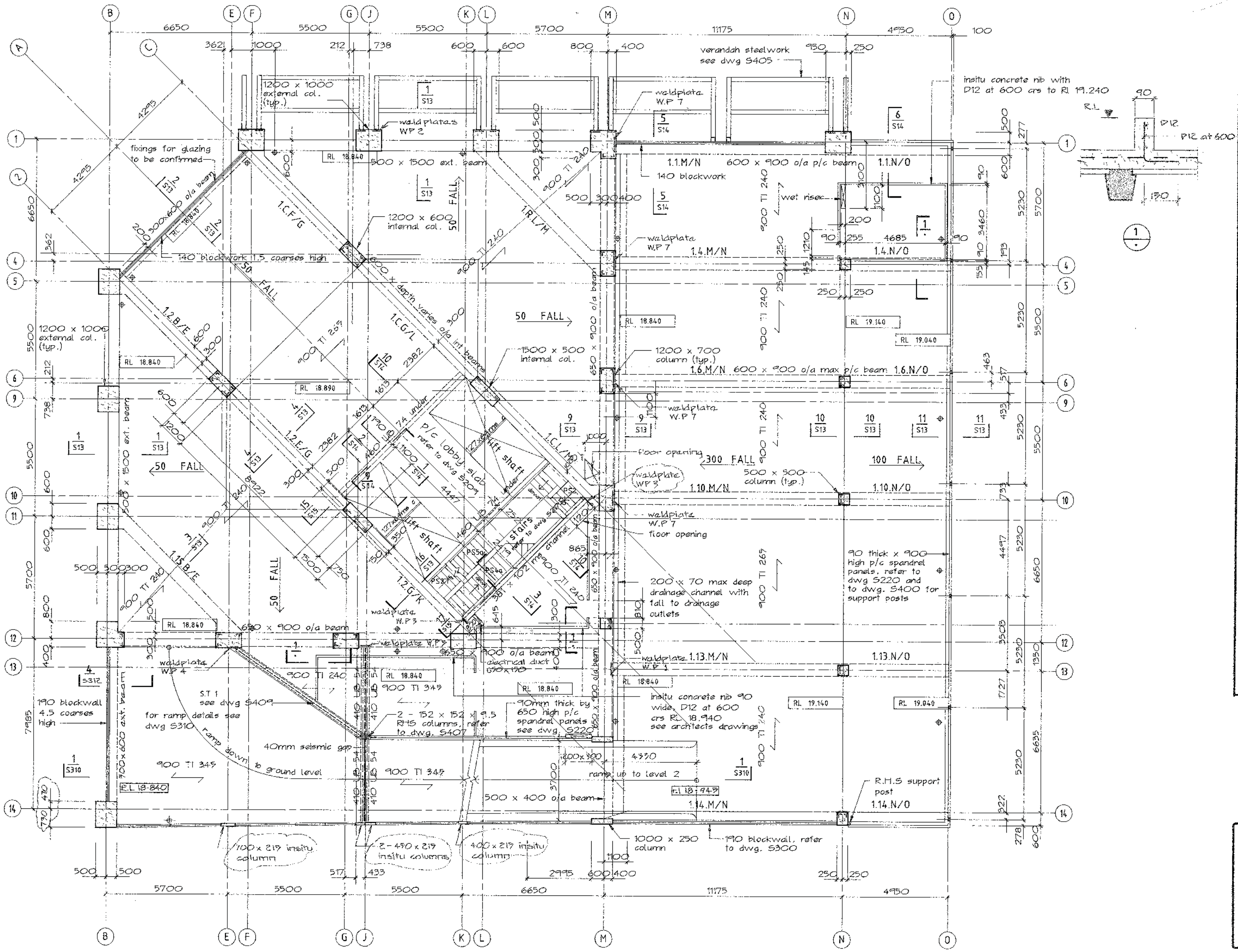
Appendix A1

Building Data

Appendix A1.1

Landmark Tower
Robert Jones House
Selected Drawings



ALL DIMENSIONS TO BE VERIFIED ON SITE BEFORE MAKING ANY SHOP DRAWINGS OR COMMENCING ANY WORK

For package number

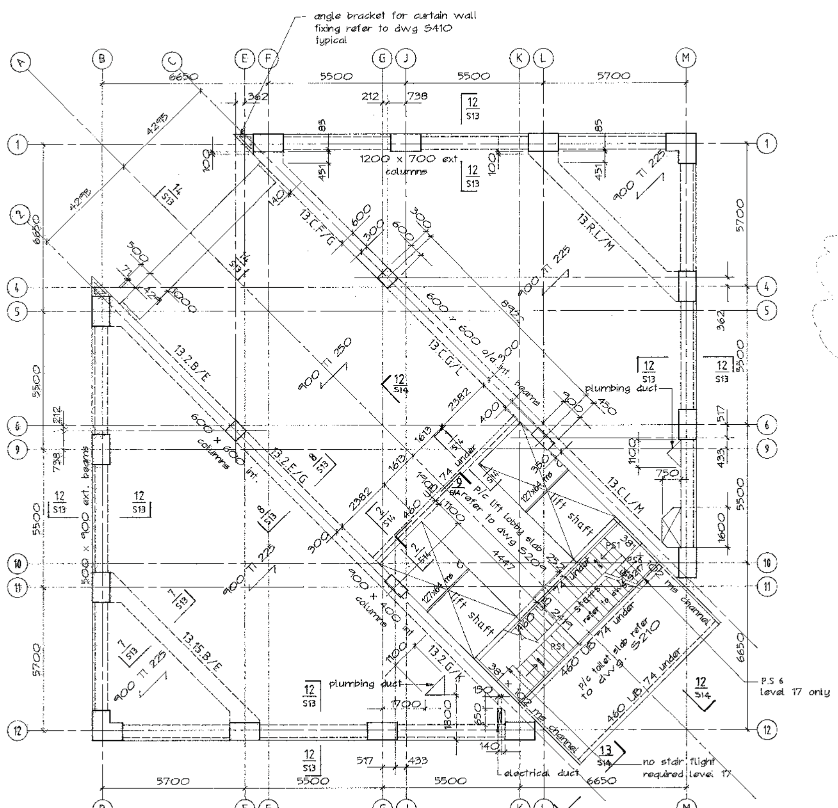
Rev.	Date	Amendment
1	22/2/88	foundation permit
2	4/3/88	permit
A	15/4/88	construction
B	19.5.88	electrical duct relocated
C	30/5/88	sections 9 and 10 added
D	22.7.88	podium hold removed

ROBERT JONES HOUSE
CHRISTCHURCH

DRAWN: SCALE: 1:100
APPROVED:

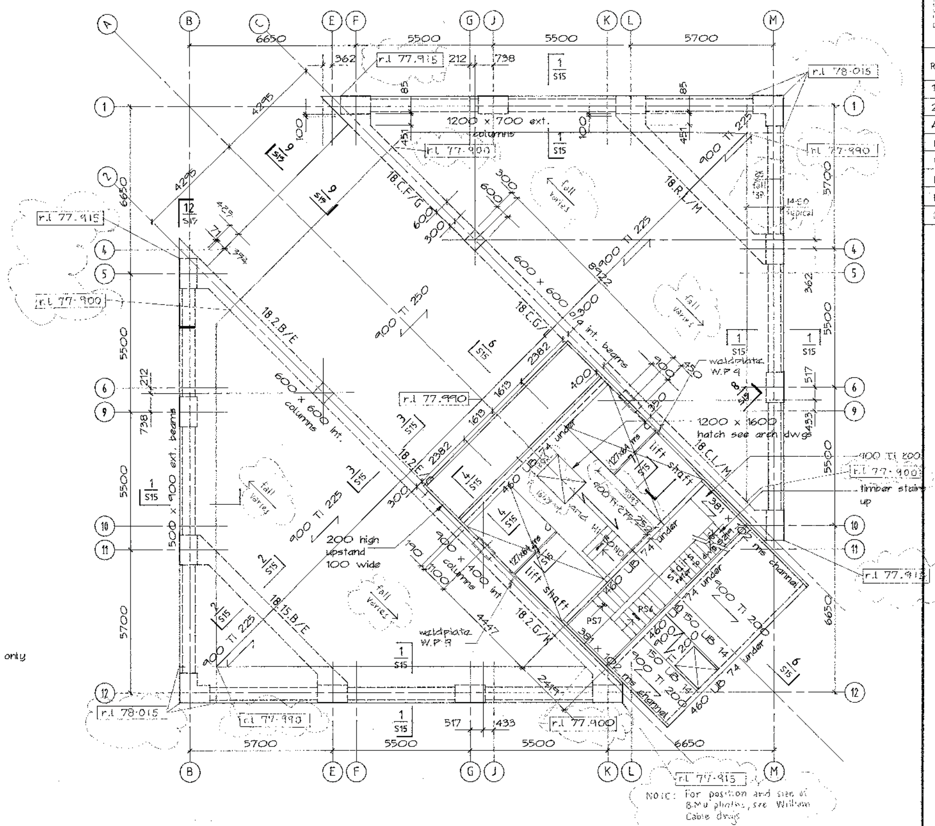
SHEET TITLE:
FLOOR PLANS
LEVEL 1
CARCASS

JOB NO: 2281 SHEET NO: S5 REV: D

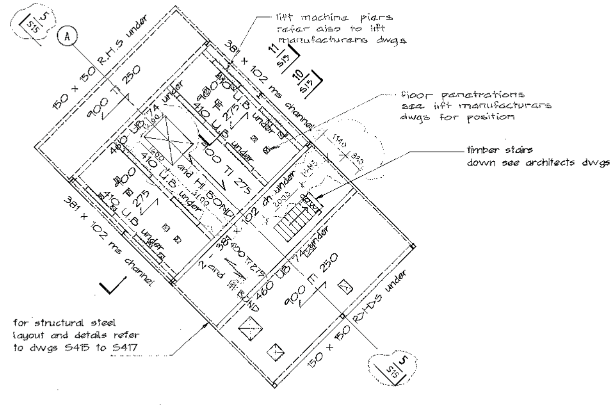


NOTE: p/c beam call up shown refers to level 13 only. beams on levels 14 - 17 prefixed with relevant floor levels

plan levels 13 - 17



plan level 18



plan level 19

ALL DIMENSIONS TO BE VERIFIED ON SITE BEFORE MAKING ANY SHOP DRAWINGS OR COMMENCING ANY WORK

For package number

4	6	7	8	9	10	11	12	13	14	15	17	31	45		

Rev.	Date	Ammendment
1	22/2/88	foundation permit
2	4/3/88	permit
A	9/5/88	construction
B	19/7/88	plumbing duct dimensions
C	3/7/88	sections 9, 12, 13, 14 added
D	1/8/88	level 18 and 19 floor
E	16.8.88	section numbers 1 & 18 changed
F	14/10/88	level 18 r l & falls, level 19 shown

ROBERT JONES HOUSE

CHRISTCHURCH

DRAWN: SCALE: 1:100

APPROVED:

SHEET TITLE: FLOOR PLANS LEVELS 13 TO 19 CARCASS

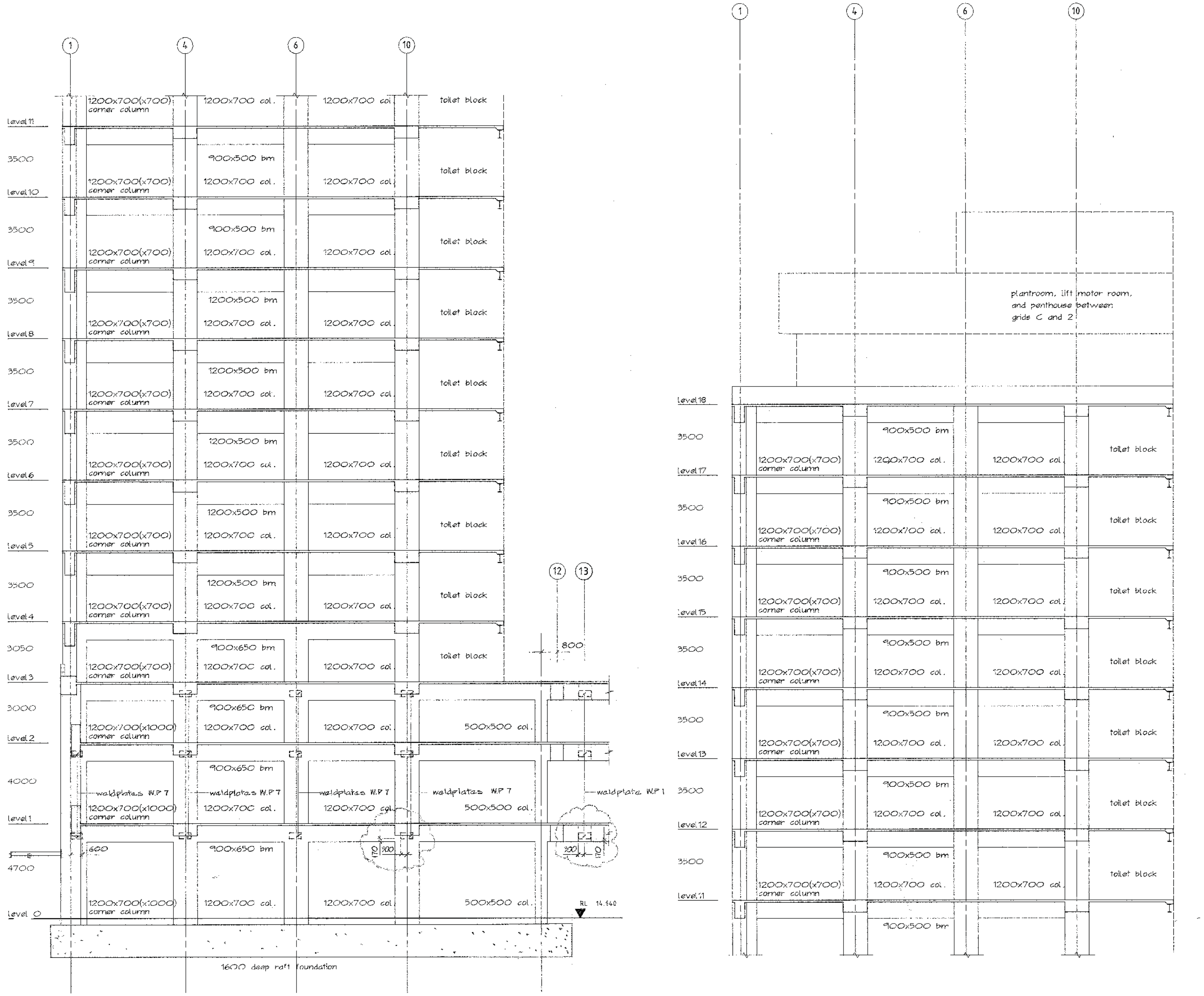
JOB NO:	SHEET NO:	REV:
2281	S10	F

ALL DIMENSIONS TO BE VERIFIED ON SITE BEFORE MAKING ANY SHOP DRAWINGS OR COMMENCING ANY WORK

For package number

4 6 7 8 9 10 11 12 13 14 15 17 31 45

Rev.	Date	Ammendment
1	22/2/88	foundation permit
2	4/3/88	Permit
A	16/5/88	Construction
B	18.5.88	Weldplate locations added
C	26.5.88	weldplate locations altered



