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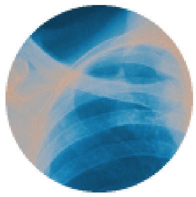
REPORT

STRUCTURAL AND CIVIL ENGINEERS



ROBERT MCDOUGALL ART GALLERY

PREPARED FOR

CHRISTCHURCH CITY COUNCIL

104653.02

SEPTEMBER 2013

ROBERT MCDOUGALL ART GALLERY – DETAILED SEISMIC ASSESSMENT REPORT

Prepared For:
CHRISTCHURCH CITY COUNCIL

Date: 2 September 2013
Project No: 104653.02
Revision No: 1

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REPORT ISSUE REGISTER

DATE	REV. NO.	REASON FOR ISSUE
2/9/13	1	Issue for review pre-detailed design.

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This report covers the structural damage sustained by the Robert McDougall Art Gallery, as a result of the series of earthquakes including the Darfield Earthquake that struck at 4:36am on 4th September, 2010 and the Lyttelton Earthquake that struck at 12.51 pm on the 22nd of February, 2011.

The current statutory requirements relevant to earthquake damaged buildings are outlined and the general form of the building and its capacity prior to the earthquakes are summarised. The capacity of the building prior to the earthquakes was found to be limited by the out-of-plane capacity of the exterior walls which resulted in the building achieving a capacity of approximately 20% current code.

The level of shaking experienced at the site is estimated from the Geonet strong motion data recorded at monitoring sites around Christchurch and is related to the fundamental periods of the building for the Lyttelton Earthquake. Given the age of the building, seismic design demands were not considered at the time the building was built. However as a general reference the strong motion data available suggests that this earthquake produced accelerations significantly in excess of the design spectra that would have been considered past assessments of this building.

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

Some cracking of foundation walls and strip footings was noted, probably as a result of minor settlement of the south-west corner. There has been little or no cracking of the unreinforced masonry walls to-date, however the heritage coverings on the interior walls has made inspection difficult up till now.

Stepped cracking of the mortar joints in the end skylight bulkhead walls has been observed and cracking of the concrete encasement around some of the skylight roof beams have been noted.

Following the repairs recommended herein, the lateral load resisting performance of the building should be restored and improved to approximately 67% of current code. This capacity is limited by the performance of the perimeter walls with out-of-plane collapse.

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

1. INTRODUCTION

Holmes Consulting Group has been engaged by the Christchurch City Council to complete a full structural review following the Canterbury Earthquakes.

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

1.1 PURPOSE

The purpose of this study was to:

- review the impact of the earthquakes on the building
- identify any significant life safety concerns
- map typical damage around the building
- identify those items requiring repairs or replacement
- design and specify repairs to comply with Christchurch City Council regulations
- provide construction monitoring for the remedial works

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

1.2 SCOPE OF WORK

The scope of work for this project included the following:-

- Review the structural drawings to determine the building structural systems and predict areas of likely damage.
- Inspect sufficient of the building structure to be able to make a determination of the behaviour of the building in the earthquake, and to map damage to the structure.
- Prepare a report detailing the proposed repairs required including extent and details.
- Prepare documentation for the repairs, and assemble a package of information for submission to the CCC Building Recovery Office.
- Assist with obtaining the Building Consent.
- Provide Construction Monitoring for the repairs, and final sign-off on completion (assumed to be a PS-4).

1.3 LIMITATIONS

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

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

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

2. STATUTORY REQUIREMENTS

2.1 BUILDING ACT

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

Section 112 - Alterations to Existing Buildings

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

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

Section 115 – Change of Use

Section 115 of the Building Act requires that the territorial authority (the Christchurch City Council) be satisfied that the building in its new use will comply with the relevant sections of the building code "as nearly as is reasonably practicable"

In relation to building earthquake strength, this section is typically interpreted by the Christchurch City Council as requiring earthquake strengthening to a minimum level of 67% of that required for an equivalent new building.

Section 122 – Meaning of Earthquake Prone Building

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

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

Section 124 – Powers of Territorial Authorities

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

Section 131 – Earthquake Prone Building Policy

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

2.2 BUILDING CODE

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

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

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

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

2.3 CHRISTCHURCH CITY COUNCIL POLICY

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

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

Section 2.3.3 – Taking Action on Earthquake-prone Buildings

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

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

Section 2.3.6 – Buildings Damaged by an Earthquake

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

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

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

2.4 CANTERBURY EARTHQUAKE RECOVERY AUTHORITY (CERA)

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

Section 38 – Works

> (4) If the chief executive gives written notice to an owner of a building, structure, or other erection on or under land that demolition work is to be carried out there, -
>
> (a) the owner must give notice to the chief executive within 10 days after the chief executive's notice is given stating whether or not the owner intends to carry out the works and, if the owner intends to do so, specifying a time within which the works will be carried out; and
>
> (b) if the owner fails to give notice under paragraph (a) or the chief executive is not satisfied with the time specified, or the works are not carried out in the time specified or otherwise agreed, then –
>
> (i) the chief executive may commission the carrying out of the works; and
>
> (ii) in the case of the demolition of a building to which section 40(1) or (2) refers, the chief executive may recover the costs of carrying out the work from the owner of the dangerous building in question; and
>
> (iii) the amount recoverable becomes a charge on the land on which the work was carried out.

Section 51 – Requiring Structural Survey

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

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

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

Typically the evaluation procedure defaults to achieving the minimum standard set out by the Earthquake Prone Building legislation, which has generally been accepted as achieving an ultimate limit state capacity equivalent to at least 33% current code. However, an additional requirement has been proposed whereby Critical Collapse Hazards (CCHs) must be specifically considered. These are discussed in detail in Appendix A.

2.5 HERITAGE CONSIDERATIONS

The Christchurch City Plan lists structures, places and objects which have a heritage value and sets out the rules for any proposed alterations. Listed historic items are divided into four groups, with Group 1 heritage items having the highest level of protection.

The rules affect proposals for demolition, alteration, removal, or additions to the listed items. The following extract from the City Plan outlines the general Resource Consent requirements:-

> If a listed building, place or object is located on the site, and demolition, alteration or removal is proposed, and/or the erection of any additional building(s) is proposed on a site containing a listed building, place or object, application will need to be made for resource consents as follows:-

	Demolition	**Alteration or Removal**	**Additional Buildings**
Group 1	Non-complying	Discretionary	Discretionary
Group 2	Non-complying	Discretionary	Discretionary
Group 3	Discretionary	Discretionary	Controlled
Group 4	Discretionary	Controlled (alteration) Discretionary (removal)	Controlled

> Applications for any alteration to, or erection of any additional building(s) on a site containing a Group 3 or Group 4 building, place or object, or any internal alteration to a Group 1 or 2 building, place or object will not require the written consent of other persons and shall be non-notified.

The Robert McDougall Art Gallery is listed as a Group 1 heritage building.

2.6 SCOPE OF THIS REPORT

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

- Repair of damage caused directly by the earthquake
- Adoption of the new loadings standard (Z=0.3)
- Strengthening to achieve 67% of full code level
- Assessment of Critical Collapse Hazards

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

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

3. PRE-EARTHQUAKE BUILDING CONDITION

This section discusses the form and capacity of the building prior to the Canterbury Earthquakes.

3.1 BUILDING FORM

The McDougall Art Gallery was designed in 1930 and building opened in 1932. The original building is predominantly one storey with basement through approximately two thirds of the footprint (the original basement only extended on the east side of the building but was extended). Above the entrance hall there is a second floor with office space. The Canaday Wing attached to the north end of the gallery is a two storey addition constructed in 1983 (Figure 3-1).

The gallery is constructed of unreinforced masonry with the walls varying from two to three wythes thick. Around the exterior walls there is a single brick veneer that is tied (with wire ties) to the two wythe main wall. The original building layout is essentially symmetrical about both principal axes with the regular layout of masonry walls providing the seismic lateral force resisting system.

The walls are generally tied together at their top by concrete (assumed to be lightly reinforced) roof slabs although these slabs are not complete plate elements due to the presence of central skylights. The main floor is an in-situ reinforced concrete slab on in-situ beams. The foundations are formed by strip footings beneath the basement wall lines (which correspond to the gallery walls above) and individual pad footings beneath the interior columns.

The Canaday Wing is constructed from a mixture of concrete block walls (assumed to be partially filled and reinforced), steel framing and timber flooring. Given the relatively new age of construction, it is expected that this will have had a level of seismic design carried out, commensurate with the building code requirements of that time (see Figure 3-2).

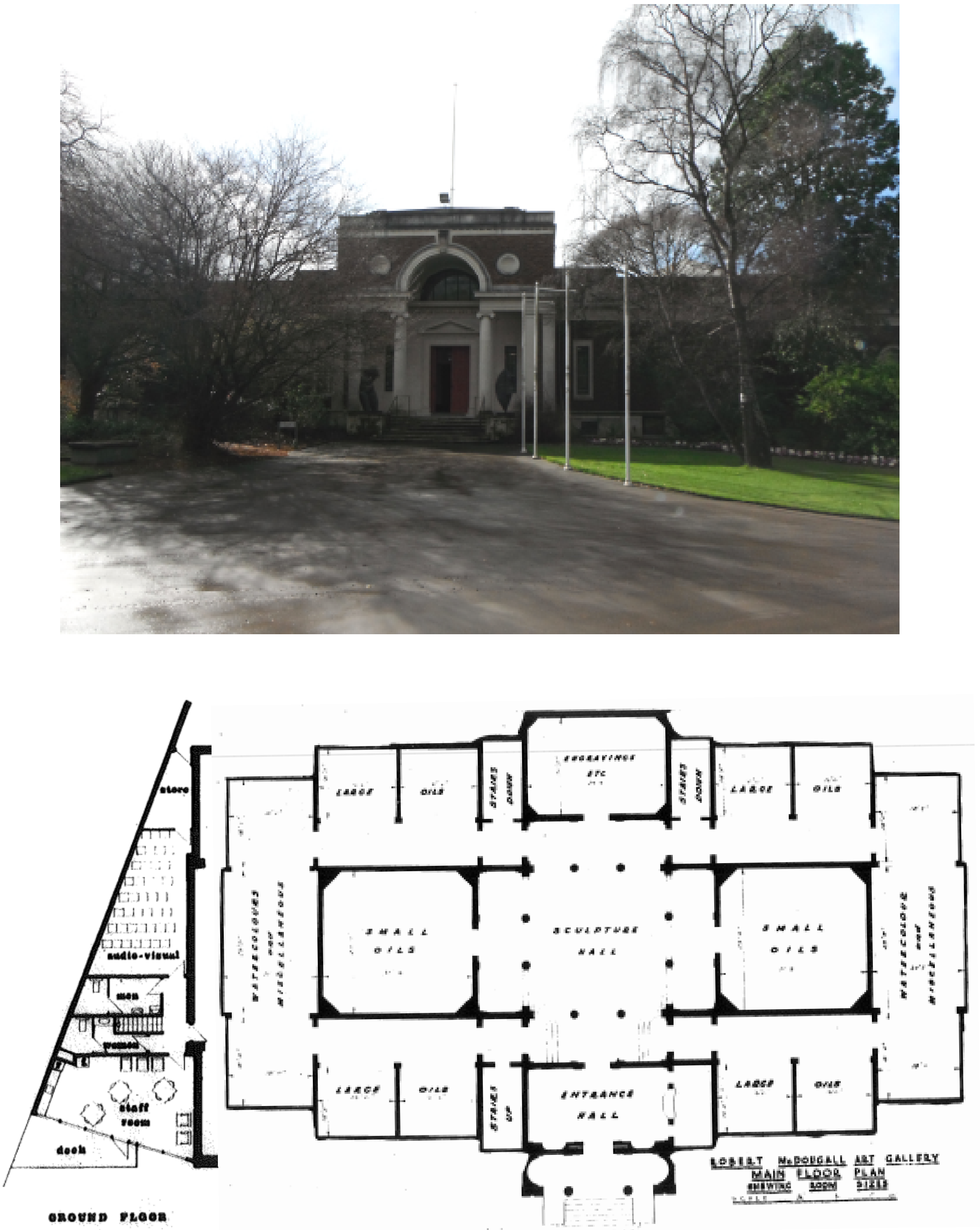


Figure 3-1: Robert McDougall Art Gallery

3.2 PRE-EARTHQUAKE BUILDING CAPACITY

The McDougall Art Gallery was designed and constructed prior to seismic design being considered in structural design practice. To this extent there is no reference design level to compare modern Code seismic requirements against.

Previous detailed assessment of the McDougall Art Gallery predicted that the primary building structure would perform relatively well in an earthquake. This assessment included time history analyses (undertaken to the current loadings standard, NZS1170.5:2004 [5]) which predicted the primary building structure to be capable of resisting an earthquake equivalent to 67% of current NZS1170.5:2004 demand by in-plane shear, and less than 33% current code by out-of-plane flexure.

The unreinforced masonry walls behave in a relatively brittle manner implying that they have little reserve capacity to sustain seismic demands greater than their yield level. Under moderate seismic demands (up to 67% of current code for an Importance Level 3 building) the main walls behave inelastically when subject to in-plane shear stresses but the level of damage is limited such that they are likely maintain a level of gravity load carrying capacity. The exterior walls are considered earthquake prone due to their low resistance (less than 33% of current code) to out-of-plane collapse.

In-situ testing of the mortar shear strength was carried out by Holmes Solutions Limited at 16 locations around the building. The full results and calculations to reference the tests back to the code recommendations are presented in Appendix F. These test results confirm the mortar strength used to determine and model the wall strength.

An indication of the current code seismic demand for the RMAG is provided in Figure 3-2. The building has a very short fundamental period in each direction (approximately 0.1 seconds) due to the number and length of the wall elements in the building. As a result the building is in the acceleration critical portion of the design spectrum and likely to suffer very high accelerations as a result.

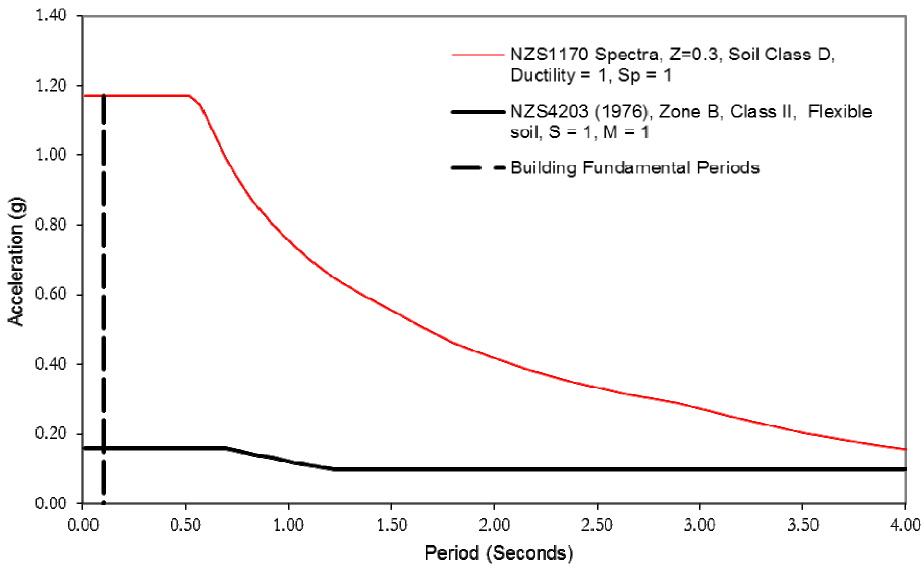


Figure 3-2: Current NZS1170.5:2004 Design Spectrum compared with NZS4203:1976 spectrum that was used for the Canaday Wing design.

3.2.1 Veneer Ties to Exterior Walls

Inspection of the veneer cavity (using a cavity inspection camera) at four separate locations (see Appendix C) has identified that veneer ties (approximately 4mm diameter wire) are present and those viewed are in reasonably good condition (i.e. the thickness of the ties does not appear to be have been reduced by corrosion). The exact size and density of the ties could not be ascertained however at each location inspected the ties were roughly five courses apart vertically, and are formed with two legs crossing the cavity at a given location.

Given that exact measurement of the tie numbers and spacing is not possible, it is difficult to make a precise estimate of the veneer tie capacity. Calculation for the capacity have been made assuming that two tie legs (4 mm diameter each) are present at one metre horizontal spacing and vertical spacing of every fifth course. The results suggest that the capacity of ties is sufficient to achieve 100% of current code demand.

3.2.2 Summary of Non-linear Time History Analysis Results

The non-linear time history assessment that was carried out utilised the numerical model shown in Figure 3-3 that was developed to run in the program ANSR. This model captures the inelastic behaviour of unreinforced masonry shear walls and therefore can be used to assess the development of damage in the walls and associated elements (lintel beams, foundation walls and pilasters). The analyses track the damage development at time intervals of 0.001 seconds through the course of each of three earthquake record pairs (i.e. both north-south and east-west recorded components of the earthquake). A full description of the analysis can be provided in an analysis specific report.

The suit of three real earthquake records, scaled according to Code specifications for this Christchurch site (allowing for soil type and proximity to a potential fault-line), were run at a range of intensities to identify the level of demand at which damage accumulation becomes critical to the buildings ability to sustain gravity loads. It was evident that this critical point occurred around 67% of current code demand for an IL3 building. Beyond this level of demand the wall elements rapidly lose their integrity and have an increased potential to collapse under gravity loads. The following Figure 3-3 provides a visual comparison the damage development with increasing seismic demand. The colouration of the elements corresponds to the damage limit state that has been exceeded by each element. The Life Safety limit state corresponds to current code Ultimate Limit State performance requirements, while Immediate Occupancy implies that the building can be occupied with minor repairs required.

Another way of viewing these results is that between 90% and 100% sufficient extents of wall reach their Collapse Limit State (CLS) to form a global failure mechanism for the building. While New Zealand design codes do not specifically refer to the CLS, if we assume a margin between Collapse and Ultimate Limit States (ULS) of 1.5x (which is approximately the margin for new buildings designed and constructed to current code) then 100%/1.5 ~ 67% which can be treated as the effective capacity as a percent of current code demand.

The non-linear time history analysis cannot capture the out-of-plane collapse capacity of the walls, therefore these analyses are carried out separately using the analysis method described in the NZSEE assessment “Red Book” [6]. The results from this phase of analysis are presented in the following section on Critical Structural Weaknesses.

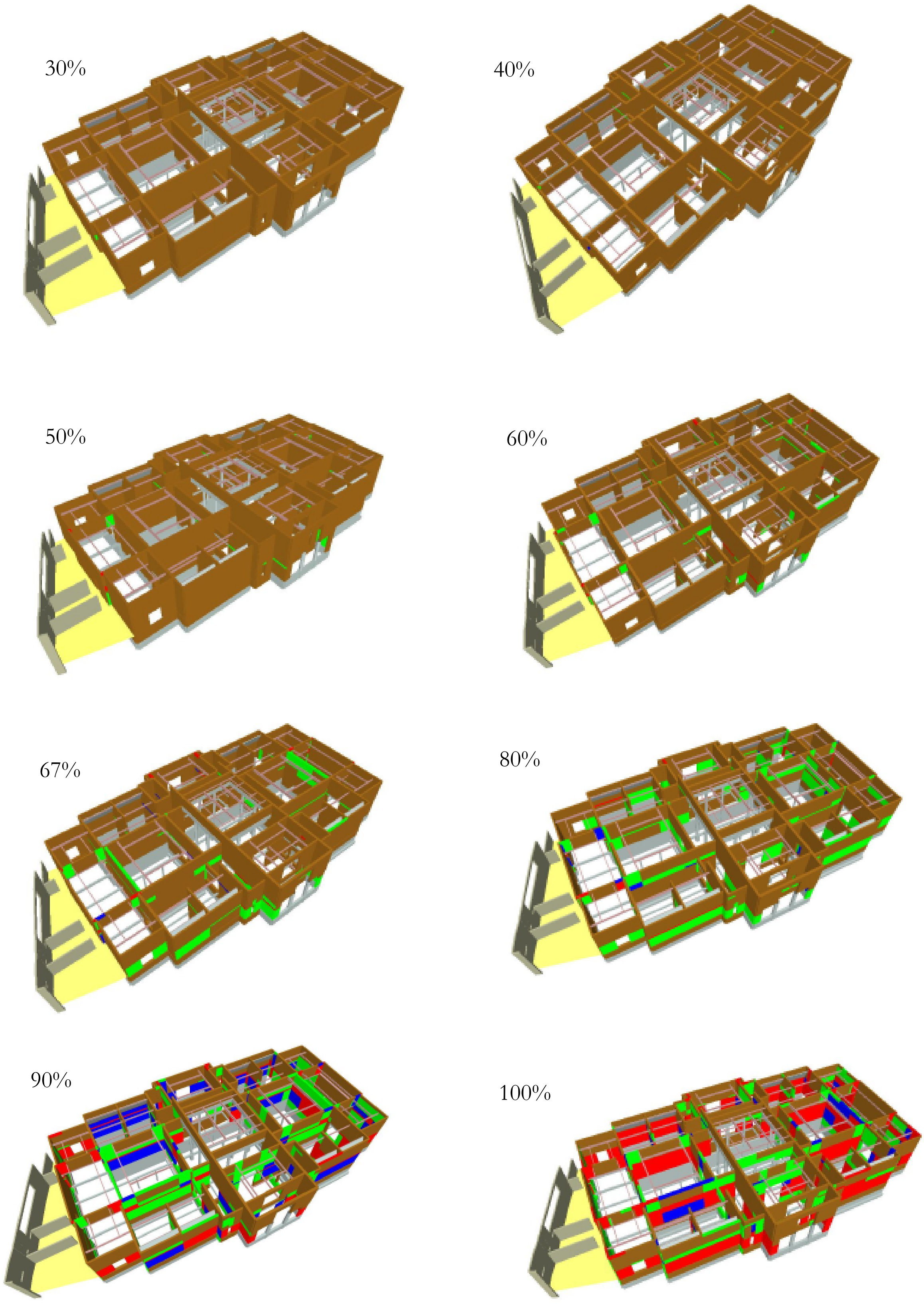


Figure 3-3: Comparison of damaged wall elements with increasing seismic intensity (% IL3 code demand). Green elements correspond to "Immediate Occupancy", Blue elements "Life Safety", Red elements "Collapse Prevention"

3.2.3 Critical Structural Weaknesses

Critical Structural Weaknesses (CSWs) were identified in both end wings where a concrete encased steel roof beam is supported on a small step in the exterior walls (see "red" circles Figure 3-4). These supporting wall regions develop severe damage at low levels of seismic demand and represent a collapse hazard for the roof skylights at these locations. Further discussion of CSWs is provided in Appendix A.

The limited out-of-plane capacity of the exterior walls around the full perimeter of the building, and four interior walls dividing the "Large Oils" alcoves (see "green" walls Figure 3-4), also represent CSWs.

The red circles in Figure 3-4 indicate the locations where existing pilaster columns formed by unreinforced masonry support skylight beams above. These pilasters have very limited ability to sustain earthquake deformations and develop severe damage at relatively low levels of earthquake intensity.

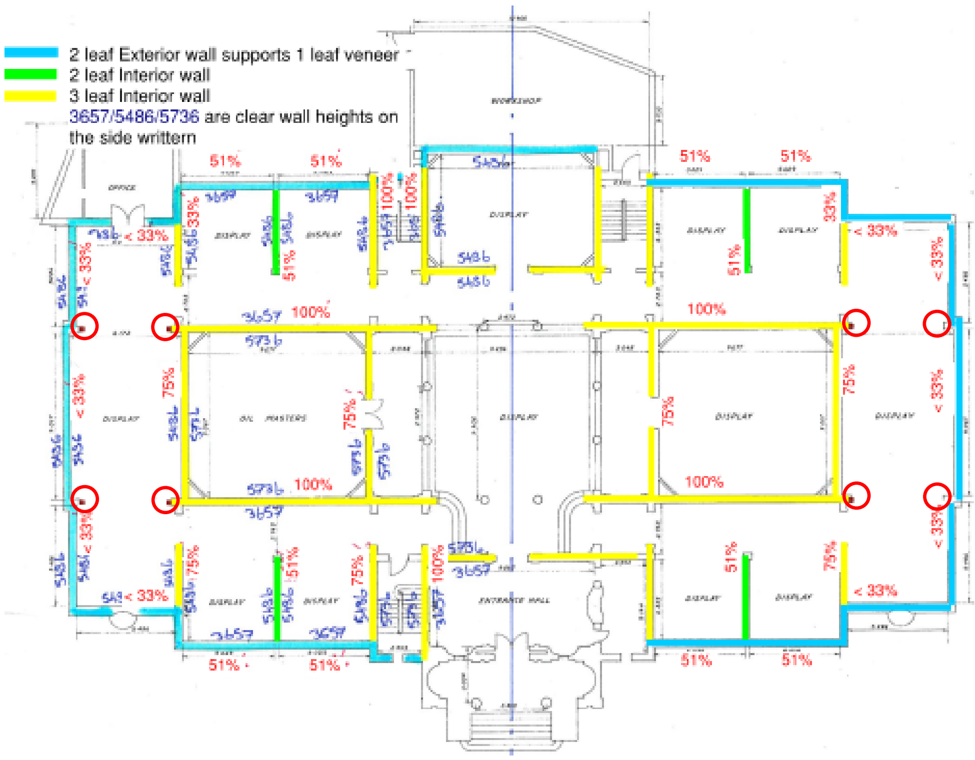


Figure 3-4 Plan indicating the % current code for out-of-plane wall capacity. The red circles indicate the locations where roof beams are supported on pilaster columns with very limited seismic capacity.

4.1 EARTHQUAKE SHAKING EXPERIENCED AT THE SITE

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

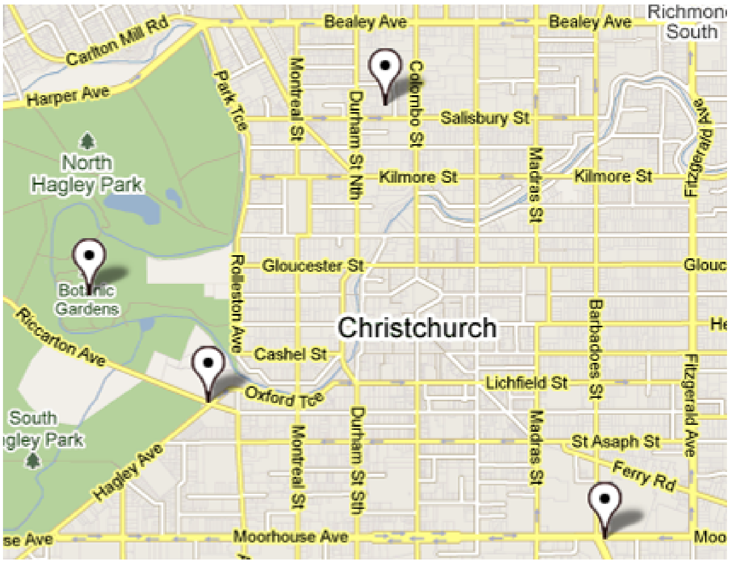


Figure 4-1: Location of Nearby Monitoring Stations

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

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

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

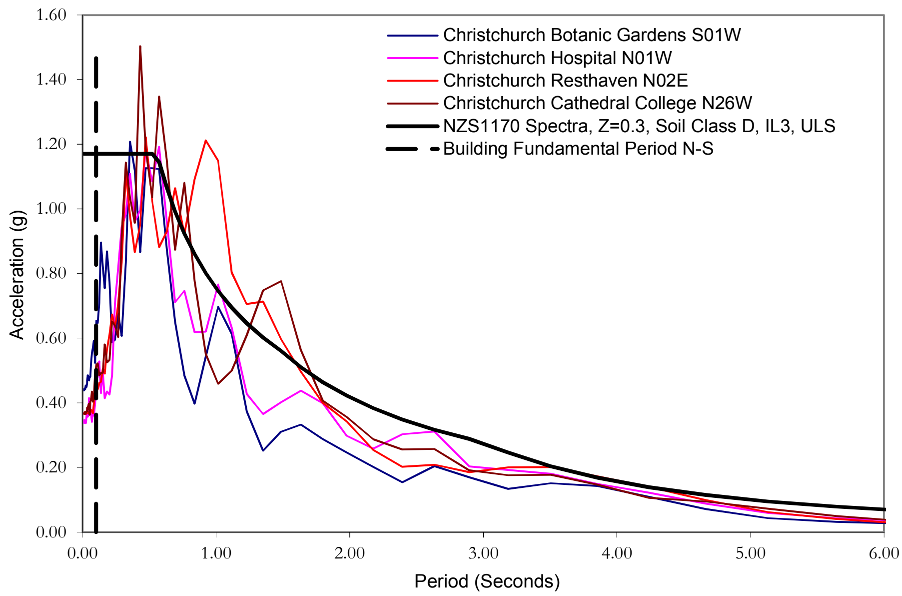


Figure 4-2: 5% Damped Spectra – North-South

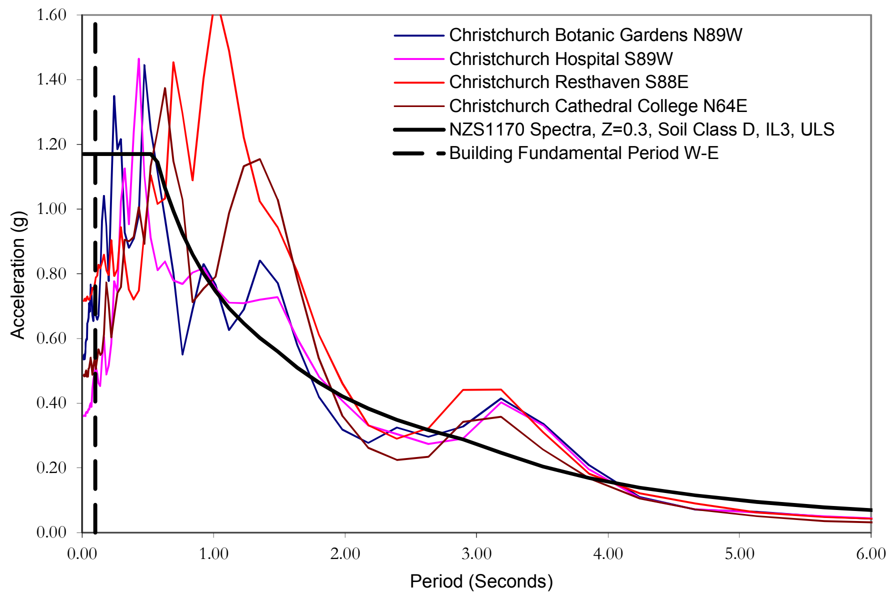


Figure 4-3: 5% Damped Spectra – East-West

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

Previous analyses of the Canterbury Centre have determined the buildings fundamental periods to be approximately 0.105 seconds (north-south) and 0.11 seconds (east-west). Based on the strong motion data downloaded, it is likely that the earthquake produced accelerations in the east-west direction equal to or greater than the current design spectra for this building.

Modern public buildings housing contents of public value, such as museums and galleries, are designed to resist an earthquake with a return period of 1000 years at the Ultimate Limit State (ULS). The magnitude of the ground accelerations recorded in the CBD are in the order of 100% to 150% of the current design code at the time of the earthquake, and are some of the strongest ever recorded in the world. This intensity of shaking is roughly equivalent to an earthquake in Christchurch having a return period of 2500 years, deemed to be the Maximum Considered Event (MCE). At this level of shaking, modern buildings designed to the current codes in place at the time are expected to be at the point of collapse.

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

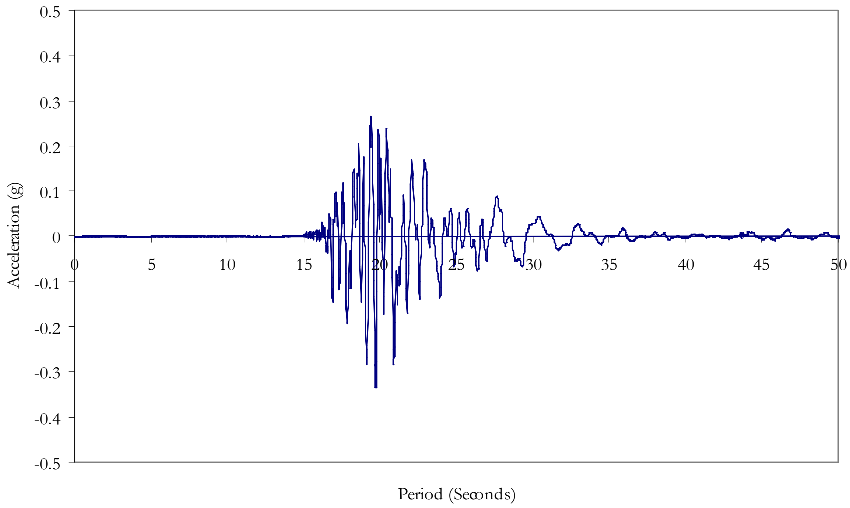


Figure 4-4: Earthquake Record from Christchurch Hospital Site

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

Due to the highly variable ground conditions around Christchurch, it is impossible to determine what the actual shaking experienced at the site was. However, based on the data described above it is likely that the shaking experienced by the building could have exceeded the current code design spectra for an IL3 building.

