Evans, Marie

From:	Michael Connolly <Michael.Connolly@collierspm.co.nz>
Sent:	Tuesday, 12 October 2010 9:58 a.m.
To:	John Hare
Cc:	Tania Sherborne
Subject:	Forsyth barr
Attachments:	105448 01FECOL0810 001.pdf

John
Please proceed with this report asap. Andy Christian of Pace has done a survey of the building so can advise on some areas of concern. I want to be sure the stairs are ok and fixed correctly. Please note some cracks were covered by the plasterer and these need to be double checked and probably filled correctly
Thanks
Mike

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HolmesConsultingGroup

STRUCTURAL AND CIVIL ENGINEERS

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Hamilton

Wellington

Queenstown

San Francisco

8 October 2010

Mike Connolly
Colliers International Property Management Ltd
PO Box 13478
CHRISTCHURCH 8141

Dear Mike

FORSYTH BARR - DETAILED SEISMIC ASSESSMENT - FEE PROPOSAL

We are pleased to provide you with a fee proposal for the structural engineering services associated with this project.

The project is the post-earthquake review for the Forsyth Barr Tower in Kilmore Street. The building is currently occupied and has been green-tagged, but a detailed assessment is required to identify any possible earthquake damage for insurance and remediation.

The scope of our review is generally as follows:

Stage 1

1. To complete a preliminary structural survey of the building to identify the general form and location of earthquake damage.
2. To complete a review of available documentation of the building to identify potential 'hot-spots' for more detailed investigation.
3. To coordinate with a contractor or maintenance staff to expose key details as required and/or commission testing if required for key elements.
4. To make an assessment of any strength reduction due to the damage and if applicable, to estimate the remaining available strength of the building in terms of Full Code Loading (%FCL), in order to establish compliance with the CCC EPB policy, and to enable an informed decision to be made regarding future reuse.

Stage 2 (if applicable)

5. To prepare remedial details and sketches for pricing.

Stage 3 (if applicable)

6. Prepare construction documentation for the repair work and obtain necessary consents. This may be a staged process, subject to tenancy matters.

Stage 4

7. Provide construction monitoring services as work is implemented and final provision of a PS-4 on completion.

Proposal Basis

Our proposal is based on our previous knowledge of the building and our discussion as to the general scope of work.

We understand the timetable for Stage 1 of this work to be 'as soon as possible'.

Subsequent Stages of the work will be undertaken according to the outcomes of Stage 1, and future timing will be subject to negotiations with insurers and tenancy matters.

Scope of Work

We have allowed for the following scope of work:

- Complete our review as outlined above.
- Prepare calculations, drawings and specification in sufficient detail to gain a Building Consent and for tendering purposes.
- Provide construction drawings and specification in sufficient detail to ensure smooth and timely completion of the construction phase.
- Advise on aspects of tenders and suitability of proprietary components offered as relevant to our role in the project.
- Carry out construction monitoring to CM3 level as per the ACENZ "Guideline on the Briefing and Engagement of Consulting Engineering Services" dated January 2004.
- Respond to and resolve any queries relating to our services that arise during the project construction.
- Provide a Producer Statement – Design (PS1).
- Provide a Producer Statement – Construction Review (PS4).

Fees

We propose initially that we work on a time and materials basis using the hourly rates given in our conditions below. As the work proceeds, we may be able to furnish fixed fees for the later stages of the project. This can be confirmed on completion of Stage 1.

For your planning purposes, our preliminary estimates of the costs of each stage are as follows (excl GST):

Stage 1	$15,000.00
Stage 2 (assuming required)	$5,000.00
Stage 3 (assuming required)	$7,500.00
Stage 4 (assuming required)	$7,500.00
Total	$35,000.00

In addition, we note that other sub-consultants may be required to investigate fire systems and mechanical and electrical services. We will assess this need when commissioned, but recommend a further allowance of $5,000 is included with the above. We assume that your lift service provider will complete a detailed inspection of the lift and shaft, including alignment of the rails.

Conditions of our Offer

- All fees and hourly rates are GST exclusive.
- We have allowed to provide up to 6 sets of the documentation at each of the major issue stages. Additional sets beyond this number will be charged for at $4.00 per A1 copy.
- Our Professional Indemnity and Public Liability insurances are both for NZ$0.5 million respectively and we limit our liability to these amounts and work we document.
- This offer is valid until 30 November 2010 beyond which we may wish to re-negotiate this offer.
- Waterproofing, site survey, structure associated with landscaping and geotechnical work are not included as part of our offer of services, but we would be pleased to assist in the briefing and engagement of these disciplines if required.
- Hourly rates applicable to changes in scope of services:

Project Director	$250/hr
Project Analyst	$200/hr
Senior Project Engineer	$175/hr
Project Engineer	$150/hr
Design Engineer	$125/hr
Project Draughtsperson	$125/hr
Draughtsperson	$100/hr

- Our preference would be to negotiate mini lump sums to carry out any alterations to our scope of services.
- Our general conditions of engagement shall be in accordance with the standard ACENZ/IPENZ/ALGENZ/TRANSIT "Conditions of Contract for Consultancy Services", August 2009 version. If you are not familiar with these conditions of contract they can be viewed on the ACENZ website (www.acenz.org.nz) or contact this office and we will send you a copy of them.
- If our scope of services alters, or our design services become protracted due to forces outside our control, or if construction has not started by March 2011, we reserve the right to re-negotiate our fees.
- A Producer Statement – Construction Review (PS4) will be supplied where required as a condition of the Building Consent, and we have been engaged to carry out construction monitoring, provided that the Contractor has supplied a full Producer Statement – Construction (PS3). You or your representative is responsible for notifying us when work commences on site.

We trust that this proposal meets with your approval. Please sign below and return fax to 03 379 2169 or email to johnh@holmesgroup.com as acceptance of this proposal.

Yours sincerely

Accepted:

John Hare
DIRECTOR

Date:

105448.01FECOL0810.001.doc

Michael Connolly

From: Michael Connolly
Sent: Thursday, 4 November 2010 7:48 a.m.
To: John Hare
Cc: Tania Sherborne; Andrew Christian
Subject: RE: Forsyth barr

Item 3.

John
Thanks For this. I have a concern about the apparent "drop" in the stairs.I assume your report will cover this and the best way to repair
Regards
Mike Connolly

From: John Hare [mailto:JohnH@holmesgroup.com]
Sent: Wednesday, 3 November 2010 4:37 p.m.
To: Michael Connolly
Cc: Tania Sherborne; Andrew Christian
Subject: RE: Forsyth barr

Item 2.

Hi Mike

An update on Forsyth Barr

Contrary to some of my worst expectations, this building has done well. Unlike some of the other similar structures around town, this one has had limited flexural hinging, not enough to warrant significant repairs, and nor have the slabs cracked and performed badly. On that basis, and subject to any individual questions that may be raised on specific locations, the building appears to have suffered no significant structural damage.

We are completing a report on this basis to give you further 'peace of mind'. but in the meantime, we are happy with the repairs that have been completed and are ongoing, but will respond direct to Andy should there be any further discovery as we go.

I will report BNZ separately, but in summary, it has some readily repairable crack damage to structural walls in the stairs and probably not too much else.

Regards

John

From: Michael Connolly [mailto:Michael.Connolly@collierspm.co.nz]
Sent: Tuesday, 12 October 2010 9:58 a.m.
To: John Hare
Cc: Tania Sherborne
Subject: Forsyth barr

Item 1.

John
Please proceed with this report asap. Andy Christian of Pace has done a survey of the building so can advise on some areas of concern. I want to be sure the stairs are ok and fixed correctly. Please note some cracks were covered by the plasterer and these need to be double checked and probably filled correctly
Thanks
Mike

FORSYTH BARR TOWER POST-EARTHQUAKE ASSESSMENT AND REPAIR REPORT

Prepared For:

764 COLOMBO ST LIMITED

Date: 29 November 2010
Project No: 105448.01
Revision No: 1

Prepared By:

Mark Sturgess
PROJECT ENGINEER

Reviewed By:

John Hare
TECHNICAL DIRECTOR

Holmes Consulting Group Limited
Christchurch Office
PO Box 25355
Christchurch 8144
Ph (03) 366 3366

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FIGURES

EXECUTIVE SUMMARY

This report covers the structural damage sustained by the Forsyth Barr Tower at 764 Colombo Street, Christchurch, as a result of the Darfield Earthquake that struck at 4:36am on 4th September, 2010.

The statutory requirements relevant to earthquake damaged buildings are outlined and the general form of the building and its capacity prior to the earthquake are summarised. As the building was designed and detailed to the relevant codes at the time of construction, the building should be considered as having the capacity to resist current code loads.

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. Given the strong motion data available, it is possible that the earthquake produced accelerations in the north-south direction in excess of the design spectra for this building.