FRAME ON GRID M

ROBERT JONES HOUSE

CHRISTCHURCH

DRAWN: SCALE: 1:100

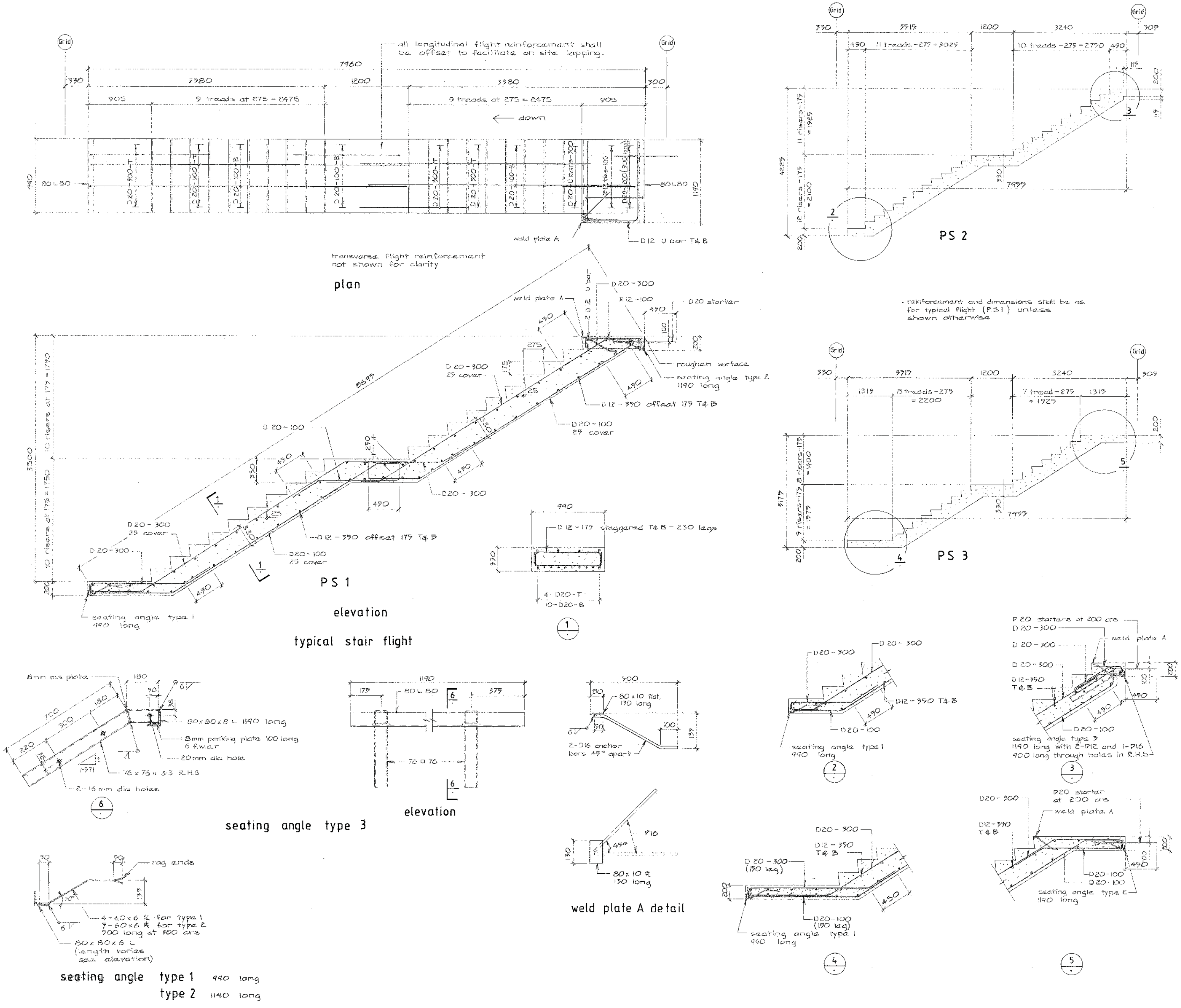
APPROVED:

SHEET TITLE:

FRAME ON GRID M
LEVELS 0 to 20
CARCASS

JOB NO: 2281 SHEET NO: S33 REV: C

DATE REC: 28 SEP 2011 FINAL BUI.COL764.0003A.6



ALL DIMENSIONS TO BE VERIFIED ON SITE BEFORE MAKING ANY SHOP DRAWINGS OR COMMENCING ANY WORK

For package number

Rev	Date	Amendment
1	4-3-88	Permit
A	7-7-88	Construction

ROBERT JONES HOUSE

CHRISTCHURCH

DRAWN: SCALE: 1:10 1:25 1:50

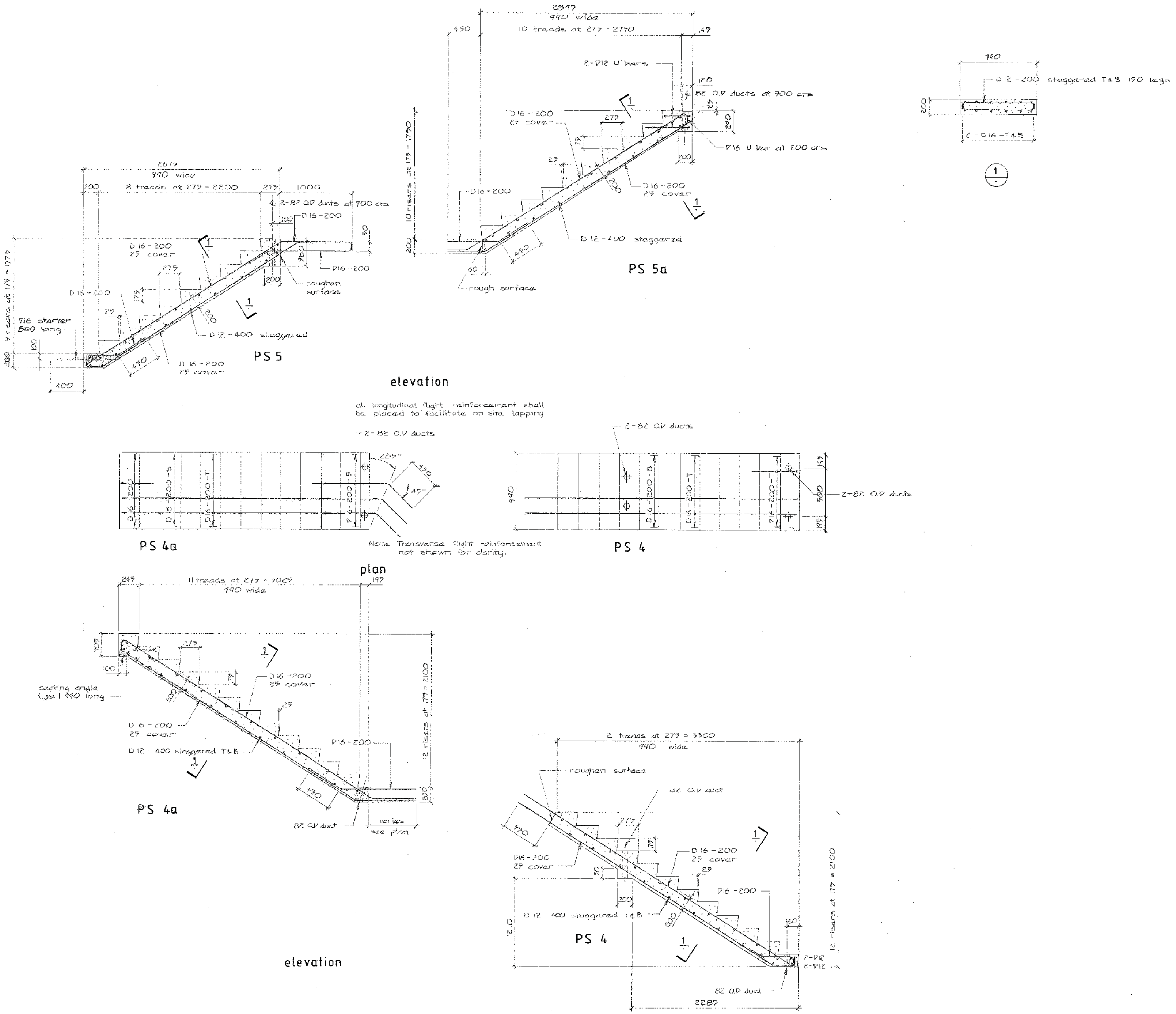
APPROVED:

SHEET TITLE:

PRECAST STAIRS PS 1 TO PS 3

JOB NO: 2281 SHEET NO: S 215 REV: A

DATE REC: 28 SEP 2011 FINAL BUI.COL764.0003A.7



ALL DIMENSIONS TO BE VERIFIED ON SITE BEFORE MAKING ANY SHOP DRAWINGS OR COMMENCING ANY WORK

For package number

4	6	7	8	9	10	11	12	13	14	15	17	18	19

Rev	Date	Amendment
1	4-3-88	Permit
A	2-9-88	Construction
B	6-5-88	PS 5a top tread removed

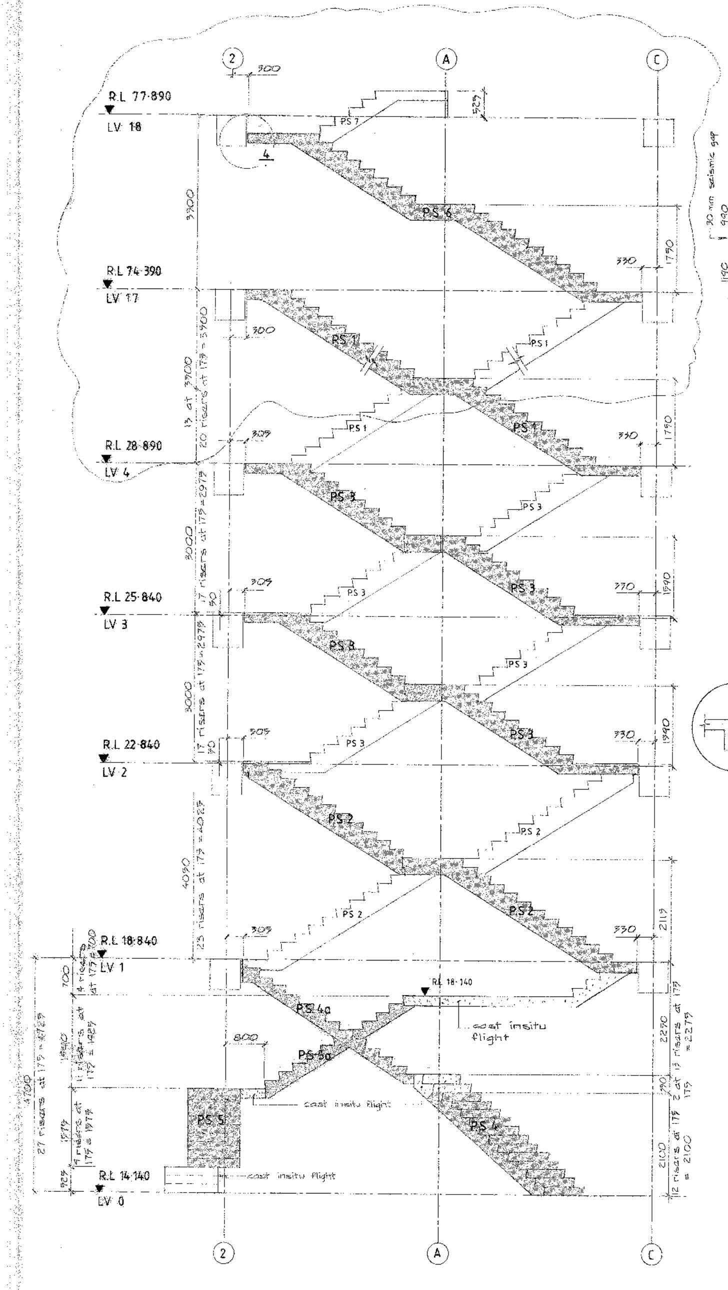
ROBERT JONES HOUSE

CHRISTCHURCH

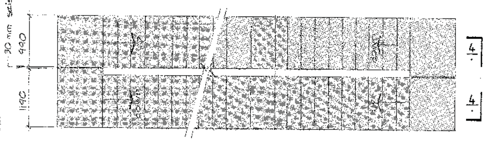
DRAWN: SCALE: 1:25
APPROVED:

SHEET TITLE:
PRECAST STAIRS
PS 4 TO PS 5

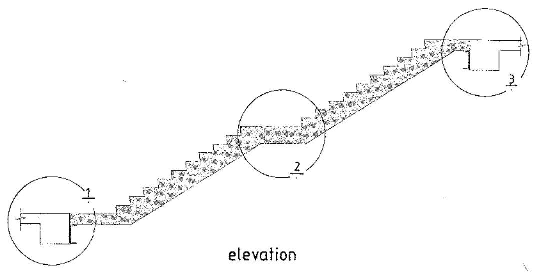
JOB NO: 2281 SHEET NO: S 216 REV: B



stair elevation

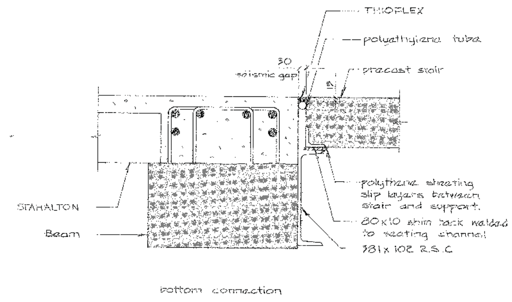


plan

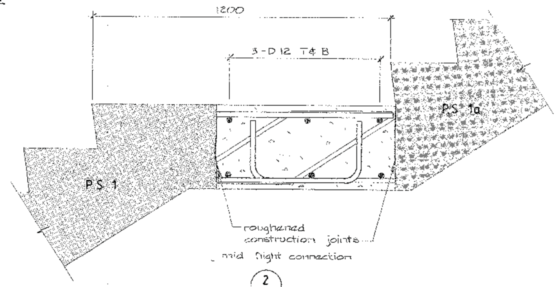


elevation

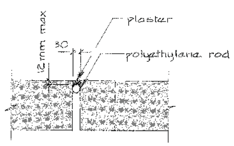
(typical stair flight connection)