4.2 PRELIMINARY INVESTIGATIONS

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

- typical damage expected for buildings of this form;

- a review of the original drawings [7];
- analysis work undertaken to date;
- damage observed following the earthquakes.

A description of typical damage expected for buildings of various construction types and periods is attached in Appendix B.

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

- cracking or crushing of the masonry piers supporting the concrete encased steel beams that support the skylights at the north and south ends of the building
- stepped shear cracking in the masonary walls
- Detachment of the brick veneers
- Signs of rocking of the parapets out-of-plane
- Cracking showing through the render layer on the inside face of the walls which might indicate out-of-plane mechanisms forming in the walls

Preliminary observations were carried out on 25/2/2011, 14/6/2011 and 17/1/2012. These identified the following primary areas of damage:-

- Cracking in the foundation walls and strip footings
- Regular cracking of the concrete steel beams in the skylights at the north and south ends of the building
- Stepping cracking in the masonry mortar joints forming the bulkhead wall between the skylights at the north and south ends of the building
- Possible movement of the floor cracks (some pre-existing) in Sculpture Hall
- Cracking in the walls and landings of the stairs

Figure 4-5: (a) Stepped mortar cracking in the skylight bulkheads (b) cracks in the encasing concrete of the skylight steel beams

Based on observations to-date the building does not appear to have suffered the extent of damage that might be expected for the intensity of ground motion that was likely experienced at this site. However as noted above, the major earthquakes were of short duration, and this type of construction tends to perform more poorly under longer duration shaking.

The damage observed though minor, is expected and related to the critical regions indicated from past and updated analysis. It should also be noted that the main walls themselves have not be fully inspected due to the lose woven/stucco lining on the interior face of the walls.

The damage to the foundations suggests there has been a limited amount of settlement in the south-west corner of the building. Floor level surveys reflect this although they are not considered particularly significant.

4.3 DETAILED OBSERVATIONS

Detailed structural observations have been carried out following the Lyttelton Earthquakes.

Following the Lyttelton Earthquake, detailed visual observations were completed on 27/7/12, 23/8/12, 28/9/12, and 7/12/12. A full record of these observations can be found in Appendix C, with reference plans describing the location labelling used found in Appendix D. A full photographic record of the observations is available electronically on request. Observations involved random visual inspections of areas identified through our preliminary review, intrusive investigations to determine the mortar shear strength at 16 locations around the building, completion of a floor level survey, and a veneer cavity camera inspection to review the existing brick veneer tie condition.

Inspection of portions of the interior face of the southern wall were also possible as a result of setting out the locations for trial centre-core testing.

4.4 SUMMARY OF BUILDING DAMAGE (LYTTELTON EARTHQUAKE)

A full description of the damage observed following the Darfield Earthquake can be found in Section 5. It should be noted that this section does not specifically distinguish between damage caused by the Darfield and Lyttelton Earthquakes. For comparison reference should be made to our previous reports.

A summary of the structural damage observed to date is as follows:-

- Cracking of the basement slab and foundation walls/footings, largely due to settlement of the south-west corner of the building
- Stepped cracking in the mortar beds of the end skylight bulkheads
- Cracking of the skylight concrete encased beams (concrete encasement cracking)
- Minor cracking of concrete lintel beams inside the gallery
- Minor cracking of unreinforced masonry pilaster columns inside the gallery
- Minor movement of the parapets and parapet capstones

5. DAMAGE OBSERVED & REPAIRS REQUIRED

Table 5-1 provides a photographic summary of the observed damage and typical repairs required. The table should be read in conjunction with Appendix C – Record of Observations and Appendix D – Location Reference Plans. Drawings containing specific details of the repairs are attached in Appendix G, with the repair Specification attached in Appendix H.

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

Given the heritage nature of this building, all repair work will need to be carried out in conjunction with guidance from the CCC Heritage Consultant team.

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

Table 5-1: McDougall Art Gallery - Photographic Summary of Primary Structural Damage Observed & Repairs Required

Damaged Item	Location	Example	Recommended Repair
1. Basement and Foundations			
1.1. Cracking of the foundation walls and beams. Some minor spalling	Noted throughout the basement, but seems to be more significant on the west side and south end.		• Epoxy inject cracks and repaint. • Patch concrete repairs where cover has spalled
1.2. Potential settlement of south-west corner	Noted that cracking in basement is more concentrated in the west and south areas of the building		Level survey does not show the building to be significantly out-of-level. Geotech report does not suggest the settlement is critical or that future settlement potential is significant. Therefore no repair recommended.
2. Exterior walls			
2.1. Concrete block walls stepped out-of-plane cracking	Part of Canaday Wing next to east entrance		Mortar patch repair per Structural Specification

Damaged Item	Location	Example	Recommended Repair
3. Concrete lintel beams over interior doors			
3.1. Minor cracking of lintel beam	Door into theatre off central sculpture hall		Review with Heritage consultants for possible repairs if necessary.
3.2. Minor cracking of lintel beam	South-east door to Large Oils gallery		Review with Heritage consultants for possible repairs if necessary.
4. Interior walls			
4.1. Unreinforced masonry walls	South-west Large Oils gallery		Review with Heritage consultants for possible repairs if necessary.

Damaged Item	Location	Example	Recommended Repair
5. Interior pilaster columns:			
5.1. Unreinforced masonry pilaster column	South-west gallery		Refer to Heritage consultant advice on appropriate repair methodology for such elements
6. Skylight bulkhead walls			
6.1. Stepped cracking in mortar beds	South and North skylights		Mortar patch repairs as necessary to reinstate mortar bond to bricks. Scrape out and replace with guidance from heritage consultants.

Damaged Item	Location	Example	Recommended Repair
7. Skylight Concrete Encased Steel beams			
7.1. Cracking of concrete up to 2mm in width	Various		Epoxy inject cracks in accordance with the Specification.
8. Exterior Concrete/Stone Roof Bond beam			
8.1. Level 1: Moderate cracking in concrete bond beam	Wall/beam corners adjacent to chimney on east elevation		Epoxy inject cracks in accordance with the Specification. Review with Heritage Consultants.

Damaged Item	Location	Example	Recommended Repair
9. Canaday Wing			
9.1. Level 1: Minor cracking in concrete wall	North wall adjacent to original McDougall Building		Epoxy inject cracks in accordance with the Specification.
10. Parapet Capstone movement and parapet rocking			
	Various capstones appear to have moved. Parapet over/by main entrance was noted to have moved following June 2011 event.		Reset capstones and parapets. Review parapets restraints to compare against current code demands. Consider introducing new strengthening/restraint to satisfy updated demands.

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

6. STRENGTHENING REQUIRED

6.1 STRENGTHENING WORKS REQUIRED TO ACHIEVE 67% NBS

Based on our detailed structural assessment, we have identified that the walls achieve 67% of current code Importance Level 3 demands without the need for strengthening. However there are a number of Critical Structural Weaknesses that limit the overall effective capacity of the building that must be addressed to comply with current regulations.

6.2 STRENGTHENING WORKS REQUIRED TO ADDRESS CSWS

Based on our detailed structural assessment, we have identified the following strengthening works that may be required to comply with current regulations.

6.2.1 Wall Out-of-Plane Strength

The primary Critical Structural Weakness identified from our analyses is the out-of-plane collapse capacity of the perimeter walls and four interior walls dividing the "Large Oils" spaces. Options for strengthening include centre-coring these walls and grouting in reinforcing bars at regular spacings from the roof down to the concrete floor, or alternatively installing structural steel framing as a series of portals and transom beams to reduce the unsupported height of these walls.

Following consultation with the Council heritage team, the centre-coring option was identified as the preferred approach. Although this method has been successfully applied in other buildings in New Zealand, and particularly Christchurch, this building poses a challenge to the method due to the walls only being two wythes thick therefore not leaving a significant margin for deviation or error in the coring. The other issue surrounding the centre-core option is the penetration of water from the coring process through the walls such that it causes damage to the interior linings and finishes. Trial cores have been carried out to assess the accuracy of the coring in the narrow wall, and also the amount of water penetration.

The decision on which method to use is yet to be confirmed by the Council and its heritage consultant team.

Associated with the outside walls is the restraint of the parapet to prevent toppling. While strengthening was carried out in 1995, given the updated loadings code requirements and local zone factor changes for Christchurch these are considered CSWs that require further restraint, particularly given that some of the details do not appear to have a satisfactory level of resilience in-light of the uncertain nature of the concrete making up the perimeter tie beam at the roof level.

Various proposals have been put forward for providing out-of-plane restraint to the parapets, with the preferred option being to extend the centre-core reinforcing bars to the top of the parapet height such that they can be attached to the back of the parapet with a mild-steel coupling plate.

A further CSW is inside with the concrete encased steel beam that supports the masonry bulkheads in the north and south skylights of the building. These steel beams are supported on small returns of masonry wall that have little reserve capacity. Should these support returns fail, the beams overhead loss their support and will potentially lead to a collapse of the skylight roof structure.

7. POST-EARTHQUAKE BUILDING CAPACITY

7.1 POST-EARTHQUAKE BUILDING CAPACITY

In its damaged state following the earthquakes, we do not consider the McDougall Art Gallery to have any reduction in gravity load resistance. The overall lateral load resisting capacity of the building has not been significantly affected, although repairs are required as outlined above. In summary, we do not consider the damage resulting from the earthquake to pose a significant structural hazard in relation to occupation of the building.

Following the recommended repair of the structural damage, the lateral load resisting performance of the structure should be restored. The capacity of the primary structural system remains for in-plane wall shear is at least 67-70% of current code. However the effective capacity of the building is limited by the Critical Structural Weaknesses identified in the preceding section. As such the out-of-plane capacity of the two wythe thick perimeter walls limits the effective capacity to less than 33% of current code implying that the building is earthquake prone.

8. REFERENCES

1 *Building Act 2004*, New Zealand Government

2 *Building Code 2011*, New Zealand Government

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

4 *Canterbury Earthquake Recovery Act, 2011*, New Zealand Government

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

6 *Assessment and Improvemet of Unreinforced Masonry Buildings for Earthquake Resistance*, New Zealand Society for Earthquake Engineering, February 2011

7 *New Art Gallery for The City of Christchurch – Architectural Drawings,* Edward Armstrong architect, 1930

APPENDIX A

Critical Collapse Hazards

APPENDIX A – CRITICAL STRUCTURAL WEAKNESSES

The following outlines the background behind Critical Structural Weaknesses (CSWs). The current design standards and the likely requirements to address CSWs are discussed herein.

CURRENT DESIGN STANDARDS

New Zealand adopts a probabilistic hazard analysis approach to seismicity, with a tiered approach to seismic design based on the following performance objectives:

1. Frequently occurring earthquakes can be resisted with a low probability of damage that would prevent the building from being used as originally intended; and

2. Rarely occurring earthquakes can be resisted with an acceptably low risk of fatality.

Objective 1 is satisfied by the serviceability limit state (SLS) requirements relating to earthquakes having an average return period of 25 years. This is commonly referred to as an 'immediate occupancy' provision, that is, the building should be able to be occupied immediately following this event.

Objective 2 is generally satisfied by designing for ultimate limit state (ULS) requirements in the Design Basis Earthquake (DBE). For normal (IL2) buildings, the DBE is the 500 year earthquake that has approximately 10% likelihood of occurrence in a 50 year building life. This is commonly described as a life safety provision, that is, all occupants should be able to safely exit the building, but it may not be repairable following this event.

It is generally implicit in the standards that a building that has been designed for life safety in the DBE event may also be subjected to a significantly larger earthquake with sufficiently low probability of collapse. This earthquake is referred to as the Maximum Considered Earthquake (MCE) and is generally accepted as an earthquake having a return period of 2500 years, with approximately 2% likelihood of occurrence over 50 years. In relative terms, the load levels from such an event are about 1.8 times as high as the DBE. This level of failure is commonly referred to as the collapse limit state (CLS).

The achievement of this performance criterion is not generally required to be verified. Instead designers are permitted to rely implicitly on the detailing requirements of the material design standards to achieve it. This is illustrated in Figure 1 below.

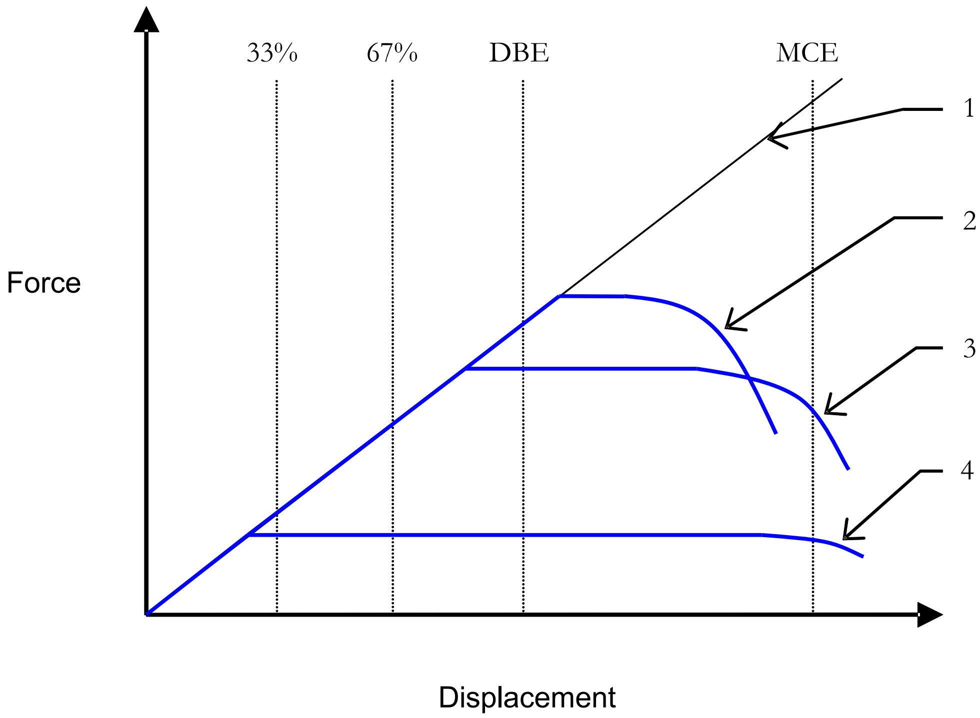


Figure 1: Force-displacement relationships for current design standards

Notes:

> Line 1 represents a fully linear elastic approach, that is, the building has been designed to simply resist the full applied load of even the MCE.
>
> Line 2 represents a structure that is designed to remain fully elastic in the DBE.
>
> Line 3 represents a building of limited ductility. If higher loads are designed for than fully ductile structures, designers may reduce the detailing standards.
>
> Line 4 represents a high ductility level. The design load is reduced according to the ductility, and capacity design is used to ensure that the building yields in a controlled fashion. The detailing provisions of the standards ensure that in the majority of cases the buildings will be capable of displacing to the full MCE displacement with acceptable risk of collapse.

Figure 1 above does not include the impact of a Critical Structural Weakness (CSW) which could result in the premature brittle failure of individual elements and lead to partial or global collapse of the building.

PRIOR DESIGN STANDARDS

In the case of buildings designed to earlier standards, or which may have limitations in detailing, this implicit margin is not available between the onset of significant damage and collapse. This is referred to as brittle behaviour and has significant consequence in the behaviour of the building.

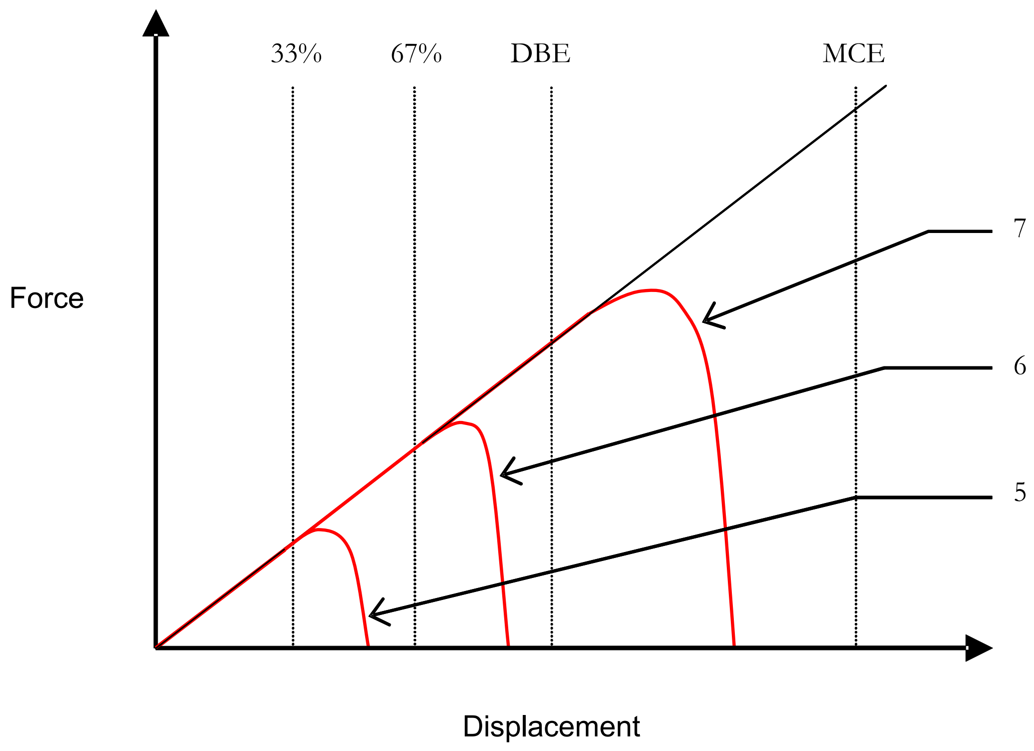


Figure 2: Force-displacement relationships for brittle buildings

Notes:

Line 5 represents a building that may just exceed the EPB threshold. Because there is so little margin between ULS and collapse, there is little resistance to anything other than a moderate earthquake or a SLS event for a modern building.

Line 6 represents a building that may have been strengthened to 67% NBS. Because there is no requirement to add ductility, the onset of collapse is still only marginally above the design load.

Line 7 represents a building designed to previous standards that may still have a brittle collapse hazard. The margin of 1.8 from ULS to collapse does not exist.

The illustration above will not apply to all older buildings. Many may perform adequately, but not necessarily by design. This may be applicable to cases where the inherent strength is simply far greater than was required, or where other mechanisms exist such as rocking of foundations, which will tend to limit loads.

CRITICAL COLLAPSE HAZARDS

Critical structural weaknesses (CSW) are details, configurations or building or site characteristics that could lead to increased damage levels in a building or the premature failure or collapse of all or part of a building. This terminology was proposed in the NZSEE *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes*, June 2006.

Potential CSWs such as plan and vertical irregularity, insufficient seismic gaps between buildings leading to pounding, and short columns can be identified by visual inspection of the building. The CSWs were then used to downgrade the building score in respect of the Initial Evaluation Procedure used to identify Earthquake Prone Buildings. This process did not typically involve plan review and therefore would not necessarily pick up detailing issues.

The more critical CSWs are those which comprise brittle mechanisms and may directly lead to collapse. Review of the structural drawings for the building can be used to identify potential collapse hazards such as inadequate diaphragm connections between the floor and walls, inadequate precast floor seating, inadequate seating or sliding capacity for stairs, precast panel connections that can not accommodate inter-storey deflections and short columns.

Detailed analysis can also be used to identify CSWs such as shear failure in wall or column elements, buckling of walls and columns and inadequate diaphragm or transfer elements.

This latter subset of CSWs can be termed Critical Collapse Hazards (CCHs)

REGULATORY REQUIREMENTS & RECOMMENDATIONS

The current EPB legislation sets a threshold of 33% NBS but does not address the requirements to minimise CCHs for buildings as discussed above. This issue is currently being investigated with respect to the Building Act.

The Canterbury Earthquake Recovery Authority (CERA), is looking to implement requirements locally for the Canterbury region to restore the relativity of ULS to CLS for the assessment of existing buildings and to require the mitigation of CCHs.

Options currently being considered include:

1. Redefining of percent New Building Strength (%NBS) to be:

$$\%NBS = \text{lesser of} \begin{cases} \%NBS_{ULS} \\ \dfrac{\%NBS_{CLS}}{1.8} \end{cases}$$

2. Increasing the factor of safety required for CSWs, such that a sudden collapse cannot occur at too low a level

APPENDIX B

Record of Observations

APPENDIX C – RECORD OF OBSERVATIONS AT: EQUITABLE HOUSE, 77 HEREFORD STREET

Inspection dates: 23/8/12, 28/9/12, 7/12/12, 30/8/13

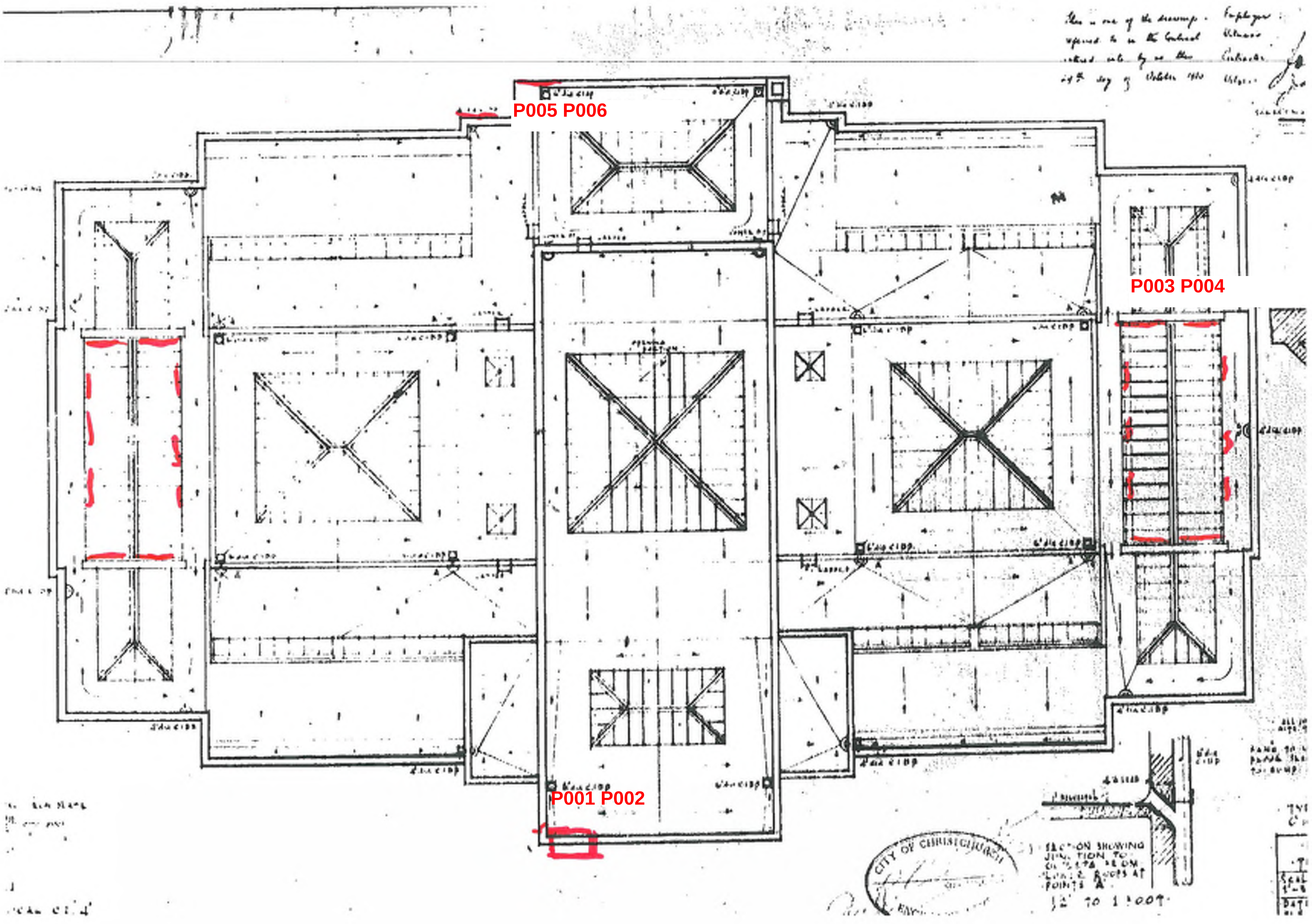
Level	Building Element	Location	Observations	Repair Required?	Photo Reference
Roof	Parapets	Various.	Some evidence of capstone movements although not easy to determine if earthquake related.	Y	
	Parapets	Over/by main entrance.	Stone parapet appears to have moved following June 2011 events.	Y	P001 P002
	Parapets	Various.	Some mortar cracks/movement however possibly due to ageing.		
	Skylight beams	Concrete encased steel framing beams – various.	Numerous cracks in concrete encasement up to 2mm in width observed in north and south skylights. Assume similar cracking will be present in other skylights that will need inspection and confirmation during repair works phase.	Y	P004
	Skylight bulkhead walls	North and South skylights.	Mortar stepped joint cracking.	Y	P003

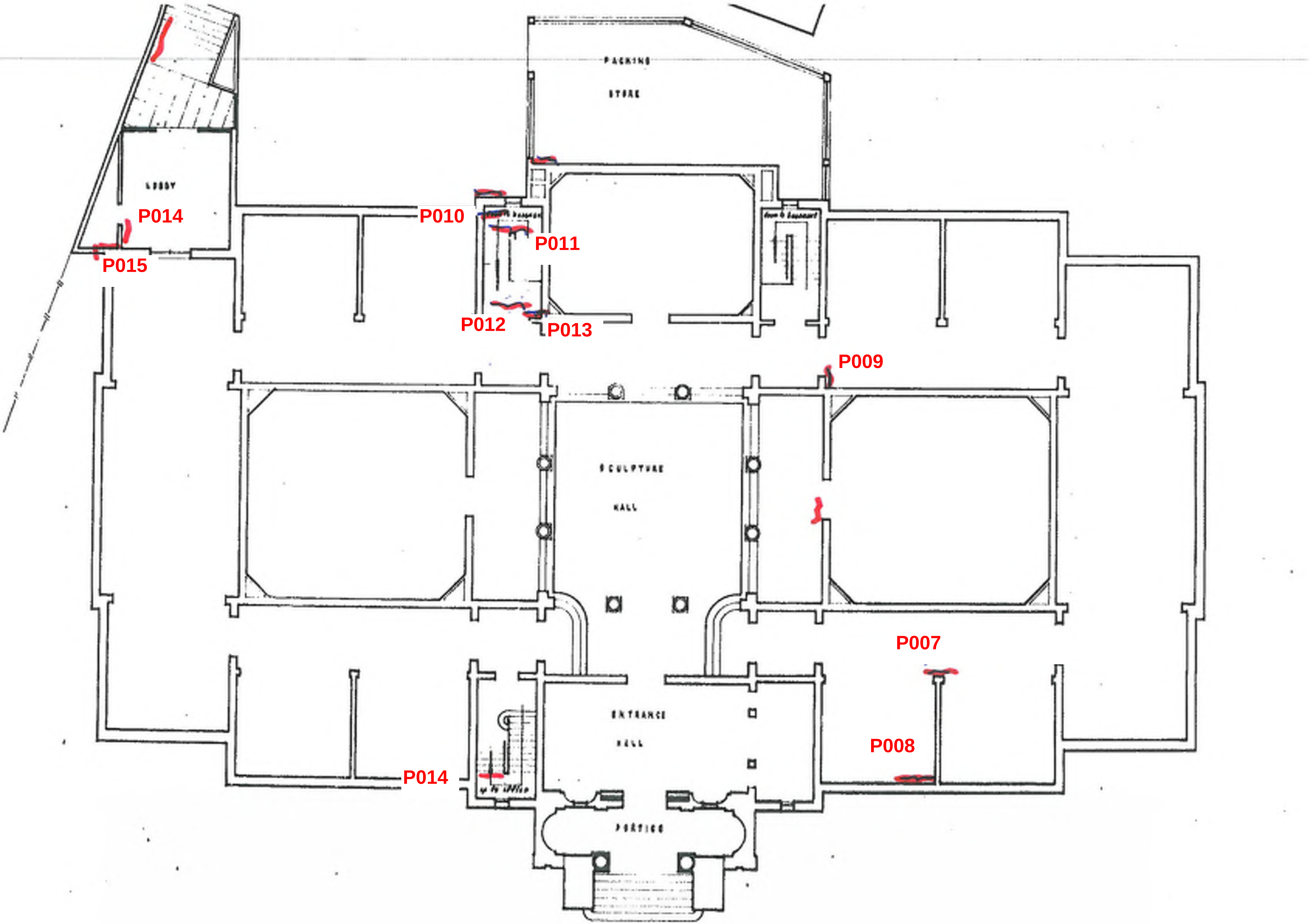
Level	Building Element	Location	Observations	Repair Required?	Photo Reference
Upstairs Floor			No significant damage observed.	N	
Main Floor	Interior Pilaster columns	South-west gallery.	Horizontal crack near top of pilaster 0.5mm width.	Y	P007
	Concrete lintel beams over doors	Various.	Minor cracking.	Y	P009
	Walls	West wall south-west gallery.	Moderate cracking at base of reinforced concrete bond beam level.	N	P008
		Opposite lift.	Moderate cracking noted above door to the lift and ceiling above Exterior walls have cracking in the concrete bond beam around this location.	Y	P010 P011 P012 P013
	Stairs	Various.	Some minor cracking in landings.	Y	P014
Basement	Perimeter basement walls	Various.	Cracking both vertical and diagonal in exterior basement walls of varying width up to 2.0mm.	Y	P017
	Interior basement walls	Various.	Range of crack patterns, particularly around door openings up to 1.0mm width. Some minor spalling of cover concrete on corners immediately above strip footings.	Y	P018 P020

Level	Building Element	Location	Observations	Repair Required?	Photo Reference
	Strip footings	South-west corner.	Indications from cracking and level survey that settlement has occurred at SW corner area of building.	N	P016 P019
Canaday Wing	Concrete block walls	Wall extension from the north-east corner of original building.	Some minor stepped cracking in mortar beds.	Y	P014

