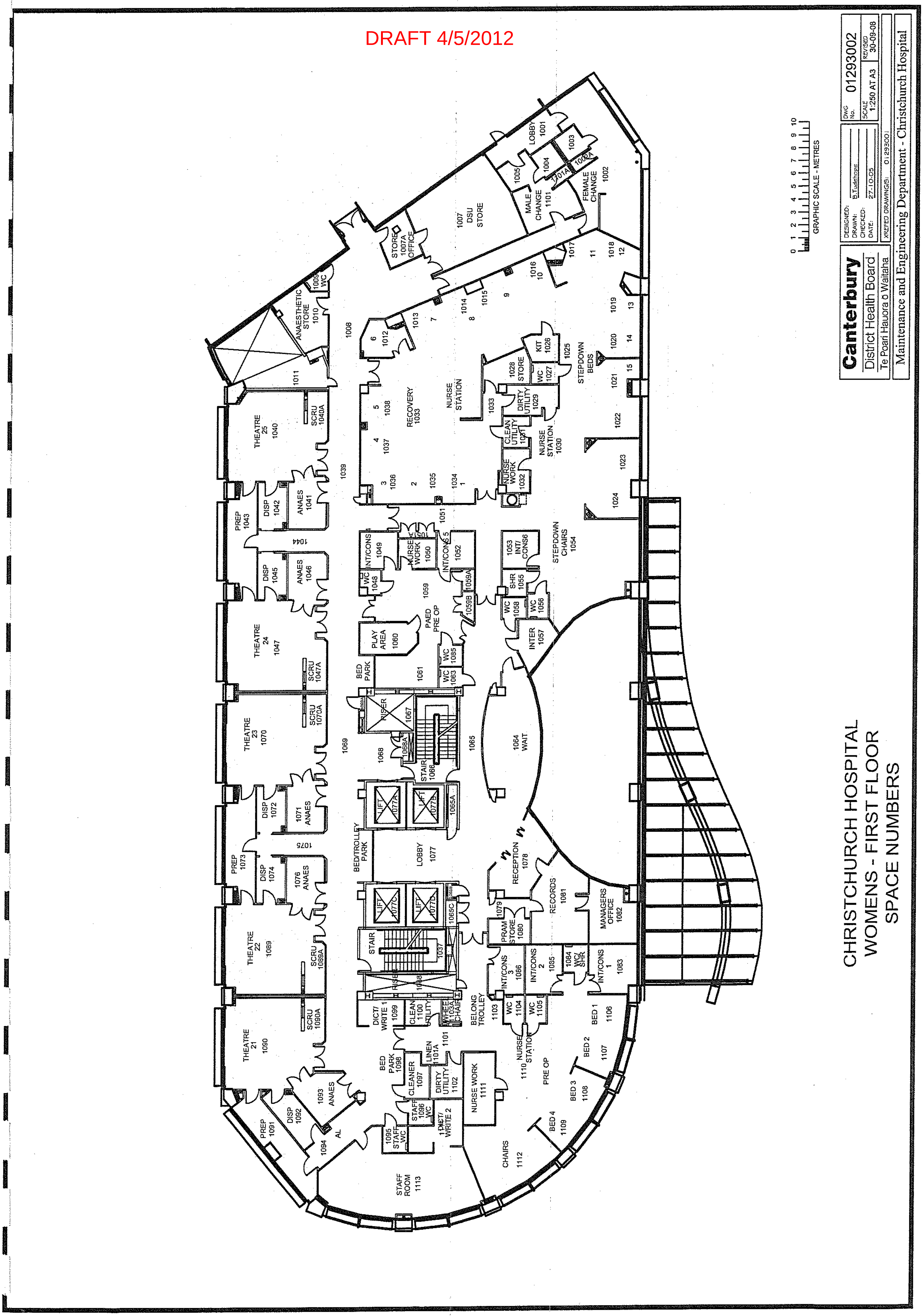

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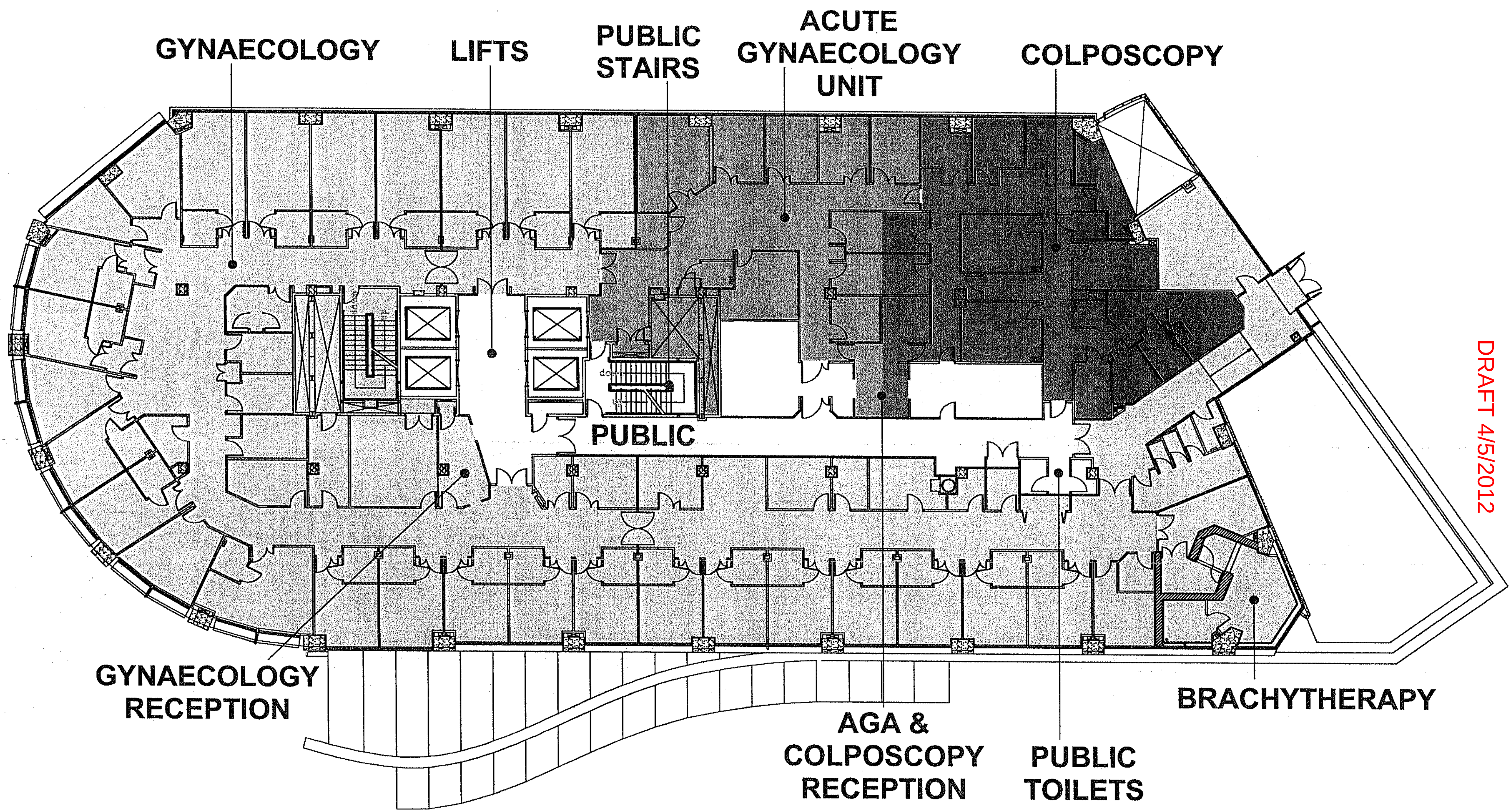