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

Minor shear and flexural cracking of the concrete beams at the beam-column interface was observed at all levels inspected, as well as minor cracking of the floor topping slab. Columns in the car park level of the structure adjacent to the ramps were identified as having sustained cracking, and some locations in the car park levels were identified that will pose long term durability concerns.

In general the structural damage sustained is considered minor and the building's capacity immediately following the earthquake is not considered to have been significantly reduced. As such, the damage resulting from the earthquake is not considered to pose a significant structural hazard in relation to occupation of the building.

Following the repairs recommended herein, the lateral load resisting performance of the building should be restored, practically to the level that existed prior to the earthquake, approximately full code loading in today's terms. Repair of the consequential damage such as column cracking will reinstate the durability performance of the building, noting that this will require future maintenance with respect to regular inspection of sealants.

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 *764 Colombo St Limited* to complete a full structural review following the Darfield Earthquake.

The earthquake of 4:36 am on 4th September has subjected the building to strong ground motions which are anecdotally close to 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 Darfield Earthquake 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 764 Colombo St Limited, its insurer, and the Christchurch City Council in its evaluation of the subject property. The findings are not intended for use by other parties, and may not contain sufficient information for the purposes of other parties or other uses.

Our observations have been visual only and limited to representative samples, as described in our record of observations. Our observations have been restricted to structural aspects only. 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 associated Building Regulations 2005 define a moderate earthquake as one that would generate loads one-third 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 CHRISTCHURCH CITY COUNCIL POLICY

The Christchurch City Council recently adopted (under urgency) their Earthquake-Prone, Dangerous and Insanitary Building Policy 2010 [2]. Amongst other things this policy has been amended to include a section of the repair of buildings damaged by earthquake, as follows:

> ***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.

Section 2.3.3 of the policy essentially requires that a building is required to be repaired to a level equating to 67% of current code loading. The Council policy adopts the recent New Zealand Society for Earthquake Engineering (NZSEE) guidelines, "Assessment and Improvement of the Structural Performance of Buildings in Earthquake" [3], for defining the technical requirements for determining a building's earthquake prone status.

3. PRE-EARTHQUAKE BUILDING CONDITION

This section discusses the form and capacity of the building prior to the Darfield Earthquake. A brief discussion of how similar structures have performed during past earthquakes is provided in Appendix A.

3.1 BUILDING FORM

The Forsyth Barr Tower was designed and constructed in the late 1980's. The building comprises seventeen floors above ground level, with the bottom three levels being car park, and the top floor a concrete roof.

Seismically, the Forsyth Barr Tower consists of perimeter ductile concrete moment resisting frames. Internal concrete gravity frames support the floors which span from the perimeter concrete frames to the internal frames.

The floors comprise precast t-beam units which are 225 deep with timber infill between with a 75mm thick reinforced concrete topping slab over the infill.

The building is founded on a raft type foundation with pad footings supporting some of the concrete columns around the buildings perimeter.

3.2 PRE-EARTHQUAKE BUILDING CAPACITY

The Forsyth Barr Tower was designed to predecessor standards of the current NZ Building Code, comprising principally NZS4203:1984 [4] (loadings) and probably DZ3101:1979 (concrete).

The loadings standard, NZS4203 has now been replaced by NZS1170.5:2004 [5]. A comparison of the load levels represented by these two standards is plotted below and shows that the seismic design load has been reduced with the introduction of the new loadings standard. Therefore, the Forsyth Barr Tower is considered to have a capacity in excess of current code levels.

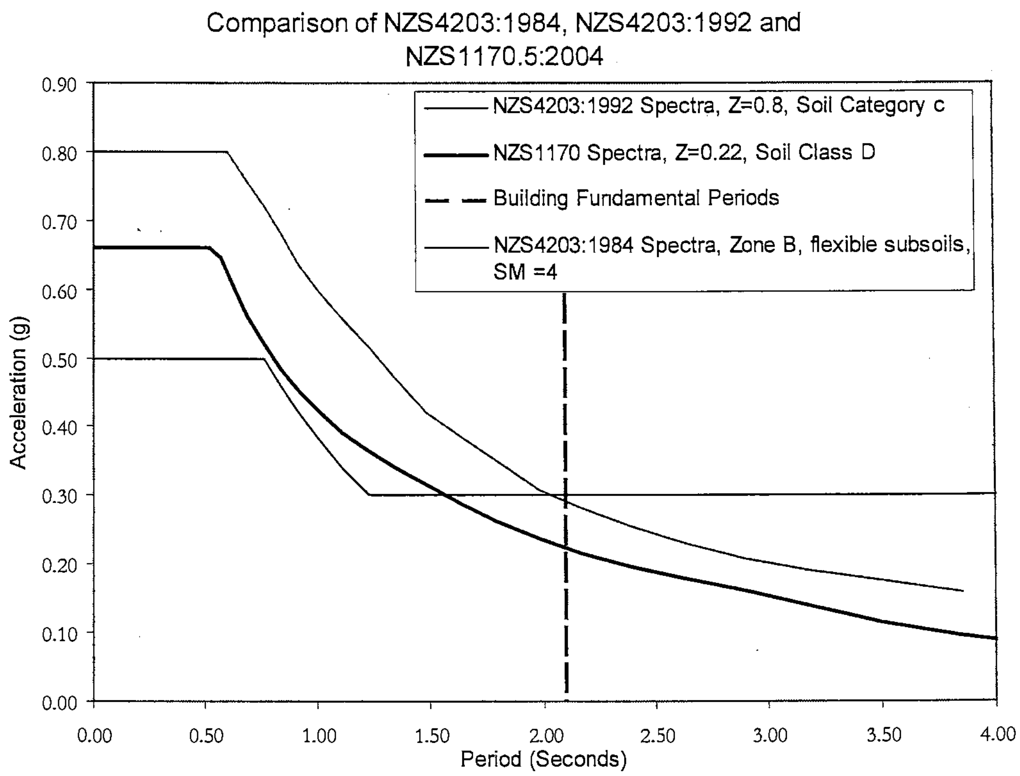


Figure 3-1: Comparison of Design Codes

4. EARTHQUAKE EVALUATION

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.

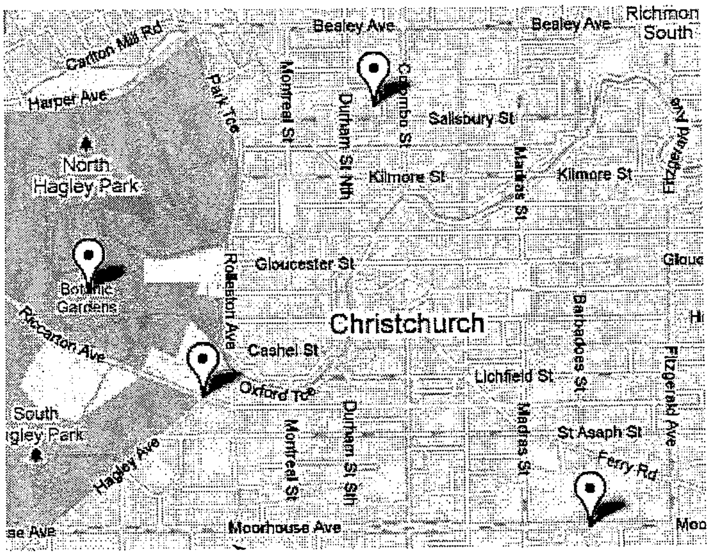


Figure 4-1: Location of Nearby Monitoring Stations

The strong motion shaking data resulting from the initial main shock at 4:36am on the 4th September 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 following graphs plot the acceleration response spectra processed from the Geonet monitoring stations, 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.