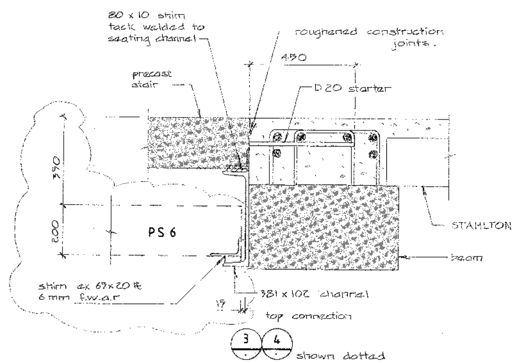


sliding detail
required at bottom of each flight



alternative midspan insitu joint
DELETED





fixed detail
required at top of each flight

ALL DIMENSIONS TO BE VERIFIED ON SITE BEFORE MAKING ANY SHOP DRAWINGS OR COMMENCING ANY WORK

For package number: 4 6 7 8 9 10 11 12 13 14 15 17 18 19

Rev	Date	Amendment
1	4-3-88	Permit
A	7-7-88	Construction
B	6-5-88	P.S 5 & P.S 5a lowered

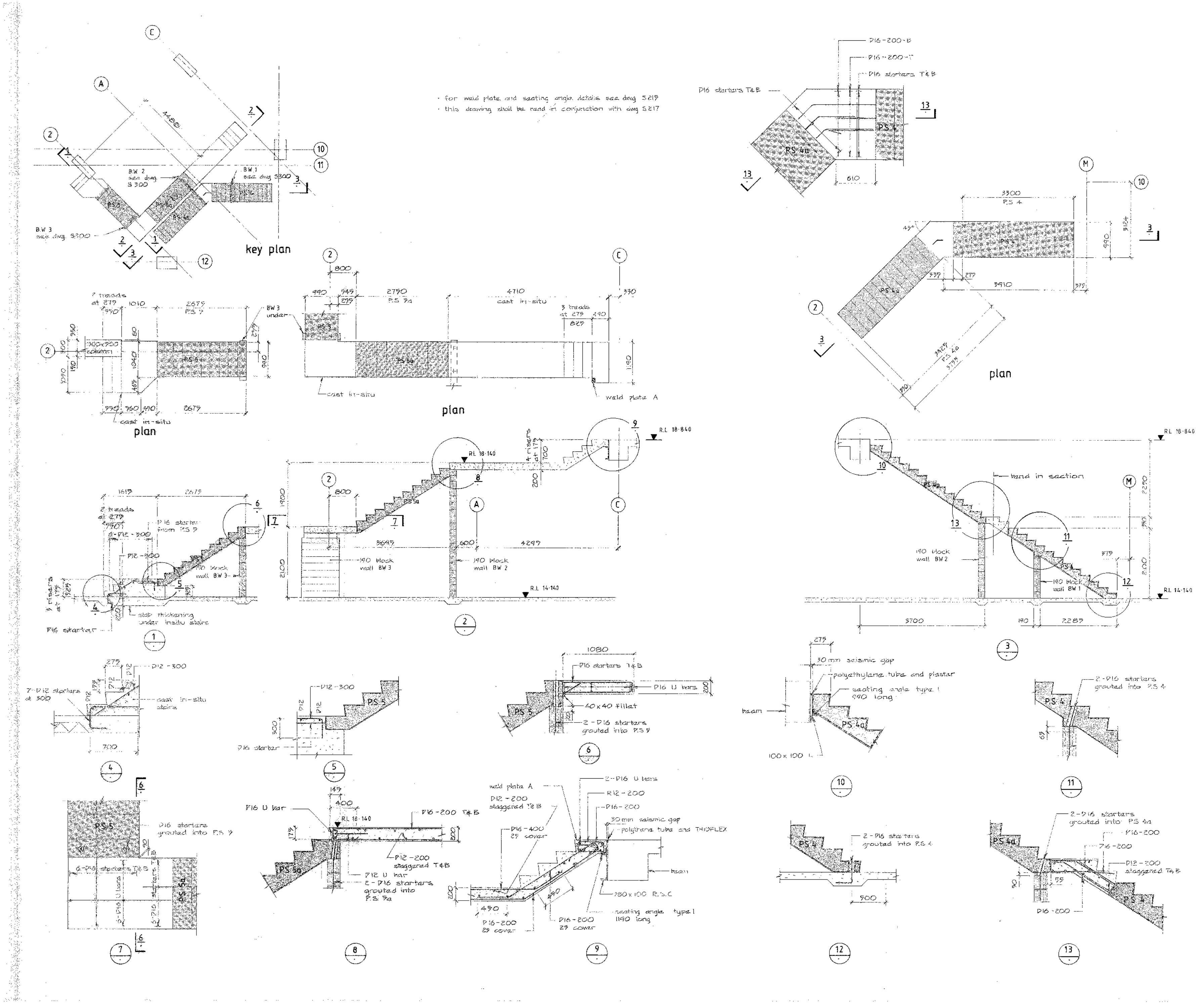
ROBERT JONES HOUSE

CHRISTCHURCH

DRAWN: SCALE 1:50 1:10
APPROVED:

SHEET TITLE: PRECAST STAIR CONNECTIONS AND LAYOUT

JOB NO: 2281 SHEET NO: S 217 REV: B



ALL DIMENSIONS TO BE VERIFIED ON SITE BEFORE MAKING ANY SHOP DRAWINGS OR COMMENCING ANY WORK

For package number

4	6	7	8	9	10	11	12	13	14	15	17	18	19

Rev	Date	Amendment
1	4-3-88	Permit
A	3-5-88	Construction
B	6-5-88	Location of PS 5, PS 5a