LEVEL 2 GYNAECOLOGY

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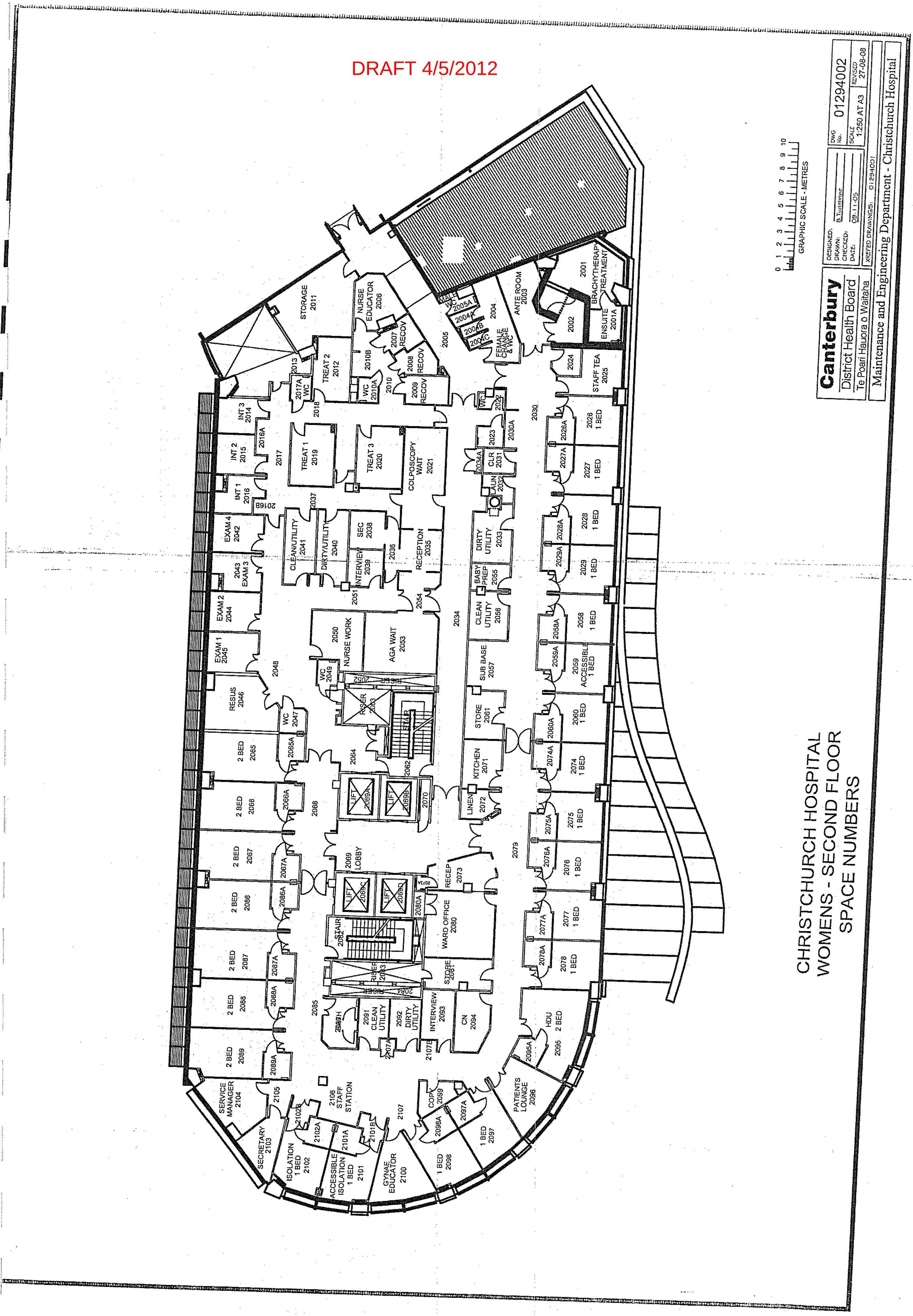

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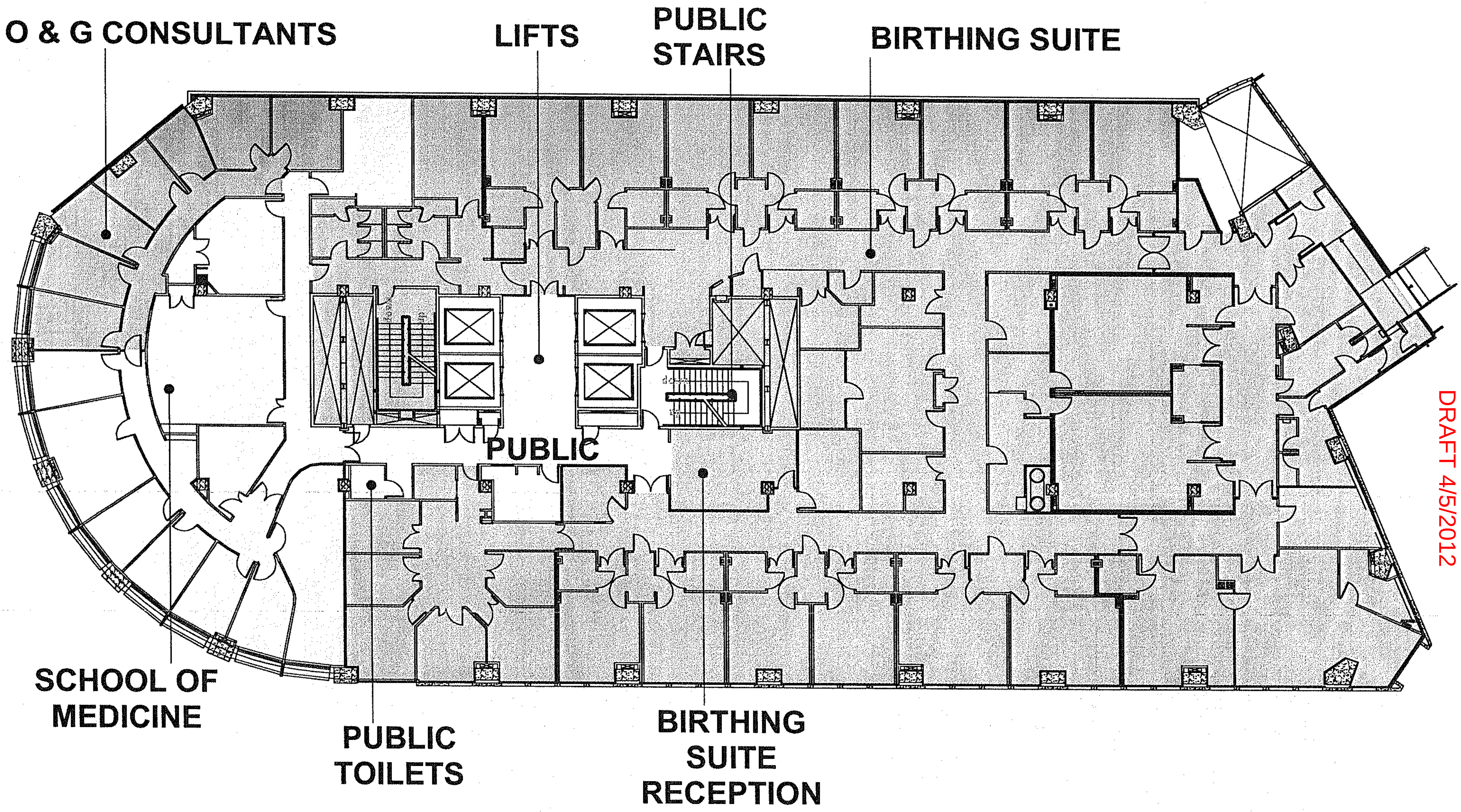

LEVEL 3

BIRTHING SUITE

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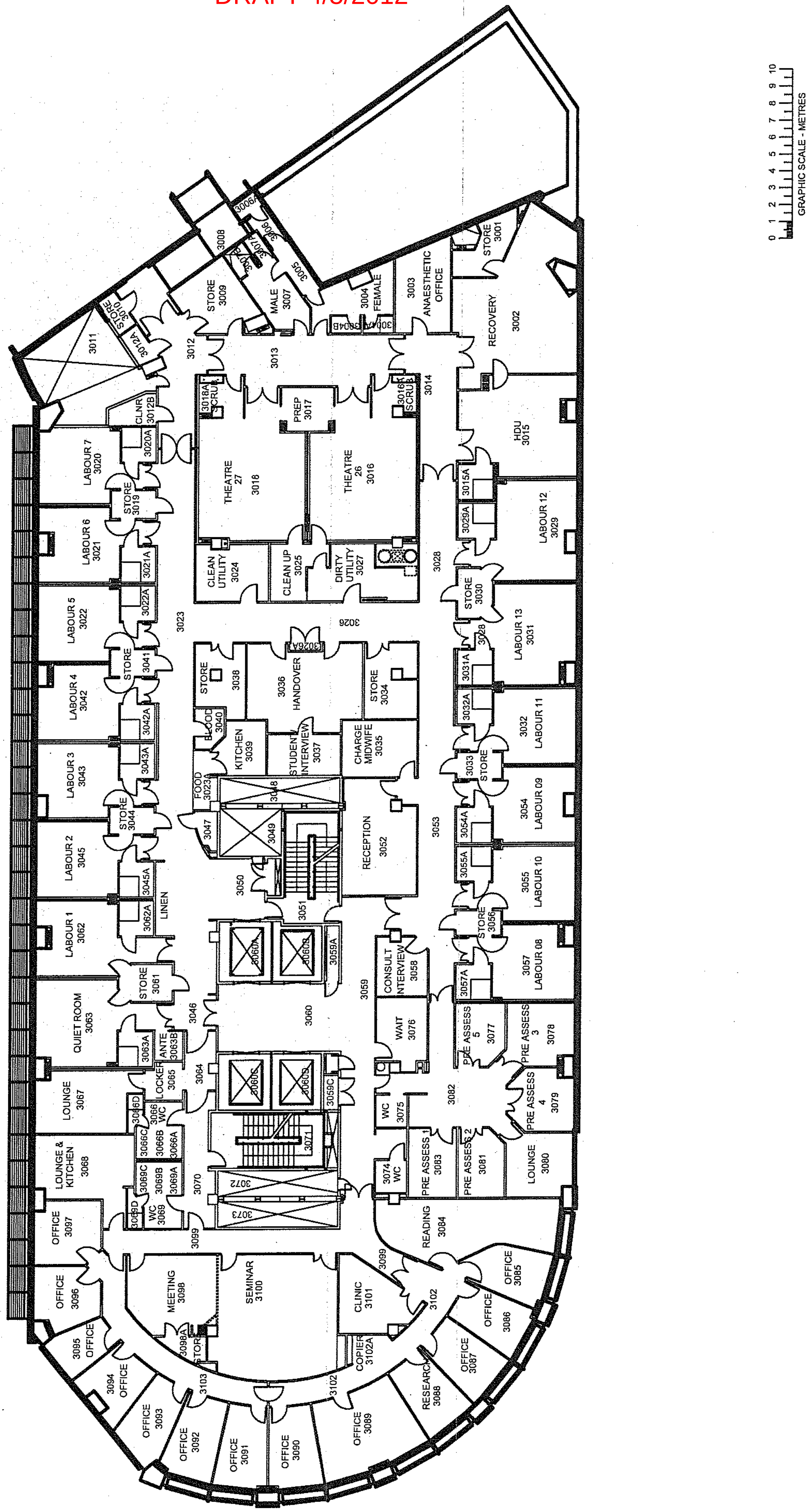