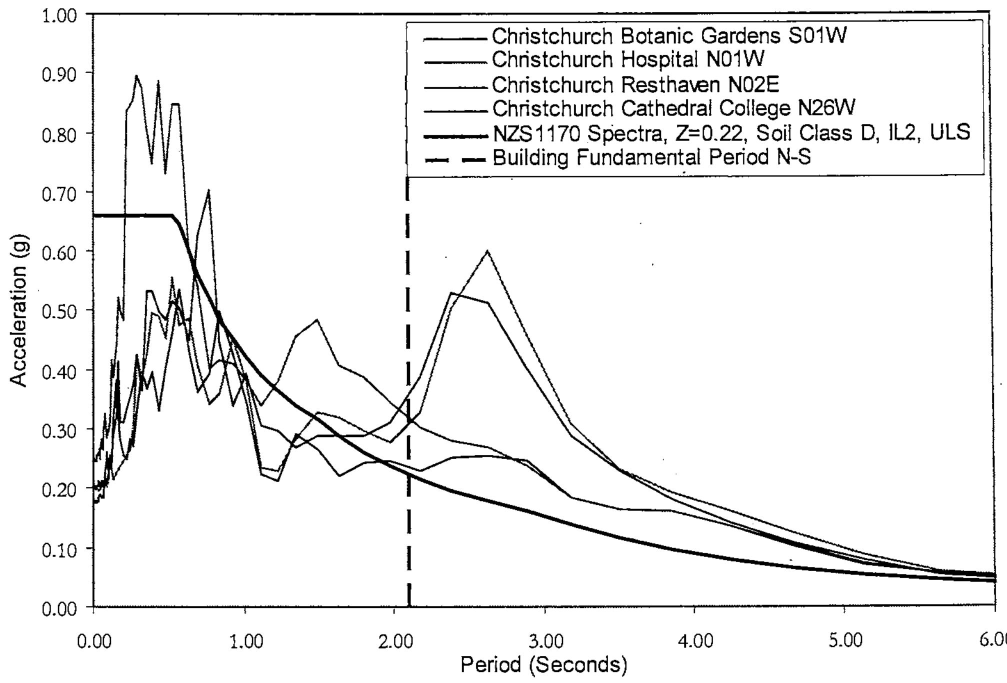


Figure 4-2: 5% Damped Spectra – North-South

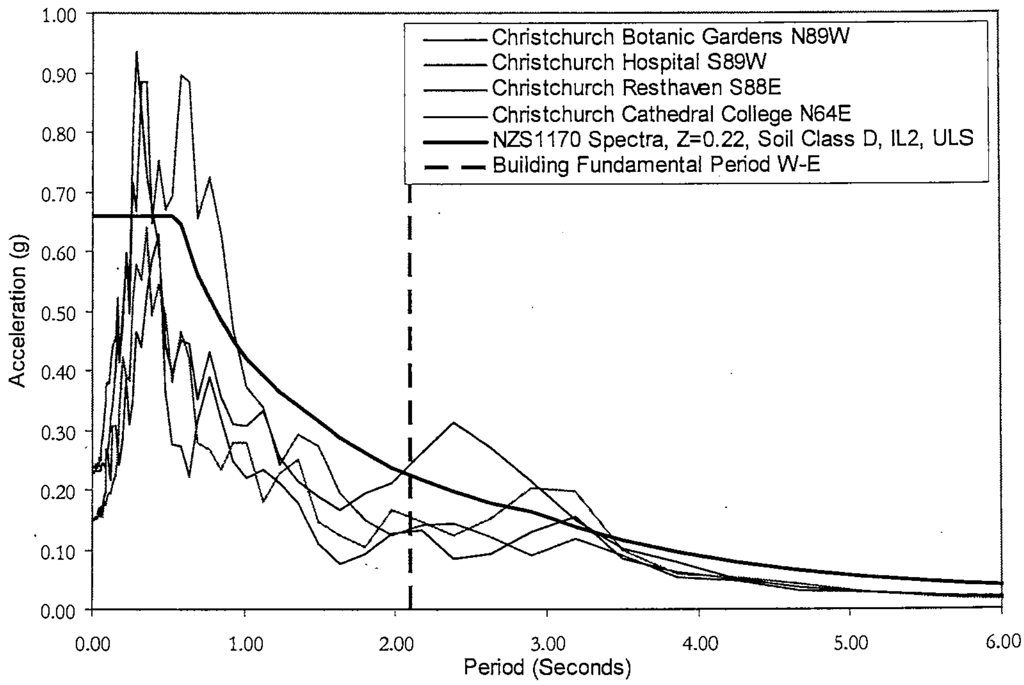


Figure 4-3: 5% Damped Spectra – East-West

It is apparent that in the North-South direction, there is significant variation in the shaking experienced at the different monitoring sites, particularly in the 2 second to 3 second period range. This is due to the highly variable ground conditions around Christchurch.

Previous analyses of the Forsyth Barr Tower have determined the buildings fundamental periods to be between 2.0 and 2.2 seconds for the primary directions. Based on the strong motion data downloaded, it is possible that the earthquake produced accelerations in the north-south direction significantly in excess of the design spectra for this building.

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 Resthaven monitoring station at 4:36am on 4th September shows that the strong motion only lasted for a duration of approximately 10-15 seconds.

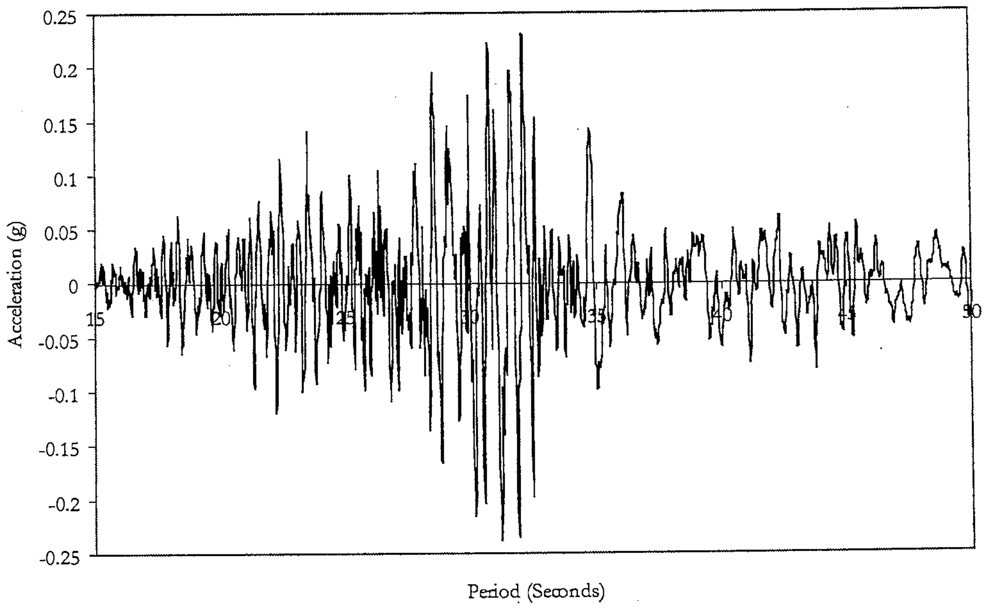


Figure 4-4: Earthquake Record from Christchurch Resthaven Site

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 strong shaking, resulting in increased damage to buildings. As an indication, a large (design level) earthquake in Christchurch 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 possible that the shaking experienced by the building could have exceeded the current code design spectra for the 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 structural drawings [6]
- damage observed during an initial walk around

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

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;

- flexural cracking of the concrete frames, particularly plastic hinging at the beam-column joint
- cracking of the floor slabs between the internal and perimeter beams due to the nature of their geometry
- damage to the upper car park ramps due to movement at seismic joints

Preliminary observations were carried out on 1st November 2010. These identified the following primary areas of damage;

- minor flexural cracking to the concrete beams at the beam-column joint within the car park.
- minor cracking of the car park floor slab. Cracking was regular at centres of either 900mm or 1800mm and was judged to be shrinkage cracking to the topping slab at the location of the concrete 'T' floor beams.
- cracking of columns adjacent to seismic gaps of the ramps and also adjacent to concrete beam in the car park
- some durability issues in the car park levels due to cracking around curbs and in the upper level car park slab
- cracking in bulkheads and linings in the upper levels at the location of beam-column joints indicating movement had occurred in these areas

In general, the building appears to have behaved well after the earthquake event, with only minor damage to the concrete frames and floor slabs noted.

4.3 DETAILED OBSERVATIONS

Based on our preliminary investigations, the following schedule of inspections was developed to complete a detailed structural assessment of the building.

Table 4-1: Detailed Inspection Schedule

Inspection Schedule	
1.	**Concrete frames**
1.1.	The beam-column joints of the concrete frames were inspected at various locations on levels 1,2,3, 9, 13 and 17 to determine how the frames performed over the height of the structure
2.	**Concrete Floors**
2.1.	Concrete floors were inspected at levels 1, 2, 3, 9 and 13 in areas where it was anticipated the most damage to the floors would be located

Inspection Schedule
3. Car Park Levels (Levels 1-3)
In addition to inspection of the frames and floors stated above, the following items were also inspected:
3.1. Structure adjacent to the seismic joint of the ramps to determine how the joint performed and if any damage was sustained by the surrounding structure
3.2. Beam to beam joints and additional columns which were terminated at the final level of the car park structure to ensure that no damage was sustained to additional podium structure
3.3. The base of the columns were inspected at level 3 to ensure no plastic hinging had occurred at the high stress areas linking the larger and stiffer carpark levels with the tower superstructure over
4. Additional inspections of floors as requested
4.1. Inspections of "uneven" floors noted at levels 7 and 9 were also performed to determine if the flooring in these areas had been affected after the earthquake

The detailed structural observations were completed on 2nd & 3rd November 2010. A full record of these observations is attached in Appendix B, with reference plans describing the location labelling used found in Appendix C. A full photographic record of the observations is available electronically on request.