APPENDIX C

Location Labelling


P005 P006
P003 P004
P001 P002
CITY OF CHRISTCHURCH
POINTS A



PACKING
STORE
LOBBY
P014
P010
P011
P015
P012
P013
P009
SCULPTURE
HALL
P007
ENTRANCE
HALL
P008
P014
PORTICO

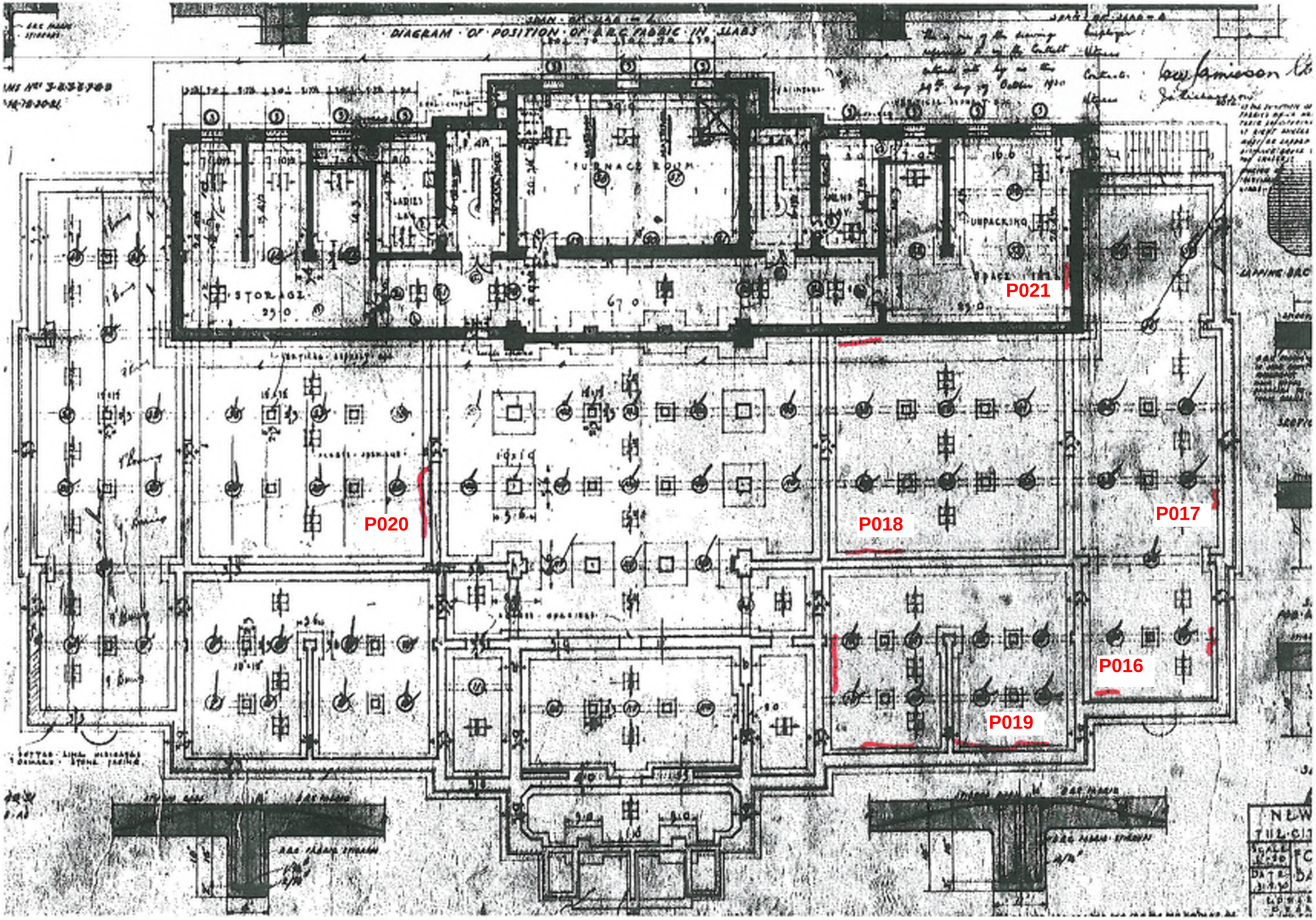
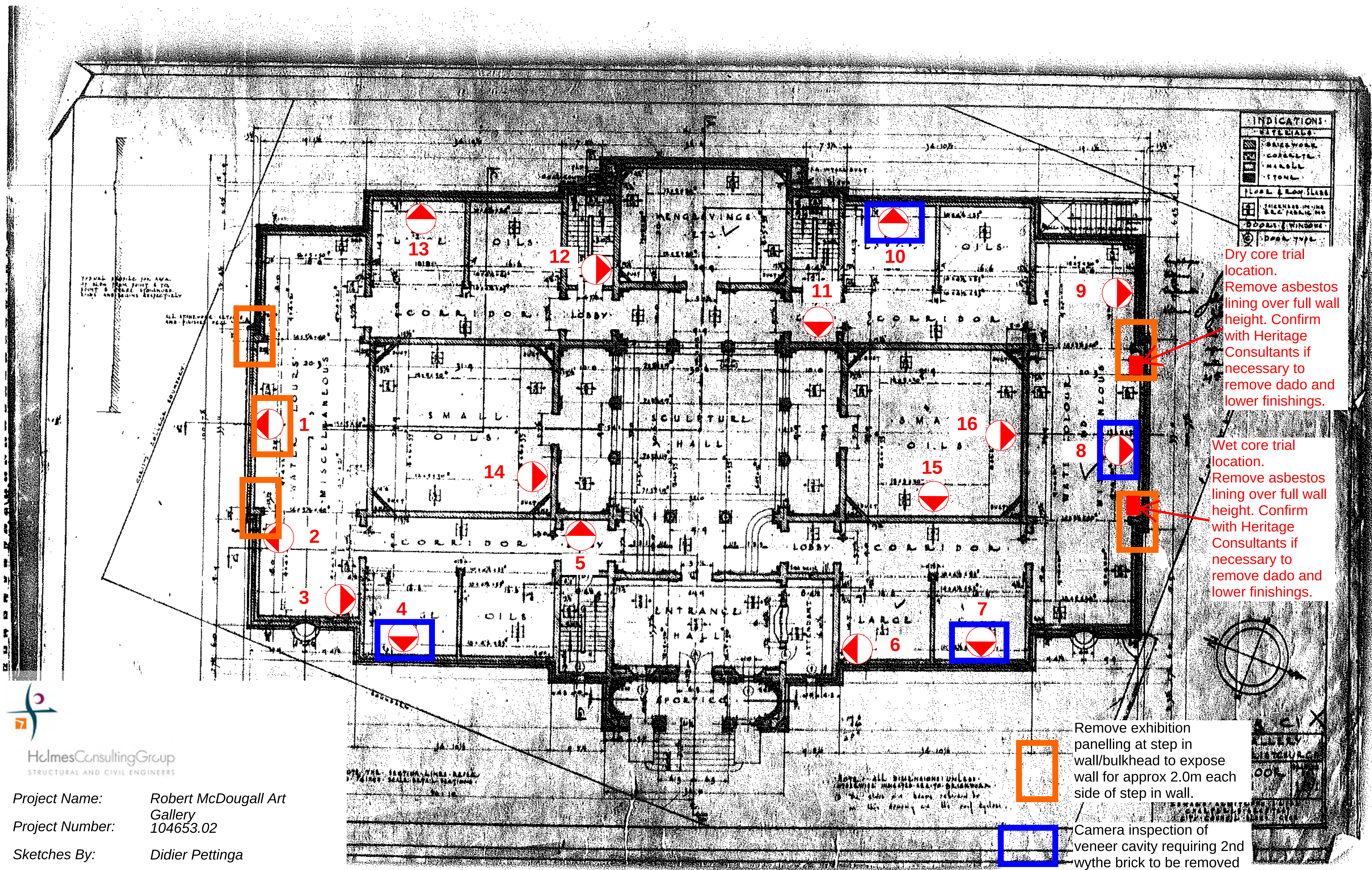


DIAGRAM OF POSITION OF B.R.C. FABRIC IN SLABS
FURNACE ROOM
STORAGE
LADIES
UNPACKING
P021
P020
P018
P017
P016
P019



HolmesConsultingGroup
STRUCTURAL AND CIVIL ENGINEERS

Project Name:	*Robert McDougall Art Gallery*
Project Number:	*104653.02*
Sketches By:	*Didier Pettinga*
Date:	*24/08/2012* and *8/4/2013*
Sketch Number:	*01*

KEY:

Remove exhibition panelling at step in wall/bulkhead to expose wall for approx 2.0m each side of step in wall.

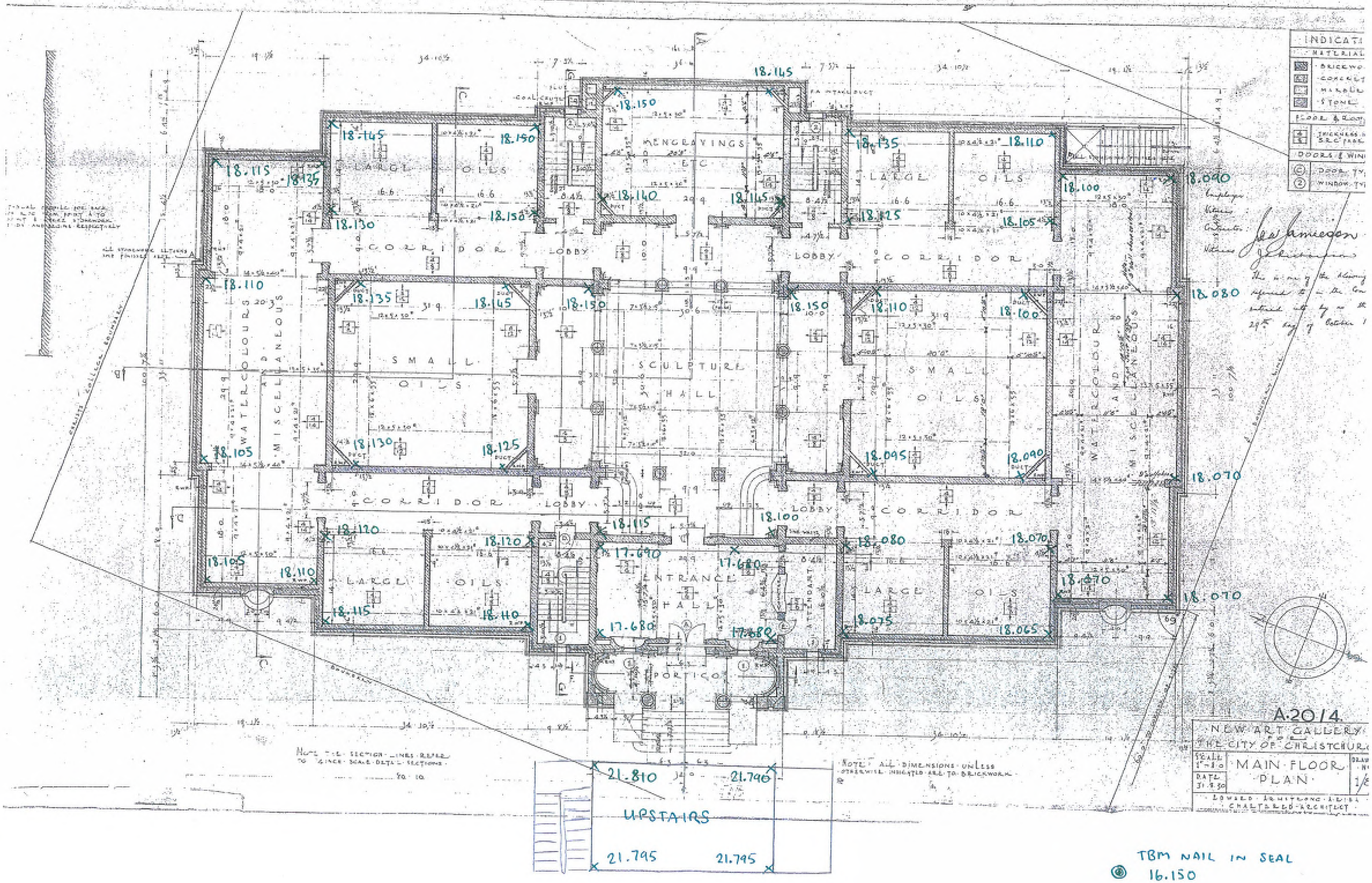
Camera inspection of veneer cavity requiring 2nd wythe brick to be removed at shear test location

in-place mortar shear test location

APPENDIX D

Level Survey

OLD ART GALLERY FLOOR LEVELS 31/08/2012



TBM NAIL IN SEAL
16.150

APPENDIX E

Geotechnical Report

REPORT

Christchurch City Council

Robert McDougall Gallery Building Geotechnical site walkover and desktop study report

Report prepared for:

Christchurch City Council

Report prepared by:

Tonkin & Taylor Ltd

Distribution:

Christchurch City Council	2 copies
Tonkin & Taylor Ltd (FILE)	1 copy

November 2012

T&T Ref: 53053.001

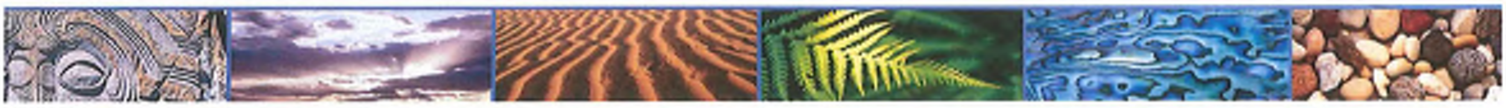


Table of contents

1 Introduction

1.1 General

This report summarises the results of a geotechnical walkover and desk study assessment of the Robert McDougall Art Gallery at 9 Rolleston Avenue, Christchurch, that has been completed by Tonkin & Taylor Ltd (T&T) for the Christchurch City Council (CCC).

The work described in this document was commissioned by CCC and has been completed in accordance with the terms and conditions outlined in T&T's letter of engagement dated 13 August 2012.

The earthquakes of 22 February 2011, and to a lesser extent those of 04 September 2010, 13 June 2011, and 23 December 2011 caused widespread land and structural damage throughout much of the Christchurch area. The purpose of our work was to conduct a site walkover inspection, review readily available building foundation drawings, and provide a preliminary assessment of foundation performance following the Canterbury Earthquake sequence.

The structural assessment of the building has been completed by Holmes Consulting Group (HCG).

1.2 Project description

The site is accessed off Rolleston Avenue and is within the Christchurch Botanic Gardens (Figure 1 in Appendix A). The building is located on an essentially flat site with an approximately rectangular footprint of 25 by 50 m. The site is situated within a meander of the Avon River which runs more than 100 m to the north and south.

The single story reinforced concrete and brick building with a partial single story basement was constructed in the early 1930's. The basement was extended to cover much of the building footprint during the 1980's. T&T understand, based on structural drawings from the 1930's provided by HCG that the building is supported on a combination of strip and pad footings.

T&T have previously completed site specific geotechnical investigations for the Canterbury Museum on the adjacent site of 11 Rolleston Avenue which have been detailed in T&T's report dated March 2012[1]. The Canterbury Museum has given permission for CCC to use the site specific geotechnical data obtained for their site.

1.3 Scope of work

The following scope of work has been completed by T&T for the purposes of this report:

- Compilation of existing geotechnical data from readily available sources near the subject site;
- A geotechnical site walkover inspection of the Robert McDougall Art Gallery building;
- Preparation of a preliminary geotechnical model for the site;
- Engineering analysis of the above model to evaluate liquefaction susceptibility and seismic settlement potential;
- A preliminary assessment of foundation damage due to the Canterbury Earthquake sequence; and,
- Preparation and issue of this geotechnical walkover and desk study assessment report.

[1] Tonkin & Taylor Ltd (March 2012) "*Canterbury Museum Geotechnical Investigation and Assessment Report.*"

2 Field investigations

2.1 Site walkover inspection

A walkover inspection of the site was conducted by T&T geotechnical engineers on 08 October 2012. The results of this work are described in Sections 3, 4 and 5 of this report.

2.2 Machine drilled boreholes[2]

Three machine-drilled boreholes at the adjacent 11 Rolleston Avenue site had been undertaken on 12 to 19 December 2011 at the approximate locations shown on the attached site plan (Figure 1, Appendix A). The boreholes were advanced to depths of between 25.0 and 25.9 m below ground level (bgl) using a sonic direct push drill rig. A detailed description of the drilling conducted, and logs of the boreholes, are presented in Appendix B.

2.3 Cone penetration testing[2]

Three Cone Penetration Tests (CPTs) at the adjacent 11 Rolleston Avenue site had been undertaken on 16 December 2011 at the approximate locations shown on the attached site plan (Figure 1, Appendix A). They were then advanced to refusal at depths of between 3.4 and 5.4 m bgl. A detailed description of the CPT testing conducted, and the CPT investigation results, are presented in Appendix B.

[2] Machine drilled borehole and Cone Penetration Test data has been generously provided for use at the Robert McDougall Art Gallery by Canterbury Museum.

3 Investigation findings

3.1 Site walkover

Relatively little damage was observed within the first floor of the Robert McDougall Art Gallery. Several cracks were present within the floor slab. However, based on conversations with Art Gallery maintenance staff T&T understand that these cracks are likely to have occurred prior to the Canterbury Earthquake sequence.

T&T observed a number of cracks within the walls and floor of the basement. No evidence of building settlement or foundation bearing capacity failure was observed during our site walkover inspection.

No liquefaction ejected sand or silt was observed on the site during T&T's site walkover inspection. Art Gallery maintenance staff said that no ejected sand or silt was present on the site following the 22 February 2011 earthquake.

3.2 Subsurface conditions

3.2.1 Published geology and geotechnical information

Published geological information[3] describes the subject site as being underlain by predominantly alluvial sand and silt overbank deposits of the Springston Formation. The Springston Formation generally consists of Holocene fluvial channel and overbank sediments composed of well-sorted gravel, sand and silt.

Environment Canterbury (ECan) bore logs are available for a number of wells in the area. A review of this data indicates that the site is likely to be underlain by:

- 1.5 to 5.0 m of sandy silt and silty sand;
- 7.0 to 8.5 m of sandy gravel;
- 10.0 to 13.0 m of interbedded sand and silt; and,
- An unknown thickness of gravel from between 20.0 to 25.0m bgl.

It should be noted that the ECan well logs were not made for geotechnical purposes, and as such, should only be used to provide a general indication of subsurface conditions.

3.2.2 T&T geotechnical investigations

T&T undertook three machine drilled boreholes and three Cone Penetration Tests (CPTs) at the Canterbury Museum site adjacent to the Robert McDougall Art Gallery site. These investigations indicate that the site is generally underlain by the following general succession of strata:

- 0.0 to 3.5 m bgl of stiff silt and loose silty sand; overlying,
- 3.5 to 4.25 m bgl of medium dense gravel;
- 4.25 to 5.25 m bgl of medium dense sand;
- 5.25 to 10.0 m bgl of dense to very dense gravel;
- 10.0 to 15.0 m bgl of dense to very dense sand;
- 15.0 to 21.0 m bgl of interbedded stiff silt and medium dense silty sand;

---

[3] LJ Brown et al, *Geology of the Christchurch Urban Area*, Institute of Geological and Nuclear Sciences Ltd, New Zealand, 1992.

- 21.0 to 23.5 m bgl of firm to very stiff silt; and,
- More than 2.5 m of very dense gravel from approximately 23.5 m bgl.

3.2.3 Generalised subsurface profile

The inferred generalised site subsurface profile is summarised in Table 1. This profile has been derived from the results of our borehole and CPT investigations undertaken at the adjacent Canterbury Museum site and is supplemented by information which is available on the ECan database and published geological information.

Table 1: Generalised subsurface profile

Soil layer No.	Soil Description	Geologic Member	Approximate depth to top of layer (m)	Approximate layer thickness (m)	Typical CPT cone tip resistance qc (MPa)	Typical SPT N_{60} value (blows/300mm)
1a	SILT, stiff and silty SAND, loose	Yaldhurst Member, Springston Formation	0.0	3.5	1 - 3	6 - 10
1b	GRAVEL, medium dense		3.5	0.75	N/A	14
1c	SAND, medium dense		4.25	1.0	-	14
1d	GRAVEL, dense to very dense		5.25	4.75	N/A	30 – 50+
2a	SAND, dense to very dense	Christchurch Formation	10.0	5.0	-	30 – 50+
2b	Interbedded SILT, stiff and silty SAND, medium dense		15.0	6.0	-	4 - 18
2c	SILT, firm to very stiff		21.0	2.5	-	7 - 28
3a	GRAVEL, very dense	Riccarton Gravel	23.5	2.5 +	N/A	>50

3.2.4 Groundwater

Groundwater was identified at a depth of approximately 3.5 m bgl within the machine drilled borehole and CPT investigations. Groundwater levels were found to be lower within borehole and CPT 03. This is inferred to be due to the higher elevation of the ground surface at this location. Groundwater levels at the site may fluctuate over time due to variations in rainfall, runoff conditions, levels in the nearby water courses and other factors.

4 Liquefaction assessment

4.1 Seismicity

The Christchurch region can be considered as a seismically active area. Recent significant earthquakes that resulted in moderate to strong ground shaking at the project site include:

- A Moment Magnitude (M_w) 7.1 event on 04 September 2010, located approximately 37 km west of the site;
- A M_w 6.2 event on 22 February 2011, located approximately 7 km southeast of the site;
- A M_w 6.0 event on 13 June 2011, located approximately 10 km east southeast of the site; and,
- A M_w 5.9 event on 23 December 2011, located approximately 10 km east of the site.

Recent research indicates that east Canterbury could experience an extended period of heightened seismic activity relative to the past century. Additionally, there is a relatively high probability of a large earthquake occurring on the Alpine Fault at some point within the next 50 years. Such an earthquake is anticipated to produce relatively strong ground shaking in the Christchurch area. Consequently, we judge that the site is likely to be subjected to moderate to strong earthquake shaking during the life of any structure located on the site.

4.2 Design earthquake scenarios

The earthquake scenarios which were used in our liquefaction analyses of the site are summarised in Table 2. The peak horizontal ground accelerations are in line with the Department of Building and Housing guidelines.

Table 2: Summary of the earthquake scenarios used in the liquefaction assessment

	Serviceability Limit State (SLS)	Ultimate Limit State (ULS)	22 February 2011 earthquake
Return period (years)	25	500	-
Moment Magnitude, M_w	7.5	7.5	6.2
Peak horizontal ground acceleration, PGA	0.13g	0.35g	0.45g$^{(1)}$ 0.32g$^{(2)}$

1 Peak ground acceleration averaged from the University of Cornell interpretations near the site.

2 Peak ground acceleration corrected using the magnitude scaling factor to derive an equivalent pga for a 7.5 magnitude earthquake

NZS 1170 Scenarios

Two design earthquake scenarios were derived from "NZS 1170 – Structural Design Actions" assuming an Importance Level 2 building with a 50 year design life. These scenarios represent the following design performance requirements:

- Serviceability Limit State (SLS) – to avoid damage that would prevent the structure from being used as originally intended, without structural repair (though maintenance may be required e.g. patching of cracks); and,
- Ultimate Limit State (ULS) – to avoid collapse of the structural system (though significant loss of serviceability may result).

In terms of NZS 1170, a site sub-soil class of D (deep soils) was assumed due to the unknown, but typically great depth to bedrock (more than 60m) in the Christchurch area.

<u>22 February 2011 Earthquake</u>

Our liquefaction analysis also included a ground motion from the 22 February 2011 earthquake interpreted by the University of Cornell and averaged from locations near to the site. The results of this analysis were used to help calibrate the estimates of liquefaction settlement under future SLS and ULS ground shaking. The peak ground acceleration has been corrected using the appropriate magnitude scaling factor to convert the M_w 6.2 earthquake to the design M_w 7.5 earthquake. This indicates that the peak ground accelerations experienced during the 22 February 2011 earthquake were slightly less than the ULS acceleration adopted for design.

4.3 Liquefaction analysis

Seismic liquefaction occurs when excess pore pressures are generated in loose, saturated, generally cohesionless soil during earthquake shaking, causing the soil to undergo a partial to complete loss of shear strength. Such a loss of shear strength can result in settlement and/or horizontal movement (lateral spreading) of the soil mass. The occurrence of liquefaction is dependent on several factors, including the intensity and duration of ground shaking, soil density, particle size distribution, and elevation of the groundwater table.

Analyses were undertaken to evaluate the liquefaction potential of the loose to medium dense sands and non-plastic/low plasticity silts found in the boreholes and CPT soundings utilising the methods recommended by Idriss and Boulanger (2008)[4]. The three earthquake scenarios described in Table 2, and a ground water level of 3.5 to 4.5 m bgl, were assumed in the analyses. A summary of the liquefaction analysis is presented in Appendix C.

The results of the analyses are presented in Table 3.

[4] Idriss, I. And Boulanger, R. (2008). "Soil liquefaction during earthquakes," Earthquake Engineering Research Institute.

Table 3: Liquefaction susceptibility under SLS and ULS seismic shaking

Soil layer No.	Soil Description	Approximate depth to top of layer (m)	Approximate layer thickness (m)	Liquefiable under SLS level earthquake	Liquefiable under ULS level earthquake
1a	SILT, stiff and silty SAND, loose	0.0	3.5	No[1]	No[1]
1b	GRAVEL, medium dense	3.5	0.75	No	No
1c	SAND, medium dense	4.25	1.0	Discrete lenses	Yes
1d	GRAVEL, dense to very dense	5.25	4.75	No	No
2a	SAND, dense to very dense	10.0	5.0	No	No
2b	Interbedded SILT, stiff and silty SAND, medium dense	15.0	6.0	Unlikely[2]	Unlikely[2]
2c	SILT, firm to very stiff	21.0	2.5	No	No
3a	GRAVEL, very dense	23.5	2.5 +	No	No

1 Potentially liquefiable if ground water level raises within this layer.
2 Unlikely to liquefy due to depth of soil layer.

4.3.1 Settlement

Estimates of settlement induced by liquefaction of the subsurface materials are presented in Table 4. These estimates were made using the methodology developed by Zhang, Robertson and Brachman (2002)[5].

It should be noted that the settlement values presented in Table 4 are total, free field settlement estimations. This describes the settlement of ground not occupied by a building, occurring due to dissipation of excess pore water pressure generated during earthquake shaking. An additional component of building settlement may also occur due to yield of the liquefied soils under foundation loading. This component of settlement is very difficult to predict and depends on the interaction of the building and the soil it is founded on.

5 Zhang, Robertson and Brachman (2002). "Estimating liquefaction-induced ground settlements from CPT for level ground," Canadian Geotechnical Journal.

Table 4: Summary of estimates of liquefaction induced free-field settlement

Description	SLS Seismic Event (M=7.5, PGA=0.13g)	ULS Seismic Event (M=7.5, PGA=0.35g)	22 February 2011 Seismic Event (M=6.2, PGA=0.45g)
Estimated liquefaction induced total settlement	5 - 25 mm	60 - 70 mm	60 - 70 mm
Estimated liquefaction induced differential settlement at surface	Less than 15 mm	Up to 40 mm	Up to 40 mm

The above settlements were calculated using simplified methods based largely on empirical data from homogenous soil sites. It must also be noted that while estimates of settlements are provided above for SLS and ULS level earthquake events, settlement can occur before a SLS level event has occurred and significant settlement (similar to the above ULS values) can occur before a ULS level earthquake has been reached.

Based on T&T's post-earthquake observations, the above predicted settlements may be conservative (an over-estimate) for some sites in Christchurch. In addition, subsurface conditions and soil properties may vary substantially across the site making accurate predictions of future seismic settlement extremely difficult. Therefore, engineering judgment should be applied when interpreting the computed settlements presented above.

In terms of the New Zealand Geotechnical Society[6] (NZGS) guidelines, the level of liquefaction estimated to occur at the site under ULS loading can be considered to correspond to a *Liquefaction Performance Level* of L2 ("moderate") – "Liquefaction occurs in layers of limited thickness (small proportion of the deposit); ground deformation results in differential settlements."

Under SLS loading, the site can be classified as *Liquefaction Performance Level* of L1 ("mild") – "Limited excess pore water pressures without complete liquefaction; relatively small deformation of the ground with relatively small settlements (few tens of millimetres)".

The observed damage at the site following the 22 February 2011 earthquake was generally consistent with an NZGS performance level of L1 ("mild").

4.3.2 Sand Boils

Sand boils occur when liquefied soils at depth break through to the ground surface through fissures, cracking and/or weak crustal soils. This phenomenon can lead to bearing capacity failure and the creation of voids in subsoil zones, and is a significant cause of differential settlement beneath foundations, slabs, roads, etc.

Empirical correlations have been developed by Ishihara[7] to quantify the thickness of non-liquefiable surface crust required to prevent the formation of sand boils resulting from the liquefaction of underlying soil layers. These correlations indicate that for a given thickness of liquefiable soil, as the peak ground acceleration increases a greater surface crust thickness of non-liquefiable soil is required to prevent liquefaction damage from manifesting on the surface.

[6] New Zealand Geotechnical Society, *Geotechnical earthquake engineering practice, Module 1 – Guideline for the identification, assessment and mitigation of liquefaction hazards*, July 2010

[7] Ishihara, K. (1985). "Stability of natural deposits during earthquakes," Theme lecture, Proc. 11th Int. Conf. On Soil Mechanics and Foundation Engineering, San Francisco, 2, 321-376pp.

The results from CPT soundings and the Ishihara correlations indicate that the thickness of the non-liquefiable surface crust overlying the liquefiable layers at the site may be sufficient to prevent sand boils and abrupt differential settlement at the ground surface during a SLS, ULS and 22 February 2011 earthquake event.

Sand boils were not observed at the site during our site walkover of 08 October 2012. Additionally, Robert McDougall Art Gallery maintenance staff said no sand boils were observed onsite following the 22 February 2011 earthquake. Aerial photos taken shortly after the 22 February 2011 earthquake[8] do not show any evidence of sand boils in the immediate area surrounding the site. The potential for loss of foundation ground support on the site due to sand boiling during a future SLS or ULS seismic event is judged to be low.

4.3.3 Lateral Spreading

Lateral spreading is generally defined as the horizontal displacement of surficial blocks of soil towards an open slope face as a result of liquefaction of the underlying soils. The occurrence of lateral spreading generally requires the presence of a relatively continuous liquefiable layer extending to an open slope face such as a river bank or open channel. Displacements can range from a few centimetres to a metre or more.

The nearest open slope or river channel is the Avon River, which runs more than 100 m to the north and south of the site.

Lateral spreading was not observed following the 04 September 2010, 22 February, and 13 June 2011 earthquakes. This is likely due to the distance to an open slope face. The risk of lateral spreading in a future SLS and ULS earthquake event is considered to be low.

[8] Publicly available from koordinates.com

5 Foundations

5.1 General

T&T understand, based on the original structural drawings from the 1930's, that the Robert McDougall Art Gallery is supported on pad footing at column locations with strip footings connecting the pad footings. The dimensions of the existing strip and pad footings are shown to vary over the building footprint. Limited information is available on the drawings regarding the width, depth and thickness of the footings. The original section of the basement appears to have the same foundation type as the rest of the structure.

The building's basement has been extended in the 1980's to encompass much of the building footprint. Structural drawings showing the extensions to the basement have not been available for review. Observations during T&T's walkover inspection indicate that the extensions to the basement are likely founded on strip and pad footings beneath the columns and walls in a similar layout to the original 1930's drawings. However, the depth, width and thickness of these foundations are unknown.

5.2 Foundation performance

No evidence of significant foundation settlement or bearing capacity failure was observed during T&T's walkover inspection. It is unlikely that significant voids have been created beneath the building's floor slabs or foundations due to seismic settlement or the formation of sand boils.

T&T conclude that it is unlikely that damage to the Robert McDougall Art Gallery building following the Canterbury Earthquake sequence was due to geotechnical foundation issues.

6 Conclusions and recommendations

6.1 General

Excess pore water pressures for the recent earthquakes and associated aftershocks are expected to have dissipated. The strength of the soil underlying the site is expected to have returned close to the pre-earthquake levels.

The inferred generalised site subsurface profile is summarised as:

- 3.5 m of stiff silt and loose sand; overlying, (Layer 1a)
- 0.75 m of medium dense gravel; (Layer 1b)
- 1.0 m of medium dense sand; (Layer 1c)
- 4.75 m of dense to very dense gravel; (Layer 1d)
- 5.0 m of dense to very dense sand; (Layer 2a)
- 6.0 m of interbedded stiff silt and medium dense silty sand; (Layer 2b)
- 2.5 m of firm to very stiff silt; (Layer 2c)
- Very dense gravel from approximately 23.5 m bgl for at least 2.5 m. (Layer 3a)

Much of the medium dense sand (Layer 1c) is likely to liquefy under ULS earthquake shaking with portions of this layer likely to liquefy under SLS earthquake shaking. Additionally, the stiff silt and loose silty sand (Layer 1a) may liquefy under ULS and SLS earthquake shaking if it is below the ground water table.

In terms of the New Zealand Geotechnical Society[9] (NZGS) guidelines, the level of liquefaction estimated to occur at the site under ULS loading can be considered to correspond to a *Liquefaction Performance Level* of L2 ("moderate") – "Liquefaction occurs in layers of limited thickness (small proportion of the deposit); ground deformation results in differential settlements."

Under SLS loading, the site is classified in term of the NZGS guideline as Performance Level L1 ("mild") – "Limited excess pore water pressures without complete liquefaction; relatively small deformation of the ground with relatively small settlements (few tens of millimetres)".

6.2 Foundations

T&T understand that the Robert McDougall Art Gallery building is supported on strip and pad footings. The width, depth and thickness of the building's foundations are currently unknown. It is recommended that foundation dimension be confirmed in order to assess likely foundation capacities.

No evidence of significant foundation settlement or bearing capacity failure was observed during T&T's walkover inspection. It is unlikely that significant voids have been created beneath the building's floor slabs or foundations due to seismic settlement or the formation of sand boils.

T&T conclude that it is unlikely that damage to the Robert McDougall Art Gallery building following the Canterbury Earthquake sequence was due to geotechnical foundation issues.

[9] New Zealand Geotechnical Society, *Geotechnical earthquake engineering practice, Module 1 – Guideline for the identification, assessment and mitigation of liquefaction hazards*, July 2010

7 Applicability

This report was prepared for the benefit of Christchurch City Council with respect to the particular brief given to us and it may not be relied upon in any other context or for any other purpose without our prior review and written agreement.

The recommendations and opinions which are contained in this report are based upon data from the currently available geotechnical investigations, and observations of surface features. The nature and continuity of sub-surface conditions away from the investigation locations is inferred, and it must be appreciated that the actual conditions may vary from the assumed model.