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CHECKED:
DATE: 16-11-05

XREFED DRAWINGS: 01295001

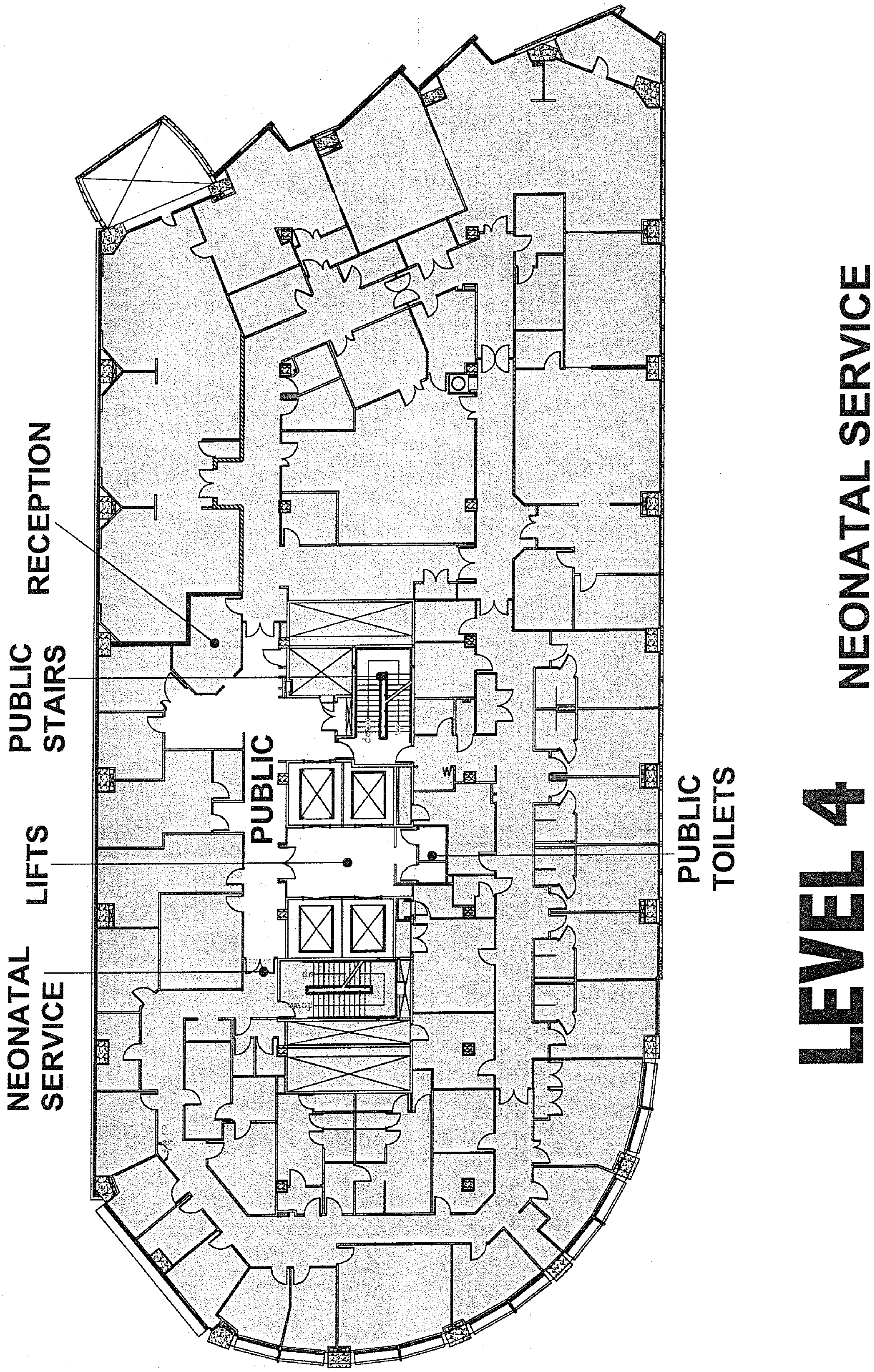
Canterbury District Health Board

Te Poari Hauora ō Waitaha

Maintenance and Engineering Department - Christchurch Hospital

CHRISTCHURCH HOSPITAL
WOMENS - THIRD FLOOR
SPACE NUMBERS

DRAFT 4/5/2012



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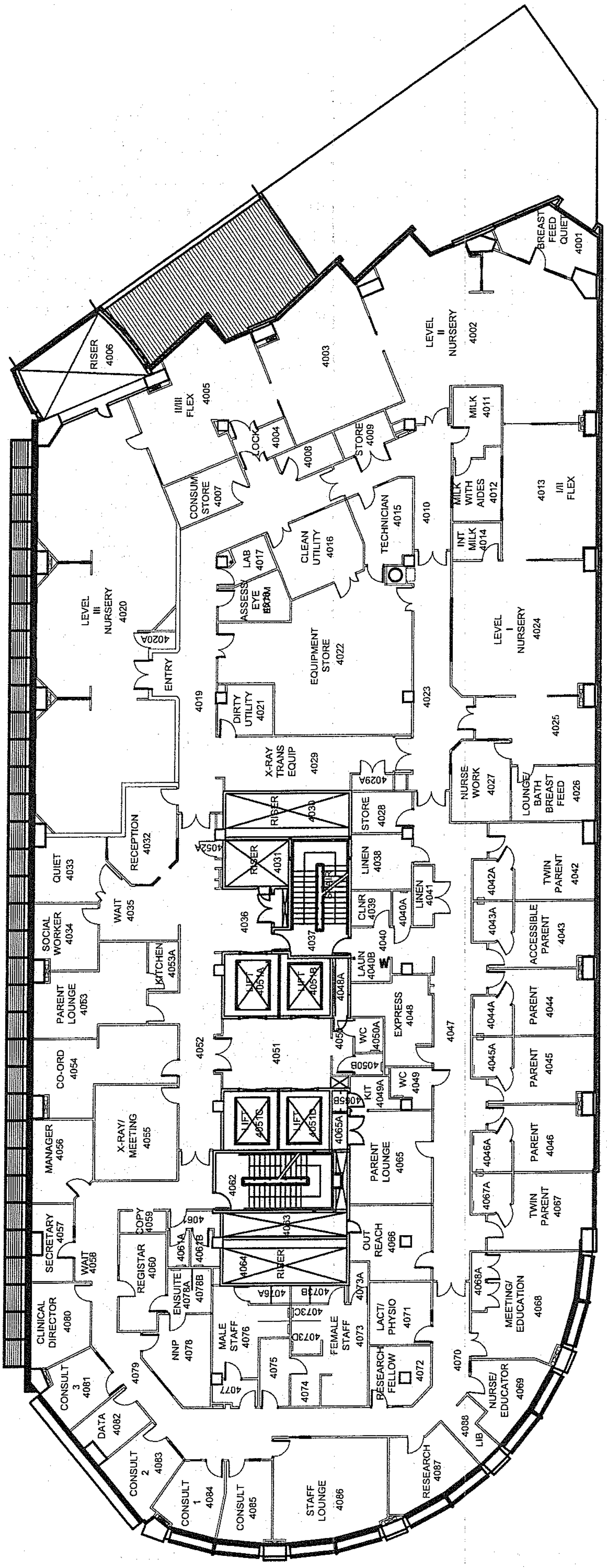

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GRAPHIC SCALE - METRES

Canterbury District Health Board
Te Poari Hauora ō Waitaha

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CHECKED:
DATE: 16-11-05
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DWG No. 01296002
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REVISED 18-11-05