4.4 SUMMARY OF BUILDING DAMAGE

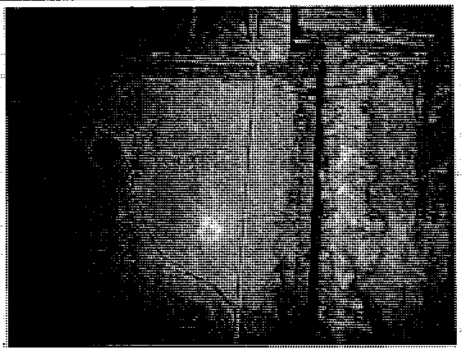
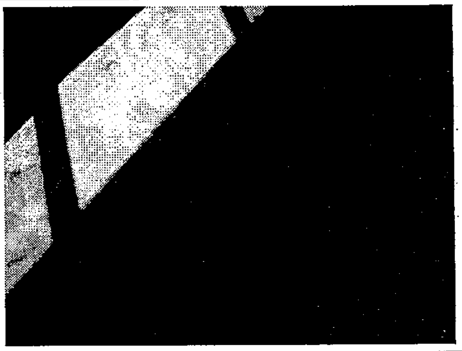
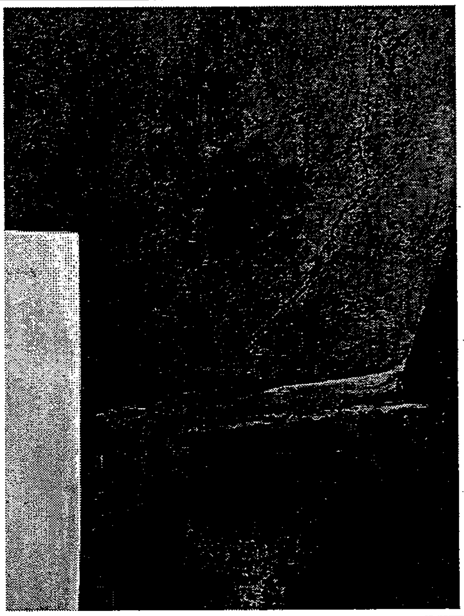
The following is a summary of our observations of the Forsyth Barr Tower, and our conclusions as to its condition and seismic load resisting capacity.

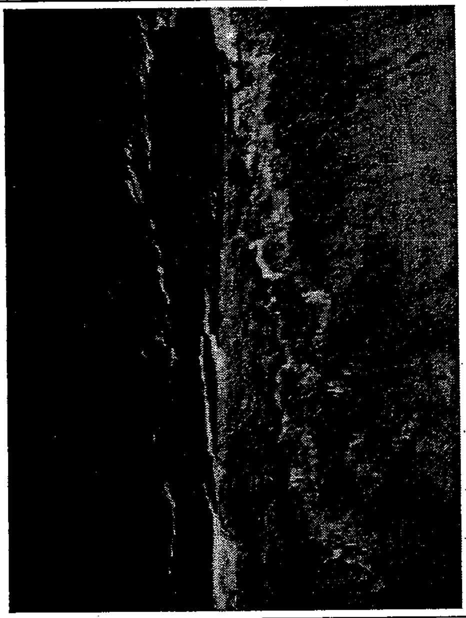
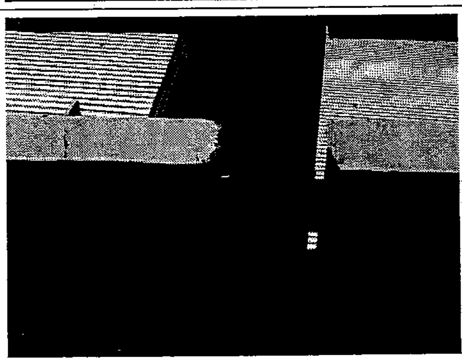
For the main tower over level 3, minor flexural cracking was observed in the beams at the beam column joints. Minor cracking in the floor topping was observed at the levels inspected adjacent to the concrete frames.

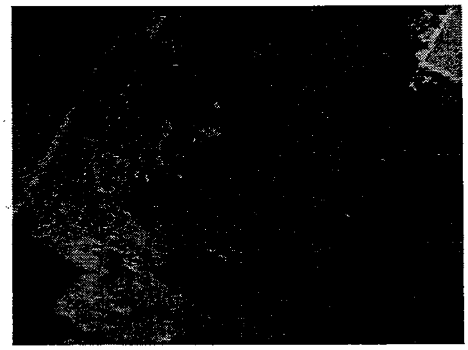
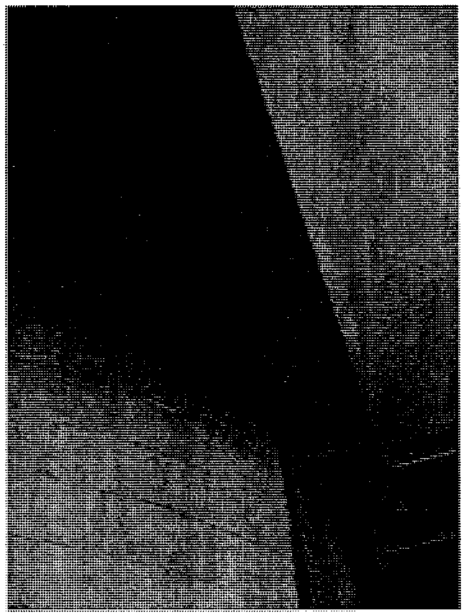
Some cracking to the car park structure was observed and will require repair, particularly relating to column elements near the ramps. Cracking was also noted at beam to beam joints in the car park level and also cracking at the balustrade fixings in the car park which will require repair works.

The following table provides a photographic summary of the primary damage observed.

Table 4-2: Photographic Summary of Primary Damage Observed

Damaged Item	Example
1. Minor flexural cracking was observed in the concrete beams at the beam-column joints on all floors inspected (cracking shown in example photograph highlighted by permanent marker). All cracks measured were less than 0.2mm.	
2. Cracking to slab adjacent to concrete frames for levels 9 and 13 (cracking shown in example photograph highlighted by permanent marker). Cracks measured were less than 0.2mm and required no repair.	
3. Cracking to columns within car park levels	
3.1. Column on grid K between grids 13 and 14 had cracks measuring up to 0.5mm at the base of the column at the location of the seismic joint.	

Damaged Item	Example
3.2. Column on grid M between grid 13 and 14 had cracks measuring up to 0.8mm.	
4. Other items specific to car park levels (Levels 1-3)	
4.1. Minor cracking and spalling was observed at the beam to beam joint on grid M/13. Staining due to water leaching through the joint was observed.	
4.2. Cracking in the curb at the location of the balustrade fixing to the car park level slabs was observed. Some rust staining to the bolts and cleat plate was observed, both above the slab as shown in the example photograph and also to the cleats and bolts fixing the balustrade to the underside of the slab.	

Damaged Item	Example
5. Inspection of Level 7 and 9 floors as requested	
5.1. The infill slab over the previous stair void sustained only minor cracking in one corner of the infill slab. The cracks measured were less than 0.2mm in width. The example photograph shows the joint between the infill and main level 7 slab.	
5.2. The floor topping over the main floor slab causing the uneven floor at level 9 showed no signs of damage.	
6. Cracking had occurred to the masonry wall at the entry to the car park. The steel beam shown supports the ramp between levels 1 and 2. At the time of the inspection, it was unable to be determined if the wall was load bearing or if the beam was fixed to the concrete beam or column shown in the photograph.	

5. REMEDIATION

5.1 REPAIRS REQUIRED

Based on our detailed structural assessment, we have identified the following repairs that are required. These are based on repairing damage caused by the earthquake as well as complying with the CCC regulations requiring buildings being repaired to have capacity in excess of 67% of current code.

Drawings containing specific details of the repairs are attached in Appendix D, with the repair Specification attached in Appendix E.