The susceptibility analyses carried out represent probabilistic analyses of empirical liquefaction databases under various earthquakes. Earthquakes are unique and impose different levels of shaking in different directions on different sites. The results of the liquefaction susceptibility analyses and the estimates of consequences presented within this document are based on regional seismic demand and published analysis methods, but it is important to understand that the actual performance may vary from that calculated.

It is important that Tonkin & Taylor Ltd be immediately contacted if there is any variation in subsoil conditions from those which are described in this report.

Tonkin & Taylor Ltd

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MIPENZ, CPEng

Authorised for Tonkin & Taylor Ltd by:

Peter Millar
Project Director
FIPENZ, CPEng

adw

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APPENDIX F

Mortar Shear Strength Testing

Determination of Shear Strength of Unreinforced Masonry

RMAG
104653.02
JDP
11/09/2012

Using shear test results provided by Holmes Solutions

Slip load taken as average of Left and Right displacements reported in test results

Mortar Dimensions for shear area			**In place gravity load**		
Backfill height	80	mm	Approx.	50	kPa
Width	220	mm			
Depth	105	mm			

Following ASCE41-13 Draft recommentions. These are then used to determine an equivalent shear strength using NZSEE "Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Resistance" Feb 2011

Test #	Slip Load L	Slip Load R	Slip Stress		v_{t0}		v_{te}	
	kN	kN	MPa		MPa		MPa	
MCD1	46	40	0.674		0.624		0.616	OK below 100psi limit
MCD2	22.5	29	0.404		0.354			
MCD3	50	50	0.784		0.734		v_{me}	
MCD4	43	3.5	0.364		0.314		MPa	
MCD5	52	52	0.815		0.765		0.26	
MCD6	47	50	0.760		0.710			
MCD7	35	35	0.549		0.499			
MCD8	51	51	0.799		0.749			
MCD9	52	40	0.721		0.671			
MCD10	42	32	0.580		0.530			
MCD11	51	51	0.799		0.749			
MCD12	52	52	0.815		0.765			
MCD13	40	25	0.509		0.459			
MCD14	51	47	0.768		0.718			
MCD15	32	32	0.502		0.452			
MCD16	52	52	0.815		0.765			
Average		**42.5**	**0.666**	MPa	**0.616**	MPa		
Lower 20th Percentile		**32.0**	**0.509**	MPa	**0.459**	MPa		

→ Need to determine f_j' per NZSEE 2/2011 doc.

$c = 0.045 f_j'$

$b_c = v_{me}$.

$\therefore$ w/ $c = v_{me} = 0.26$ MPa.

$f_j' = 0.26/_{0.045} = 5.8$ MPa

$\mu_f = 0.65$.

$f_j' = 5.8$ MPa $\simeq$ Medium hardness (5.5 MPa)

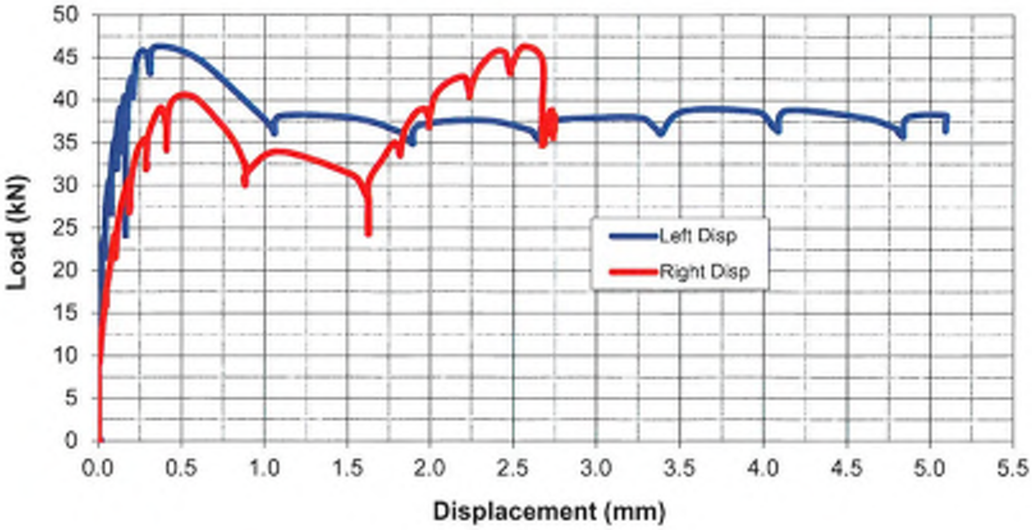
$\therefore f_{jl}' = 3.2$ MPa

But do we use average "expected" props seeing this is based on testing

yes, as recommended in guidelines p.2-5.

$\therefore f_j' = 5.8$ MPa

MCD1



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Displacement (mm)
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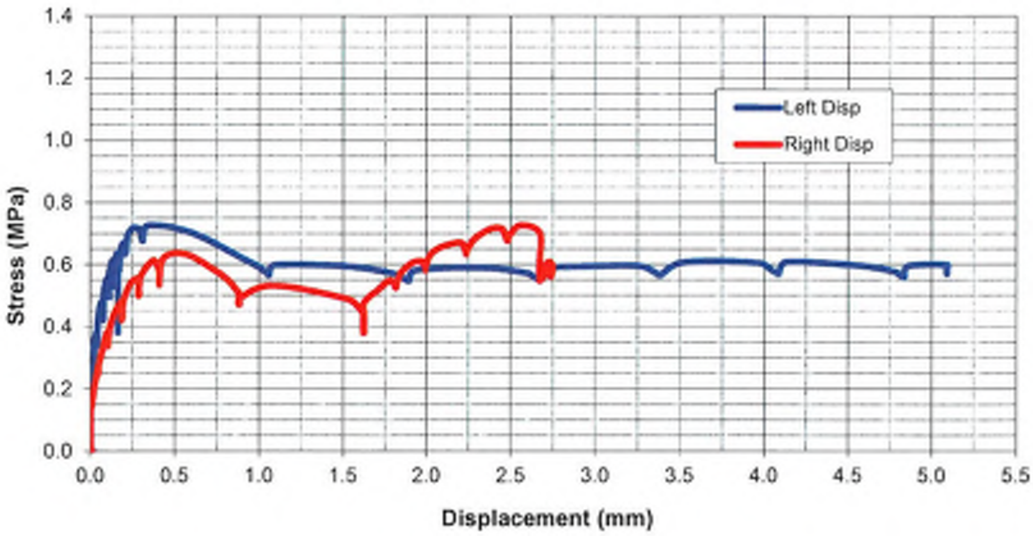
Right Disp



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Displacement (mm)
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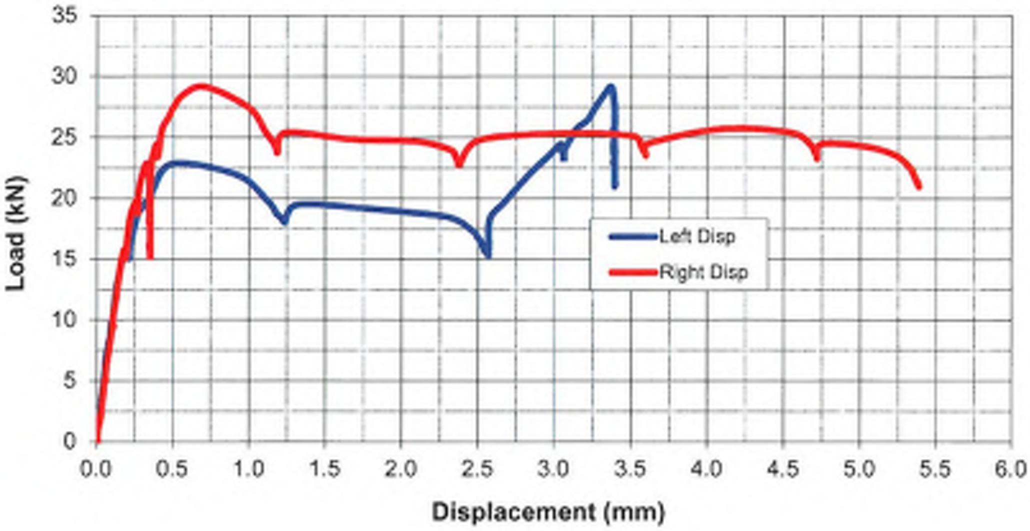
MCD2



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Displacement (mm)
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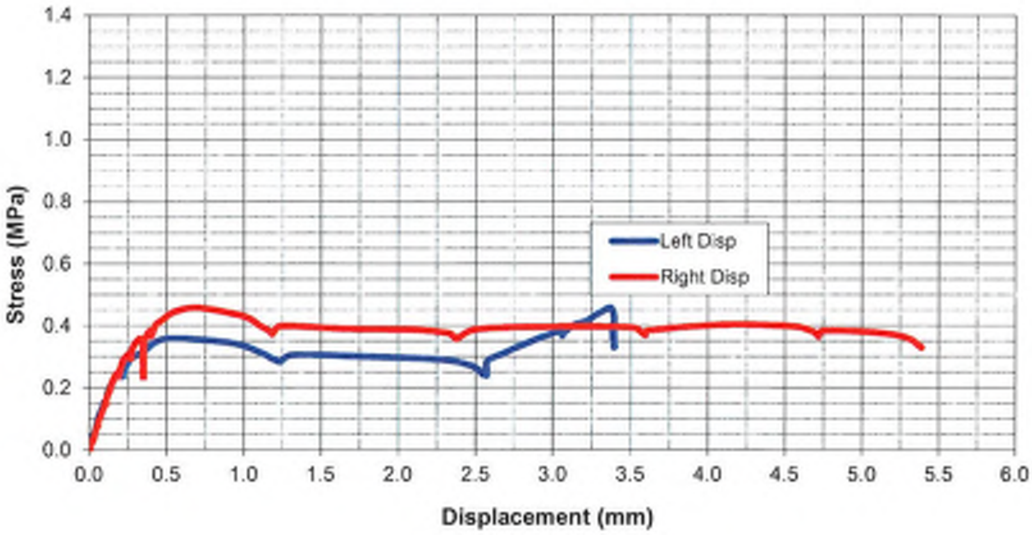

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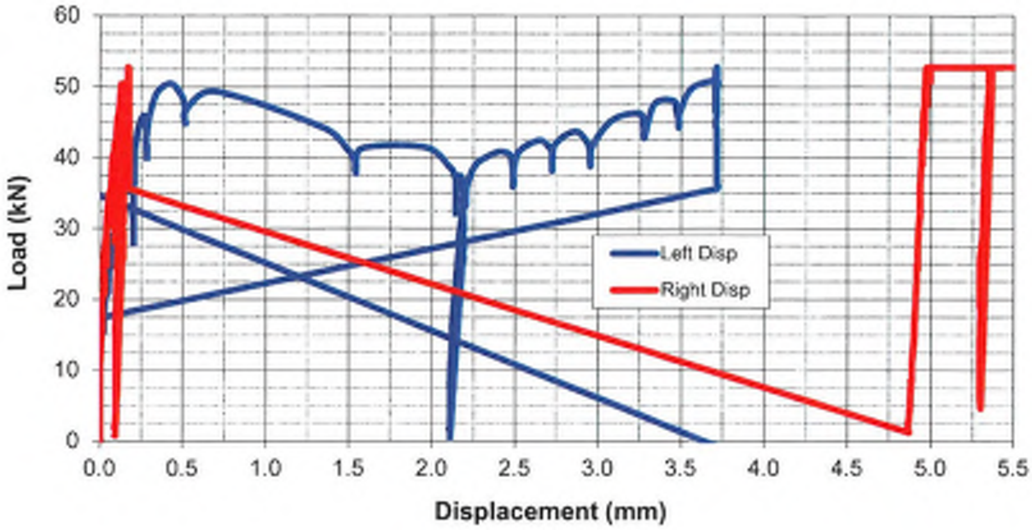
Displacement (mm)
Left Disp
Right Disp

MCD3



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Left Disp
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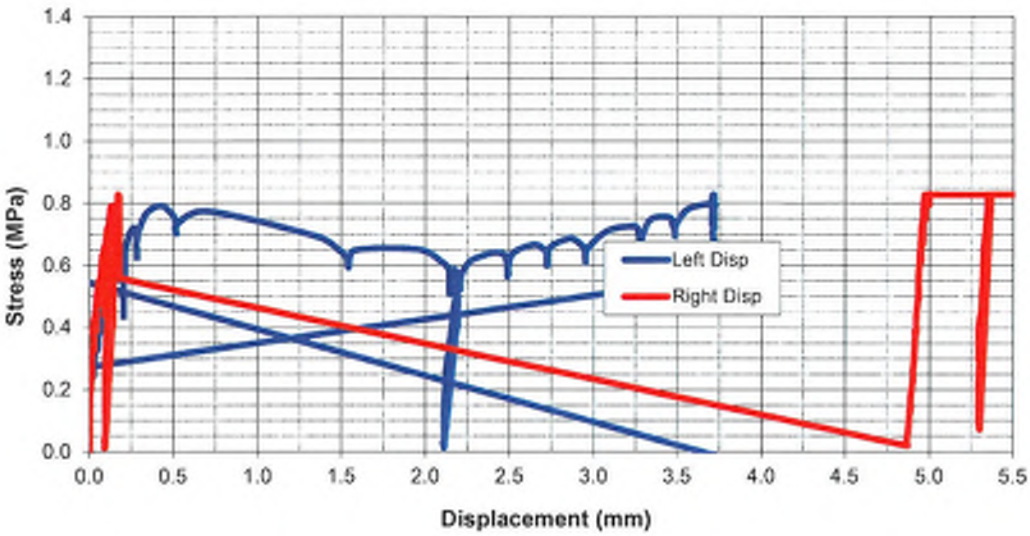

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Displacement (mm)
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Right Disp

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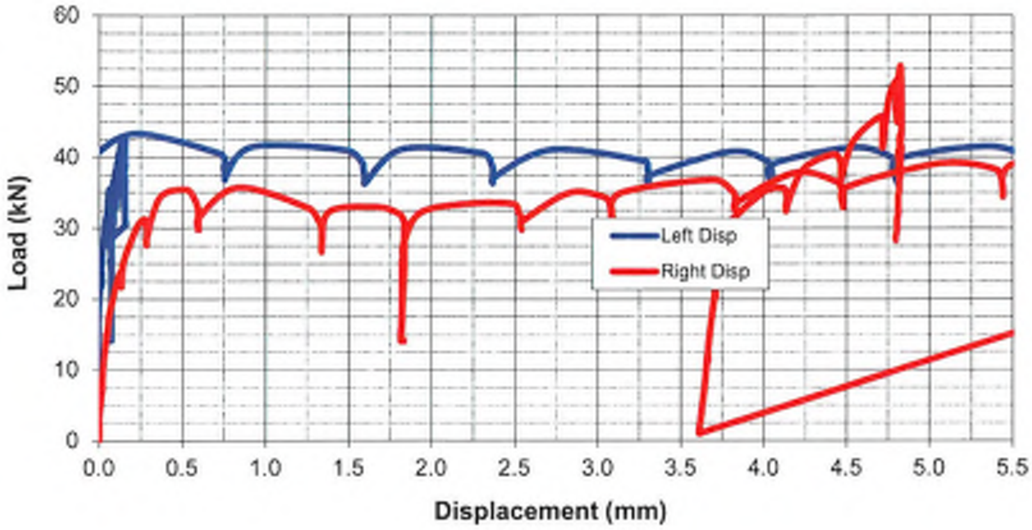
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4.0
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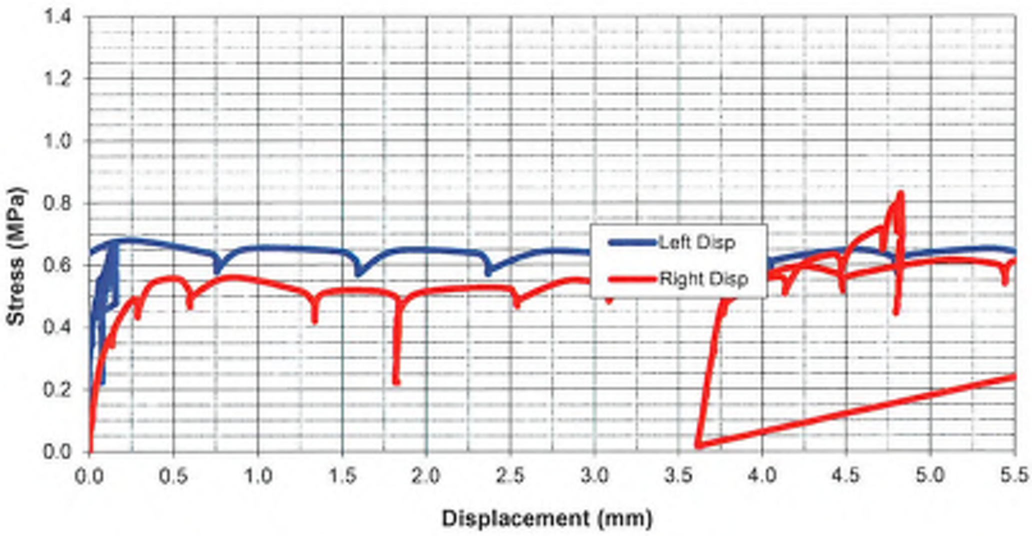
5.5
Displacement (mm)
Left Disp
Right Disp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4.5
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Displacement (mm)
Left Disp
Right Disp

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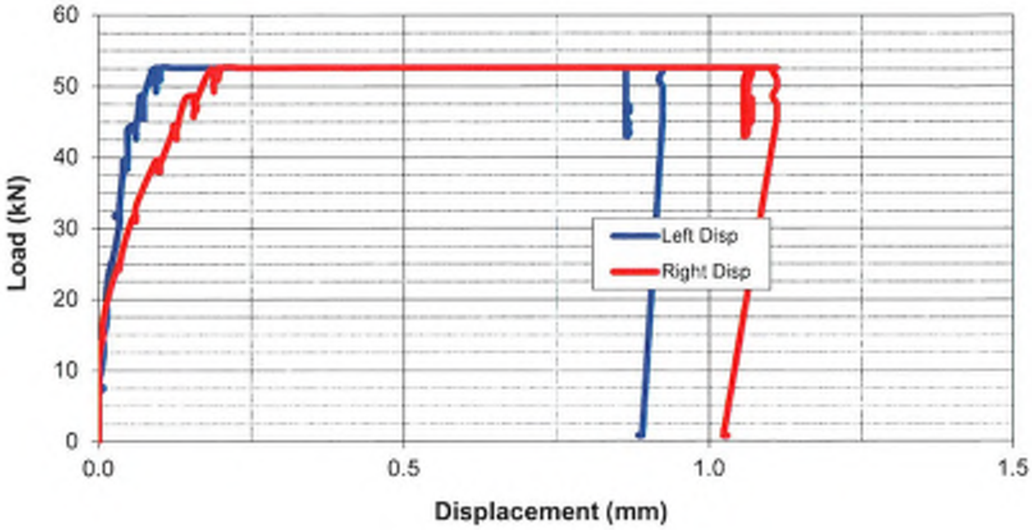
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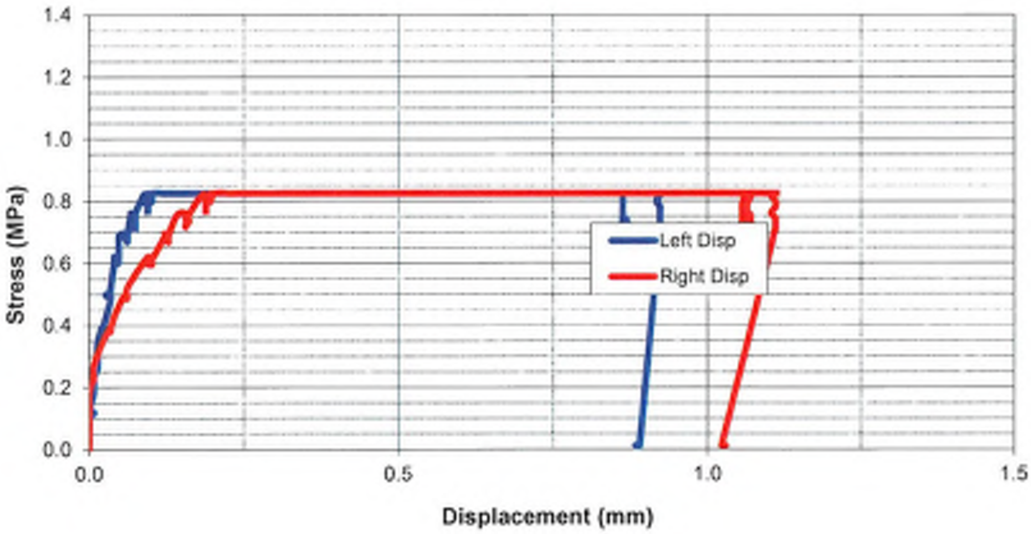
Displacement (mm)
Left Disp
Right Disp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Displacement (mm)
Left Disp
Right Disp

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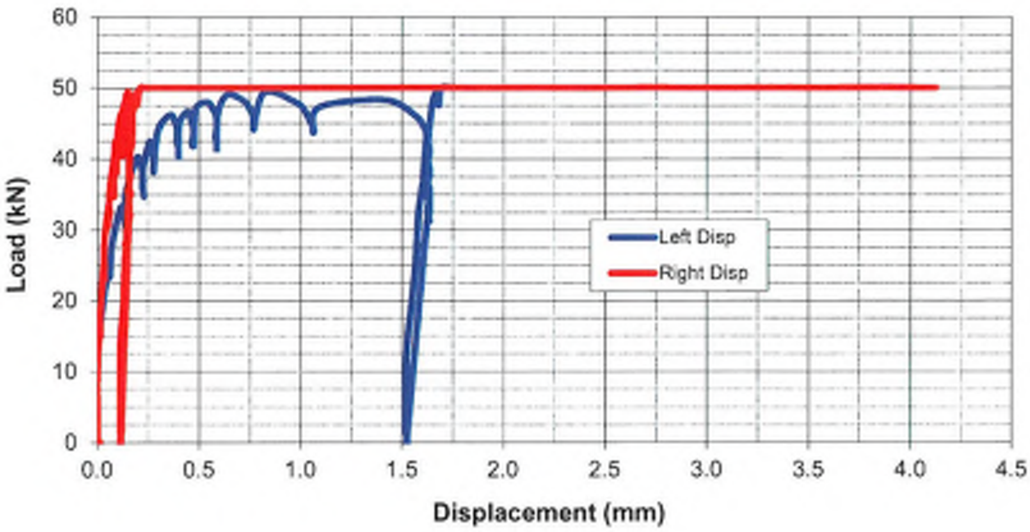

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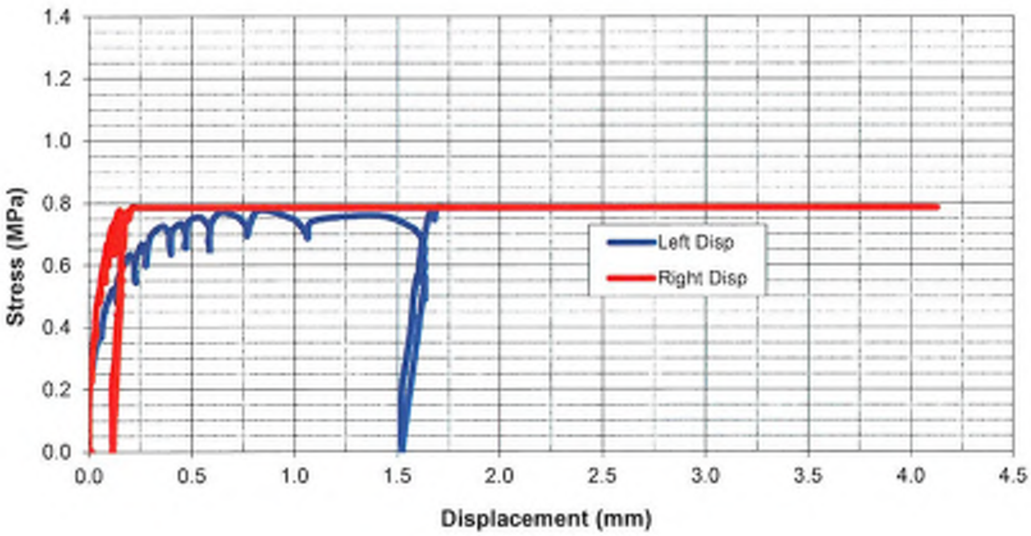

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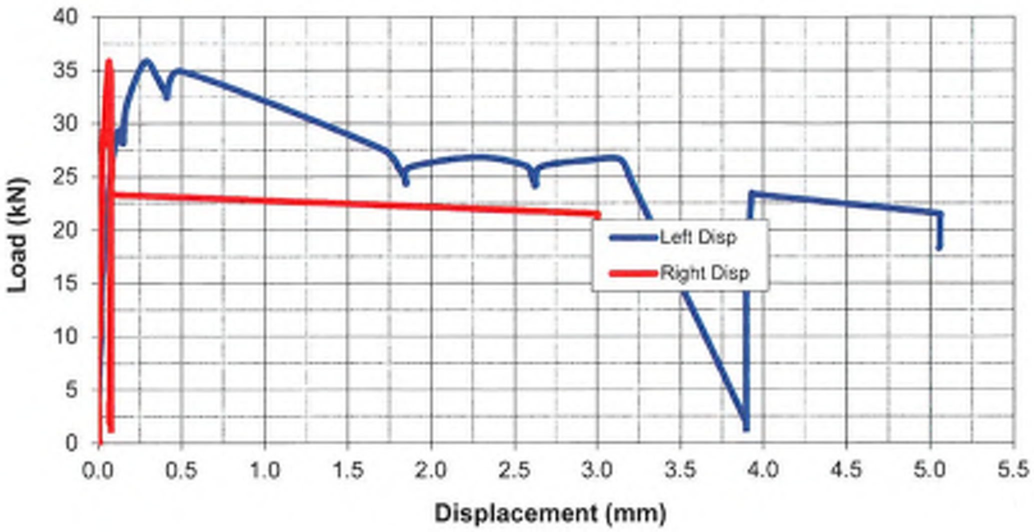
MCD7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Right Disp
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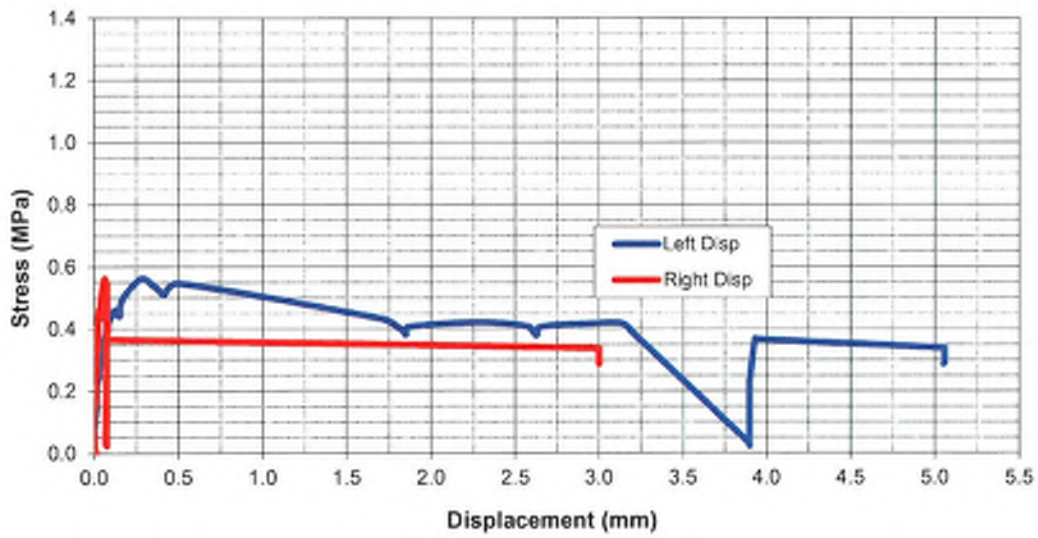
5.0
5.5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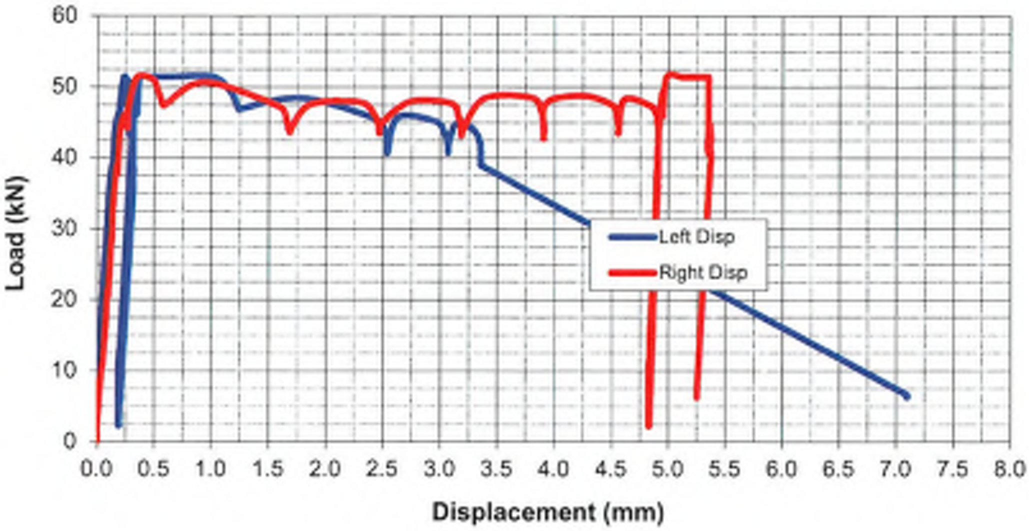
MCD8



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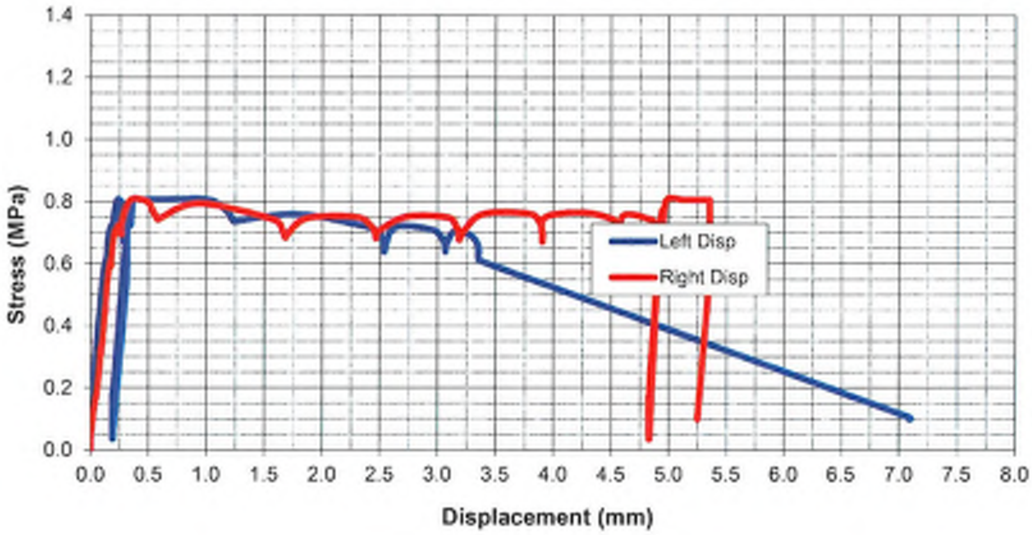

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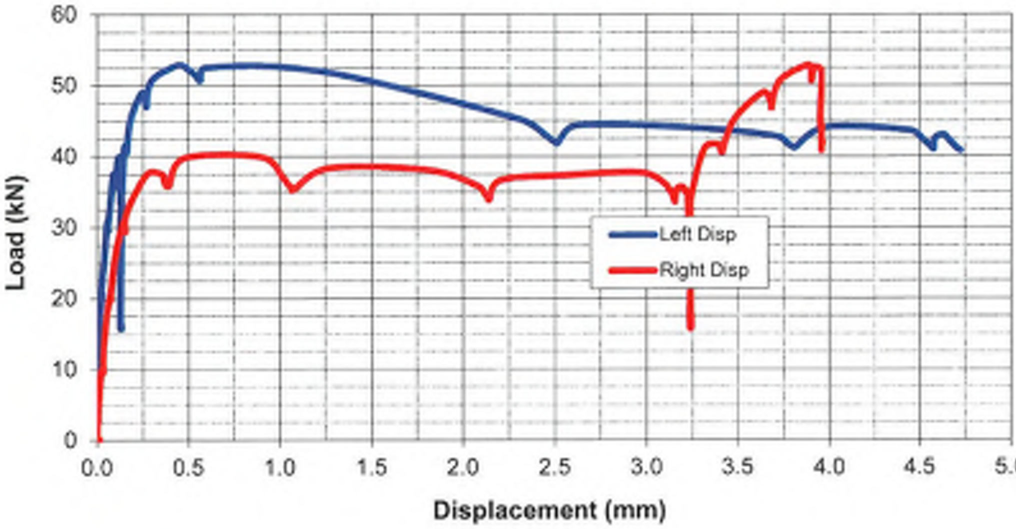
MCD9