ROBERT JONES HOUSE

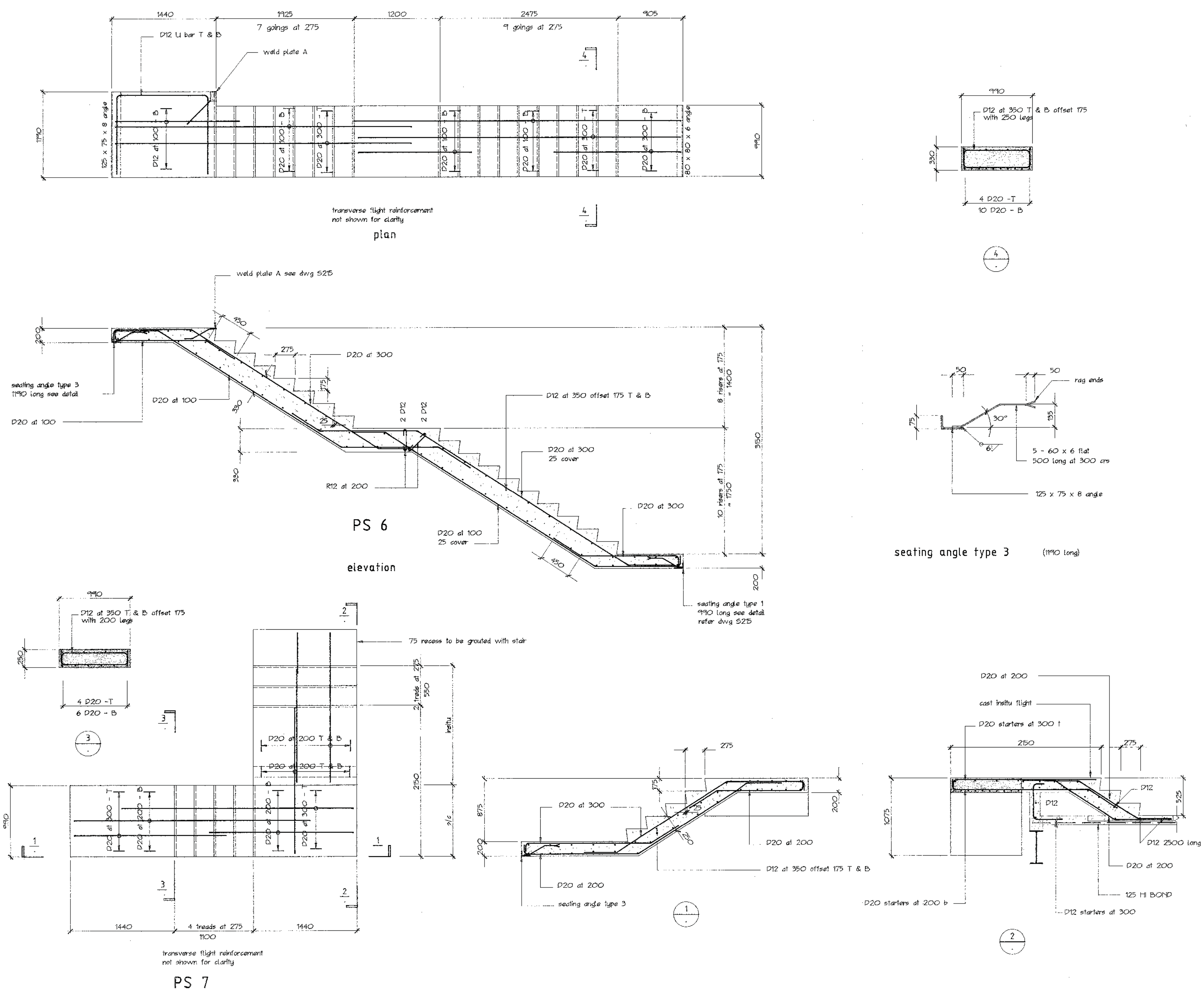
CHRISTCHURCH

DRAWN:
SCALE: 1:100 1:50 1:25
APPROVED:

SHEET TITLE:
PRECAST STAIR CONNECTIONS AND IN-SITU STAIR DETAILS

JOB NO: 2281
SHEET NO: S218
REV: B

DATE REC: 28 SEP 2011 FINAL BUI.COL764.0003A.10



ALL DIMENSIONS TO BE VERIFIED ON SITE BEFORE MAKING ANY SHOP DRAWINGS OR COMMENCING ANY WORK

For package number

Rev.	Date	Amendment
A		Construction

ROBERT JONES HOUSE CHRISTCHURCH

DRAWN: SCALE: 1:25

APPROVED:

SHEET TITLE: PRECAST STAIRS PS 6 AND PS 7

JOB NO: 2281 SHEET NO: S219 REV: A

Appendix A2

Reports on Structure

Appendix A2.1

Level 2 Rapid Assessment 5th September 2010

Christchurch Eq RAPID Assessment Form - LEVEL 2

Inspector Initials:
Territorial Authority: Christchurch City
Date: 5/9/10
Time: 12:30
Final Posting (e.g. UNSAFE): ~~Restricted Use (Y2)~~ Now G2, refer attached

ORIGINAL

Building Name: Forsyth Barr
Short Name:
Address: Armagh Street
GPS Co-ordinates: S° E°
Contact Name:
Contact Phone:
Storeys at and above ground level: 17
Below ground level: 1
Total gross floor area (m²):
Year built:
No of residential Units:
Photo Taken: (Yes) No

Type of Construction

- [ ] Timber frame
- [ ] Steel frame
- [ ] Tilt-up concrete
- [x] Concrete frame
- [ ] RC frame with masonry infill
- [ ] Concrete shear wall
- [ ] Unreinforced masonry
- [ ] Reinforced masonry
- [ ] Confined masonry
- [ ] Other:

Primary Occupancy

- [ ] Dwelling
- [ ] Other residential
- [ ] Public assembly
- [ ] School
- [ ] Religious
- [x] Commercial/ Offices
- [ ] Industrial
- [ ] Government
- [ ] Heritage Listed
- [ ] Other

Investigate the building for the conditions listed on page 1 and 2, and check the appropriate column. A sketch may be added on page 3

Overall Hazards / Damage	Minor/None	Moderate	Severe
Collapse, partial collapse, off foundation	☑	☐	☐
Building or storey leaning	☑	☐	☐
Wall or other structural damage	☑	☐	☐
Overhead falling hazard	☐	☑	☐
Ground movement, settlement, slips	☐	☑	☐
Neighbouring building hazard	☑	☐	☐
Electrical, gas, sewerage, water, hazmats	☑	☐	☐

Comments: Stairs have generally settled and may be unstable.

Record any existing placard on this building:
Existing Placard Type (e.g. UNSAFE): Inspected

Choose a new posting based on the new evaluation and team judgement. Severe conditions affecting the whole building are grounds for an UNSAFE posting. Localised Severe and overall Moderate conditions may require a RESTRICTED USE. Place INSPECTED placard at main entrance. Post all other placards at every significant entrance. Transfer the chosen posting to the top of this page.

INSPECTED GREEN: G1 | (G2)
RESTRICTED USE YELLOW: Y1 | Y2
UNSAFE RED: R1 | R2 | R3

Record any restriction on use or entry:

Further Action Recommended:

Tick the boxes below only if further actions are recommended

- [ ] Barricades are needed (state location):
- [ ] Detailed engineering evaluation recommended
  - [x] Structural
  - [ ] Geotechnical
  - [ ] Other:
- [x] Other recommendations: Stair landing bulkheads need to be removed to allow investigation

Estimated Overall Building Damage (Exclude Contents)

None	☐		
0-1 %	☐	31-60 %	☐
2-10 %	☑	61-99 %	☐
11-30 %	☐	100 %	☐

Inspection ID: __________ (Office Use Only)

Structural Hazards/ Damage	Minor/None	Moderate	Severe	Comments
Foundations	☑	☐	☐	
Roofs, floors (vertical load)	☑	☐	☐	
Columns, pilasters, corbels	☑	☐	☐	
Diaphragms, horizontal bracing	☑	☐	☐	
Pre-cast connections	☐	☑	☐	- Apparent Damage to base of sta[ir]
Beam	☒	☐	☑	- steel Beam supporting car ramp sheared through welded connection ~ 30mm settlement
Non-structural Hazards / Damage				
Parapets, ornamentation	☑	☐	☐	
Cladding, glazing	☑	☐	☐	
Ceilings, light fixtures	☑	☐	☐	
Interior walls, partitions	☑	☐	☐	Minor damage to roof tiles
Elevators	☐ N/A	☐	☐	Not Inspected
Stairs/ Exits	☐	☐	☑	stair settlement ~ 40mm under supports at level 7.
Utilities (eg. gas, electricity, water)	☑	☐	☐	
Other	☐	☐	☐	
Geotechnical Hazards / Damage				
Slope failure, debris	☑	☐	☐	
Ground movement, fissures	☑	☐	☐	
Soil bulging, liquefaction	☑	☐	☐	

General Comment • Stair supports need to be investigated prior to upgrading to 'green'
• Beam under car ramp needs propping

Usability Category

Damage Intensity	Posting	Usability Category	Remarks
Light damage *Low risk*	Inspected (Green)	G1. Occupiable, no immediate further investigation required	
		G2. Occupiable, repairs required	
Medium damage *Medium risk*	Restricted Use (Yellow)	Y1. Short term entry	
		Y2. No entry to parts until repaired or demolished (circled)	
Heavy damage *High risk*	Unsafe (Red)	R1. Significant damage: repairs, strengthening possible	
		R2. Severe damage: demolition likely	
		R3. At risk from adjacent premises or from ground failure	

 Inspection ID: ________ *(Office Use Only)*

Sketch (optional)
Provide a sketch of the entire building or damage points. Indicate damage points.