Table 5-1: Repairs Required

Damaged Item	Recommendation	Sketch reference
1. Minor cracking to the beam at the beam-column joint at all floors inspected	No repair required as all cracks measured less than 0.2mm	No Repair Required
2. Cracking to the slabs adjacent to the concrete frames for levels 9 and 13	No repair required as all cracks measured were less than 0.2mm	No Repair Required
3. Cracking to various columns within car park levels		
3.1. Cracking to the column on grid K between grids 13 and 14 at the location of the seismic joint	Epoxy inject cracks greater than 0.2mm	Specification
3.2. Cracking to the column on grid M between grids 13 and 14	Epoxy inject cracks greater than 0.2mm	Specification
4. Other items specific to car park levels		
4.1. Cracking and spalling at the beam to beam joint on grid M/13	Clean the surface of beams to remove staining. Breakout any areas of spalling and clean the face of the concrete. Reinstate concrete	Specification

Damaged Item	Recommendation	Sketch reference
	with repair mortar. Epoxy inject cracks greater than 0.2mm. Seal the joint using a mastic sealant or topping at level above to prevent further moisture penetration into the joint.	
4.2. Cracking in the curb at the location of the balustrade fixings	Repair in accordance with detail by removing existing cleats and bolts, repairing cracked and spalled concrete, and then fix the spandrel to the RHS post using the angle bracket as specified.	Detail D01
5. Inspection of floors at Level 7 and 9 as requested		
5.1. Joint between the infill slab over the previous stair void penetration and the main level 7 slab	No repair required as all cracks measured less than 0.2mm	No Repair Required
5.2. Topping slab causing uneven floor at level 9	No cracking found and as such no repair required	No Repair Required
6. Cracking to the masonry wall at the entry to the car park around the steel support beam	Remove blockwork adjacent to the steel beam to allow inspection of the connection and to determine how the steel beam is supported. Contact Holmes Consulting Group to arrange an inspection. NOTE: Do NOT remove the blockwork directly beneath the beam unless propping to the beam is installed at the wall prior to removal.	Remove blockwork and arrange further inspection

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

5.2 POST-EARTHQUAKE BUILDING CAPACITY

In its damaged state following the earthquake, we do not consider the Forsyth Barr Tower 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 building is expected to be slightly more flexible than previously, but its overall capacity will be unchanged following the repairs outlined above and should be considered in excess of 67% current code.

Following the recommended repair of the consequential damage to the gravity system and repair of the durability concerns outlined above, the durability performance of the structure should be restored. However this should continue to be monitored throughout the life of the building to ensure that the sealants are maintained to prevent moisture penetration and that any future cracking is repaired.

6. REFERENCES

1. *Building Act 2004*

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

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

4. *Code of Practice for General Structural Design and Design Loadings for Buildings, NZS4203:1984*, Standards New Zealand, 1984.

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

6. *Robert Jones House, Christchurch*, Holmes Consulting Group, 1988.

APPENDIX A

APPENDIX A – TYPICAL BUILDING FORMS

The following outlines the generic performance and damage expected of a variety of building forms, constructed at different periods of New Zealand's construction history.

DUCTILE CONCRETE MOMENT RESISTING FRAMES

Ductile Concrete Moment Resisting Frames (DCMRFs) are buildings that have some to full modern detailing and are designed with practices that account for seismic attack. Largely restricted to the CBDs of the main cities, DCMRFs were constructed from about 1975 to the present.

In terms of New Zealand Standards for Concrete Structures: NZS 3101: in 1982, the first version, there was an enormous leap in design and detailing practices for seismic performance of buildings. In 1995, there were significant improvements in detailing for robustness of structures; in 2006, further improvements were made. The sections of the Ministry of Works and a few leading structural engineers were developing and employing what was to become the accepted modern seismic engineering principles from 1975 onwards.

The lateral load resisting mechanism is typically frame action on all sides.

The seismic performance should be acceptable in most cases as detailing for ductility was employed and, through "capacity design", acceptable plastic mechanisms should have been selected.

Frame action should result in the preferred weak beam-strong column mechanism. In a limited number of cases, for buildings three storeys or less, ductile column sidesway mechanisms, may be acceptable.

Prior to NZS 3101:1995, the design of interior columns was not up to full ductility detailing. If the columns are in buildings with high lateral drift then these columns may have insufficient ductility and gravity capacity in a major seismic event.

Lift shafts had evolved away from reinforced concrete cores to sheathed timber partitions. These partitions have little lateral capacity; however, the stairs and lift guides, in these cores, can be significantly damaged due to the relatively large interstorey drifts expected in these MRFs. The presence of heavy reinforced concrete stairs can alter the behaviour of the building, acting as stiff props between floors (as do ramps). Up until the late 1990s, the stairs are prone to collapse due to jamming between floors; subsequently, detailing of the stairs (sliding at one end) became the accepted feature.

Early floors and roofs are usually cast insitu concrete flat slabs, though at this time precast concrete floors with cast-in-place concrete toppings were emerging. By the early 1980s, most floors and roofs in commercial buildings were prestressed precast concrete units with concrete topping. The issues with precast concrete floors are highlighted in a section specifically written on these.

Problem	Fix	Impact
1. Columns (typically interior) have insufficient ductility and shear capacity.	a. Wrap the columns with steel plates or reinforced concrete or FRP jackets.	Intrusive, with disruption to the fit-out of each floor affected. If an exterior column, a very intrusive solution. May be impractical in many cases, where cladding impedes access, or where beam-column joints are inaccessible due to concrete floors or two-way frames.
	b. Supplementary columns added, to carry a portion of the gravity load.	Very intrusive on fit-out and architecture. No enhancement of the lateral capacity of the building, typically.
2. Column sidesway mechanism, ***not specifically designed for***, results in excessive ductility and shear demand on columns.	a. Add separate stiffer lateral load resisting system to reduce displacement.	Very intrusive solution. New system requires new load path, so that diaphragm and collectors need to be reassessed, and new foundations will be required.

Problem	Fix	Impact
	b. Introduce supplemental damping into the structure to reduce demand on frames	Dampers tend to be very expensive although less intrusive than complete new supplemental structure. If using hysteretic dampers, load to foundations increase significantly requiring upgrade.
	c. Strengthen columns and beam-column joints to force beam mechanisms	Very intrusive particularly on external frames. May be impractical in many cases, where cladding impedes access or where joints are inaccessible due to concrete floors or two-way frames.
3. Inadequate connections of floor and roof diaphragms to MRFs – common where the MRFs are adjacent to lifts and stair and hence separated from main diaphragm support	a. Disconnect diaphragm altogether if alternative load paths exist.	Only possible in a limited number of cases. Care needs to be taken to ensure that out of building load support to MRFs is still provided.
	b. Strengthen diaphragm in areas affected with steel straps, concrete or FRP overlay.	FRP least intrusive if possible. Concrete overlay thickness makes stairs etc a problem due to height rise. Steel straps difficult to fix appropriately.
4. Inadequate stiffness of the structure as a whole meaning that	a. Add separate stiffer lateral load resisting system to reduce displacement.	Very intrusive solution. New system requires new load

Problem	Fix	Impact
the building exceeds drift limits.		path, so that diaphragm and collectors need to be reassessed, and new foundations will be required.
	b. Introduce supplemental damping into the structure to reduce displacement.	Dampers tend to be very expensive although less intrusive than complete new supplemental structure. If using hysteretic dampers, load to foundations increase significantly requiring upgrade.
5. Torsional behaviour through secondary structures (walls, stairs or ramps) which are incompatible with displacements of the moment resisting frame structures.	a. Modify structure that is inducing the torsional response (stairs or ramps or concrete stair).	Moderate work may be required. Cutting one end of stairs/ramps, possibly providing additional gravity support structure.
	b. Introduce stiffer load elements in parallel frames such as braced frames to reduce eccentricity	Significant intrusion into the existing space. May increase foundation loads to affected frames requiring expensive foundation work.
	c. Remove the concrete cores	Very extensive work will be required. If the core was part of the exterior fabric, can introduce weatherproofing issues in boundary walls.

FULLY FILLED REINFORCED CONCRETE MASONRY

Fully (solid) filled reinforced concrete masonry was used from the mid-1970s. As the cells or the flues are fully filled with concrete grout, these walls are stronger that the lightly reinforced partially filled concrete masonry walls and behave similarly to a reinforced cast-in-place wall of the same dimensions.

Fully filled reinforced masonry walls are an alternative way of building structural walls. Therefore the performance issues of structural concrete walls will apply to these concrete masonry walls.

Poor performance of buildings with fully filled reinforced concrete masonry walls can be attributed to:

- Inadequate flexural strength
- Inadequate shear strength.
- Inadequate foundations, not sized for forces and displacements that are expected for a major earthquake.
- The connections of concrete floor diaphragms to walls may be compromised because of:
    - Stair and lift penetrations through the adjacent floor
    - Inadequate design of reinforcement across the floors and in to the walls
    - Displacements of the walls (such as by rocking, by design or by inadequate foundations) can damage the floor to wall connections. The structure being restrained by the walls can disconnect from the walls and collapse.
    - Floors disconnecting from the walls due to inadequate connection hardware or the face shells of the blocks separating from the grouted flues.

Fully filled reinforced concrete masonry walls, constructed from the mid-1990s, are not expected to have major damage. However, a remaining issue will be the integrity of the connections of the floors to the walls (though improved over that used for earlier walls).

Problem	Fix	Impact
1. Inadequate shear strength	a. Build a new reinforced wall or skin against the	Highly intrusive solution.