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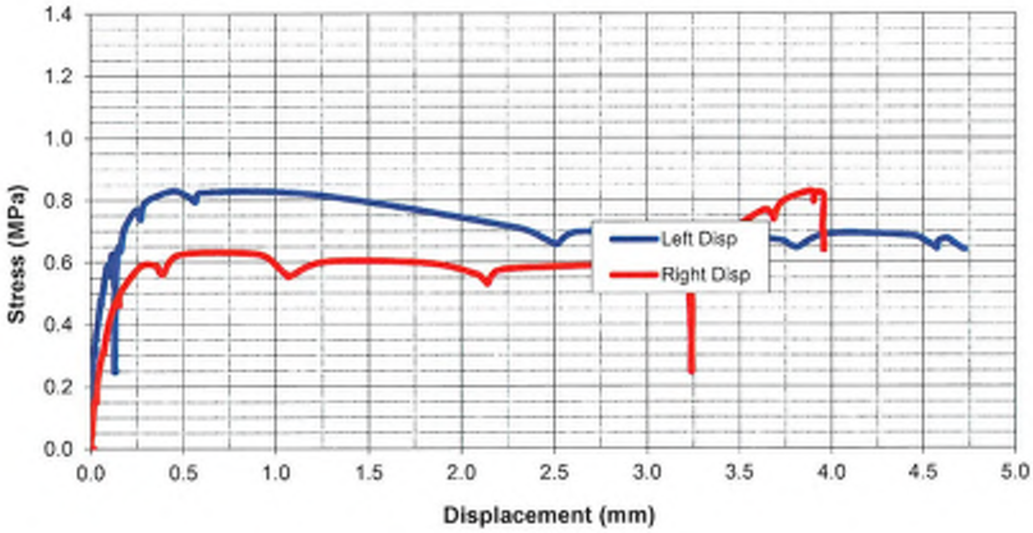
Displacement (mm)
Left Disp
Right Disp



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Displacement (mm)
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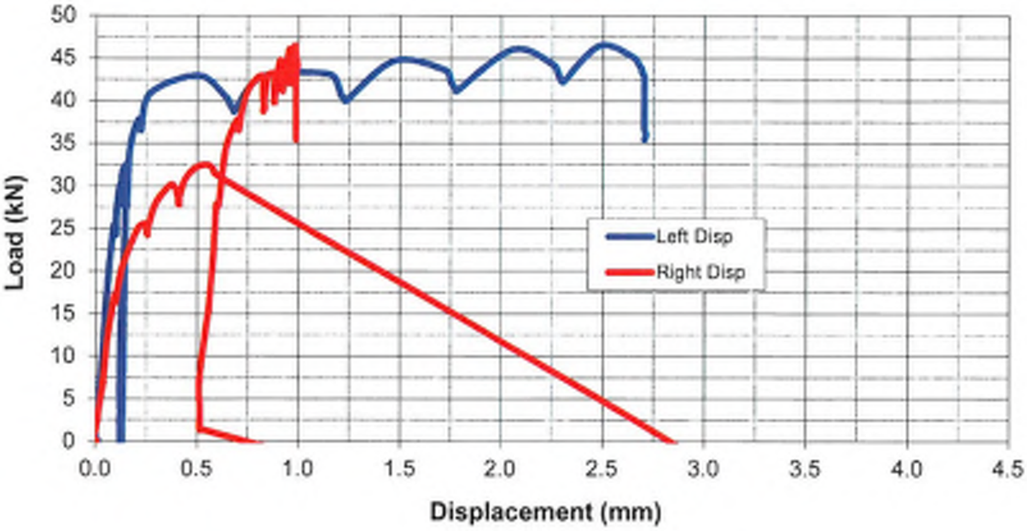
Right Disp

MCD10



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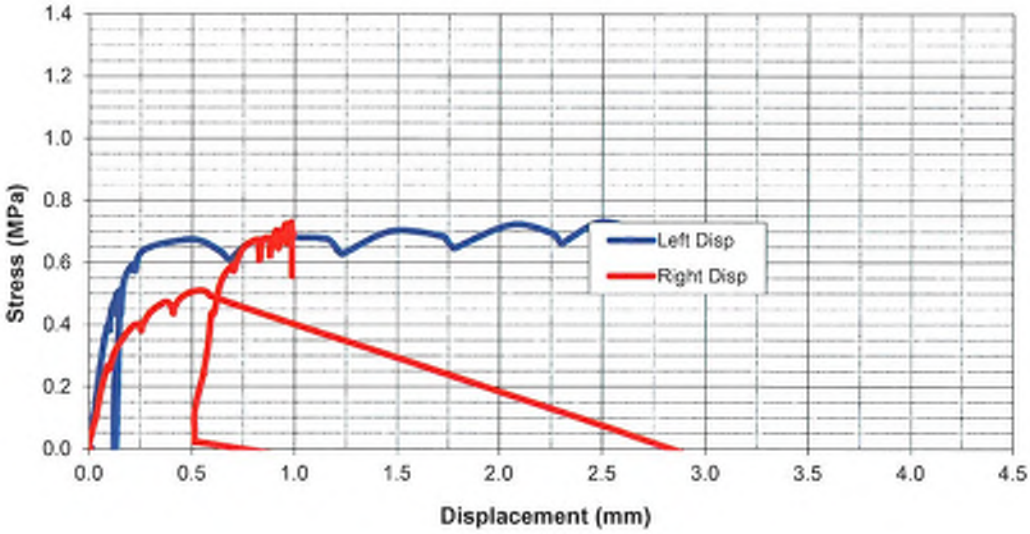
Displacement (mm)
Left Disp
Right Disp



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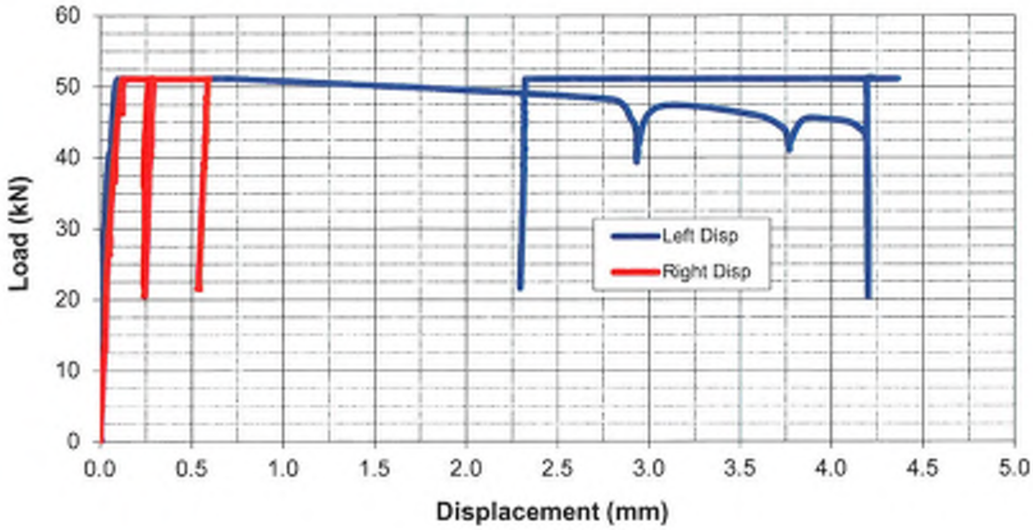
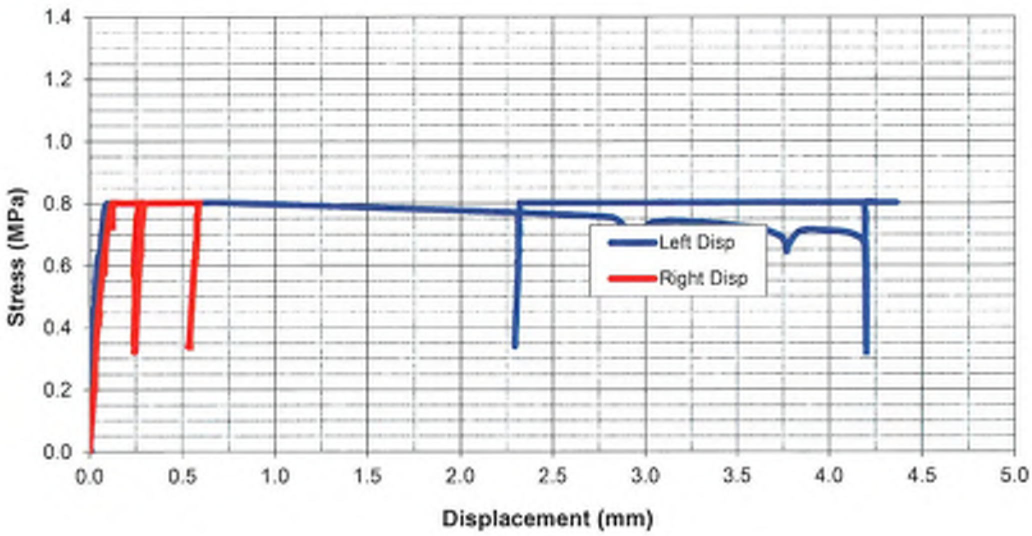
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
Displacement (mm)
Left Disp
Right Disp

MCD11

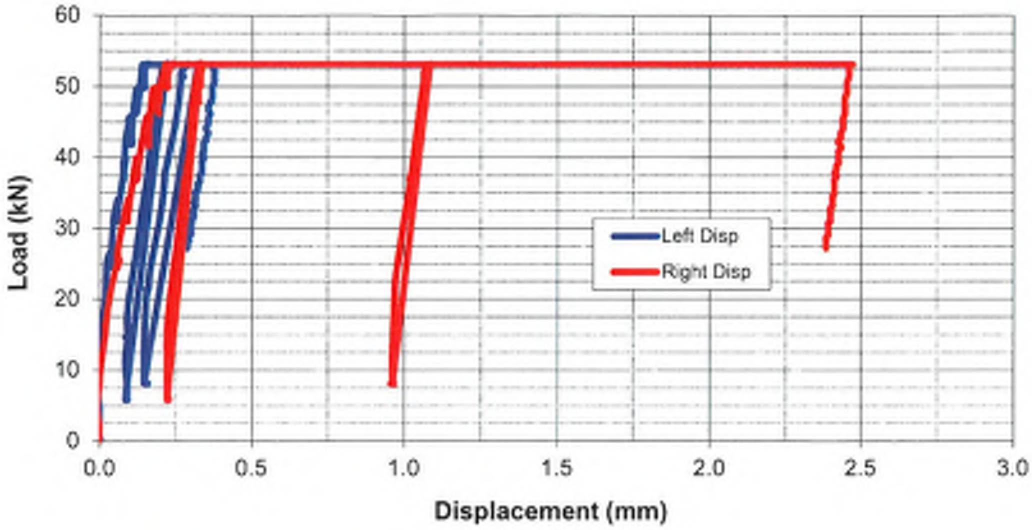


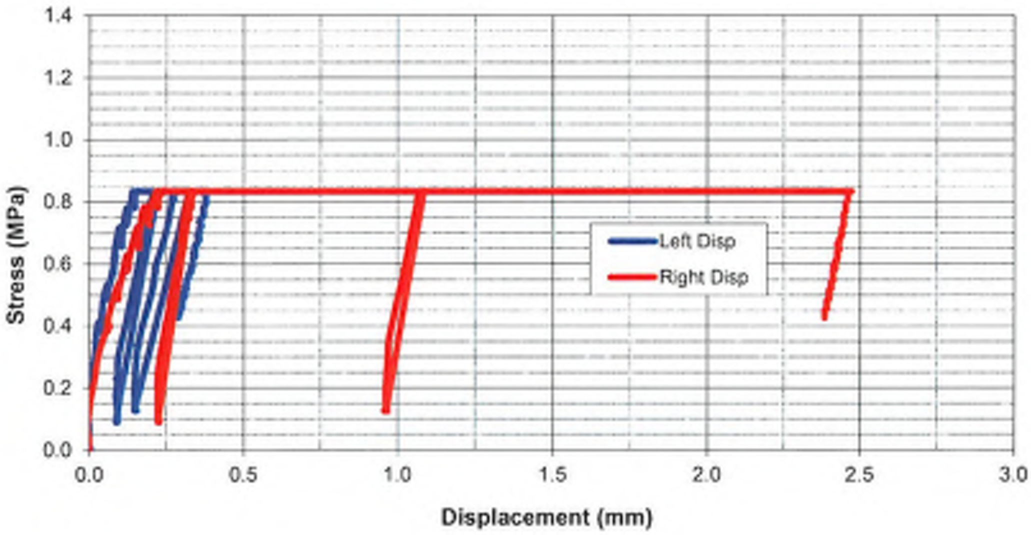
60
50
40
30
20
10
0
Load (kN)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
Displacement (mm)
Left Disp
Right Disp



1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
Stress (MPa)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
Displacement (mm)
Left Disp
Right Disp

MCD12


60
50
40
30
20
10
0
Load (kN)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
Displacement (mm)
Left Disp
Right Disp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1.2
1.0
0.8
0.6
0.4
0.2
0.0
Stress (MPa)
0.0
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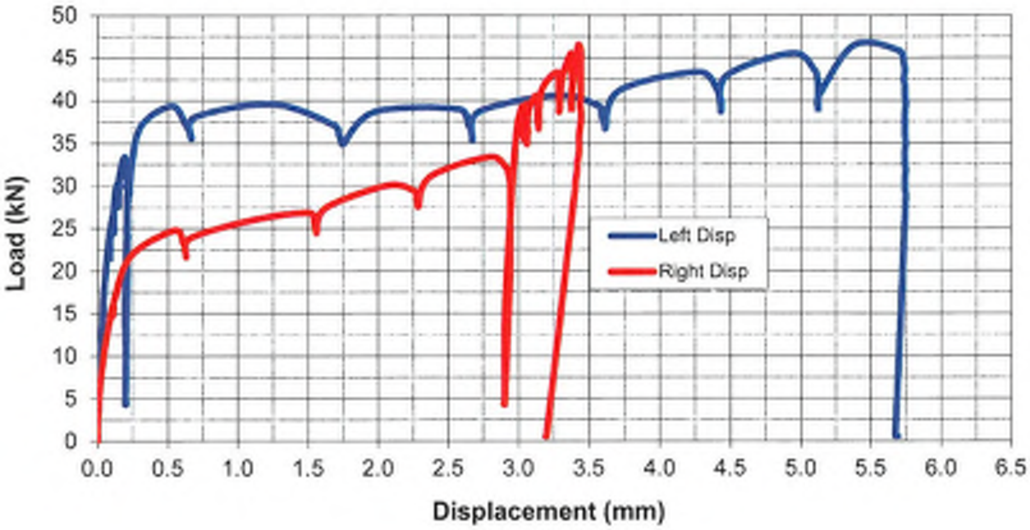
1.0
1.5
2.0
2.5
3.0
Displacement (mm)
Left Disp
Right Disp

MCD13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45
40
35
30
25
20
15
10
5
0
Load (kN)
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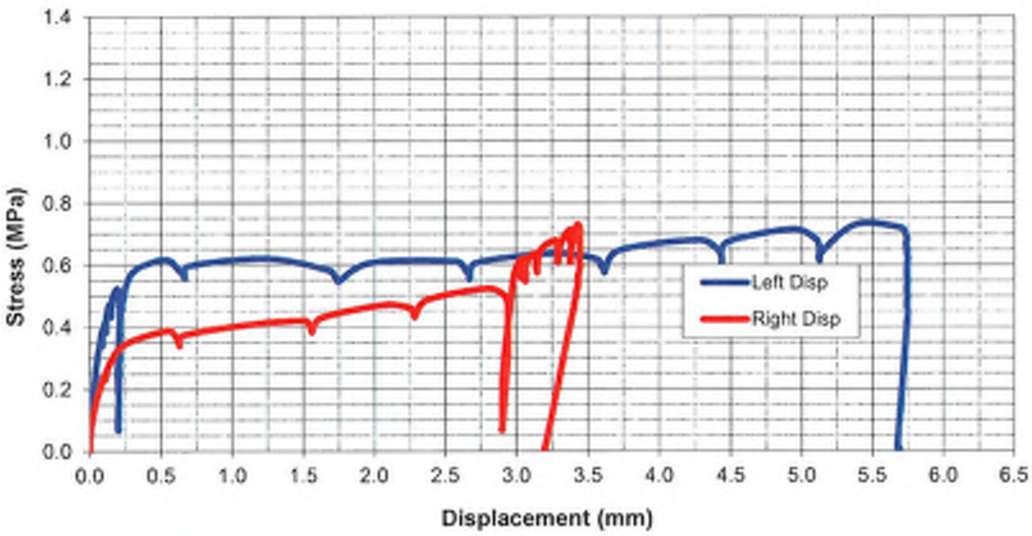
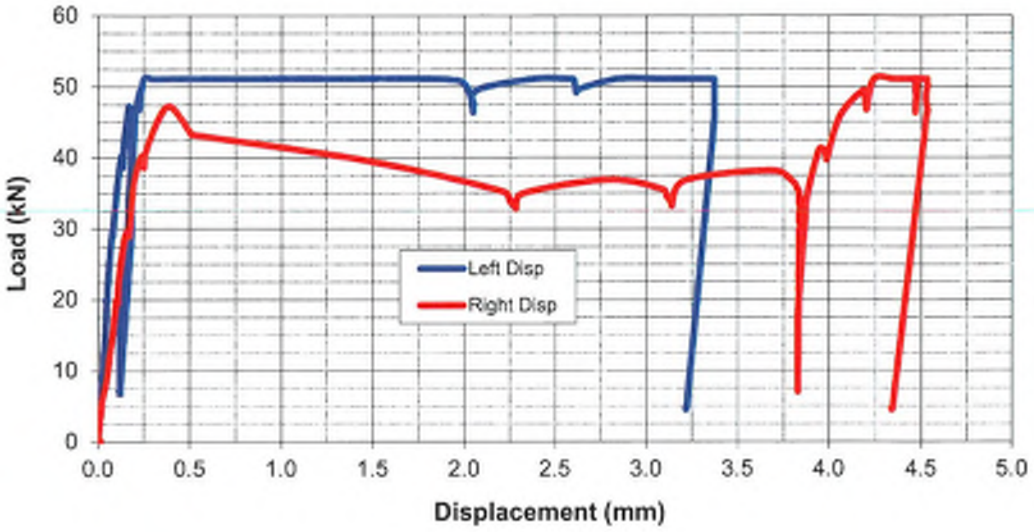
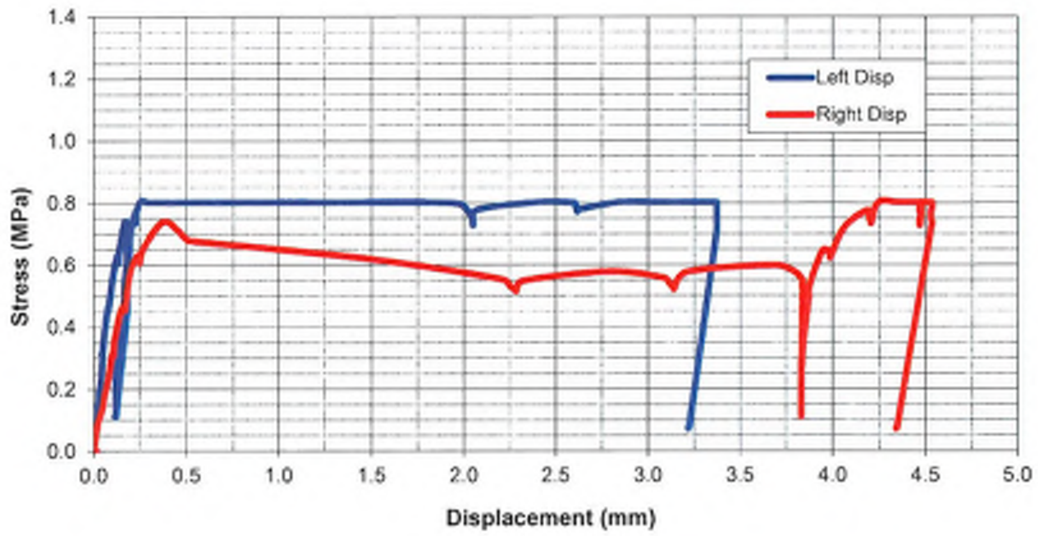
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
5.5
6.0
6.5
Displacement (mm)
Left Disp
Right Disp


1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
Stress (MPa)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
5.5
6.0
6.5
Displacement (mm)
Left Disp
Right Disp

MCD14

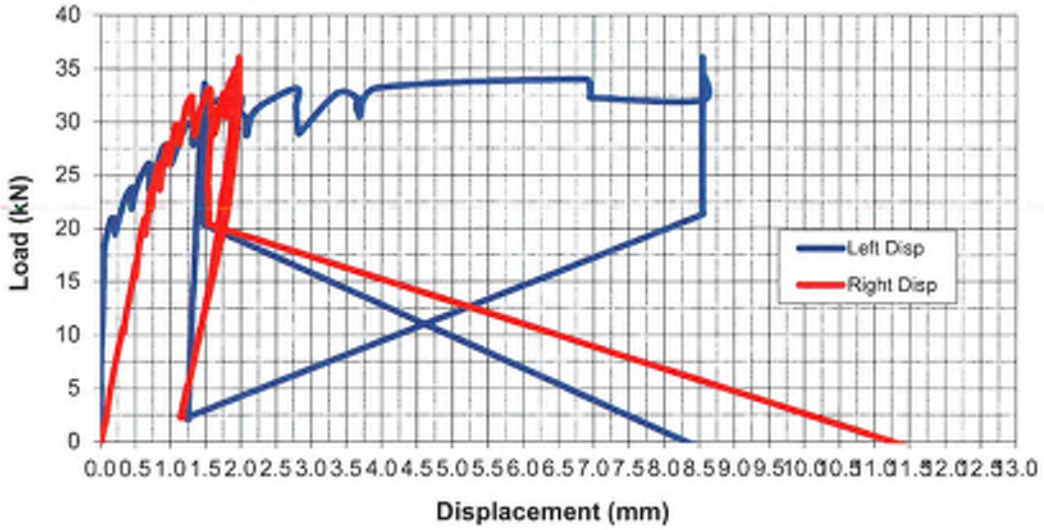


Load (kN)
Displacement (mm)
Left Disp
Right Disp

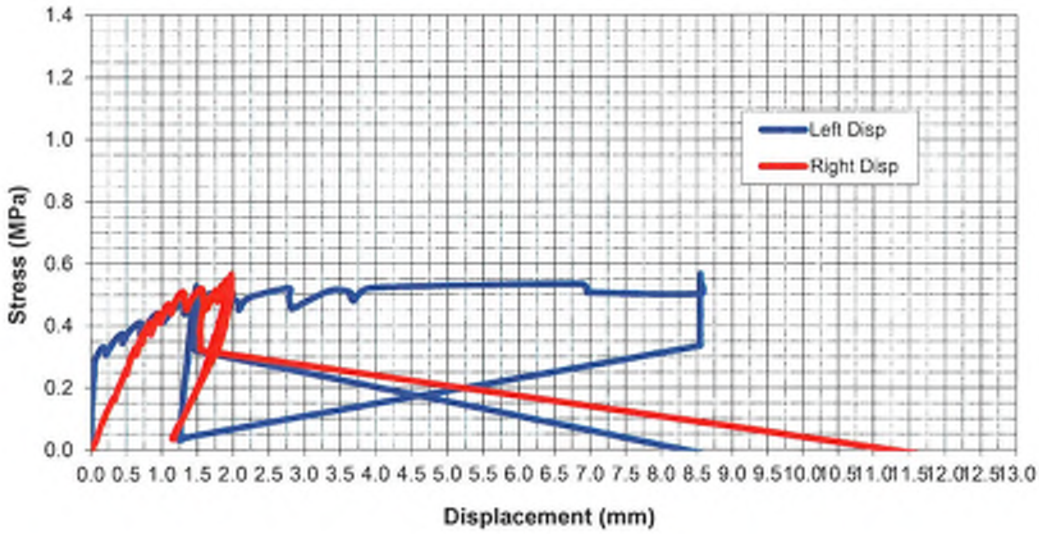


Stress (MPa)
Displacement (mm)
Left Disp
Right Disp

MCD15

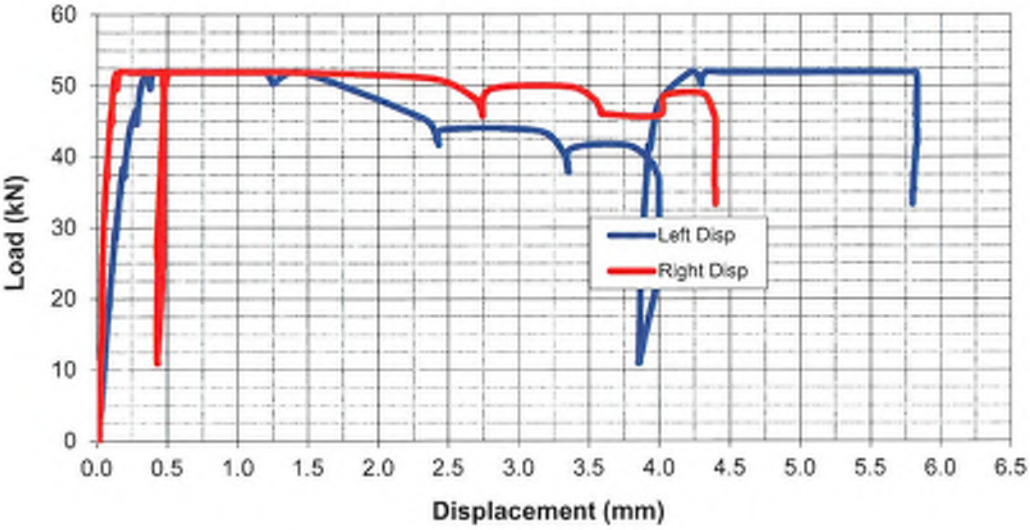


Load (kN)
Displacement (mm)
Left Disp
Right Disp

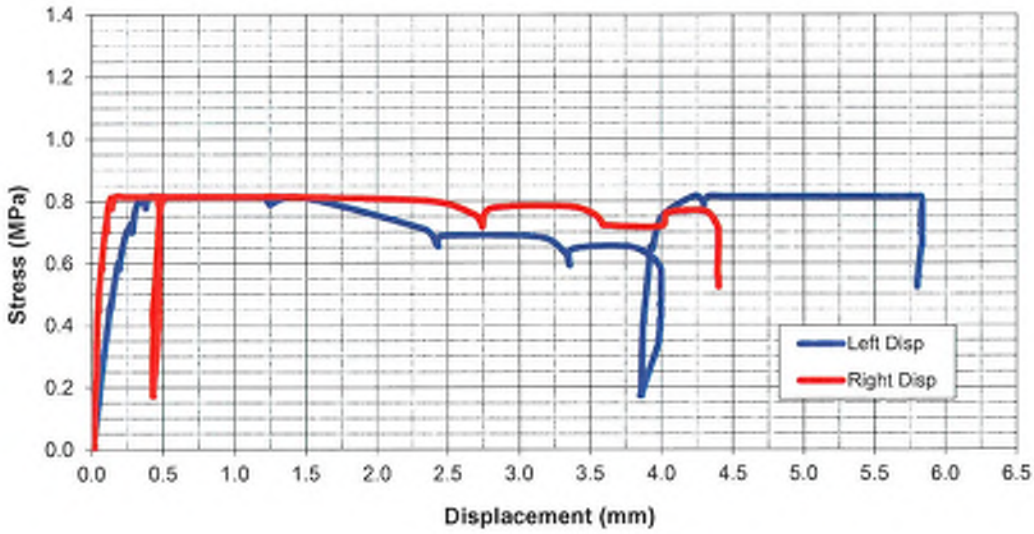


Stress (MPa)
Displacement (mm)
Left Disp
Right Disp

MCD16



60
50
40
30
20
10
0
Load (kN)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
5.5
6.0
6.5
Displacement (mm)
Left Disp
Right Disp



1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
Stress (MPa)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
5.5
6.0
6.5
Displacement (mm)
Left Disp
Right Disp

APPENDIX G

Repair and Strengthening Concept Sketches

Title: RMAG Out-of-Plane Strengthening

Job Name: RMAG

Job #: 104653.02 SSK#: SSK01-01

Date: 2/9/2013 Rev: 1

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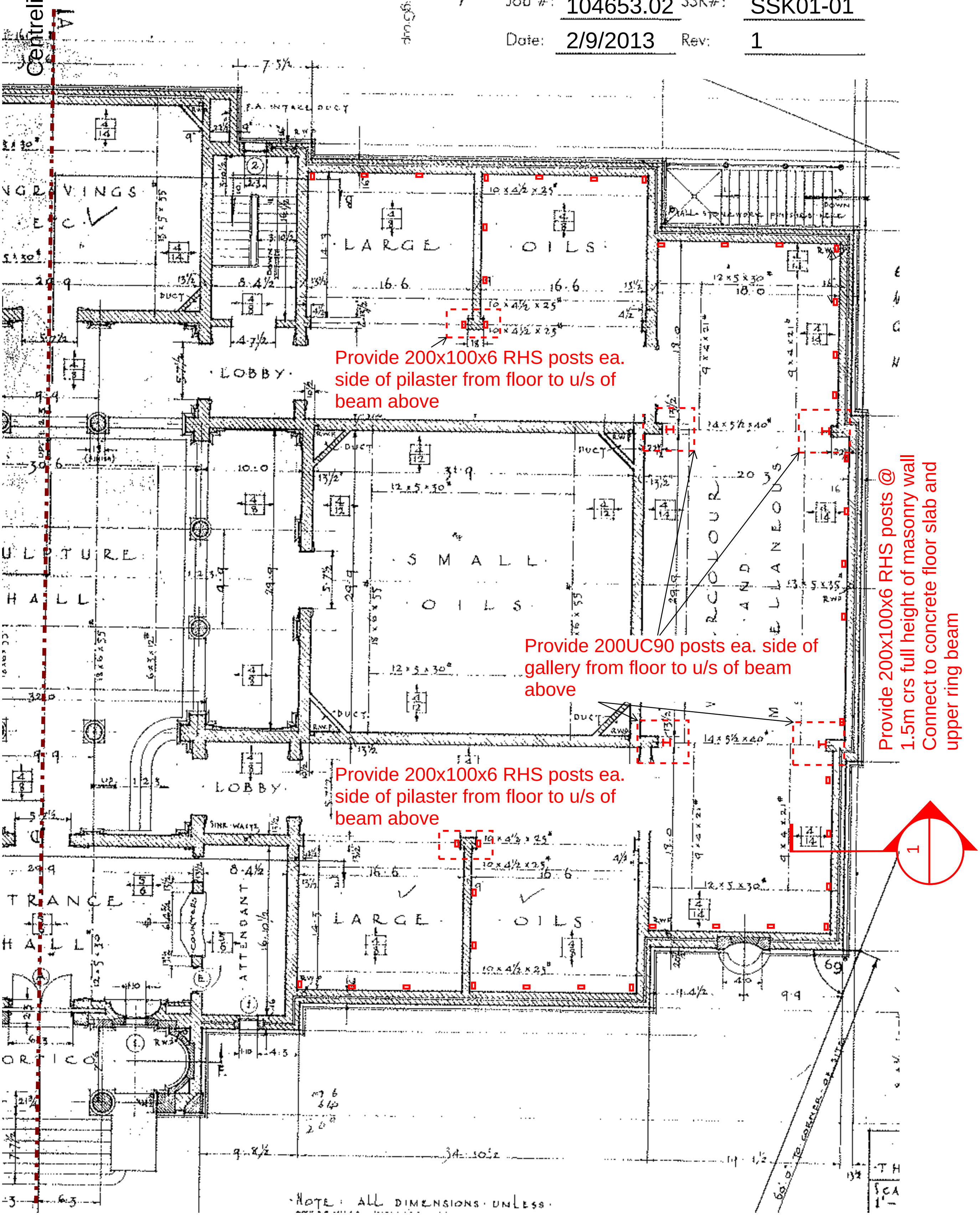