Maintenance and Engineering Department - Christchurch Hospital

CHRISTCHURCH HOSPITAL
WOMENS - FOURTH FLOOR
SPACE NUMBERS

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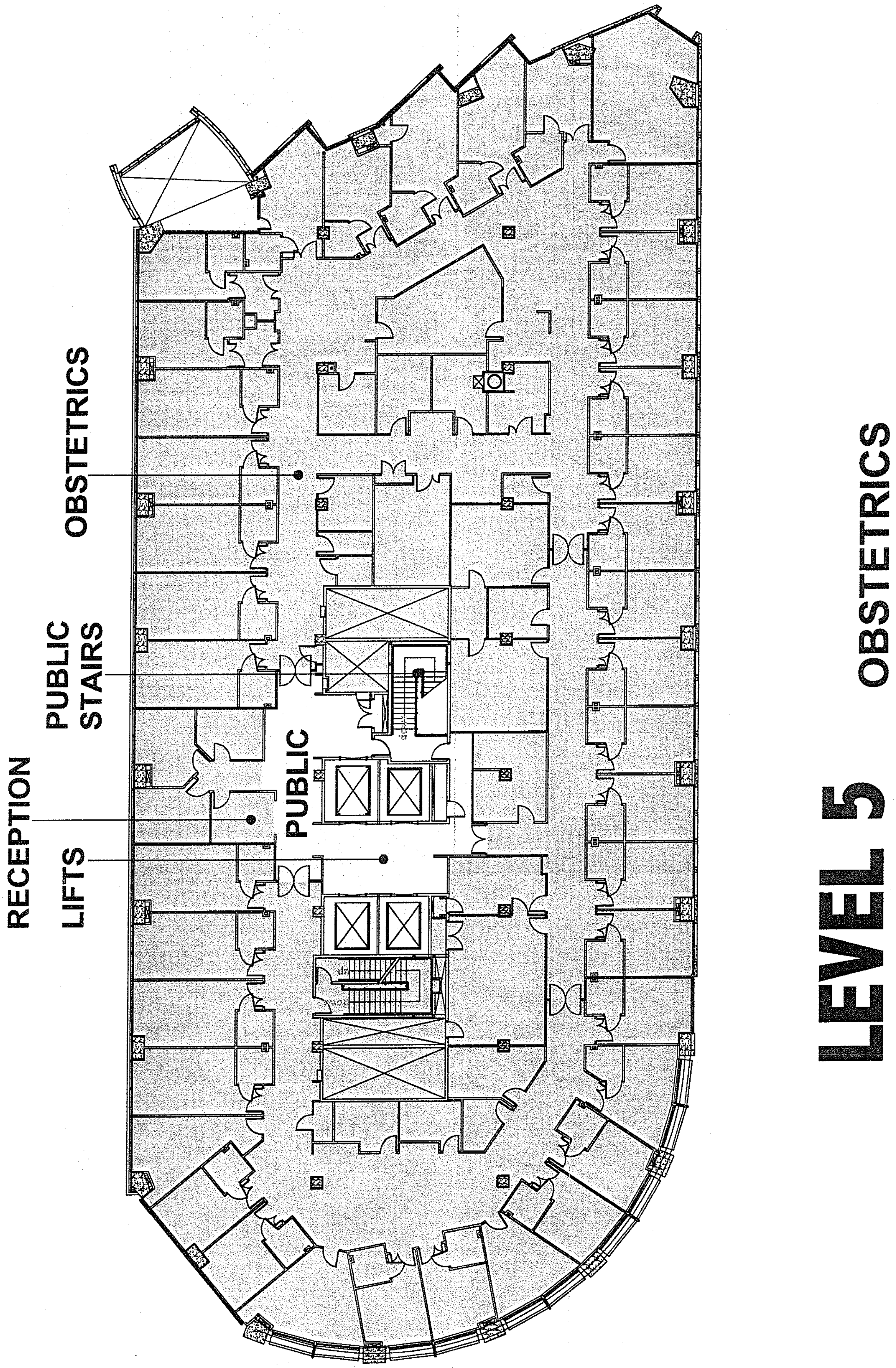
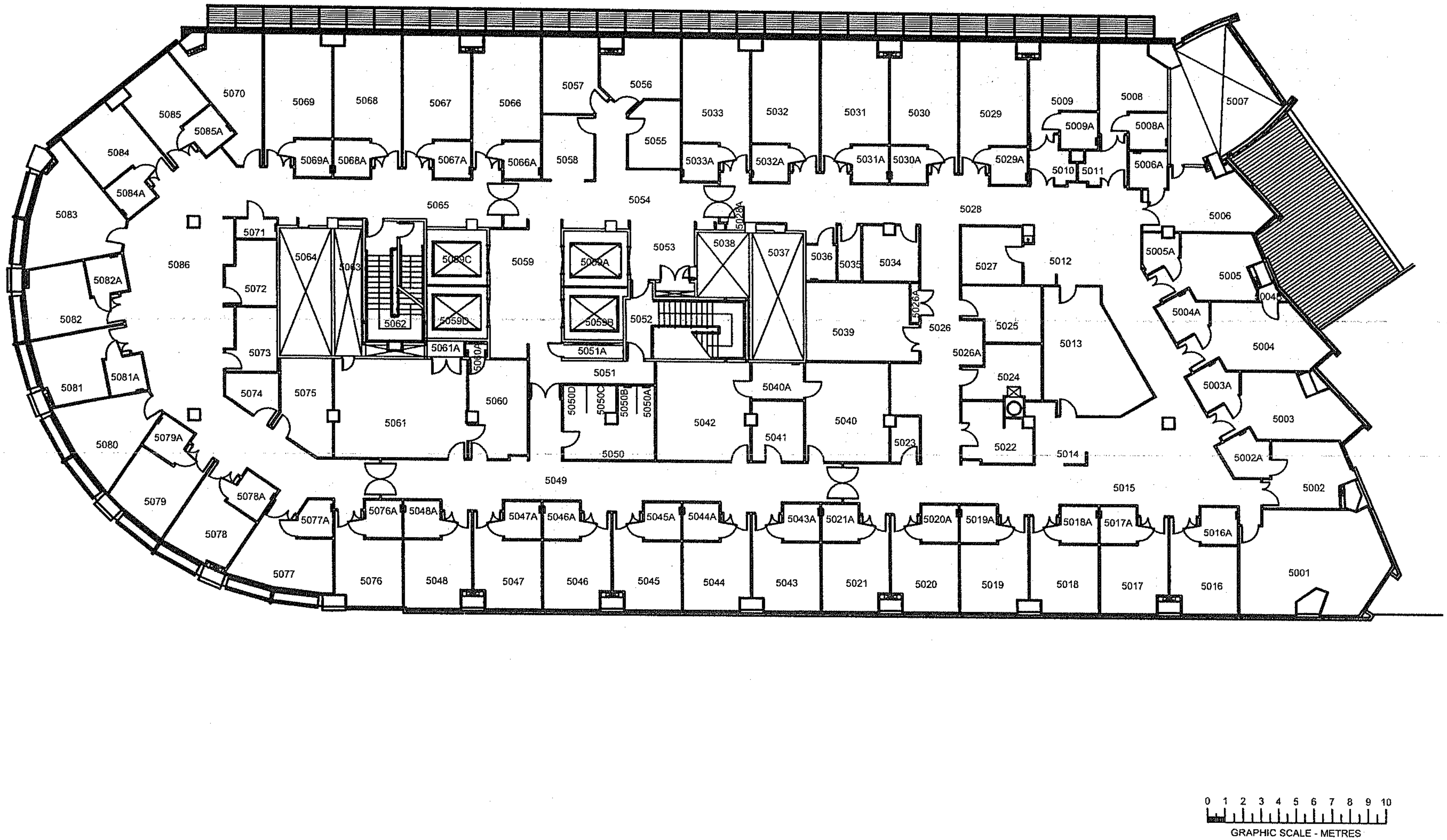


DRAFT 4/5/2012



CHRISTCHURCH HOSPITAL
WOMENS - FIFTH FLOOR
SPACE NUMBERS

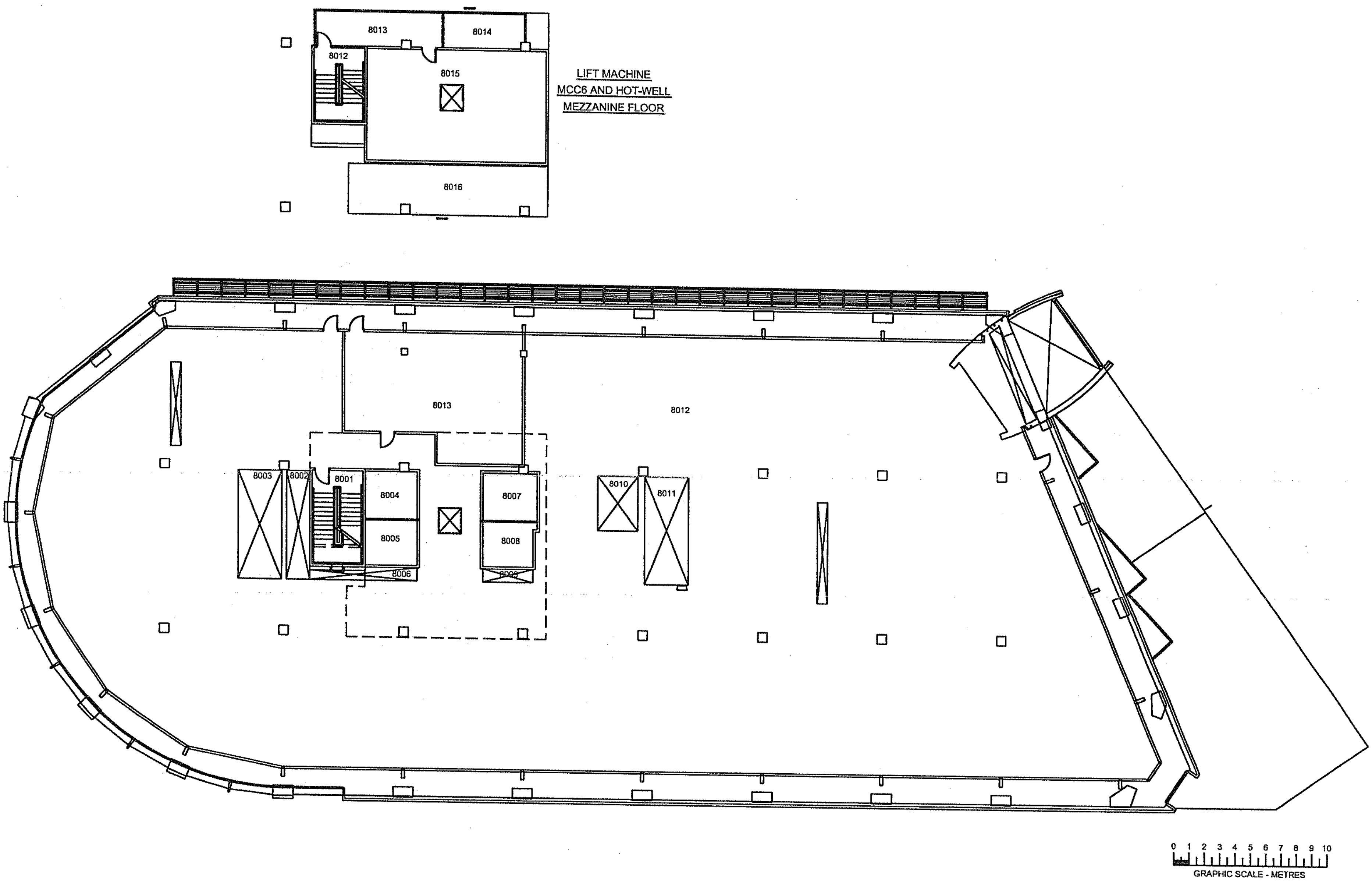
Canterbury
District Health Board
Te Poari Hauora ō Waitaha