Problem	Fix	Impact
	existing wall – New concrete and reinforcement needs to be placed.	
	b. Apply a new skin – FRP typically , though steel plates can be used.	Moderately intrusive.
	c. FRP or steel strips strapped to the walls. Expoxying the strips to the wall.	Moderately intrusive.
	d. Selective weakening, by cutting some or all of the vertical bars in the wall.	Moderately intrusive. Limited use: usually requires addition main structure to be added elsewhere.
2. Inadequate foundations	a. Build new foundations, possibly including piles	Very highly intrusive
	b. Selective weakening, by cutting some or all of the vertical bars in the wall.	Moderately intrusive. Limited use: usually requires addition main structure to be added elsewhere.
3. Inadequate connections of floor and roof diaphragms to the walls.	a. Disconnect diaphragm altogether if alternative load paths exist.	Only possible in a limited number of cases. Care needs to be taken to ensure that face load support to walls is still provided.
	b. Strengthen diaphragm in	FRP and ply wood

Problem	Fix	Impact
	areas affected with steel straps, concrete or FRP overlay. Plywood overlay on timber floors also.	least intrusive if possible. Concrete overlay thickness makes stairs etc a problem due to height rise. Steel straps difficult to fix appropriately.
4. Inadequate flexural strength	a. Provide tension capacity by FRP, reinforcing rods or flat steel plate bonded to the wall (epoxied and bolted).	Moderately intrusive
	b. Build new boundary elements attached to the wall, reinforced vertically and transversely.	Highly intrusive
	c. Typically will require new foundations as a result of 4.a. and 4.b.	Very highly intrusive

PRECAST CONCRETE FLOOR SYSTEMS

Early floors and roofs are usually cast insitu concrete flat slabs, though at this time precast concrete floors with cast-in-place concrete toppings were emerging. By the late 1970s, most floors and roofs in commercial buildings were prestressed precast concrete units with concrete topping.

Floors and roofs must act as large flat elements (diaphragms) that tie the vertical parts of the building together and transfer forces generated by the earthquake or wind across the building to the vertical lateral force resisting structures.

A precast concrete floor system may be a slab, a hollowcore unit, "rib and timber" infill, or single or double tee units. All the variations will have reinforced cast-in-place topping (50 – 70 mm thick, and on occasions, up to 150 mm thick).

Precast concrete floors started in around 1965; these where typically short spans (≈ 6 m) and conventional reinforced. From the early 1970s, prestressing of the precast floor units started, permitting longer spans.

Prior to 1998, the minimum seating for precast floors was typically 50 mm. Post-1995, the seatings are specified as a minimum of 75 mm. Observation in the field shows that the seatings were less than these specified minima, in each time period, mainly due to construction tolerances and poor design.

From the mid-1970s through to 1995, for flat units (slab and hollowcore), the provided seating on site ranged between 25 to 50 mm. For stem supported Tees, the seatings ranged between 75 and 150 mm. For rib and timber infill the seating range from 25 to 75 mm.

Each floor type has some common structural performance traits:

- Typically supported on the unreinforced cover concrete. Though reinforced ledges (armoured and unarmoured) have been used to support relatively long and/or heavily loaded floors.

- Lack of alternative load paths (redundancy) should local overload/collapse occur.

- Loss of gravity capacity during moderate to large earthquakes – a function of the overall building characteristics and the support connection details of the floor to the main structure.

  - Loss of support through spalling of the units and supports, and pulling off the support by neighbouring beams undergoing plastic elongation.

  - Catastrophic failure of the floor when deformations are imposed on the floor (unaccounted for in the design of the floors) by the neighbouring parts of the structure (warping of the floor, rocking walls, prising apart of the units or the topping off the units and significant bending causing tension on the top of the floor).

Concrete and steel Moment Resisting Frames are expected to displace laterally at or exceeding the Loading Code limits (those design from mid 1970s onwards).. If theses frames form plastic hinges that undergo plastic elongation, sections of floor can become unsupported. Sections of floors drop on to the floor below. If one unit falls, it is unlikely to overload the floor below. Should a significant section of floor fall, then it is likely that the lower floor below will fail and fall with the first floor on to the next causing a cascading collapse of all floors below.

The elongation of beams and associated reduction of seating is a function of the drift of the MRFs. Further or compounding causes of loss of support, in all structures, is the distortion of the supports. Each building should be assessed for critical weaknesses and performance features including what was the as-built seating available to support the floors.

Floors and roofs need to act a "diaphragms". To date, the design of diaphragms has been simplistic and do not cover all the critical behaviour (maintaining load paths, detailing the floor to structure connections and dealing with large penetrations through the diaphragms, for stairs and lifts). Older cast-in-place conventionally reinforced slabs are expected to perform better than the topped precast concrete floors. This is due to the brittle nature of hollowcore and some tee units and the relatively narrow ledges supporting floor units. The reinforcement in the topping, up until 2004, was typically a non-ductile cold-drawn wire mesh. After 2004, the reinforcement was required to be ductile. Though under very limited circumstances, the non-ductile mesh could be used).

Load paths across the floors were not visualised well up until 2000. The additional reinforcement needed along these load paths was not sized or placed correctly or not consider at all. Though improved, this design feature is still being done inadequately in modern structures.

Problem	Fix	Impact
1. Inadequate support: seating length and unreinforced cover concrete	a. Build an additional ledge (steel angle, typically) or hanger (structural steel cleat or "U" shaped support).	Low to medium intrusive solution. Depends on access to the plenum space below each floor. Lowest cost of the three options here.
	b. Install vertical reinforcement, "hangers", through the critical areas of the floor. Steel rods, bolts or FRP.	Medium intrusive solution. Medium cost
	c. Install catch frames of steel beams or trusses under the floors.	Highly intrusive solution. Relatively high cost

Problem	Fix	Impact
2. Moment resisting frames – inadequate stiffness of the structure meaning that the building exceeds drift limits, causing loss of support.	Refer to the section on Ductile Concrete Moment Resisting Frames	
3. Inadequate connections of floor and roof diaphragms to the vertical structure.	a. Disconnect diaphragm altogether if alternative load paths exist.	Only possible in a limited number of cases. Care needs to be taken to ensure that face load support to walls is still provided.
	b. Strengthen diaphragm in areas affected with steel straps, concrete or FRP overlay.	FRP least intrusive if possible. Concrete overlay thickness makes stairs etc a problem due to height rise. Steel straps difficult to fix appropriately.
4. Inadequate tension capacity across zones of the floors.	a. provide tension bands or "collectors: FRP, reinforcing rods or flat steel; plate cut in to the floor (epoxied and bolted). Steel members fixed in place under the floors.	FRP - moderately intrusive Rebar or flat plate - moderate to highly intrusive Steel members underneath - very highly intrusive.

PRECAST CLADDING SYSTEMS

Precast cladding became common with the advent of ready-mix concrete, and larger cranes, at which time architects began experimenting with precast concrete as an alternative to cast-in-place or built-up cladding systems. Early examples date from the early 60's.

Although seismic loadings and design techniques became more formalised with the 1965 code, it was not really until 1976 that the considerations of parts and portions loading was more clearly articulated, along with the need to provide adequate clearances to structural members to allow for the deformation of the main building frames. Coupled with this was the understanding of the significant forces that the connection may be subject to.

Another significant issue affecting early precast cladding systems is corrosion. This manifests in two ways – firstly in the lack of cover concrete leading to corrosion of the reinforcement, leading in turn to spalling and cracking of the units. Secondly in corrosion of the connections, many of which are simple drilled-in or cast-in mild steel anchors, in positions that were not as waterproof as may have been anticipated.

Although these systems may not impact on the performance of the structure as a whole, there are in some cases life safety implications from these elements that could or should be addressed. Notwithstanding, failure of the panels will not generally cause failure of the main structure, so buildings with unsafe panel systems will not necessarily be EPB's because of this. The only exception would be if the panels engage with the main structure and modify its behaviour enough to cause failure.

For the sake of completeness, some issues and fixes are listed below:

Problem	Fix	Impact
1. Concrete cancer has weakened panels to the extent that large pieces are able to fall in event of earthquake.	a. Break out and repair affected areas of panels	Expensive and difficult, as extent of damage is difficult to determine.
	b. Remove panels and reclad building	Very expensive solution and very intrusive as will involve linings also.
2. Connections are weak and/or corroded.	a. Replace connections.	May be difficult if connections are inaccessible, and/or expensive if it requires removal of linings.

Problem	Fix	Impact
	b. Remove panels and reclad building	Very expensive solution and very intrusive as will involve linings also.
3. Panels have inadequate clearance to structure	a. Cut back or replace panels to ensure no impact can occur	Very expensive and/or intrusive as likely to impact internal linings.