Recommendations for Repair and Reconstruction or Demolition (Optional)

 Inspection ID: __________ (Office Use Only)

Forsyth Barr House – Level 2 Seismic Assessment

By:

Date: 6 September 2010

Subject: Forsyth Barr House Level 2 Seismic Assessment

Our Ref: 5320000

Level 2 Seismic Assessment

Scissor stair flights

Following the first level 2 assessment completed on 5 September 2010, further access was required to assess the damage to the scissor stair around the landing area as noted in the assessment. A contractor was brought in to assist with removal of the stair bulkhead on the level 7 landing, which we believed to be the most damaged stair. The following points were observed:

- Beam and connections supporting the base of the stair (~380-PFC) appeared to be in good condition.
- Fire proofing material was intact.
- Flexural cracking in the base of the lower knee of the scissor flight has resulted in residual deformation of the stair, with the stair settled by ~40mm at that location.

The majority of the stair flights had similar damage, although it is believed that level 7 was the most damaged and therefore representative of the remaining stairs. Although the deformations in the stairs are significant, we believe that the stairs still contain sufficient capacity for normal use.

Car Ramp

As noted in the Level 2 assessment, a failed weld in the beam supporting the car ramp on Level 2 has failed resulting in ~40mm of settlement. Temporary Propping has been provided ~~will be required from the beam down to the~~ down to the level 2 slab, then also down from level 2 to level 1 slab. This propping will allow pedestrian but not vehicle access to the supported ramp.

Recommendations

- Cleaning of loose debris from the seismic separations at the base of all stairs.
- Propping of Car ramp beam to level 2 slab below, and a further prop from that slab to the level1 slab below that.
- The scissor stairs are available for normal use after health and safety issues have been addressed.
- The loose debris should be cleared from the seismic separation gaps at the end of each stairflight to allow movement as originally intended.
- The current temporary propping to the carpark ramp beam at level 2 will allow pedestrian access but not vehicle access to the supported ramp.

File Note

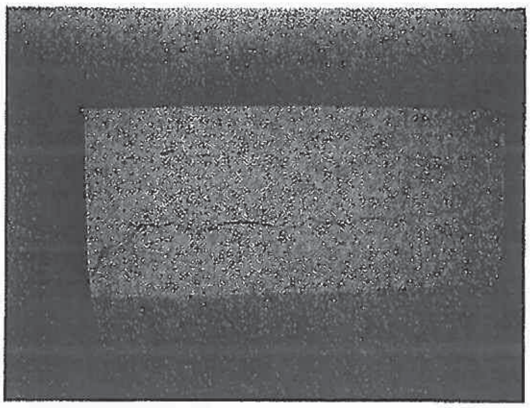


Photo 1. Flexural cracking in base of stair


Photo 2. Spalling of concrete at stair knee.

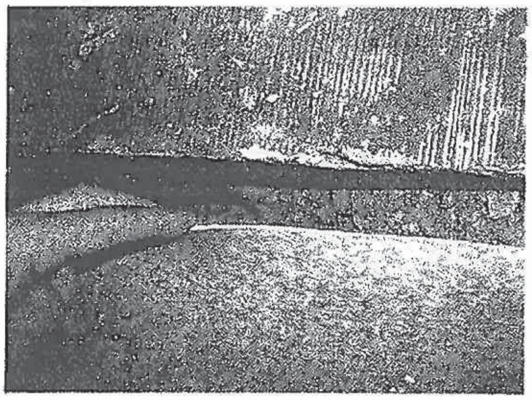


Photo 3. Debris in Seismic separation

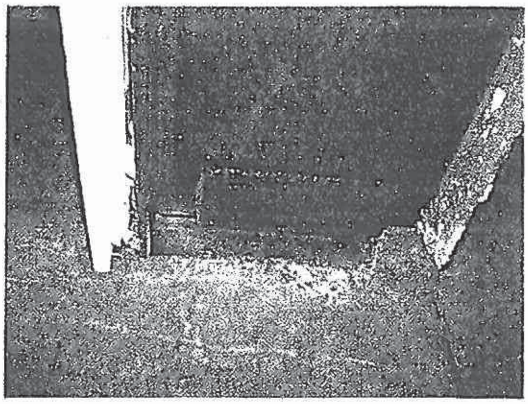


Photo 4. Broken Weld

RE-INSPECTED 15/9 TO INVESTIGATE LEVEL 7 INFILL FLOOR SAG.

Christchurch Eq RAPID Assessment Form - LEVEL 2

Inspector Initials		Date	5/9/10	Final Posting (e.g. UNSAFE)	Restricted Use (Y2)
Territorial Authority	Christchurch City	Time	12:30		

Now G2, refer attached

OFFICIAL

Building Name	Forsyth Barr
Short Name	
Address	Armagh Street
GPS Co-ordinates	S° E°
Contact Name	
Contact Phone	379 6280
Storeys at and above ground level	17
Below ground level	1
Total gross floor area (m²)	
Year built	
No of residential Units	
Photo Taken	(Yes) No

Type of Construction

- ☐ Timber frame
- ☐ Steel frame
- ☐ Tilt-up concrete
- ☑ Concrete frame
- ☐ RC frame with masonry infill
- ☐ Concrete shear wall
- ☐ Unreinforced masonry
- ☐ Reinforced masonry
- ☐ Confined masonry
- ☐ Other:

Primary Occupancy

- ☐ Dwelling
- ☐ Other residential
- ☐ Public assembly
- ☐ School
- ☐ Religious
- ☑ Commercial/ Offices
- ☐ Industrial
- ☐ Government
- ☐ Heritage Listed
- ☐ Other

Investigate the building for the conditions listed on page 1 and 2, and check the appropriate column. A sketch may be added on page 3

Overall Hazards / Damage	Minor/None	Moderate	Severe
Collapse, partial collapse, off foundation	☑	☐	☐
Building or storey leaning	☑	☐	☐
Wall or other structural damage	☑	☐	☐
Overhead falling hazard	☐	☑	☐
Ground movement, settlement, slips	☐	☑	☐
Neighbouring building hazard	☑	☐	☐
Electrical, gas, sewerage, water, hazmats	☑	☐	☐

Comments: Stairs have generally settled and may be unstable.