DESIGNED:		DWG No.	01297002
DRAWN:	B.Tudehope	SCALE	1:250 AT A3
CHECKED:		REVISED	18-11-05
DATE:	18-11-05		
XREFED DRAWING/S:	01297001		

Maintenance and Engineering Department - Christchurch Hospital

BUI.RIC002.0001.56

DRAFT 4/5/2012

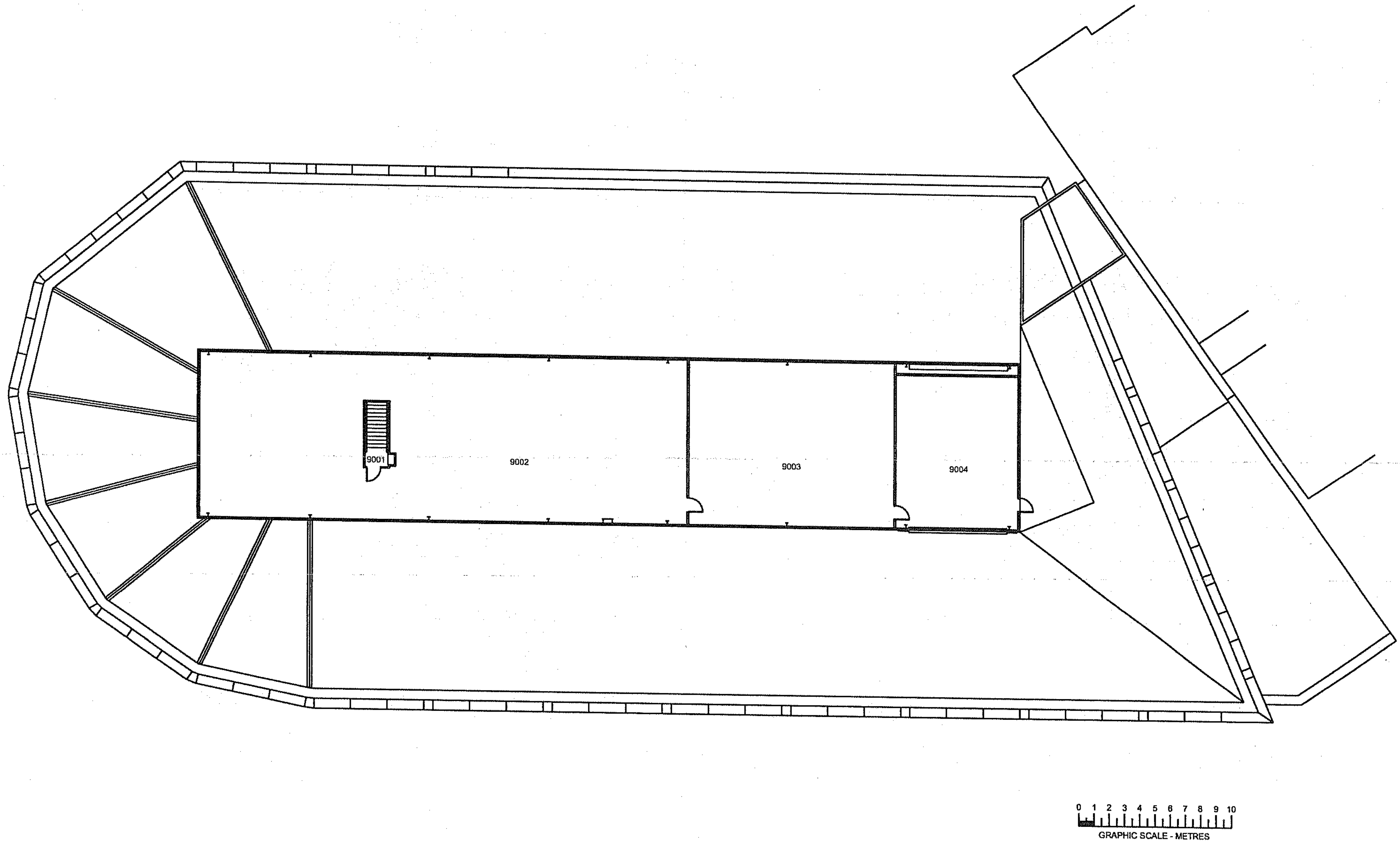


CHRISTCHURCH HOSPITAL
WOMENS - SIXTH FLOOR
SPACE NUMBERS

Canterbury District Health Board Te Poari Hauora ō Waitaha	DESIGNED: DRAWN: B.Tudehope CHECKED: DATE: 11-12-06	DWG No. 01298002 SCALE 1:250 AT A3 REVISED 11-12-06
	XREFED DRAWING/S: 01298001	