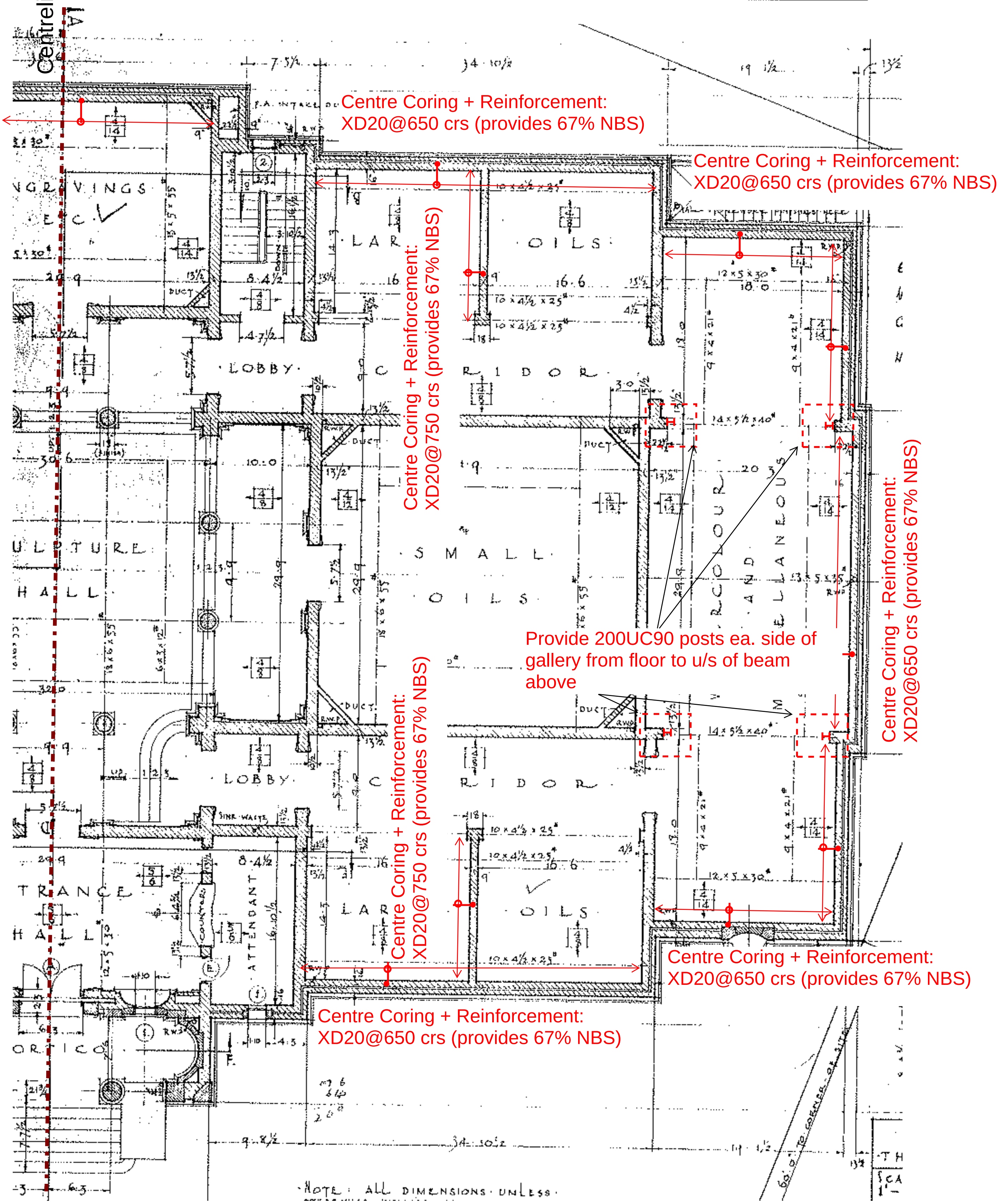
Holmes Consulting Group

Title: RMAG Out-of-Plane Strengthening

Job Name: RMAG

Job #: 104653.02 SSK#: SSK01-02

Date: 2/9/2013 Rev: 1



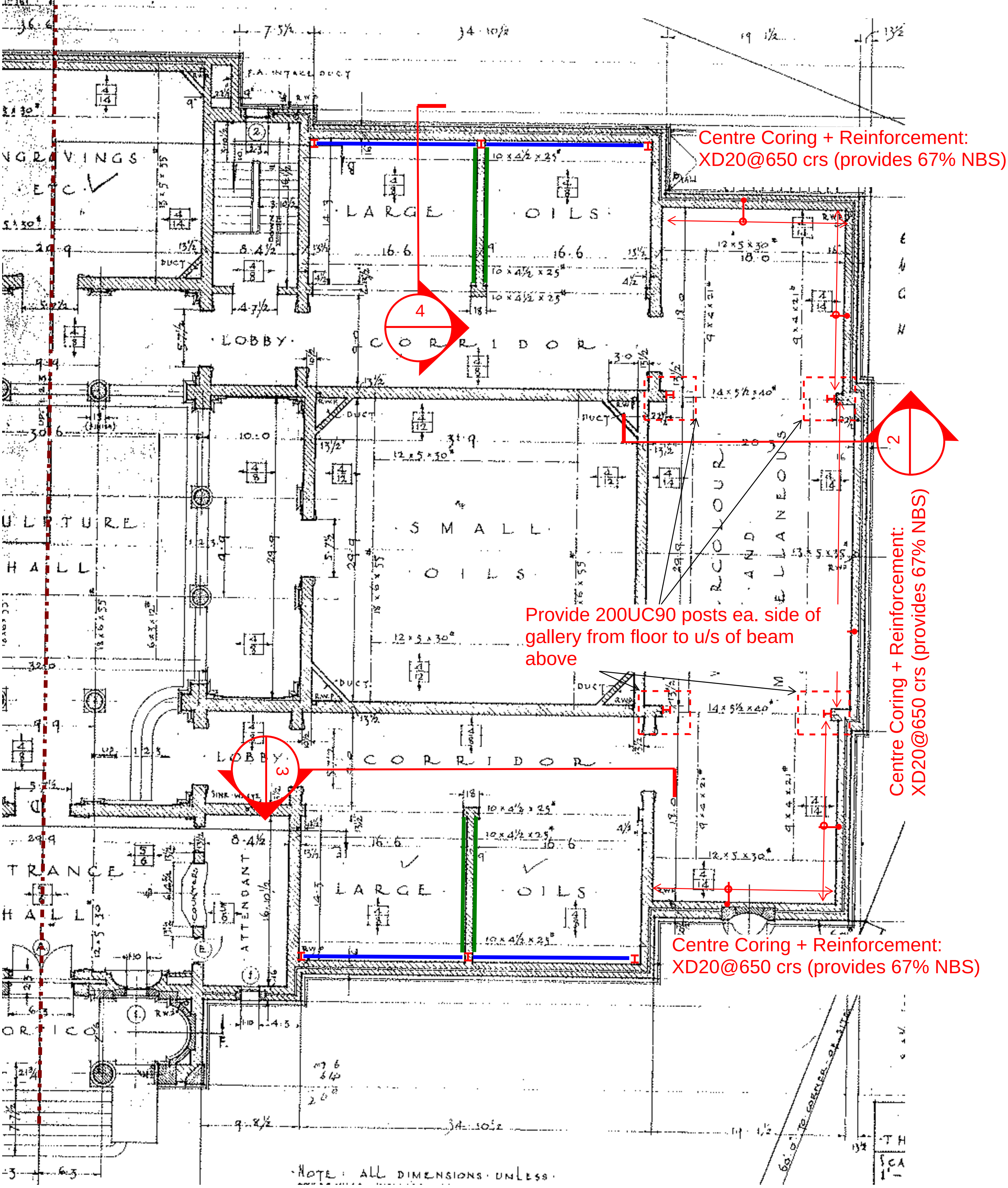
Title: RMAG Out-of-Plane Strengthening

Job Name: RMAG

Job #: 104653.02 SSK#: SSK01-03

Date: 2/9/2013 Rev: 1

Holmes Consulting Group



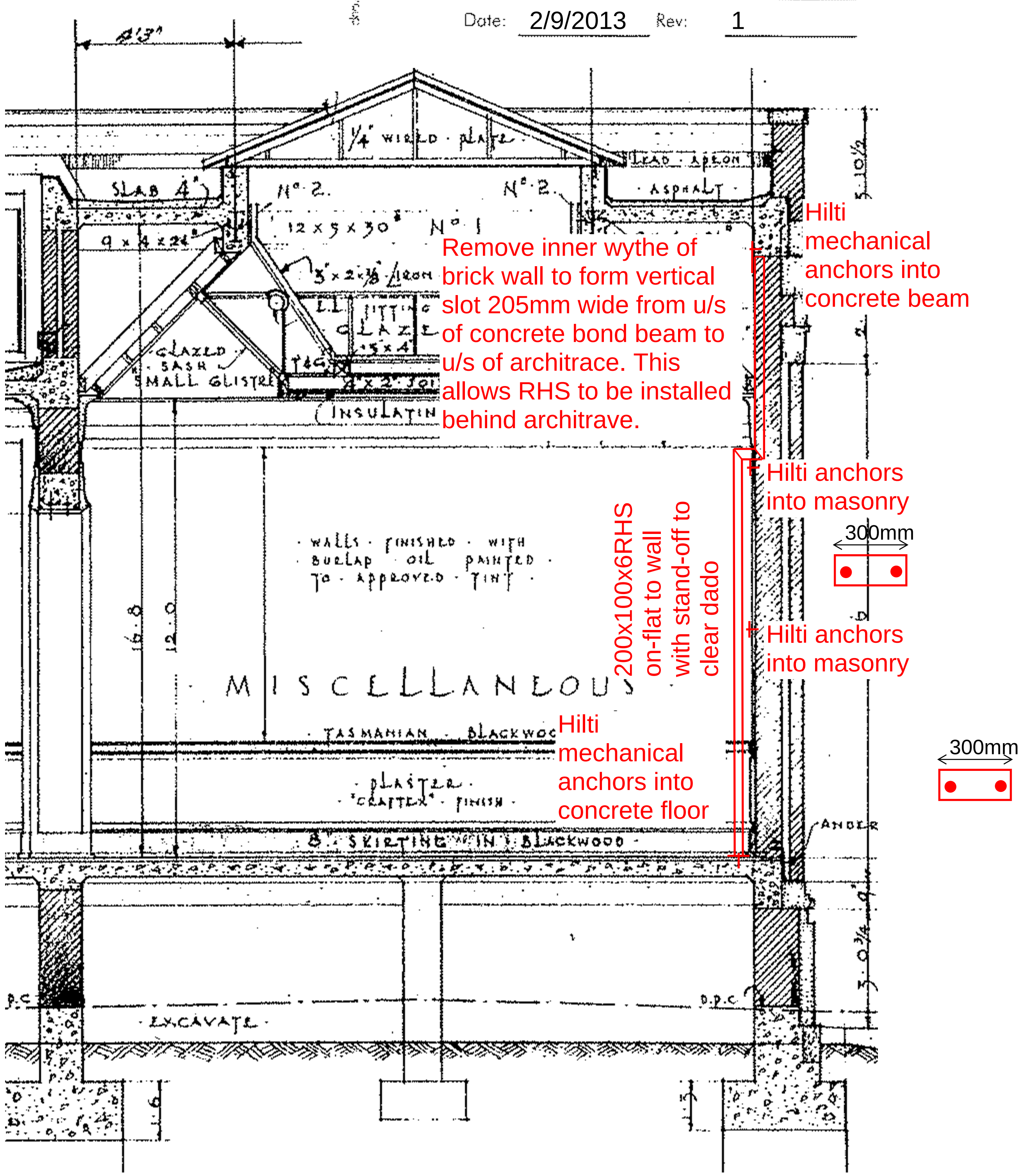
Holmes Consulting Group

Title: RMAG Out-of-Plane Strengthening

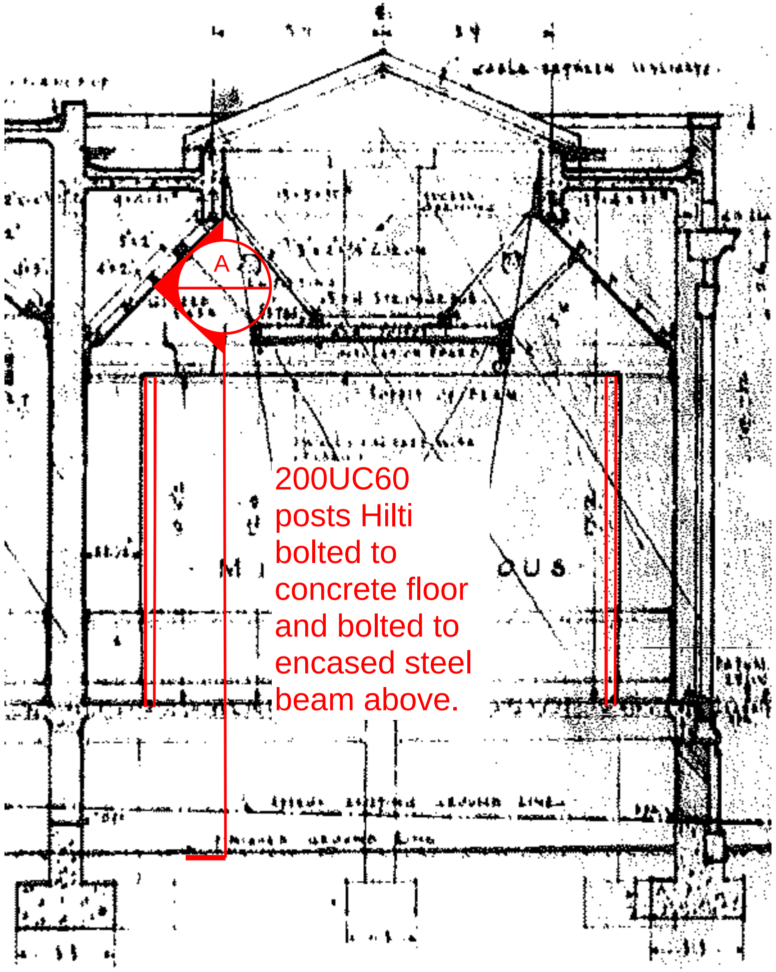
Job Name: RMAG

Job #: 104653.02 SSK#: SSK03-01

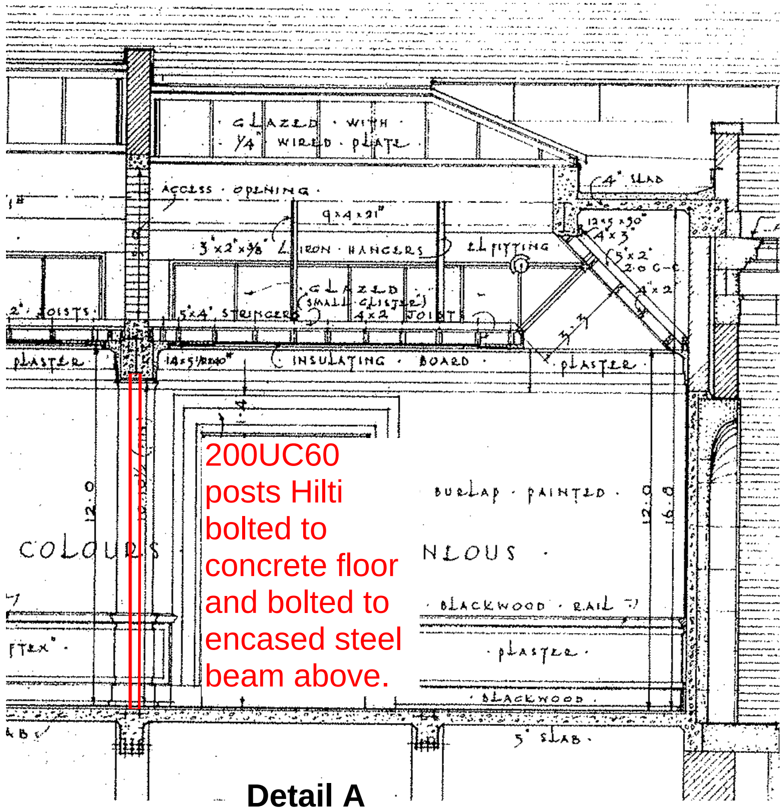
Date: 2/9/2013 Rev: 1



Detail 1



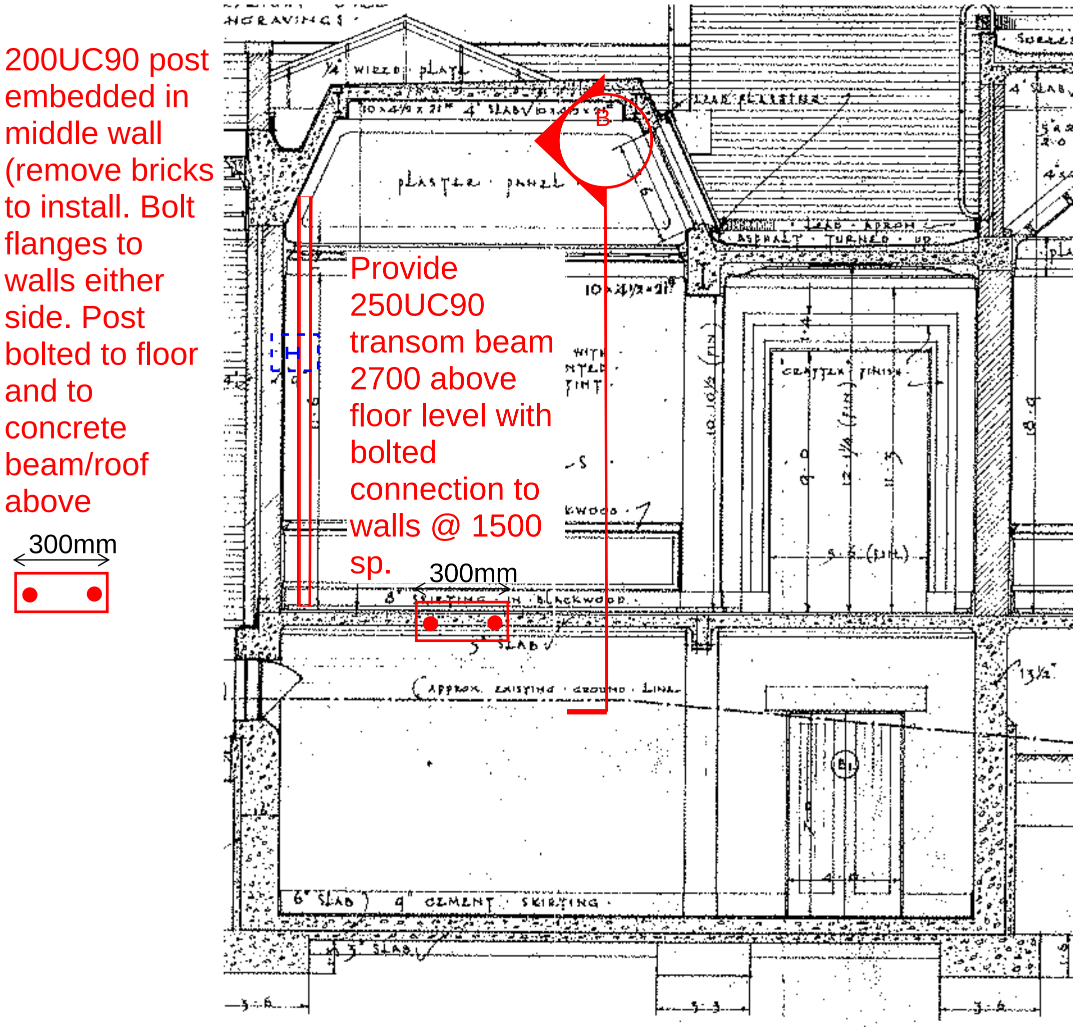
Detail 2



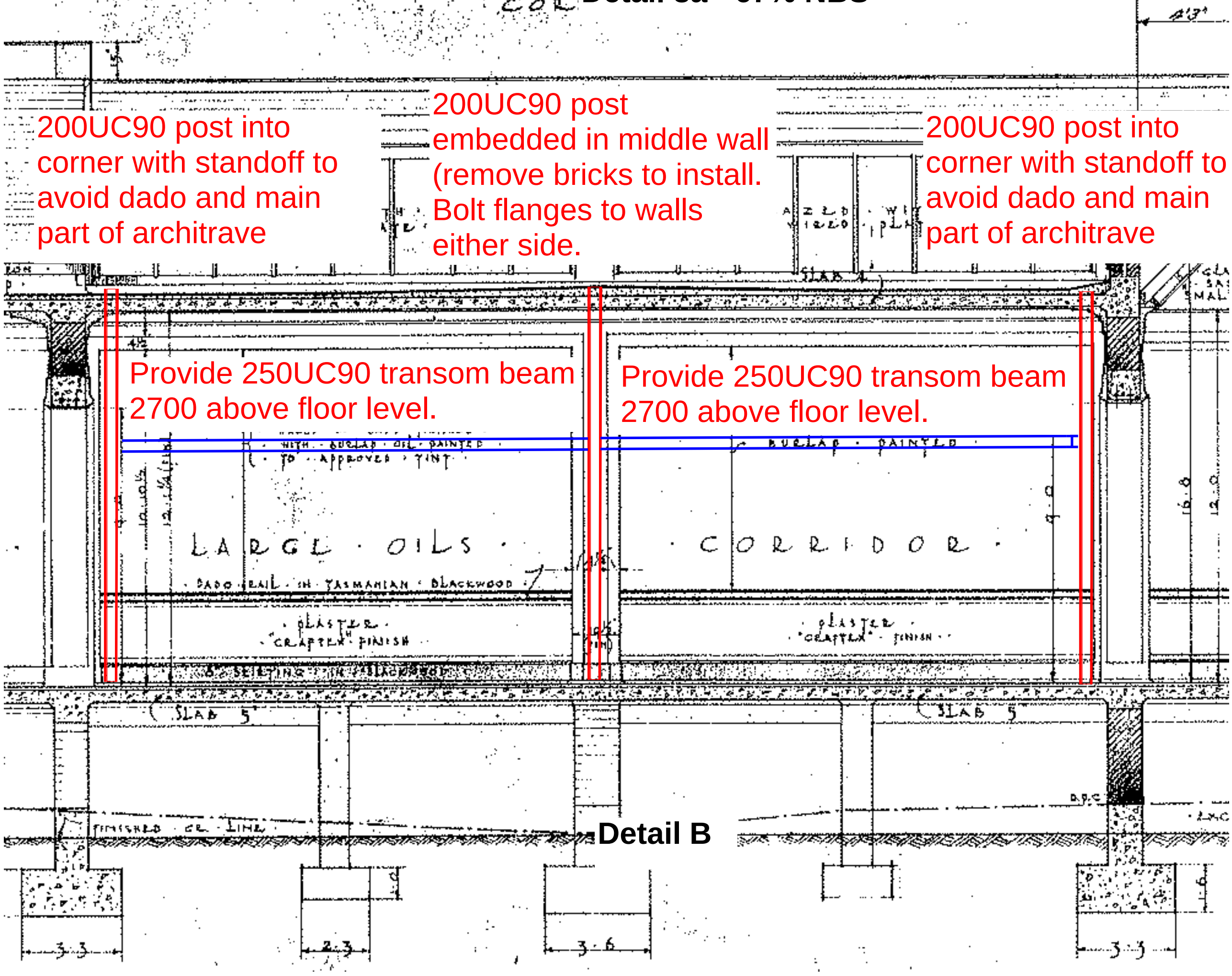
Detail A

Title:	RMAG Out-of-Plane Strengthening		
Job Name:	RMAG		
Job #:	104653.02	SSK#:	SSK03-02
Date:	2/9/2013	Rev:	1