APPENDIX B

APPENDIX B – RECORD OF OBSERVATIONS

Inspection date: 02 November 2010

NOTE: The level referenced in this appendix is the level below the beam joint (i.e. if level 17 is referenced in the level column the beam-column joint is at the underside of level 18.) For floors, the level referenced is the level of the floor inspected (i.e. if level 13 is referenced this is the floor slab of level 13). Where a column is reffered to, it is the column over the floor refernced (i.e. a column on level 2 spans from level 2 to level 3).

Level	Building Element	Location	Observations	Repair Required?	Photo Reference
L17	Perimeter Beam to Column Connection	Grid B/12	No cracking formed at joint.	N	01-05
	Perimeter Beam to Column Connection	Grid M/4	Hairline cracking (<0.2mm) formed at joint.	N	06-10
L13	Perimeter Beam to Column Connection	Grid B/12	Hairline cracking (<0.2mm) formed in beams at joint (i.e. at beam spanning between grids 11 and 12 and beam spanning between grids B to E).	N	11-17
	Perimeter Beam to Column Connection	Grid B/9	Hairline cracking (<0.2mm) formed at beam to column joint. Photo's taken are on the Southern side of the joint.	N	18-21
	Perimeter Beam to Column Connection	Grid M/4	Hairline cracking (<0.2mm) formed on Southern side (i.e. grid 5 side) of internal beam spanning between grids L and 4 (photos 22-25). No cracking in perimeter beam spanning from grids 4-6 (photo 26).	N	22-26
	Perimeter Beam to Column Connection	Grid M/1	Hairline cracking (<0.2mm) formed on beam spanning North-South (grids 1-4) at column joint (photos 27-29). A	N	27-31

Level	Building Element	Location	Observations	Repair Required?	Photo Reference
			hairline crack was also found in the column (photo 30), and no cracking was found on the beam spanning East-West (grid L to M, photo 31).		
	Perimeter Beam to Column Connection	Grid J/1	Hairline cracking (<0.2mm) was found in the perimeter beam on the Eastern side (spanning from grids J to K). Joint with beam spanning from grid F-G not inspected.	N	32
	Floor Slab	Grid C/F-K	Cracking (up to 0.2mm) was found in the top of the slab when the carpet was removed.	N	33-37
L9	Perimeter Beam to Column Connection	Grid M/1	Hairline cracking (<0.2mm) was found in both the beams at the joint (photos 38-40 show cracking in the beam spanning between grids L-M and 41-43 in beam spanning grids 1-4).	N	38-43
	Perimeter Beam to Column Connection	Grid J/1	Hairline cracking (<0.2mm) found in beam on grids J-K side. No cracking in column, beam on other side of column not inspected.	N	44
	Perimeter Beam to Column Connection	Grid M/4	Hairline cracking (<0.2mm) was found in Southern side of internal beam at column joint (photos 45 and 46), no cracking found in perimeter beam at column joint for beam spanning from grids 5-6 (photo 47). Joint on other side of column not inspected.	N	45-47
	Perimeter Beam to Column Connection	Grid B/12	Hairline cracking (<0.2mm) was found in both beams connected to column.	N	48,49
	Perimeter Beam to Column Connection	Grid B/9	Hairline cracking (<0.2mm) was found in beam at connection (cracking in beam on grid 9-10, other side of column not inspected). Some minor spalling had occurred in the beam, however this appeared to be a result of poor fixing	N	50,51

Level	Building Element	Location	Observations	Repair Required?	Photo Reference
			of timber framing rather than due to earthquake loading.		
	Floor Slab	Grids 2/5-6	Hairline cracking (<0.2mm) was found in floor after carpet was removed.	N	52,53
L3	Perimeter Beam to Column Connection	Grids B/12	Hairline cracking (<0.2mm) was found in beam spanning between grids B-E at column connection.	N	54,55
L2	Column	Grids M/13-14	Cracking up to 0.8mm was found in the column at the base of the ramp. Patching of the column will be required.	Y	56,57 (see also 62-64 taken 03.11.10)

Inspection date: 03 November 2010

Level	Building Element	Location	Observations	Repair Required?	Photo Reference
L3	Car park Barrier Fixing	Fixings for length of Grid O	A crack had formed in the curb at the fixing of the carpark barrier. The crack appeared to emanate from the bolt fixing the barrier.	Y	58,59,72,86,87
L2-L3 Ramp	Cracking in Ramp Slab	Ramp between grids E-M/13-14	Cracking had occurred in the joint of the ramp in the concrete column on the Northern side.	Y	60,61
L2	Junction of Beams	Junction of beams at grids M/13	Leaching water through the slab at the beam joint was occurring. Minor spalling had occurred on the Northern side of the beam joint.	Y	65-68
L1	Column	Grids M/13-14	Cracking up to 0.3mm was found in the column at the base of the ramp. Patching of the column will be required.	Y	69-71

Level	Building Element	Location	Observations	Repair Required?	Photo Reference
GL	Steel Beam Connection	Grids E/14	Cracking in the masonry surrounding the connection to a steel beam was found at the entrance of the car park. Further investigation will be required to determine if the masonry is load bearing and as such what repair is required..	TBC	73,74
L9	Floor Slab	Grids 2/10	Carpet was removed to investigate floor which appeared uneven. This was the result of an existing topping and was judged to be a non-structural issue.	N	75-77
L7	Floor Slab	Grids A/5-6	Carpet was removed to investigate floor which appeared to be uneven. The floor was an infill which occurred when a stair void was filled. A single hairline crack (<0.2mm) was found in the topping of the infill slab, a joint was found around the perimeter of this floor. An inspection of the soffit of the slab (performed by removing ceiling tiles in Level 6 showed no movement of the seating of the infill slab).	N	78-85

APPENDIX C

HolmesConsultingGroup

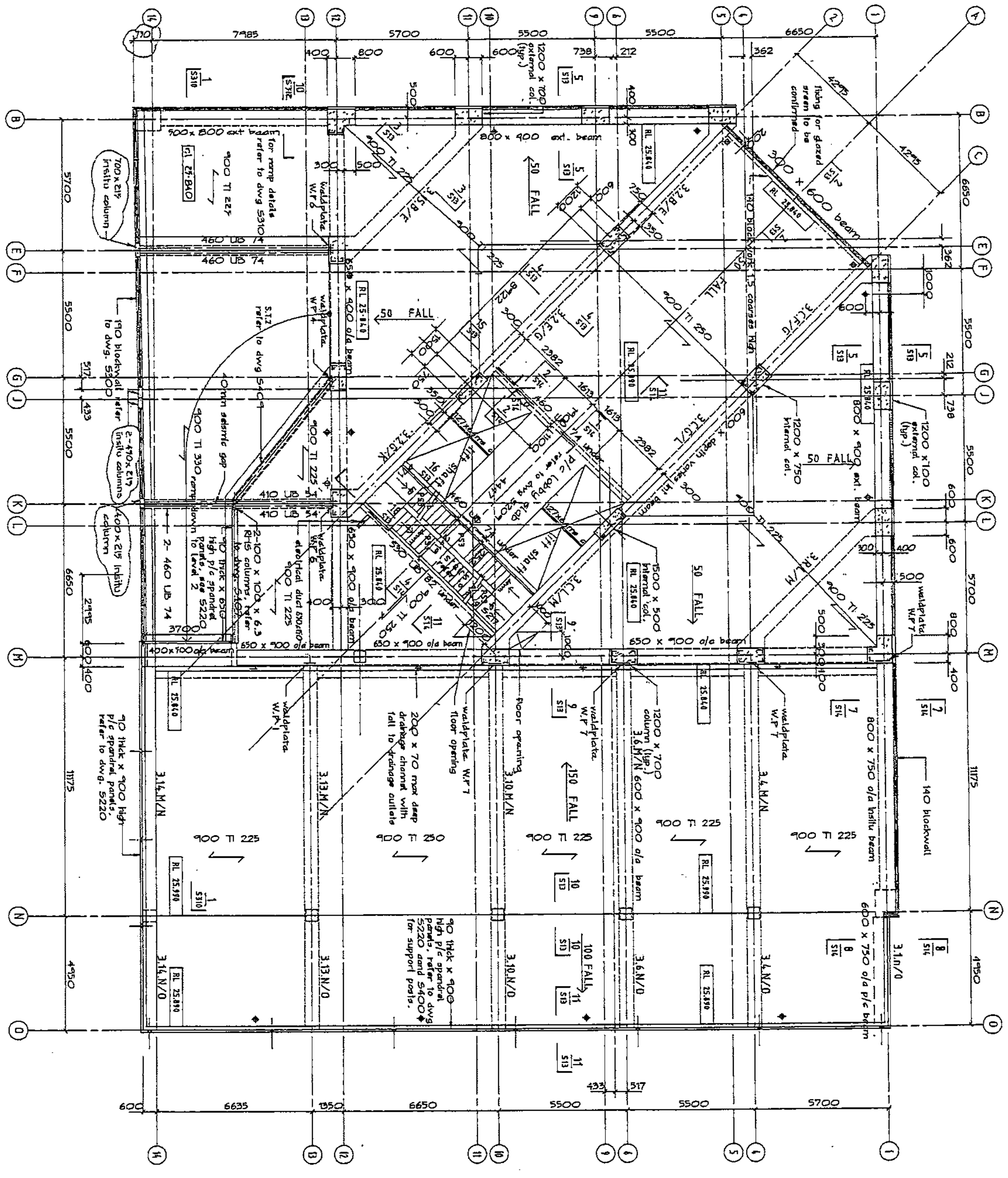
Project Name: FORSYTH BARR DETAILED EQ REVIEW

Project #: 105448.01

Sketch #: C01

Title: LOCATION LABELLING PLAN

Date: 18.11.10 Drawn: MS.