Maintenance and Engineering Department - Christchurch Hospital

DRAFT 4/5/2012



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

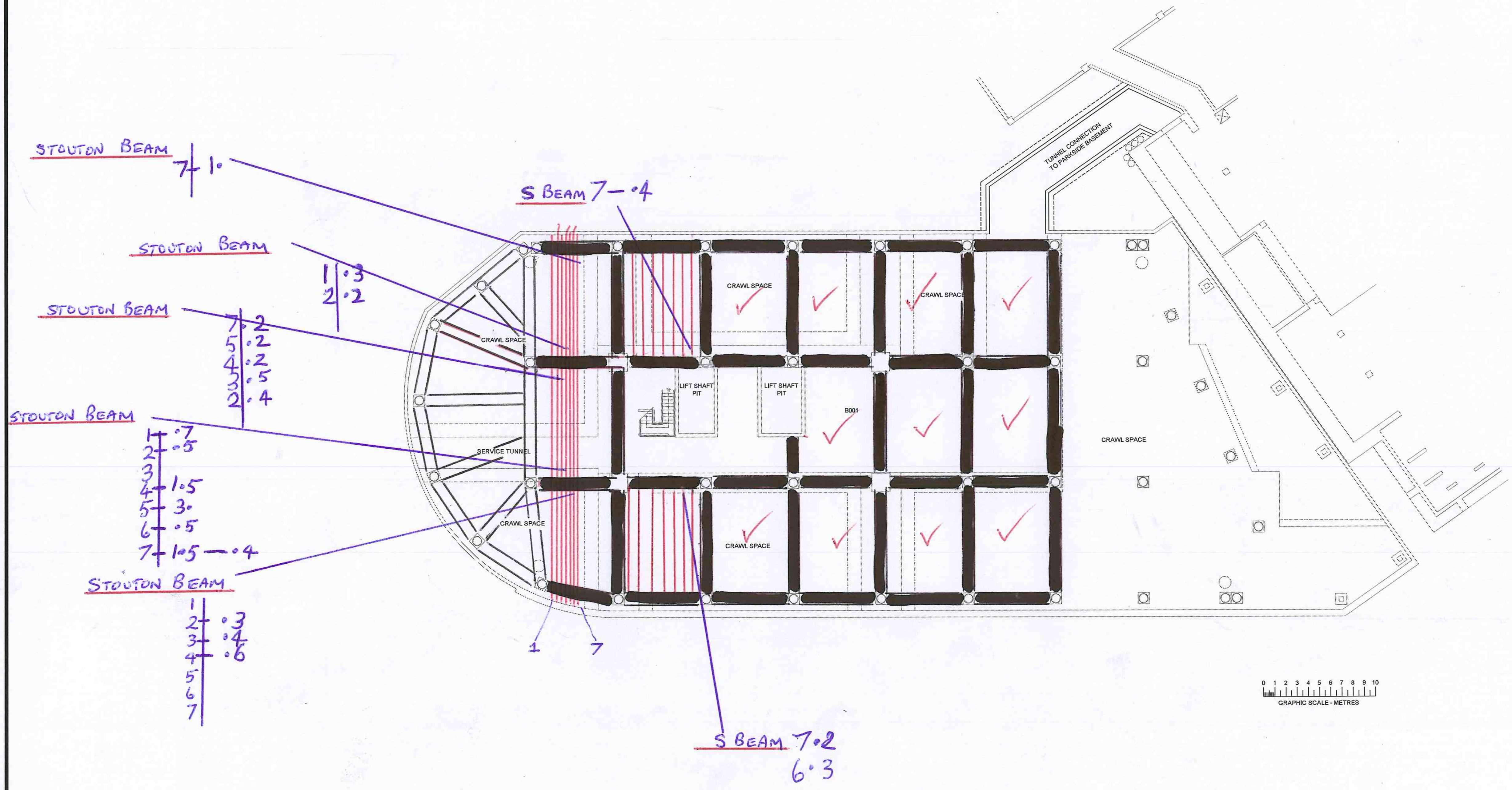
DRAFT 4/5/2012

APPENDIX C

Remediation Sketches

SKS-C1 & Preliminary SKS-C2

DRAFT 4/5/2012



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

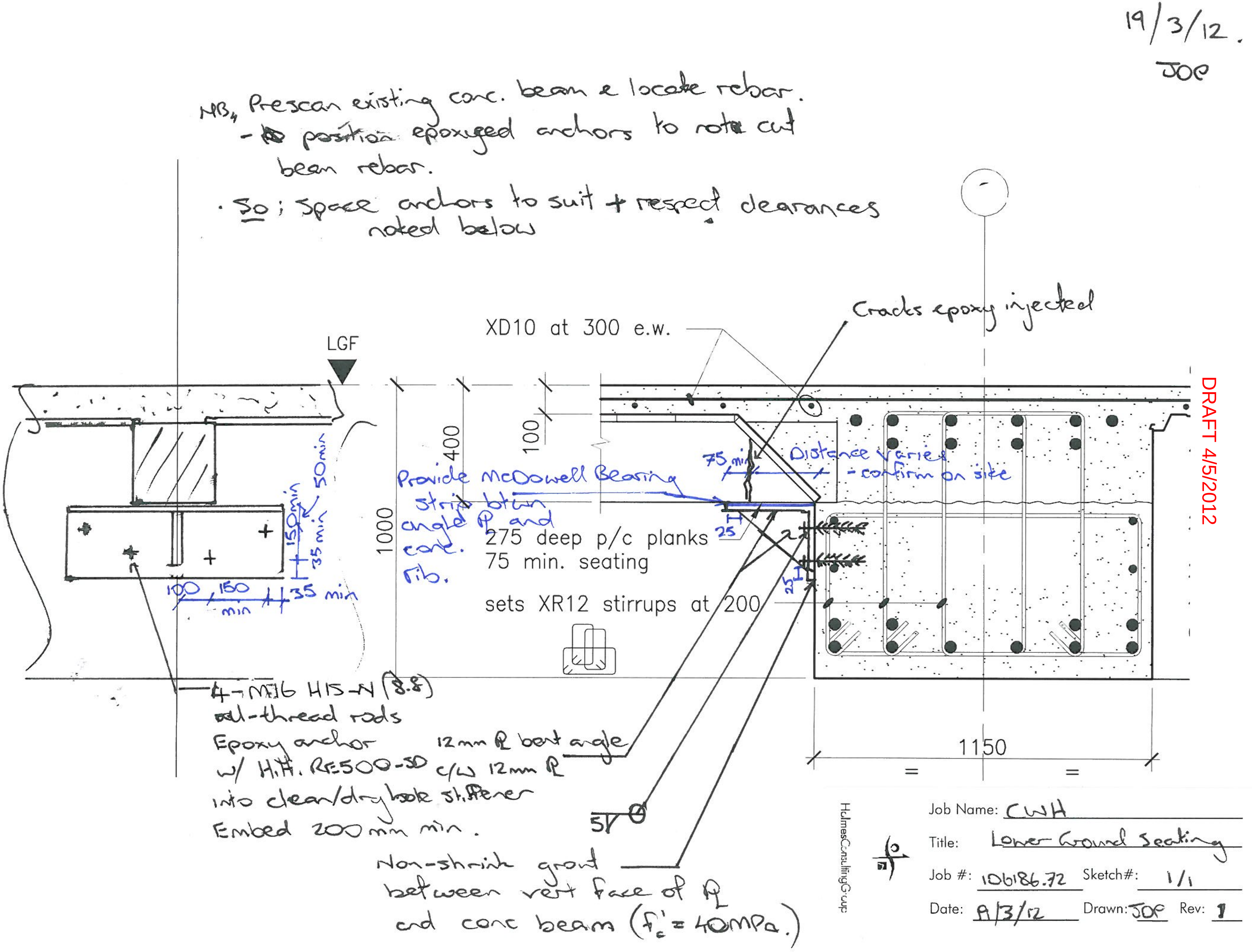
CHRISTCHURCH HOSPITAL
WOMENS - BASEMENT
FLOOR PLAN

Canterbury District Health Board
Te Poari Hauora ō Waitaha

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

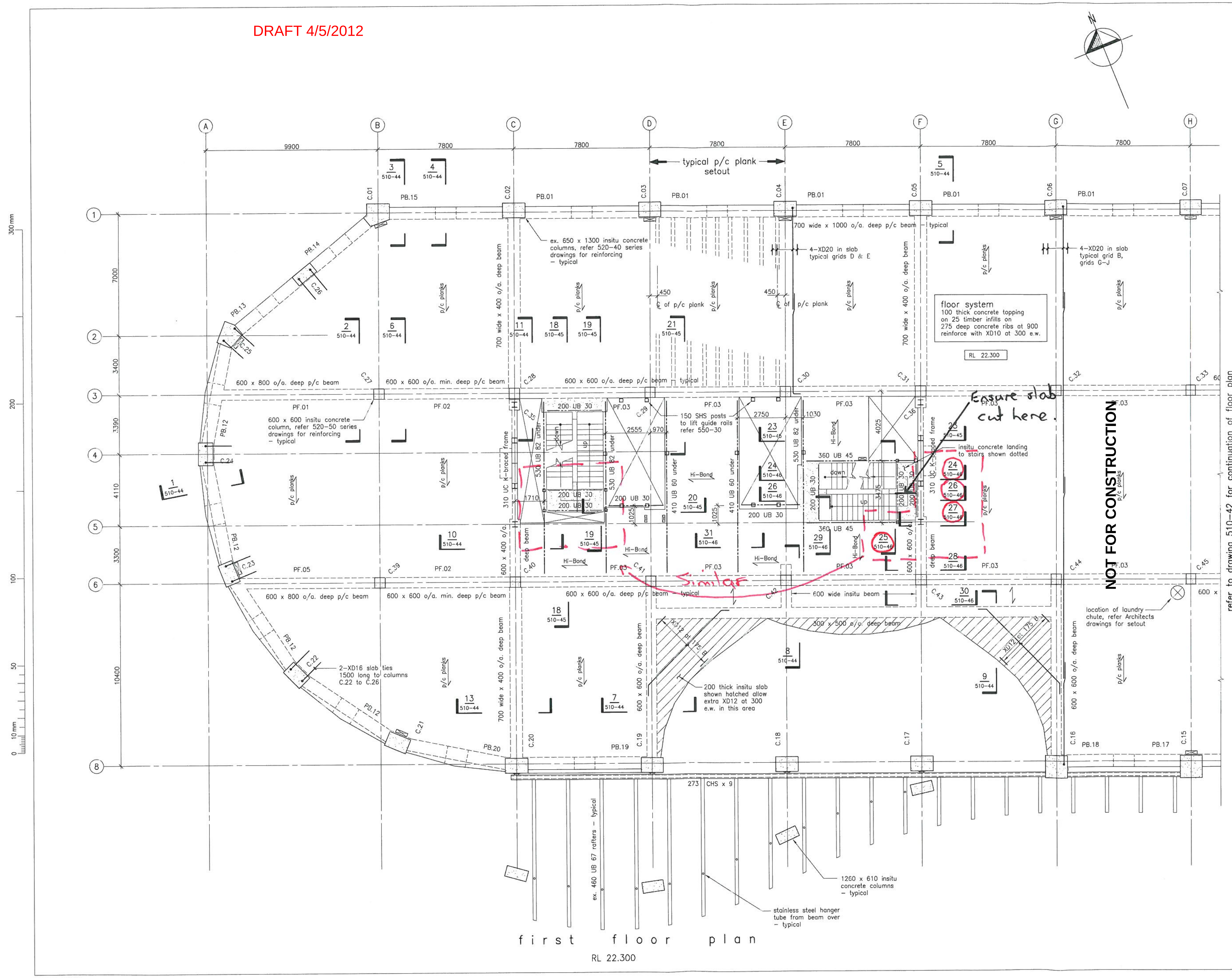
DWG No. 01291006
SCALE 1: 200 AT A1
REVISED 15-11-05

Maintenance and Engineering Department - Christchurch Hospital



19/3/12.
JOE
NB, Prescan existing conc. beam & locate rebar.
- position epoxied anchors to not cut beam rebar.
· SO; Space anchors to suit + respect clearances noted below
LGF
XD10 at 300 e.w.
Cracks epoxy injected
400
100
1000
75 min
Distance varies - confirm on site
Provide McDowell Bearing strip btwn angle PL and conc. rib.
275 deep p/c planks
75 min. seating
25
25
sets XR12 stirrups at 200
50 min
150 min
35 min
100
150 min
35 min
4-M16 H15-N (8.8) all-thread rods
Epoxy anchor w/ H.H. RE500-SD into clean/dry hole
Embed 200mm 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
=
=
HolmesConsultingGroup
Job Name: CWH
Title: Lower Ground Seating
Job #: 106186.72
Sketch#: 1/1
Date: 19/3/12
Drawn: JOE
Rev: 1

DRAFT 4/5/2012



AMENDMENT	APP'D	DATE
A Construction Issue	DJM	08.11.02
3 Tender Issue	DJM	02.09.02
2 Quantity Surveyor Issue		20.05.02
1 3rd drawing review		22.04.02