HolmesConsultingGroup



Detail 3a - 67% NBS



Detail B

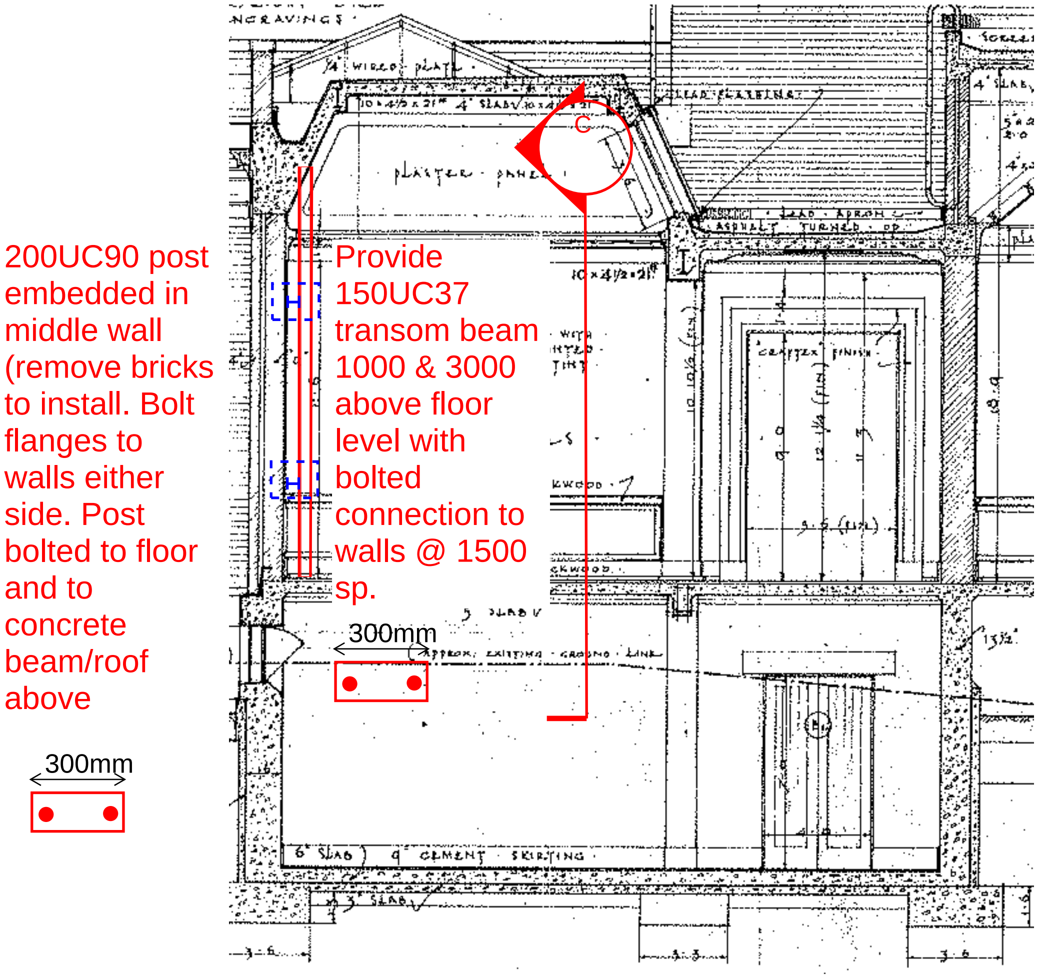
Title: RMAG Out-of-Plane Strengthening

Job Name: RMAG

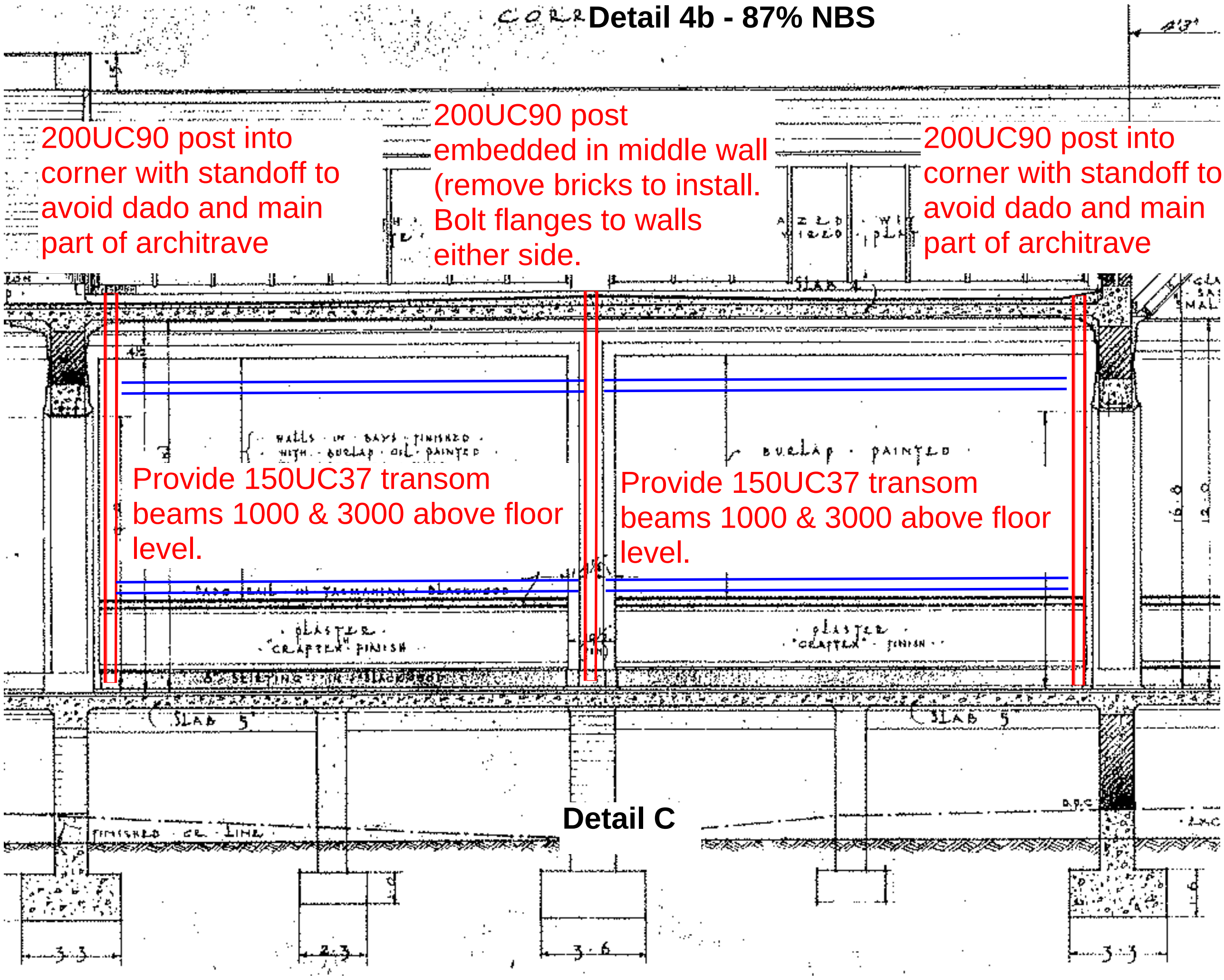
Job #: 104653.02 SSK#: SSK03-03

Date: 2/9/2013 Rev: 1

HolmesConsultingGroup



Detail 4b - 87% NBS



Detail C

Title: RMAG Out-of-Plane Strengthening

Job Name: RMAG

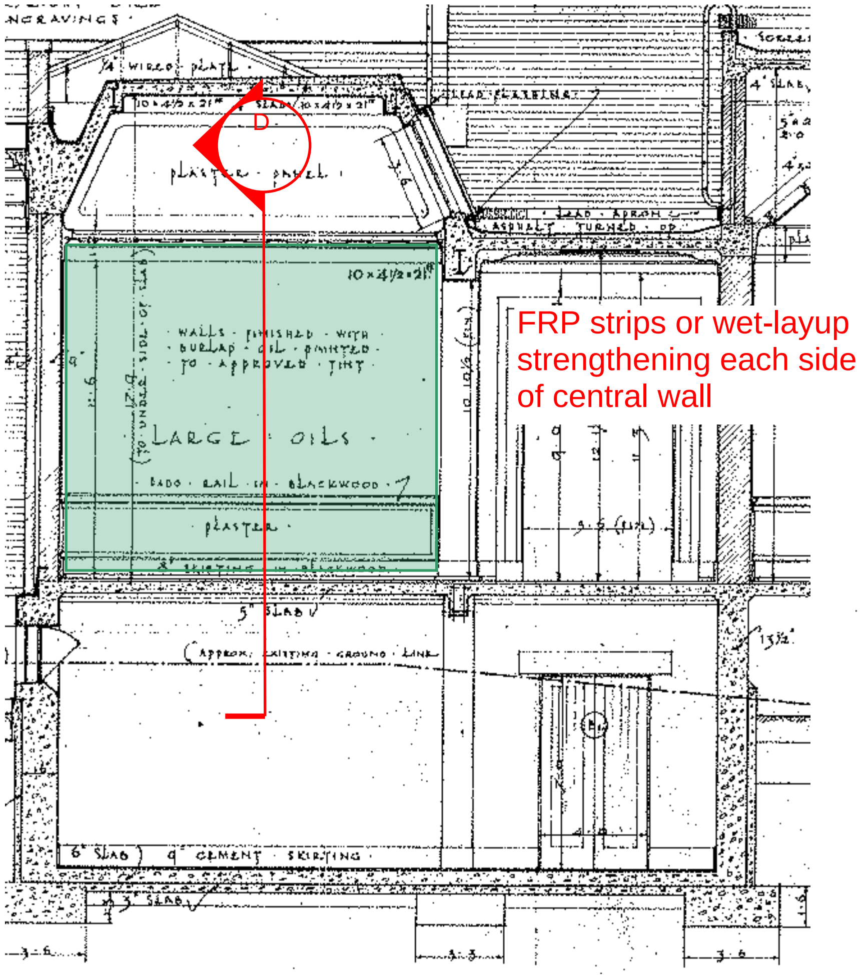
Job #: 104653.02

SSK#: SSK03-04

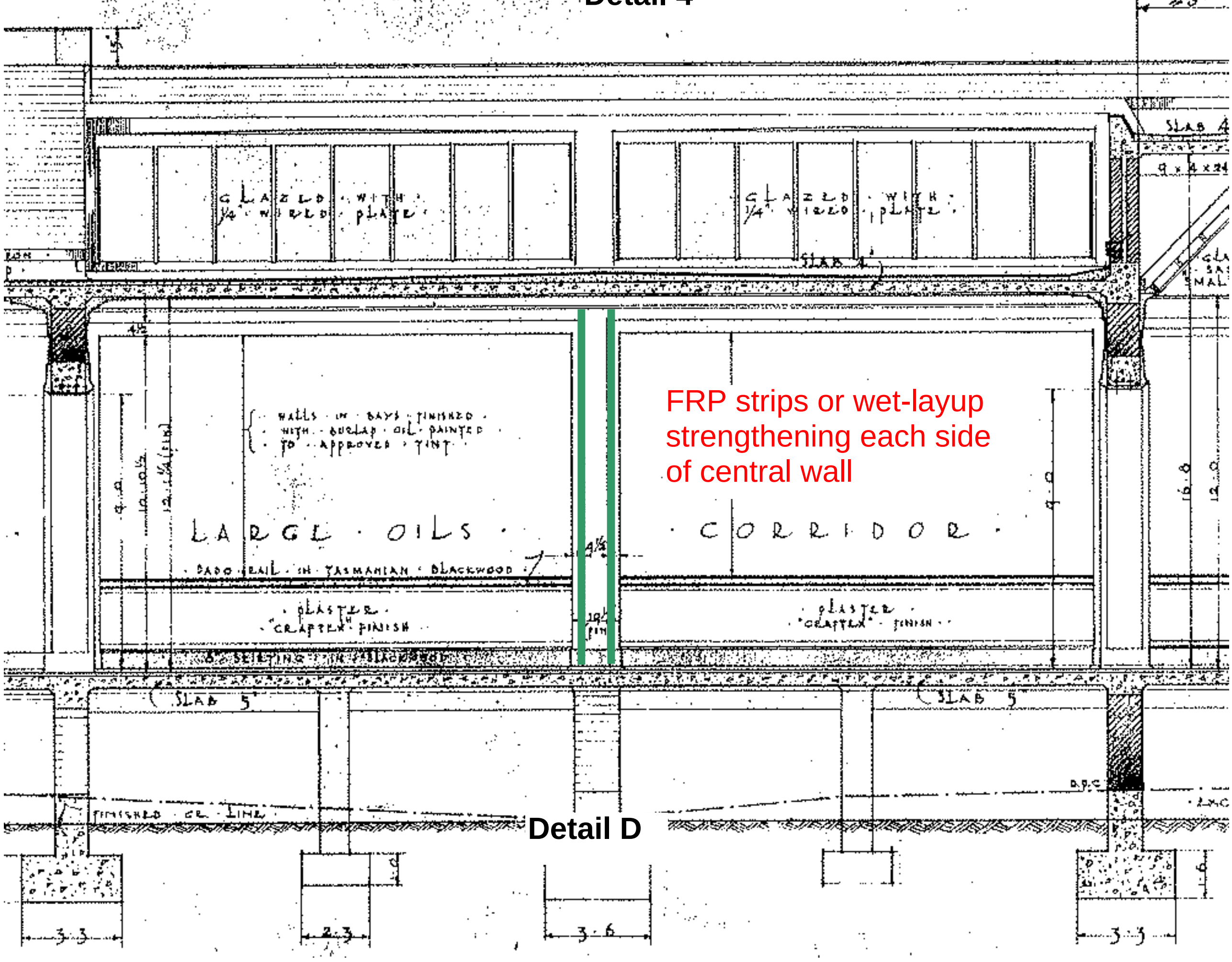
Date: 2/9/2013

Rev: 1

HolmesConsultingGroup



Detail 4



Detail D

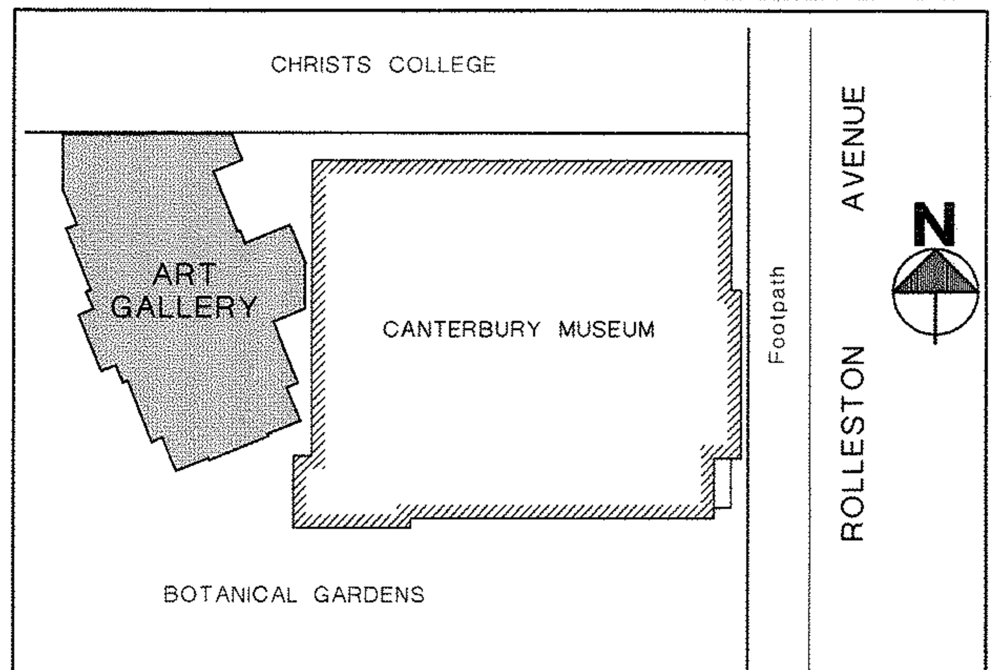
Title: RMAG Out-of-Plane Strengthening

Job Name: RMAG

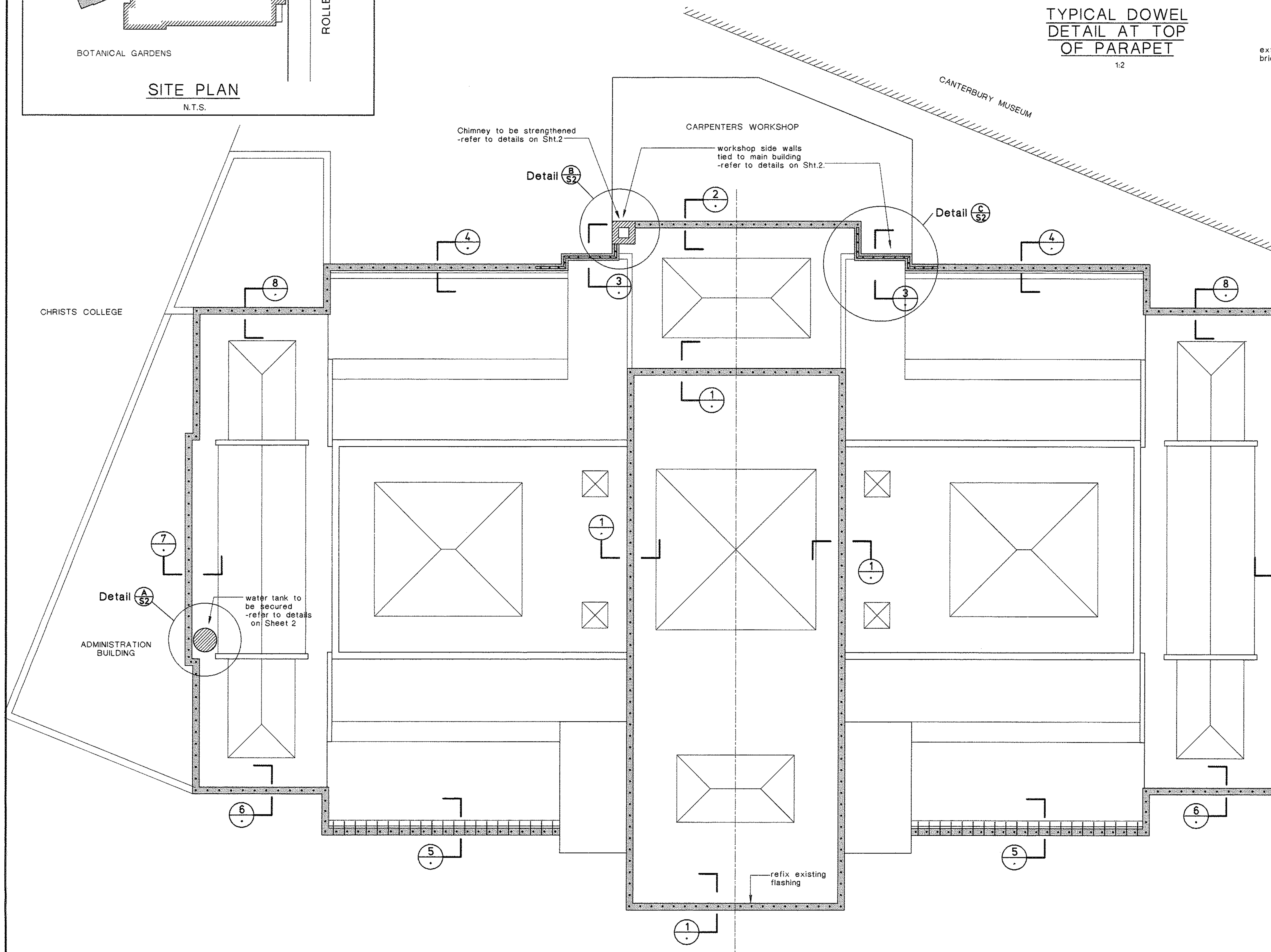
Job #: 104653.02 SSK #: SSK03-05

Date: 2/9/2013 Rev: 1

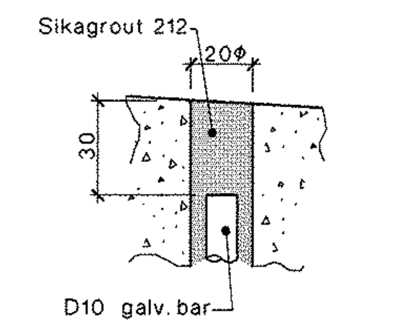
HolmesConsultingGroup



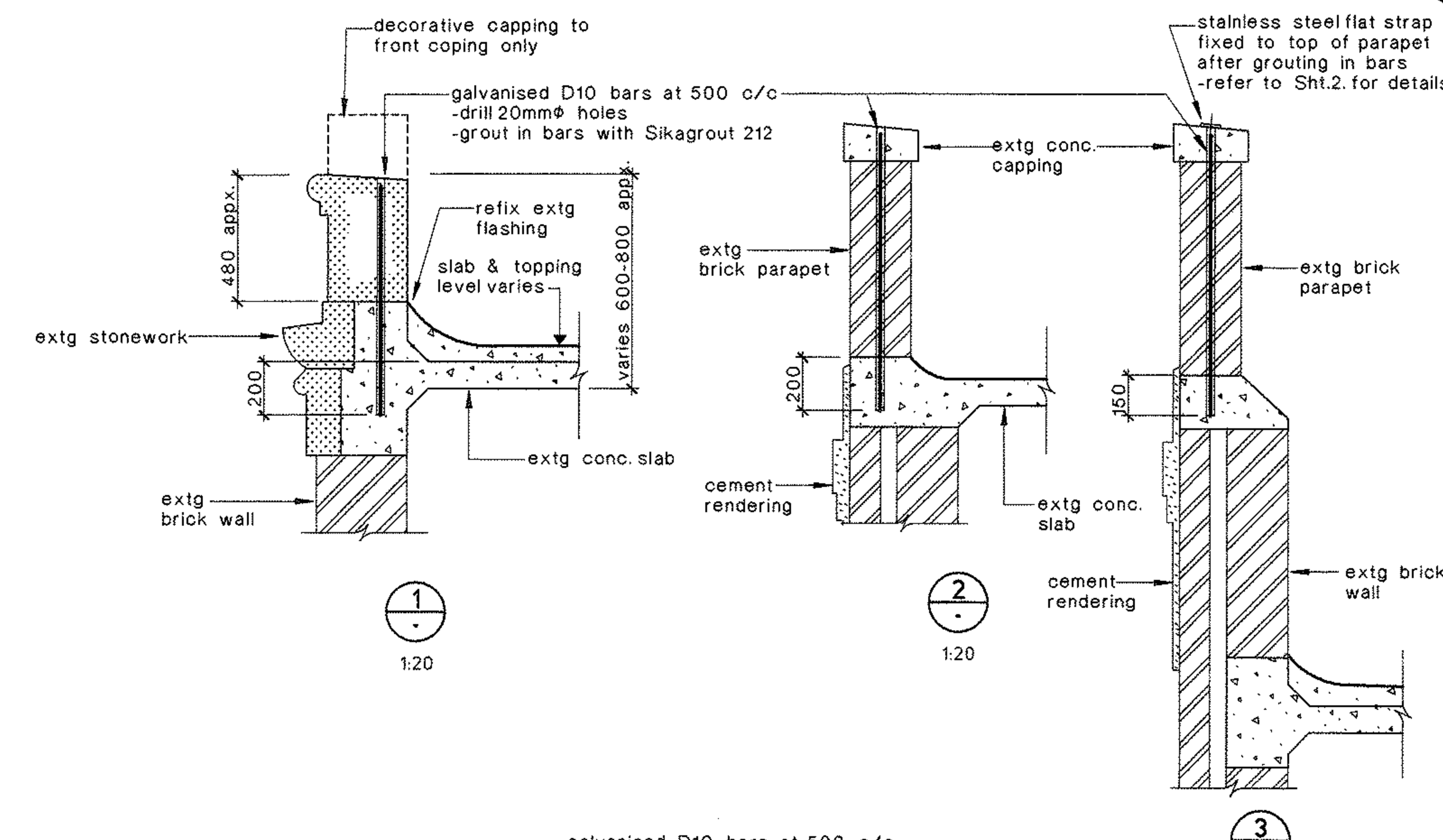
SITE PLAN
N.T.S.

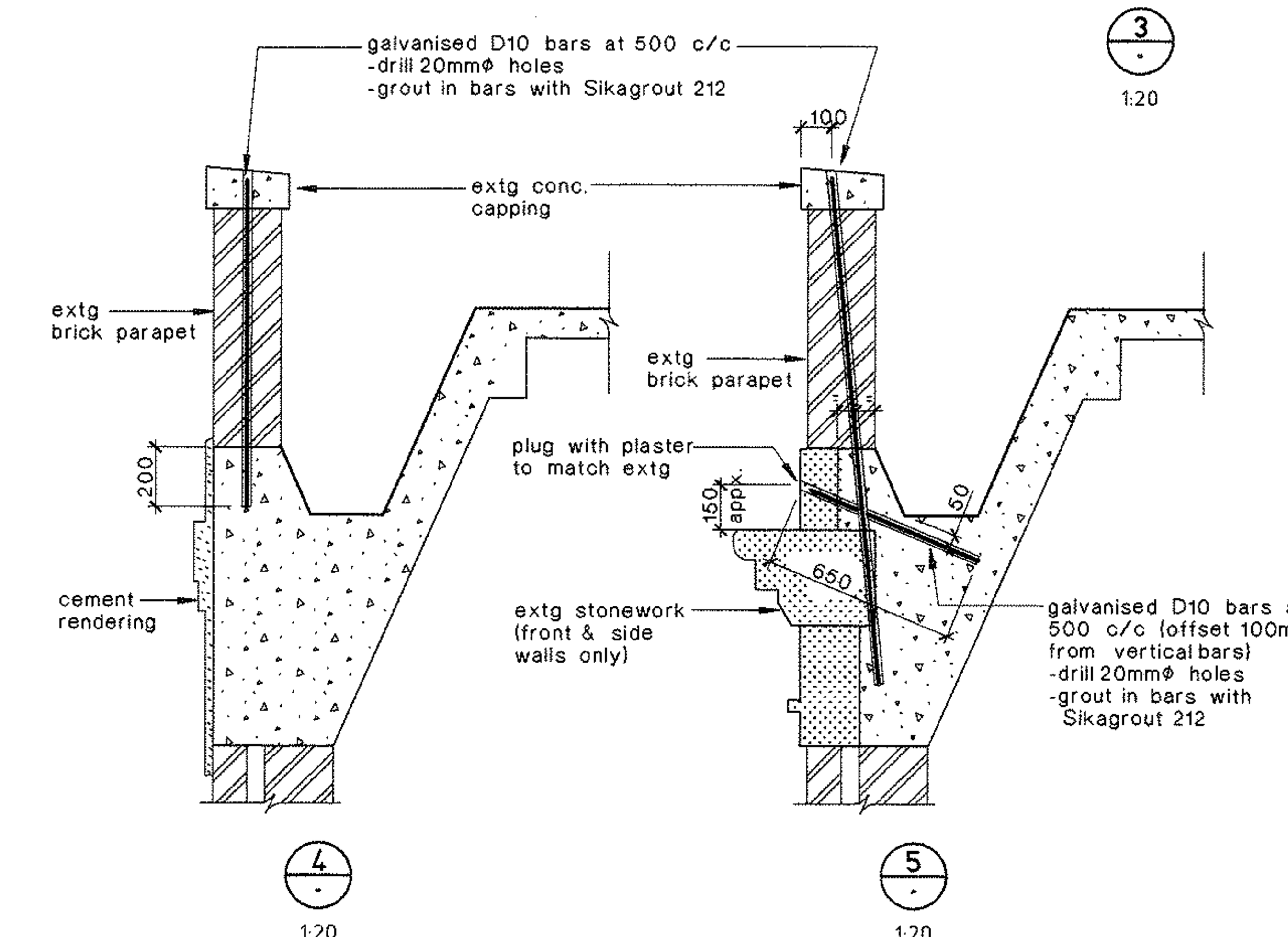


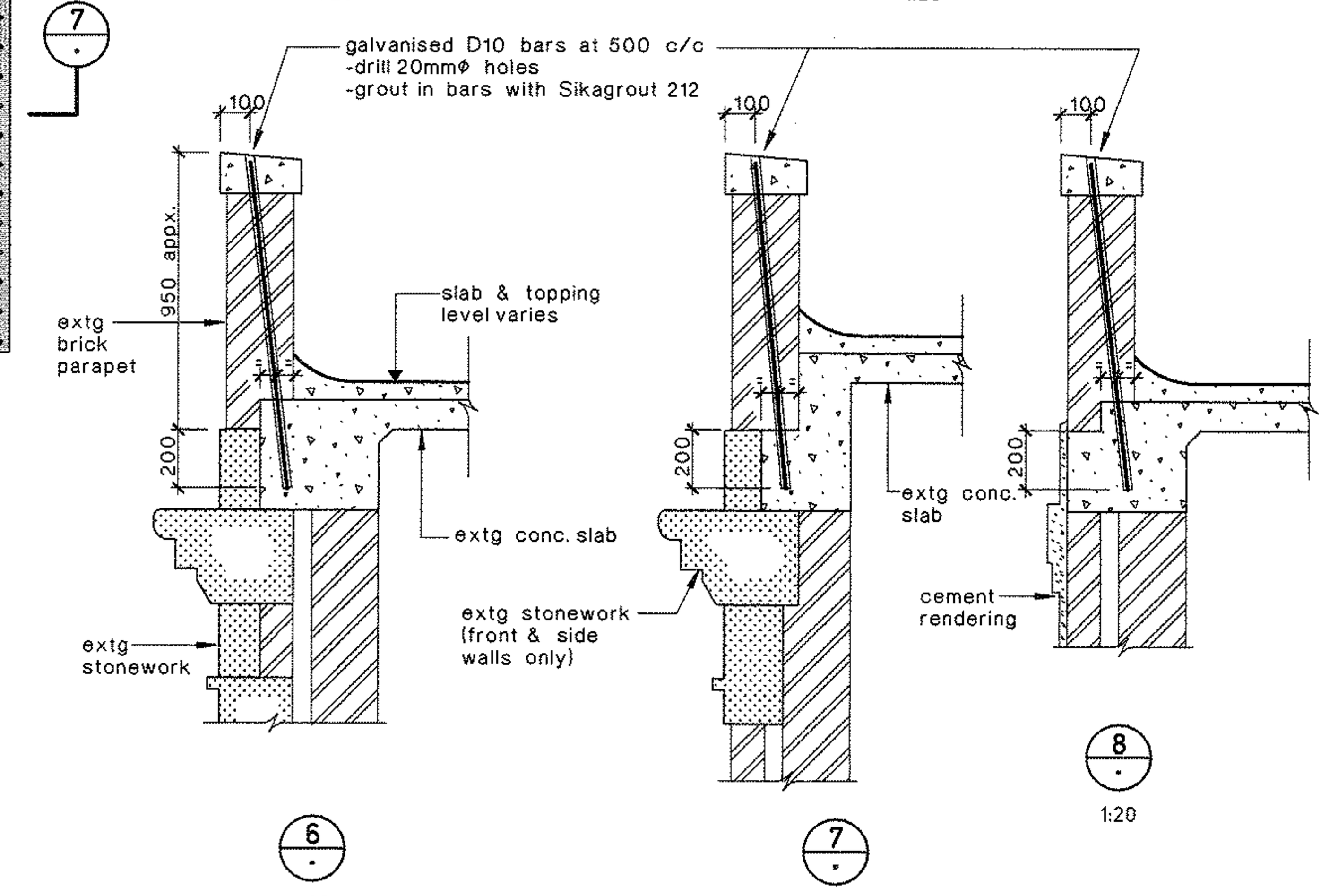
PLAN
1:100
NOTE: Parapets to be strengthened are shown shaded



TYPICAL DOWEL DETAIL AT TOP OF PARAPET
1:2







NOTES:
1) All holes to be diamond core drilled
2) Alternative grouts to Sikagrout 212 may be used, subject to the Engineers approval

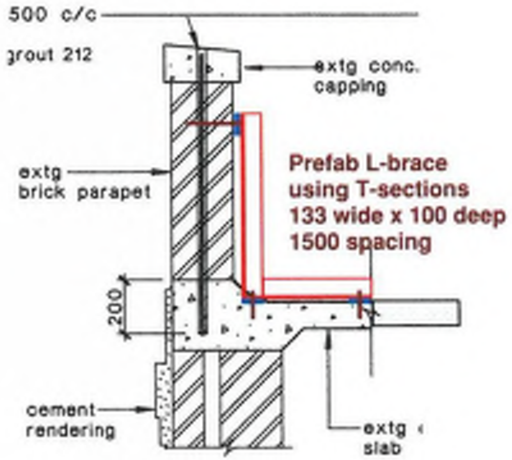
DESIGN SERVICES UNIT		INITIALS	DATE			APPROVED
CHRISTCHURCH THE GARDEN CITY *The city that shines*	DESIGNED	S.D.Smith	3/95	DATUM		/ / DATE
	DRAWN	A.O'B.	3/95	BENCH MK.		
	TRACED			SURVEY FB		
	DRW. CHK.		4/95	SURVEY LB		
	DES. CHK.			CONSTN. EB		
	INDEXED	SD005401.DGN	22/3/95	CONSTN. LB		DESIGN SERVICES MANAGER

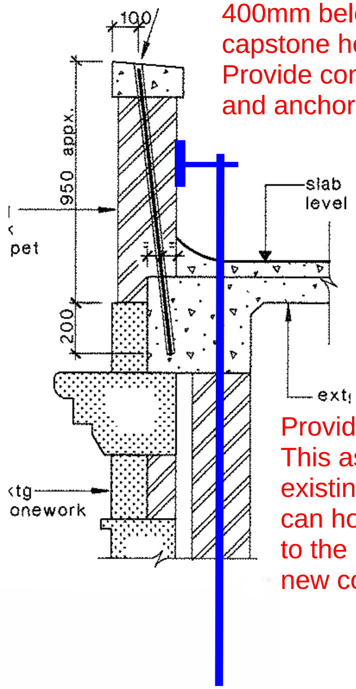
JOB TITLE
McDOUGALL ART GALLERY EARTHQUAKE STRENGTHENING

DRAWING TITLE
STRUCTURAL DETAILS

SCALES	C.N.
As shown	D.4824
	SHEET S.1 OF 2

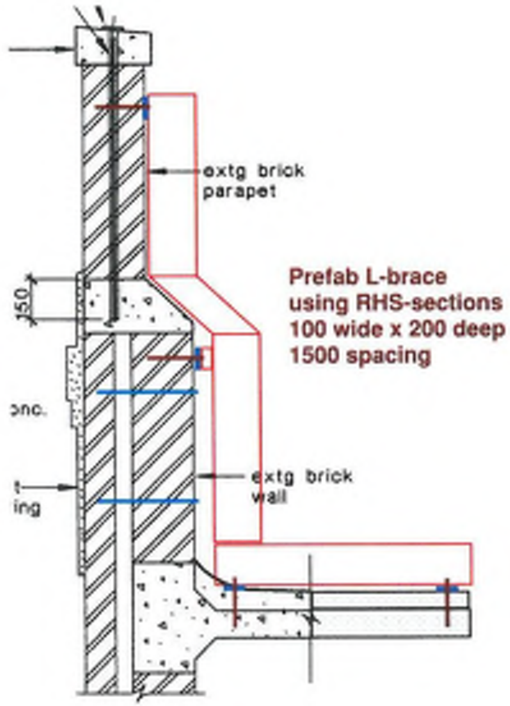
e:\au\sd005401.dgn Apr. 4, 1995 13:44:00

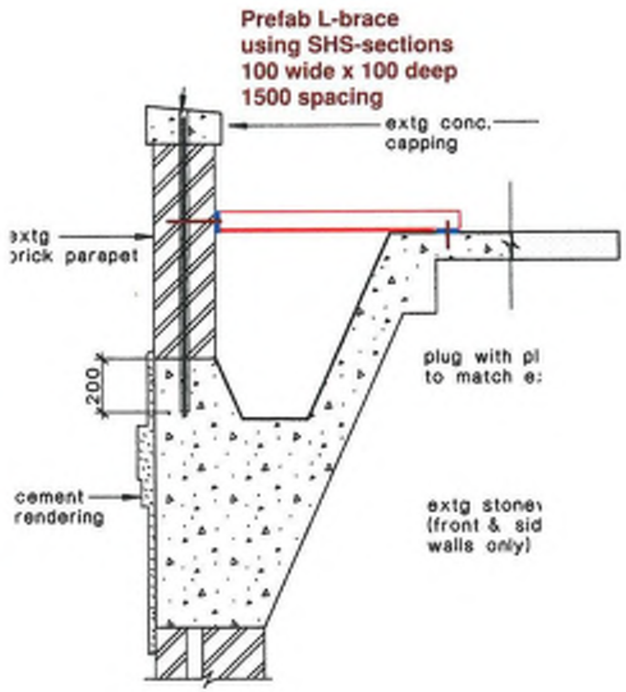




Extend centre-core rebar to 400mm below parapet capstone height.
Provide connection plate and anchors to parapet

Provide XD20 @ 500 crs
This assumes that the existing rods (from 1995) can hold parapet together to the point where the new connection is made.





HolmesConsultingGroup

Title:	RMAG Parapet Strengthening
Job Name:	RMAG
Job #:	104653.02
SSK#:	SSK03-06
Date:	2/9/2013
Rev:	1

APPENDIX H

Repair Specification

SPECIFICATION

STRUCTURAL AND CIVIL ENGINEERS

ROBERT MCDOUGAL ART GALLERY SESIMIC STRENGTHENING

PREPARED FOR

CHRISTCHRUCH CITY COUNCIL

1046536.02

18 FEBRUARY 2013

CONTENTS

1. POST-EARTHQUAKE DAMAGE REPAIR

1.1 PRELIMINARY

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

1.2 SCOPE

This Section consists of:-

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

1.3 RELATED DOCUMENTS

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

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

BS EN 459-2:2010	Building Limes Part 2: Test Methods	(BS EN)
NZSEE	Assessment & Improvement of the Structural Performance of Buildings in Earthquakes.	(NZSEE)
ASTM C 109-08	Standard Test Methods for Compressive Strength of Hydraulic Cement Mortars.	(ASTM)
ASTM C 1314-10	Standard Test Methods for Compressive Strength of Masonry Prisms.	(ASTM)
ASTM E488-90	Standard Test Methods for Strength of Anchors In Concrete and Masonry Elements.	(ASTM)

1.4 QUALITY ASSURANCE

1.4.1 General

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

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

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

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

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

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

1.4.2 Inspection

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

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

1.4.3 Producer Statement – Construction (PS3)

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

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

Refer to the Appendix for additional explanation and a sample of the form of these Statements.

1.5 TESTING

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

Allow an additional provisional sum of $1000 for additional random testing, to be instructed at the Engineer's discretion.

1.6 SAFETY

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

1.7 MATERIALS AND WORKMANSHIP

1.7.1 Materials

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

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

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

1.7.2 Workmanship

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

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

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

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

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

1.7.3 Completion

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

1.8 DAMAGE SURVEYS

An initial damage survey has been commissioned by the client. This has identified general forms of damage. We have not been able to expose all critical elements for observation, nor have we conducted a detailed survey identifying each individual crack. At the request of the engineer the Contractor shall expose areas of the structure, in order to enable detailed observations to be made of critical areas.

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

1.8.1 Record of Repairs Carried Out

Full records of repairs carried out shall be maintained by the contractor, as outlined in the relevant sections below. An example form is attached at the end of this section of the Specification outlining the level of detail expected for recording the repairs carried out.

1.8.2 Crack Damage

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

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

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

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

1.8.3 Spalling Damage

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

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

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

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

1.9 REPAIR OF CRACKS IN REINFORCED CONCRETE

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

Cracks less than 0.2mm wide are considered to be superficial and do not require specific structural repair unless directed otherwise by the Architect.

1.9.1 Repair of Hairline Cracks (< 2mm)

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

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

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

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

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

1.9.2 Repair of Large Cracks (< 5mm)

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

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

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

1.9.3 Repair of Very Large Cracks (> 5mm)

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

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

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

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

1.10 REPAIR OF CONCRETE SPALLING

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

1.10.1 Repair of Shallow Spalling (<40mm thick)

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

Clean any exposed reinforcing using a wire brush. Prepare the exposed concrete surface and reinforcing in accordance with the manufacturers instructions, applying a primer such as Sika MonoTop-910N Primer as required.

Build up the required concrete profile using a high strength repair mortar, such as Sika MonoTop-412N Mortar, and finish in accordance with the manufacturer's instructions.

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

1.10.2 Repair of Moderate Spalling (<80mm thick)

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

Clean any exposed reinforcing using a wire brush. Prepare the exposed concrete surface and reinforcing in accordance with the manufacturers instructions, applying a primer such as Sika MonoTop-910N Primer as required.

Build up the required concrete profile using a high build repair mortar, such as Sika MonoTop-352N High Build Mortar, and finish in accordance with the manufacturer's instructions.

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

1.11 REPAIR OF CRACKS IN UNREINFORCED MASONRY

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

1.11.1 Unreinforced Masonry with Cement-Lime Mortar

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

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

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

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

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

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

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

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

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

1.12 GROUT INJECTION OF CRACKS AND OPEN JOINTS IN UNREINFORCED MASONRY

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

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

1.12.1 Materials

Grout shall be Centricrete MV or approved alternative.