Rev.	Date	Amendment
1	22/2/80	foundation permit
2	4/3/80	permit
A	16/7/80	construction
B	19.5.81	electrical duct relocated
C	30.7.81	sections 9 and 11 added
D	12.7.88	podium hold removed

For package number

Warren & Mahoney Architects Ltd, Christchurch Wellington & Auckland

HOLMES CONSULTING GROUP, STRUCTURAL AND CIVIL ENGINEERS

ROBERT JONES HOUSE CHRISTCHURCH

SHEET TITLE: FLOOR PLANS LEVEL 3 CARCASS

SCALE: 1:100

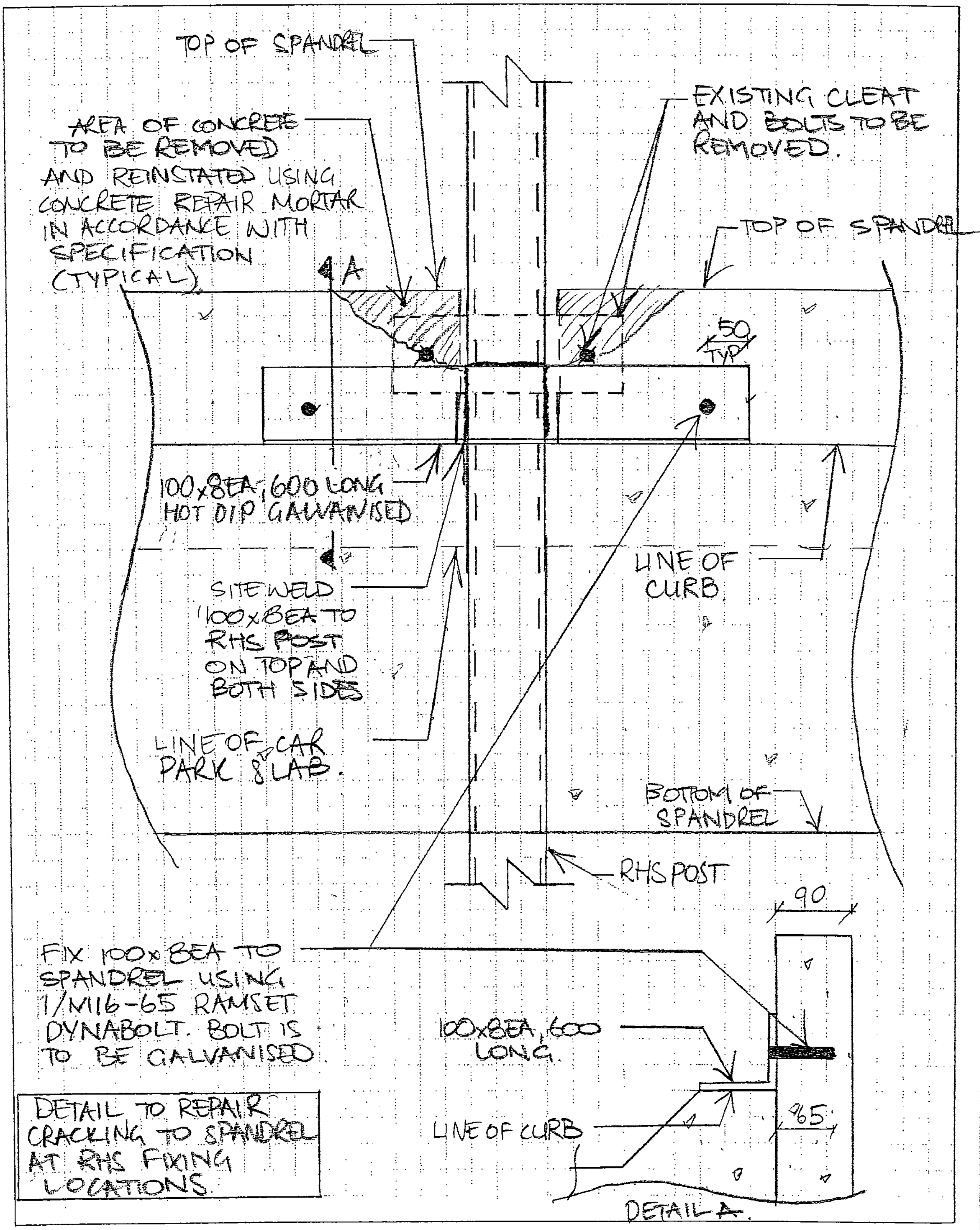
JOB NO. 2281 SHEET NO. S7 REV. D

APPENDIX D

HolmesConsultingGroup

Project Name: FORSYTH BARR EQ REVIEW
Project No: 105448.01
Calcs By: MS
Date: 22.11.10
Page No: 1
Sketch No: DETAIL D01
Revision: A

CALCS/SKETCHES



DETAIL TO REPAIR CRACKING TO SPANDREL AT RHS FIXING LOCATIONS

WIT.HAR.0003A.51

APPENDIX E

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 and blockwork.
3. Repair of concrete spalling.

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 890:1995	Specification for Building Limes	(BS EN)
NZSEE	Assessment & Improvement of the Structural Performance of Buildings in Earthquakes.	(NZSEE)
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. Photographic record of damage observed
2. Recording of repairs completed
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.

Masonry shall be erected only under the direction of a Registered Mason specialising in the laying of masonry units. Before work commences on site, the Contractor shall advise the Engineer, in writing, the name of the Registered Mason who will be responsible for the masonry construction.

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.

1.4.4 Testing

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

Measurements of materials used shall be recorded daily.

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

1.5 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).

1.6 MATERIALS AND WORKMANSHIP

1.6.1 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 efflouresence.

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

1.6.2 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.

1.6.3 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.

1.7 DAMAGE SURVEYS

We have undertaken an initial assessment that has identified general forms of damage and repairs required. 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 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.

1.7.1 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 the following elements:-

1. Perimeter beams
2. Exterior columns
3. Core walls and spandrels
4. Floor topping
5. Stairs

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

1.7.2 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 on the following elements:-

1. Perimeter beams
2. Exterior columns
3. Stairs
4. Seismic joints

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

1.7.3 Verticality Survey

The Contractor shall undertake a verticality survey of the building. The verticality survey shall ascertain whether there is any residual displacement or twist of the building. The Contractor shall submit their proposed methodology to the Engineer for approval prior to undertaking the survey.

1.8 REPAIR OF CRACKS IN REINFORCED CONCRETE AND BLOCKWORK

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.

1.8.1 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.

1.8.2 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.

1.8.3 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.

1.9 REPAIR OF CONCRETE SPALLING

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

1.9.1 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 Primer as required.

Build up the required concrete profile using a high strength repair mortar, such as Sika Monotop Structural Mortar, and finish 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 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 Primer as required.

Build up the required concrete profile using a high build repair mortar, such as Sika Monotop High Build Mortar, and finish 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 Deep 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.

Box and pour to the required concrete profile using a flowable repair mortar, such as Sika Monotop Microconcrete, and finish 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 COORDINATION

The Contractor shall coordinate all associated trades so as to ensure the correct finished relationship, both as to dimensions, details, and finishes, between concrete repair work and all other trades, in particular finishing trades who will be working in the same areas.

1.11 SUBMITTALS

The Contractor shall supply the following documentation for review, at least 10 days prior to installation of the system:

A complete list of proposed materials for the system, including the following areas and clearly identifying any proposed variances from this specification:

1. Repair product
2. Primer / filler
3. Fire resistant coating
4. Protective coating

The individual component materials proposed for the system must be confirmed by the manufacturers to be mutually compatible.

The manufacturer must be able to demonstrate compliance with the Materials section of this specification above.

The manufacturer must also be able to provide supporting evidence of adequate testing of the performance of the proposed system, to the satisfaction of the Engineer.

A complete methodology shall be provided for the system, addressing the following areas and clearly identifying any proposed variances from this specification:

1. Substrate surface preparation
2. Mixing of epoxy / grout
3. Application method
4. Curing method
5. Testing of samples