	BY	CHECKED	DATE
DESIGN			
DRAWN	D.J.M.		
APPROVED			

HolmesConsultingGroup
STRUCTURAL AND CIVIL ENGINEERS

Canterbury DHB
District Health Board
Te Poari Hauora o Waitaha

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

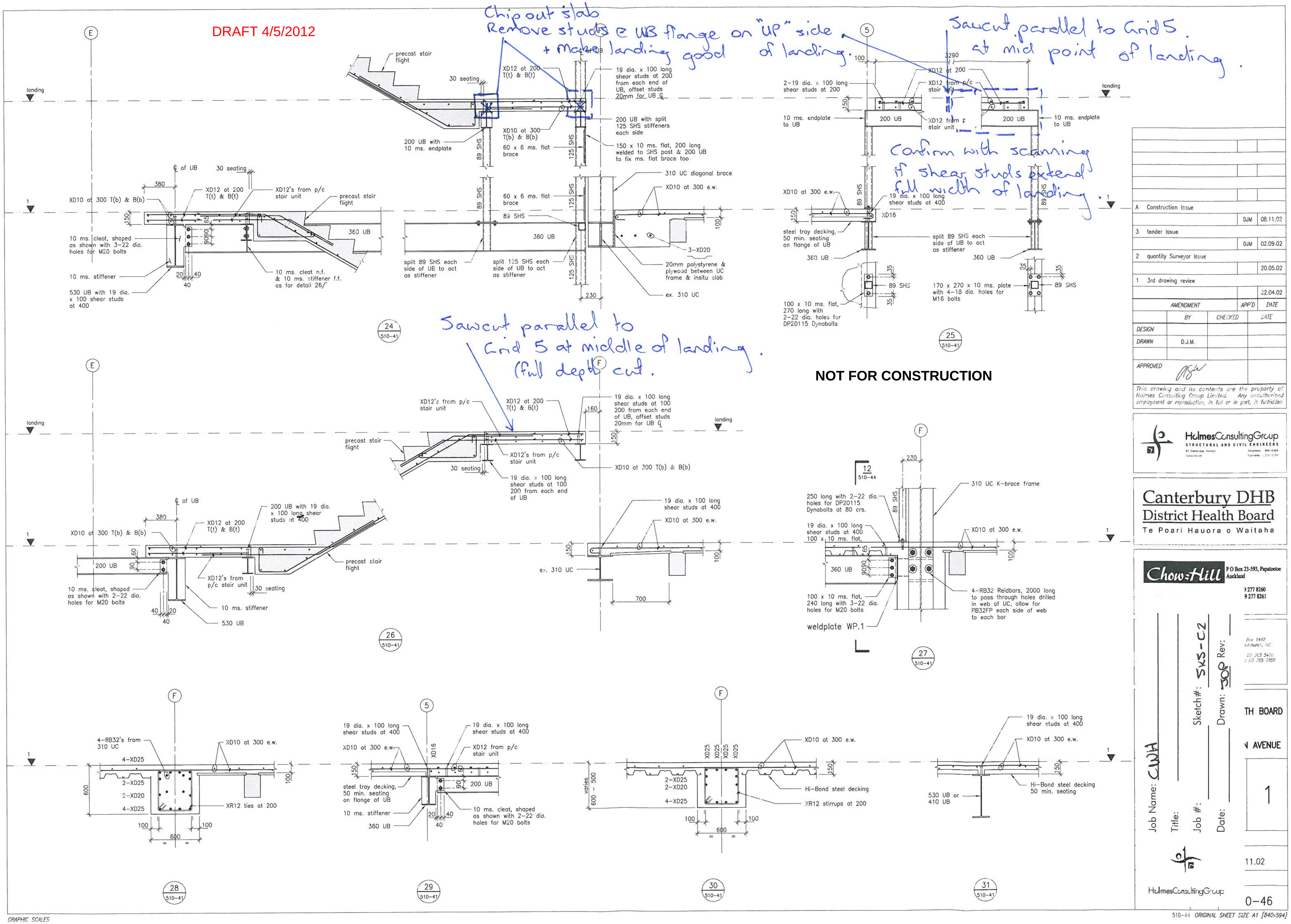
OPUS ARCHITECTURE

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

FLOOR PLAN
first floor plan
– west
1

STATUS: WORKING DRAWING
SCALE: 1 : 100
PLOT DATE: 08.11.02
FILE: 510-41
CODE:
FEATURE IDENTIFIER: 30500
REVISION: A
SHEET: 510-41

ORIGINAL SHEET SIZE A1 [840x594]


DRAFT 4/5/2012
Chip out slab
Remove studs @ UB flange on "UP" side
+ Make landing good of landing.
Saucut parallel to Grid 5
at mid point of landing.
Confirm with scanning
if shear studs extend
full width of landing.
Sawcut parallel to
Grid 5 at middle of landing.
(full depth cut.
NOT FOR CONSTRUCTION
precast stair flight
30 seating
XD12 at 200 T(t) & B(t)
19 dia. x 100 long shear studs at 200 from each end of UB, offset studs 20mm for UB ℄
200 UB with split 125 SHS stiffeners each side
XD10 at 300 T(b) & B(b)
60 x 6 ms. flat brace
150 x 10 ms. flat, 200 long welded to SHS post & 200 UB to fix ms. flat brace too
200 UB with 10 ms. endplate
310 UC diagonal brace
XD10 at 300 e.w.
360 UB
3-XD20
20mm polystyrene & plywood between UC frame & insitu slab
ex. 310 UC
split 89 SHS each side of UB to act as stiffener
split 125 SHS each side of UB to act as stiffener
10 ms. cleat, shaped as shown with 3-22 dia. holes for M20 bolts
10 ms. stiffener
530 UB with 19 dia. x 100 shear studs at 400
10 ms. cleat n.f. & 10 ms. stiffener f.f. as for detail 26/
XD12's from p/c stair unit
24 510-41
2-19 dia. x 100 long shear studs at 200
10 ms. endplate to UB
200 UB
XD12 from p/c stair unit
steel tray decking, 50 min. seating on flange of UB
19 dia. x 100 long shear studs at 400
XD16
split 89 SHS each side of UB to act as stiffener
170 x 270 x 10 ms. plate with 4-18 dia. holes for M16 bolts
100 x 10 ms. flat, 270 long with 2-22 dia. holes for DP20115 Dynabolts
25 510-41
19 dia. x 100 long shear studs at 100 200 from each end of UB, offset studs 20mm for UB ℄
XD10 at 300 T(b) & B(b)
200 UB with 19 dia. x 100 long shear studs at 400
10 ms. cleat, shaped as shown with 2-22 dia. holes for M20 bolts
530 UB
26 510-41
310 UC K-brace frame
250 long with 2-22 dia. holes for DP20115 Dynabolts at 80 crs.
100 x 10 ms. flat, 240 long with 3-22 dia. holes for M20 bolts
weldplate WP.1
4-RB32 Reidbars, 2000 long to pass through holes drilled in web of UC, allow for RB32FP each side of web to each bar
27 510-41
4-RB32's from 310 UC
4-XD25
2-XD25
2-XD20
XR12 ties at 200
28 510-41
29 510-41
Hi-Bond steel decking
XR12 stirrups at 200
30 510-41
530 UB or 410 UB
Hi-Bond steel decking 50 min. seating
31 510-41
A Construction Issue DJM 08.11.02
3 tender Issue DJM 02.09.02
2 quantity Surveyor Issue 20.05.02
1 3rd drawing review 22.04.02
AMENDMENT APP'D DATE
BY CHECKED DATE
DESIGN
DRAWN D.J.M.
APPROVED
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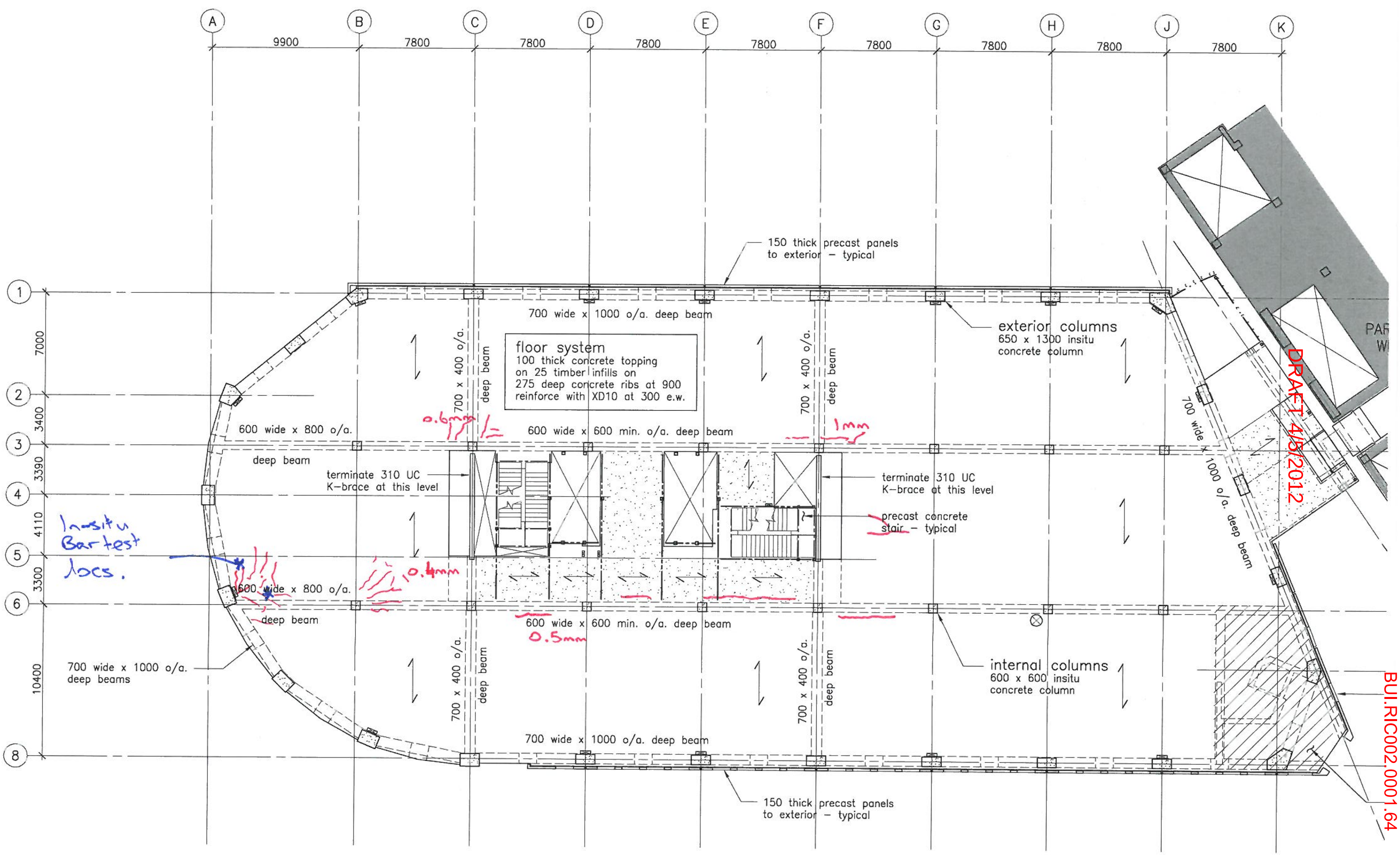
HolmesConsultingGroup STRUCTURAL AND CIVIL ENGINEERS
Canterbury DHB
District Health Board
Te Poari Hauora o Waitaha
Chow:Hill
Job Name: CWH
Title:
Job #:
Date:
Sketch #: SKS-C2
Drawn: JOP Rev:
TH BOARD
N AVENUE
1
11.02
0-46
510-46 ORIGINAL SHEET SIZE A1 [840x594]
GRAPHIC SCALES