Repair mortar shall be as specified in Section 1.11.

1.12.2 Methodology

Rake open joint to adequate depth to remove any loose mortar, but no less than 25 mm. Open joints are to be temporarily held open with timber levelling wedges as necessary

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

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

Remove levelling wedges and re-point holes and open joints. Pointing shall be undertaken to match existing.

1.13 GROUTING OF BARS INTO UNREINFORCED MASONRY

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

1.13.1 Shallow Embedment (< 750mm)

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

1. Hilti RE500 / HY150 / HY70 (brick masonry only)
2. Hilti CM 651
3. Sika Sikadur 52
4. Sika Grout 212
5. Ramset Epcon C6 (brick masonry only)
6. Ramset Premier Grout MP

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

Anchor hole diameters are detailed in Table 1.

Table 1 Hole Diameters

Anchor Size (mm)	Hole Diameter (mm) Hilti RE500 / Sikadur 52 / Epcon C6	Hilti CM 651 / Sika Grout 212/ Premier Grout MP
10	12	20
12	14	24
16	18	32
20	22	40
25	27	50

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

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

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

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

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

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

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

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

1.13.2 Deep Embedment (> 750mm Vertical)

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

1. Hilti RE500
2. Hilti CM 651
3. Sika Sikadur 52
4. Sika Grout 212
5. Ramset Epcon C6
6. Ramset Premier Grout MP

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

Anchor hole diameters are detailed in Table 2.

Table 2 Hole Diameters

Anchor Size (mm)	Hole Diameter (mm) Hilti RE500 / Sikadur 52/ Epcon C6	Hilti CM 651 / Sika Grout 212/ Premier Grout MP
10	12	20
12	14	24
16	18	32
20	22	40
25	27	50

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

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

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

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

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

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

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

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

1.13.3 Control Tests – Grouted Anchors & Reinforcing Bars

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

Torque Testing

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

12mm diameter anchors or bars 54Nm

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

No slippage shall occur under the above torques.

Direct Tension Testing

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

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

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

Reports

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

Level	Location/Room	Building Element	Description of Damage [1]	Photo of Damage	Repair Methodology [2]	Date Repaired

Notes:

1. Describe damage being repaired (e.g. for repair of cracks include crack length, crack width)
2. Describe repair being carried out - including the sketch/drawing number or specification clause as applicable, type and quantity of material used, etc

2.1 PRELIMINARY

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

2.2 INTERPRETATION

2.2.1 Design Engineer

For the purposes of this section of the Specification, the Design Engineer will be an employee of Holmes Consulting Group or a nominated representative.

2.2.2 Construction Reviewer

For the purpose of this section of the specification, the role of Construction Reviewer will be undertaken by the Design Engineer.

2.3 SCOPE

This section consists of:-

1. The supply, fabrication, surface treatment, delivery and erection of the structural steel and related items necessary to complete the work indicated on the drawings and as further specified.
2. The supply, fabrication and finishing of all weldplates, bolts and cleats etc. Attendance on site as necessary to complete fixing and painting of connections. Provision of all scaffolding, ladders and planks etc required to carry out the work.

The following items are included in this section:-

1. Out-of-plane unreinforced masonry wall supports.
2. Columns.
3. All other structural steelwork shown on the drawings and required for completion of the building including cleats, weldplates, bolts and other fixings.

2.4 RELATED DOCUMENTS

In this section of the specification, reference is made to the latest revisions of the following documents:-

The New Zealand Building Code (NZBC)

AS/NZS 1170	Structural Design Actions
AS/NZS 1252	High-strength steel bolts with associated nuts and washers for structural engineering
AS/NZS 1554	Structural Steel Welding
AS/NZS 2312	Guide to the Protection of Structural Steel against Atmospheric corrosion by the use of protective coatings and related documents (Refer Section 1.4 of the Standard)
NZS 3404:1997	Steel Structures Standard and related documents (Refer to Appendix A of the Standard for specific referenced documents)
AS/NZS 4600	Cold-formed steel structures

2.5 QUALITY ASSURANCE

2.5.1 General

The Structural Steelworker's quality assurance procedures should encompass all aspects of the structural steel construction including, but not necessarily limited to:

1. Compliance for materials with relevant standards.
2. Weld preparation and welding procedures.
3. Weld testing and inspection.
4. Fabrication.
5. Steel preparation prior to coating.
6. Quality of painting/coating.
7. Transportation, handling, and storage.
8. Erection procedures and equipment.

The Structural Steelworker shall advise the Construction Reviewer in writing the name of a suitably experienced and qualified representative from their organisation, to be responsible for the quality control of all structural steelwork.

The Structural Steelworker shall provide details of the fabrication and erection quality control procedures to the Contractor for forwarding to, and approval of, the Construction Reviewer. These procedures should encompass all aspects of fabrication.

2.5.2 Producer Statement – Construction (PS3)

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

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

Refer to the Appendix for additional explanation and a sample of the form of these Statements.

2.6 INDEPENDENT COMPLIANCE INPECTION

2.6.1 General

These clauses outline the requirements and scope of independent inspection to check, test, and certify that structural steelwork on the project complies with this section of the specification, plus all related standard specifications.

The Compliance Inspector will act as the Construction Reviewer for the aspects of the structural steelwork as outlined in Extent of Work below.

2.6.2 Relationship to Structural Steelworker

The Compliance Inspector will have full authority and responsibility to issue instructions to the Structural Steelworker relating to quality assurance procedures and compliance matters. The Compliance Inspector will reject all work that does not comply with this specification. All work redone is required to be retested so that compliance can be ascertained.

The Compliance Inspector must be independent of the Structural Steelworker.

2.6.3 Extent of Work

Testing and certification of steelwork shall cover the following aspects of the structural steelwork:-

1. Review and approve the Structural Steelwork shop drawings for descriptions of weld preparations, preheating requirements, and fully detailed welding descriptions.

2. Review and approve the Structural Steelwork quality assurance plan and procedures.

3. Check for compliance with relevant materials codes.

4. Inspection, testing, and any retesting of welds required to ensure compliance with this specification, AS/NZS 1554, and the contract drawings.

5. Steel preparation prior to painting.

6. Quality and thickness of the prime coat.

7. Review and approve the Structural Steelworkers shop and site welding procedures. Inspect, test, and retest shop and site welds as necessary to ensure compliance.

8. Check all bolting procedures for compliance.

9. The Compliance Inspector is required to provide regular reports.

10. The Compliance Inspector is required to issue a Structural Steelwork Compliance Certificate.

The Compliance Inspector is not required to check dimensional accuracy of the steelwork, nor certify the dimensional accuracy. However, if the Structural Steelworker's work is rejected due to dimensional inaccuracy, use of incorrect sections, or lack of fit, then the Compliance Inspector shall inspect and test the remedial works as part of this contract.

2.6.4 Familiarisation

By tendering for this work it shall be deemed that the Compliance Inspector has familiarised himself with all details pertaining to the contract including the drawings and the Structural Steelwork section of this Specification.

Furthermore, the Compliance Inspector is required to be familiar with the quality performance that can be expected of the various Structural Steelworkers bidding for the work. The tendered sum for compliance inspection shall be taken to include all necessary re-inspection and retesting that the Compliance Inspector deems may be required during this contract.

2.6.5 Personal and Operator Requirements

The Compliance Inspector responsible for the implementation, interpretation, evaluation, and reporting of non-destructive testing shall, for visual, magnetic particle, and dye-penetrant inspection, have the qualifications and experience appropriate to the testing concerned and for radiographic and ultrasonic examination, shall hold signatory approval for such tests from the Testing Laboratory Registration Council of New Zealand. Compliance Inspectors should hold welding inspector certification from the Certification Board for Inspection Personnel or an equivalent qualification acceptable to CBIP. The Design Engineer may require evidence of these qualifications, experience, and signatory approvals. The Design Engineer may require evidence that the Compliance Inspector has sufficient equipment and personnel to discharge his duties under this contract as part of the tender submission.

2.6.6 Inspection and Non-Destructive Examination

Inspection of shop work by the Compliance Inspector shall be performed in the Structural Steelworker's shop to the fullest extent possible, unless agreed otherwise with the Structural Steelworker. Such inspections shall be in sequence, timely and performed in such a manner as to minimise disruptions in operations and to permit the repair of all non-conforming work while the work is in the process of fabrication.

Inspection of site work shall be carried out promptly, so that corrections of non-complying work can be made without unnecessary delays to the progress of the project.

For all non-destructive examination (NDE) the process, extent, technique, and standards of acceptance shall comply with AS/NZS 1554 and Appendix D of NZS 3404, except as modified herein.

Test percentages shall be based on the number of similar joints as opposed to a portion of each joint.

The amount of NDE required shall be generally as suggested in Table D1 of NZS 3404, except that the minimum amount of radiography or ultrasonic testing for grade SP butt welds shall be 100% and grade SP fillet welds shall be 10%, generally in accordance with the flowchart at the end of this section.

All inspection done by the Compliance Inspector is additional to, and independent of, such inspection as is conducted by the Structural Steelworker. However, the Structural Steelworker's inspection procedures shall be taken into account by the Compliance Inspector when setting the overall levels of inspection and NDE required.

When during one inspection, more than 2.5% of the total amount of weld examined exceeds the levels of weld imperfection in AS/NZS 1554 Tables 6.1 and 6.2 and is classed as unacceptable; the Compliance Inspector shall carry out a programme of additional testing. When additional testing is required, it shall conform to the NDT inspection programme described in the flowchart at the end of this section, adapted from Figure 7.2.3.2 of HERA Design Guides Volume 2, Section 17. The cost of all additional testing or retesting shall be borne by the Contractor.

2.6.7 Instructions and Reporting

All instructions to the Structural Steelworker must be given in writing by the Compliance Inspector during the relevant site visit. A copy of those instructions must be sent by facsimile or email to the Contractor and the Design Engineer within 2 hours of the site visit when the instructions were given. Instructions can be neatly handwritten.

Reports are required to be provided regularly to the Contractor and the Design Engineer. The first report is due within two weeks from the date of receipt of the first of the shop drawings and subsequent reports at two weekly intervals until all the steelwork is in place including steel purlins, brace channels, etc.

These reports shall summarise the extent of the structural steelwork carried out over the reporting period, the extent of inspection and NDT work carried out over the preceding period, and a summary of the extent of any non-conforming work and remedial actions taken or required.

2.6.8 Producer Statement – Construction (PS3)

When the works are sufficiently complete that they are ready for application to the Territorial Authority for a Code Compliance Certificate, or otherwise at key handover dates for this section of the works, the Compliance Inspector will be required to certify to the main Contractor that all compliance items covered by this section of the specification have been carried out in full accordance with all Contract Documents and Contract Instructions in the form of a Producer Statement - Construction. This statement will be required to be completed prior to the issue of the Producer Statement – Construction Review by the Design Engineer for the whole or sections of the works as appropriate.

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

Refer to the Appendix for additional explanation and a sample of the form of these Statements.

2.6.9 Other Issues

Issues such as notice for inspection, order of work, etc. shall be by mutual agreement between the Structural Steelworker and the Compliance Inspector.

2.7 SHOP DRAWINGS

2.7.1 General

The Design Engineer's drawings provide overall dimensioning member sizes and typical connections only.

Shop drawings shall be prepared by the Structural Steelworker at their expense from the information presented in the structural and architectural drawings and any other relevant documents to show full construction details.

The fabrication programme shall incorporate adequate time for preparation, review, and revision of shop drawings prior to commencing fabrication. The programme shall allow at least 10 working days for shop drawings review by the Construction Reviewer.

Where discrepancies are noted in the drawings, it shall be the duty of the Contractor to notify the Design Engineer of these discrepancies as soon as they become evident. Failure to do so will not constitute an excuse for failure to perform to programme.

The drawings shall be reviewed by the Construction Reviewer for design concept and general arrangement only. The accuracy and adequacy of the shop drawings are the Contractors responsibility.

Shop drawings shall be prepared insofar as is practicable in accordance with "Detailing for Steel Construction", American Institute of Steel Construction.

Aspects to be covered by the shop drawings shall include, but are not limited to the following:-

1. Dimensions of overall assemblies and individual components.
2. Full component drawings, showing all end preparations required for following work
3. Weld preparation, preheating requirements, and fully detailed welding descriptions. These drawings shall clearly distinguish between shop and site welds.
4. Component assembly details, both for shop assembly and site assembly. All associated bolting, accessories, and/or joining details shall be shown on these drawings.
5. Finishes, including surface preparation and recoating time.

The Contractor shall check all tolerances and clearances between steelwork components and other building elements to ensure a satisfactory fit between all elements. Notify the Design Engineer of any locations where tolerances or clearances need to be increased to ensure satisfactory construction procedures.

2.7.2 Requirements for Electronic Format Drawings

If shop drawings are to be provided in an electronic format, an appropriate viewer shall also be provided at the Structural Steelworkers expense, including licences as appropriate.

The viewer must be suitable for Windows based workstations and be compatible with both 32 and 64 bit versions of Windows XP and Windows 7.

2.8 WORKMANSHIP AND MATERIALS

2.8.1 General

The Contractor shall adhere to all relevant requirements of NZS 3404: 1997 "Steel Structures Standard", and to AS/NZS 1554 "Structural Steel Welding" for supply of all materials and in workmanship both on and off the site.

2.8.2 Steel

Steel shall be Grade 300 of approved origin and conforming to NZS 3404. Any variation of steel supply or source from the specification shall be notified at time of tender.

Hollow Sections shall be grade C350, unless noted otherwise.

Note that this project includes steel for members subject to the seismic design requirements of Section 12 of NZS 3404. Members have been designed as category 4 and the material requirements of Table 12.4 of NZS 3404, duplicated below, shall be met.

	Category 4 Members
Maximum specified yield stress	450 MPa
Maximum ratio of (fy/fu)	0.90
Minimum percentage elongation required on a gauge length complying with ISO 2566.1	15

Where steel is supplied that is not strictly the same grade as that specified, it shall be the contractors responsibility to demonstrate that the steel supplied complies in full with the additional requirements of the Contract Documents, this specification and its nominated references. In particular, where mill certificates are supplied, adequate margin over the nominated yield strength is required in accordance with NZS 3404, Section 17.5.

2.8.3 Alternative Sections

The Contractor shall ascertain at time of tendering whether the steel sizes detailed on the drawings will be available for the job. Any tender based on substitute sizes must be accompanied by a statement listing the proposed substitutions. Substitute sizes will be permitted only with the approval of the Design Engineer. Extra costs of substitute sizes required will be borne by the Contractor.

2.8.4 Welding Consumables

Welding electrodes shall be selected for the grade of steel being welded as set out in AS/NZS 1554.1, unless noted otherwise.

All site welding shall be done using Hydrogen controlled electrodes, unless authorised otherwise by the Design Engineer.

2.8.5 Bolts

Unless noted otherwise on the drawings, bolts and nuts shall be Grade 8.8 high strength, to AS/NZS 1252.

Grade 4.6 bolts and screws shall be mild steel to AS 1111, and nuts shall be to AS 1112.

At least one washer shall be provided under the rotating component of each bolt assembly, and shall be not less than twice the nominal bolt size in diameter. Where necessary to ensure even bearing, tapered washers to BS 4320 shall be used.

The bolts shall be selected so that the projection beyond the nut is not less than two threads and not more than 10mm. There shall be at least one clear run of thread beneath the nut after tightening.

The durability treatment and surface finish of bolts, nuts, and washers shall match that of the components being connected.

2.9 FABRICATION

2.9.1 General

The Contractor and Structural Steelworker shall confirm, by site measurement where possible, all dimensions that affect fabrication or set out of all structures and their individual components.

Fabrication shall comply with Section 14 of NZS 3404.

2.9.2 Cutting

All cutting shall be to NZS 3404 Section 14.3.3

Unless specified otherwise, steel may be cut by sawing, shearing, cropping, machining, or thermal cutting. Hand thermal cutting shall be confined to cutting of section shapes, copes, repairs and other work where machine cutting is not possible.

Surfaces produced by cutting shall be finished square (unless noted otherwise), true to the required dimensions, and free from such defects as excessive roughness which would impair its function or interfere with subsequent fabrication.

Re-entrant corners shall be shaped notch free to a minimum radius of 10mm.

2.9.3 Welding

2.9.3.1 General

All welded connections shall be metal arc welded as shown on the drawings. Unless noted otherwise, all welding shall comply with AS/NZS 1554.1 and the additional clauses of NZS 3404.

Unless noted otherwise, all welds shall be category SP.

2.9.3.2 Site Welding

All site welding shall be done using Hydrogen controlled electrodes, unless authorised by the Construction Reviewer.

Where site welding is required, facilities shall be provided to obtain the same standard of workmanship there as in the shop. Welding in the air shall be reduced to a minimum by assembly and erection procedures. All welding in the air shall be from properly positioned platforms and wherever possible shall be designed to avoid overhead welding. Parts to be welded shall be firmly held by erection bolts. Tacking bolts or cleats, other than those detailed, shall be provided as needed but only after discussion with the Construction Reviewer. If required, tacking cleats will be removed after erection and erection bolt holes filled by welding.

2.9.3.3 Welding Inspection

The Construction Reviewer shall be given reasonable notice when each section of the work is prepared and ready for welding, and shall be given every opportunity to arrange for inspection and to satisfy himself as to the quality of the work and competence of the operators.

Welding inspection may include non-destructive examination. The Contractor shall supply all necessary facilities, ladders, and light scaffolding required for adequate inspection and non-destructive testing. The sequence of work shall be arranged where requested, to facilitate random inspection and non-destructive testing. The steelworker shall prepare welds to the required standards that will permit inspection and/or testing as instructed by the Construction Reviewer.

The Construction Reviewer may arrange for specialist welding advice and inspections to amplify his own inspections. The Structural Steelworker shall allow in his tender a provisional sum of $1000 for specialist welding inspection as instructed by the Construction Reviewer.

For all non-destructive examination (NDE) the process, extent, technique, and standards of acceptance shall comply with AS/NZS 1554 and Appendix D of NZS 3404, except as modified herein.

The amount of NDE required shall be generally as suggested in Table D1 of NZS 3404, except that the minimum amount of radiography or ultrasonic testing for grade SP butt welds shall be 100% and grade SP fillet welds shall be 10%, generally in accordance with the flowchart at the rear of this section.

2.9.3.4 Welding Defects

Welding defects disclosed by inspection or other investigation shall be assessed by the Construction Reviewer and if he so instructs, be cut out and remade.

Any joints so cut out shall be examined and passed by the Construction Reviewer before re-welding.

When welding defects are disclosed, testing of further welds may be ordered at the Structural Steelworker's expense. If stiffeners or other concealing details have been added, these may be required to be removed to permit this additional testing. Re-testing shall comply with the flowchart included at the rear of this section.

2.9.4 Holing

Drill all holes required for all fixings shown or implied on the drawings, including those to be used by other trades.

Holes for bolts shall be drilled, punched, or machine flame cut to NZS 3404 Section 4.3.5.

Edge distances shall be as indicated on the drawings, but in any case not less than 2D, where D is the nominal bolt diameter, from the centre of the bolt to the edge of the steel.

The minimum distance between adjacent bolts shall be as indicated on the drawings, but in any case not less than 2.5D.

Standard holes shall be D + 2 mm for bolts not exceeding 24mm in diameter, or D + 3 mm for larger bolts, unless otherwise noted.

Standard holes for baseplates may be D + 3 mm maximum, unless accompanied by a special flat washer, or otherwise indicated on the drawings. If accompanied by a special flat washer, the hole may be D + 6 mm. The washer shall be square or round with a minimum plan dimension of 2.25D, except that it must fully cover the hole when installed. The washer shall be at least 6mm thick mild steel, have a standard hole, and be welded all round to the baseplate. The washer thickness shall be confirmed by the Design Engineer to suit the loading configuration of the connection.

Slotted holes shall be the appropriate hole size as noted above in width, and the greater of 1.33D or D + 10mm long, unless otherwise noted.

2.9.5 Bolting

2.9.5.1 General

Supply and fix all bolts, nuts and washers necessary for completion of the steelwork, including those to be cast into concrete.

Bolted connections marked on the drawings with the suffix '/S' need be snug tightened only.

Bolted connections marked on the drawings with the suffix '/X' shall be sized and installed to have the threaded portion of the shank excluded from the shear plane of the connection. These connections have been specifically designed and may have inadequate strength if the shear plane of the connection passes through the threaded portion of the bolt.

Bolted connections marked on the drawings with the suffix '/TB' or '/TF' shall be fully tensioned in accordance with NZS 3404 Clause 15.2.5, using the "part-turn method of tensioning. When using the part turn method, location marks shall be permanent, and clearly identifiable for subsequent inspection.

Tensioning of fully tensioned joints shall proceed form the stiffest point, typically the centre, towards the outer free edges of the joint, to ensure that all bolts carry an equal proportion of the load.

2.9.5.2 Bolting to Existing Structures

Before bolting, the existing steelwork shall be thoroughly cleaned to a minimum of 50mm on either side of the connection. All dust, dirt etc. shall be removed to a sound protective coating, unless bolts in '/TF' or '/TB' mode are being used, in which case the coating shall be removed also. Where no coating is present, all rust and scale shall be cleaned off, and new finish applied to the treated area, in accordance with the appropriate section of Finishes below.

2.9.6 Finishes

2.9.6.1 General

The Contractor shall be responsible for the design, specification, and application of protective finishes for steelwork in accordance with the performance specification contained herein. The required performance standards in accordance with AS/NZS 2312 are nominated below in Schedule of Surface Finishes.

In particular, the Contractor must ensure that steelwork which has had a protective treatment system applied is adequately protected during transportation, erection and temporary exposure. Where erection sequence or the programme for subsequent closing in will dictate extended periods of exposure in conditions which the protective system has not been designed for in its final use, the contractor shall be responsible for providing temporary cover, repairing the system to its new condition, or re-specifying the protective treatment to suit the final condition.

Unless noted otherwise in Schedule of Surface Finishes herein, all protective finishes shall comply fully with AS/NZS 2312 in selection, application, and repair. The Contractor is responsible for ensuring that the protective system in its finished state complies in full with the provisions of this specification, as well as the standard.

Where there is any conflict between the two documents, clarification must be sought from the Design Engineer.

Where steelwork is exposed to view in its final condition, the Contractor shall be responsible for ensuring that the protective system used complies fully with the appropriate provisions of the Architect's Specification for gloss and colour. Where there is any conflict between the Architect's Specification and this document, clarification must be sought from the Design Engineer.

Note that in some cases, two top coats may be necessary to achieve the full colour depth for exposed finishes, to the complete satisfaction of the Architect. The Contractor shall allow for this in the tender.

2.9.6.2 Preparation

All welds shall have slag removed, and welds exposed in the finished building shall have spatter removed and be ground to a neat clean finish.

Steelwork to be sprayed for fire protection, or to be cast more than 50mm into concrete, shall have all rust and mill scale etc. removed by power or manual wire brushing and shall be left un-primed.

Faying (contact) surfaces for high strength friction grip bolting (refer Bolting above) shall be masked, unless the contractor can verify by tests that the required performance can be developed with the painted surface.

Surface preparation such as abrasive blasting or wire brushing shall be carried out after fabrication of major elements has taken place, and the appropriate coating applied as soon as possible after preparation, in accordance with the manufacturer's specification, but in any case within 4 hours.

In all cases the total coating shall be applied in the shop in accordance with manufacturer's recommendations. On site painting shall be kept to a minimum adjacent to necessary site joints. These areas shall be made good and painted in accordance with the manufacturer's recommendations. Adjacent areas shall be protected during welding.

2.9.6.3 Hot Dip Galvanising or Metal Spraying

Where required, steel to be galvanised shall be clearly noted on the drawings.

Preparation, coating thickness, appearance and acceptable quality shall be as set out in the Galvanising Manual of the Galvanisers Association of New Zealand.

Zinc metal spraying may be a suitable alternative for galvanising in some locations. Prior approval of the Design Engineer must be obtained before substituting zinc metal spraying for galvanising.

Where required by the Architect, or otherwise in accordance with Schedule of Surface Finishes below, galvanised steel shall be painted.

2.9.6.4 Intumescent Paint Finishes

Where an intumescent paint finish is specified to meet the fire rating requirements, preparation shall be generally in accordance with Finishes-General above.

Steelwork preparation and paint application shall be strictly in accordance with the manufacturers' recommendations for the paint system selected.

Refer to the Fire Resisting Treatmentsection of this specification for guidance on where specific fire ratings are required.

2.9.6.5 Submittals

The Contractor shall submit details of each of the proposed protective finish systems to the Construction Reviewer for review, with the shop drawings ensuring that the Construction Reviewer has 10 working days to complete the review, with additional time to incorporate any alterations, if required, before commencing fabrication.

The following details must be included in the submittal:

- Full details of each system, including preparation requirements, method of application, recoating intervals, and site touch-up and repair methods.
- Full manufacturer's specifications of all coatings.
- A method statement for the temporary protection or otherwise for steelwork during construction.
- A maintenance schedule for the completed system.

The Design Engineer's approval of the protective treatment systems must be received prior to commencing any surface treatment.

2.9.6.6 Schedule of Surface Finishes

Interior steelwork concealed in its completed state	ALK1
Interior steelwork exposed in its completed state (refer also to the Architects specification)	ALK4
Exterior steelwork	HDG600P7

Steelwork requiring intumescent finishes. To meet the fire ratings specified, and to match the exposure classifications for ordinary steelwork in the same location - as above.

Alternative systems may be considered, provided that the nominated system achieves a similar level of performance in accordance with AS/NZS 2312. Alternative systems offering a differing level of protection may be considered, provided that a complying system is offered, and whole-of-life costing information is presented, taking into account maintenance costs.

2.9.7 Storage, Handling and Delivery

Steelwork shall be handled and stored by methods or appliances that will not deform or overstress the steel or damage the finish. In particular, during delivery, care shall be taken to stiffen free ends and otherwise protect steelwork from distortion.

Fabricated steelwork shall be delivered to site in such sequence as shall minimise time for erection, and exposure to potential damage. Where exposure times exceed the protective treatment manufacturer's recommendations, the Contractor shall make arrangements for temporary protection, alter the treatment specification accordingly, or allow for the appropriate maintenance treatment before closing in.

Make all arrangements necessary with relevant authorities for transportation of steelwork.

2.10 ERECTION

2.10.1 General

Erection procedures shall be agreed in advance with the Design Engineer.

Erection shall comply with Section 15 of NZS 3404.

The Contractor shall provide adequate temporary bracing and anchorage as necessary to stabilise the structure.

Every effort shall be made to keep steelwork true to dimension, plumb and level. Final welding of erection connections shall be delayed until each section of the structure is proved true. Final welding up of all steelwork shall be completed before any further loads are added to the structure.

The Steelworker is to co-operate with other trades in erection of steelwork.

2.10.2 Tolerances

Unless noted otherwise herein, tolerances for erection of steelwork shall comply with Clause 15.3 of NZS 3404.

2.10.3 Safety

The requirements of Statutory Authorities, Labour Department, and relevant Acts and Laws shall be adhered to at all times.

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

During erection, the structure shall be maintained in a stable condition by use of temporary bracing and/or guy ropes. Design of temporary support structure shall be the responsibility of the contractor.

On completion of erection of the steelwork, the structure shall be left in a stable condition, pending completion of the whole structure.

2.10.4 Lifting Equipment

Cranes and lifting equipment shall be of adequate capacity to safely lift and maintain work in a stable condition until it is securely braced. Construction loads imparted to the structure during erection or temporary storage of steelwork shall be checked by the contractor. Any damage caused shall be repaired at no cost to the Principal.

2.10.5 Baseplates

Packing under steel bases shall be steel.

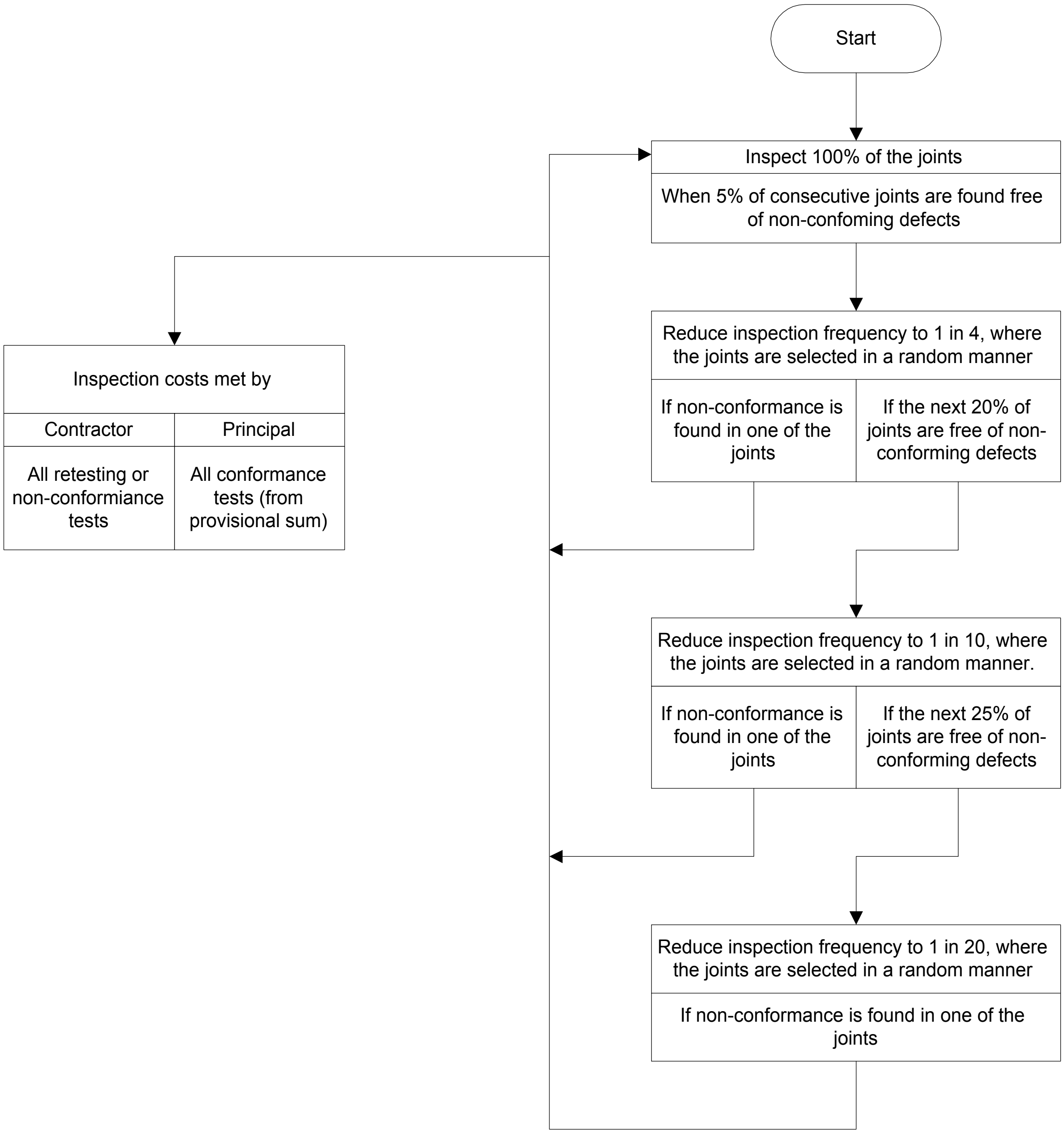
After erection of steelwork is complete, high strength non-shrink grout under baseplates and elsewhere as indicated on the drawings.

2.11 PROVISIONAL SUMS

2.11.1 Welding Inspection

Refer to clause Welding Inspection above for this sum.

NDT INSPECTION PROGRAMME



APPENDIX A

Producer Statement – Construction PS3
(Subcontractor)

PRODUCER STATEMENT - CONSTRUCTION PS3 (SUBCONTRACTOR)

ISSUED BY: ....
(Subcontractor)

TO: ....
(Contractor)

TO BE SUPPLIED TO: ....
(Territorial Authority)

IN RESPECT OF: ....
(Description of Subcontract Work)

AT: ....
(Address)

UNDER: ....
(Building Consent Number)

.... has been contracted by ....
(Subcontractor) (Contractor)

to carry out and complete certain Contract works in accordance with the Contract, titled

....
(Project)

I ....
(Name of Duly Authorised Agent)

a duly authorised representative of the Subcontractor believe on reasonable grounds that the Subcontractor has carried out and completed ☐ ALL ☐ PART ONLY as specified in the Attached Particulars of the Subcontractors Work in accordance with the plans, specifications, and authorised directions of the Principal in accordance with the Contract.

....
(Signature of Authorised Agent on behalf of) (Date)

....
(Subcontractor)