DRAFT 4/5/2012

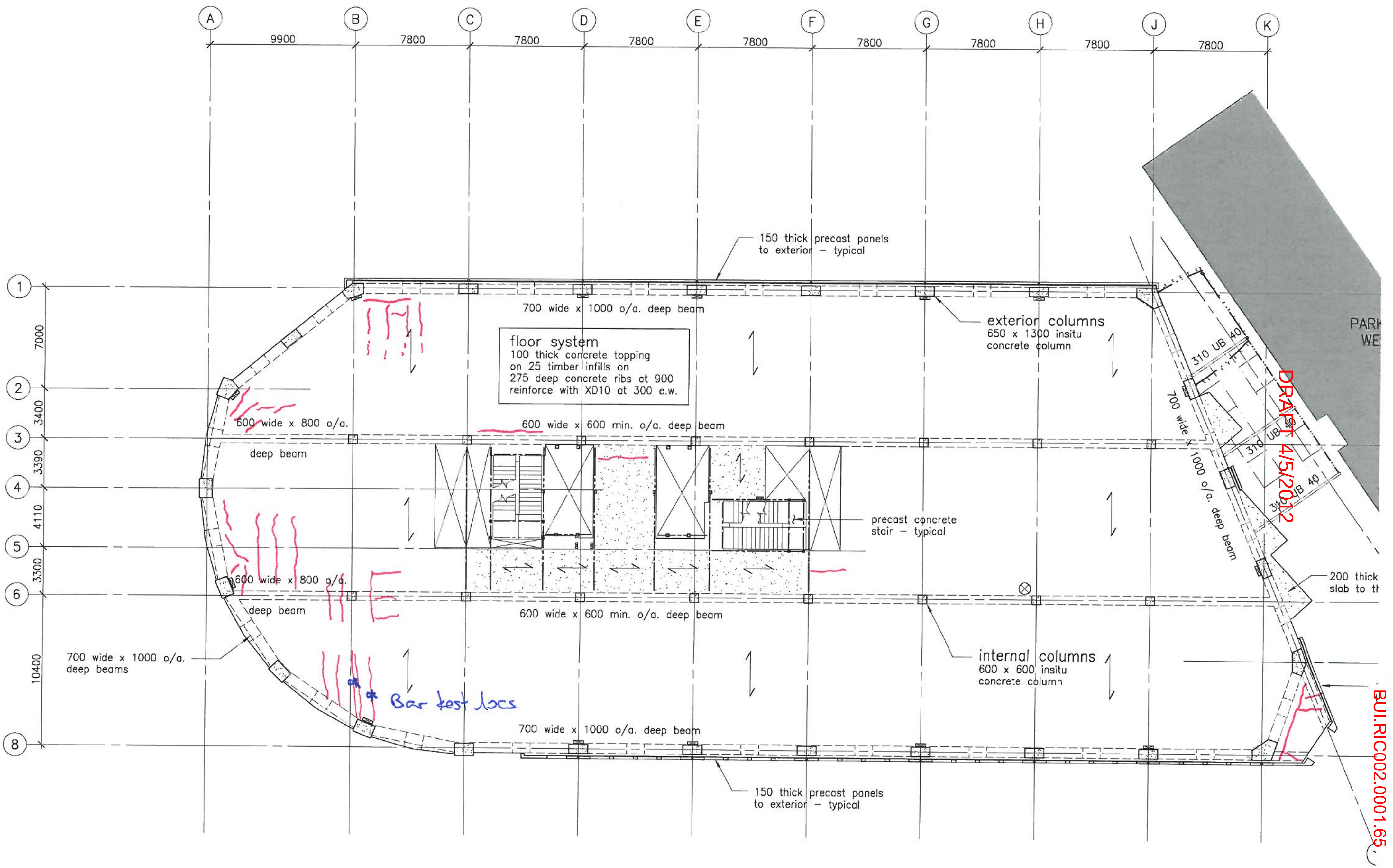
APPENDIX D

Approximate Crack Maps Levels 3, 4 and 5

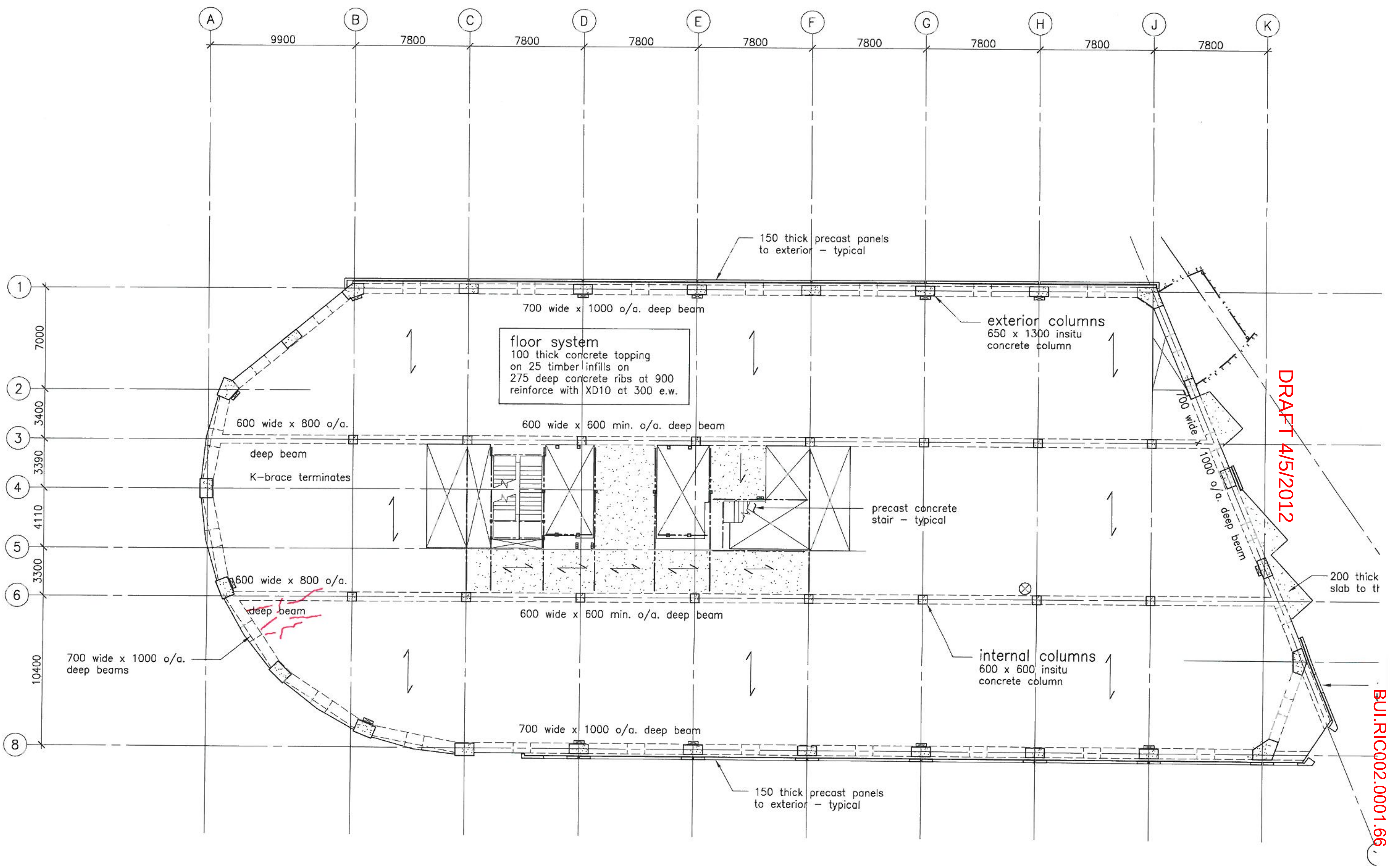
DRAFT 4/5/2012



THIRD FLOOR PLAN: APPROX CRACK MAP FOR AREAS INSPECTED UP TO 1/5/2012



FOURTH FLOOR PLAN: APPROX CRACK MAP FOR AREAS INSPECTED UP TO 1/5/2012



FIFTH FLOOR PLAN: APPROX CRACK MAP FOR AREAS INSPECTED UP TO 1/5/2012