Record any existing placard on this building:

Existing Placard Type (e.g. UNSAFE): Inspected

Choose a new posting based on the new evaluation and team judgement. Severe conditions affecting the whole building are grounds for an UNSAFE posting. Localised Severe and overall Moderate conditions may require a RESTRICTED USE. Place INSPECTED placard at main entrance. Post all other placards at every significant entrance. Transfer the chosen posting to the top of this page.

INSPECTED *GREEN* G1 (G2) RESTRICTED USE *YELLOW* Y1 Y2 UNSAFE *RED* R1 R2 R3

Record any restriction on use or entry:

Further Action Recommended:

Tick the boxes below only if further actions are recommended

- ☐ Barricades are needed (state location):
- ☐ Detailed engineering evaluation recommended
  - ☑ Structural ☐ Geotechnical ☐ Other:
- ☑ Other recommendations: Stair landing bulkheads need to be removed to allow investigation

Estimated Overall Building Damage (Exclude Contents)

None	☐		
0-1 %	☐	31-60 %	☐
2-10 %	☑	61-99 %	☐
11-30 %	☐	100 %	☐

Inspection ID: ________ *(Office Use Only)*

Structural Hazards/ Damage	Minor/None	Moderate	Severe	Comments
Foundations	☑	☐	☐	
Roofs, floors (vertical load)	☑	☐	☐	*LVL 7 INFILL FLOOR HAS SAGGED ≈ 10mm IN ONE PLACE (NON-STRUCTURAL)*
Columns, pilasters, corbels	☑	☐	☐	
Diaphragms, horizontal bracing	☑	☐	☐	
Pre-cast connections	☐	☑	☐	*- Apparent Damage to base of sta*
Beam	☒ (crossed out)	☐	☑	*- Steel Beam supporting car ramp sheared through welded connection ~ 30mm settlement*
Non-structural Hazards / Damage				
Parapets, ornamentation	☑	☐	☐	
Cladding, glazing	☑	☐	☐	
Ceilings, light fixtures	☑	☐	☐	*Minor damage to roof tiles*
Interior walls, partitions	☑	☐	☐	
Elevators	☐ *N/A*	☐	☐	*Not Inspected*
Stairs/ Exits	☐	☐	☑	*stair settlement ~40mm under supports at level 7.*
Utilities (eg. gas, electricity, water)	☑	☐	☐	
Other	☐	☐	☐	
Geotechnical Hazards / Damage				
...pe failure, debris	☑	☐	☐	
Ground movement, fissures	☑	☐	☐	
Soil bulging, liquefaction	☑	☐	☐	

General Comment
• Stair supports need to be investigated prior to upgrading to 'green'
• Beam under car ramp needs propping

Usability Category

Damage Intensity	Posting	Usability Category	Remarks
Light damage *Low risk*	Inspected (Green)	G1. Occupiable, no immediate further investigation required	
		G2. Occupiable, repairs required	
Medium damage *Medium risk*	Restricted Use (Yellow)	Y1. Short term entry	
		Y2. No entry to parts until repaired or demolished (circled)	
Heavy damage *High risk*	Unsafe (Red)	R1. Significant damage: repairs, strengthening possible	
		R2. Severe damage: demolition likely	
		R3. At risk from adjacent premises or from ground failure	

 Inspection ID:__________ (Office Use Only)

Sketch (optional)
Provide a sketch of the entire building or damage points. Indicate damage points.

Recommendations for Repair and Reconstruction or Demolition (Optional)

★ Inspection on Mon 13th Sept of internal stairs between L6 & L7
~~[illegible]~~ Stairs have been previously removed (15 yrs +) and infilled
Tenants noticed slight sag in the floor by door (L7). could not
inspect without removing carpet - recommend further
inspection with carpet removed and through suspended
ceiling on L6.

LEVEL 7 FLOOR SAG ~~[illegible]~~ LIKELY CAUSED BY INFILL (~~[illegible]~~ INTERSPAN ONTO STEEL BEAMS)
~~[illegible]~~ SAGGING DUE TO DEFLECTION OF STEEL SUPPORT BEAMS.
SAG IS MINIMAL - STEEL BEAM CONNECTIONS COULD NOT BE
INSPECTED DUE TO ACCESS LIMITATIONS. SUGGEST AREA OF FLOOR
IS RE-LEVELLED IF DEEMED NECESSARY.

 Inspection ID: __________ (Office Use Only)

Appendix A2.2

Post Earthquake Assessment and Repair Report
29th November 2010

FORSYTH BARR TOWER POST-EARTHQUAKE ASSESSMENT AND REPAIR REPORT

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

CONTENTS

APPENDICES

TABLES

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

has been engaged by 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 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.

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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.

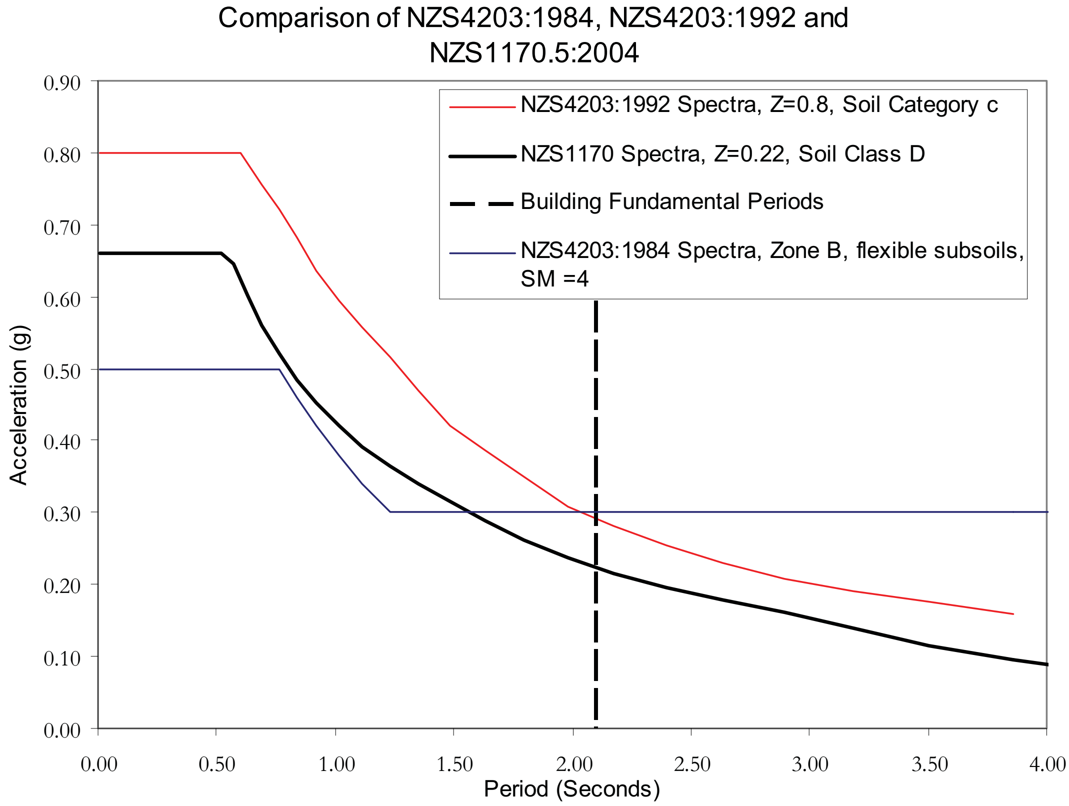


Figure 3-1: Comparison of Design Codes

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.

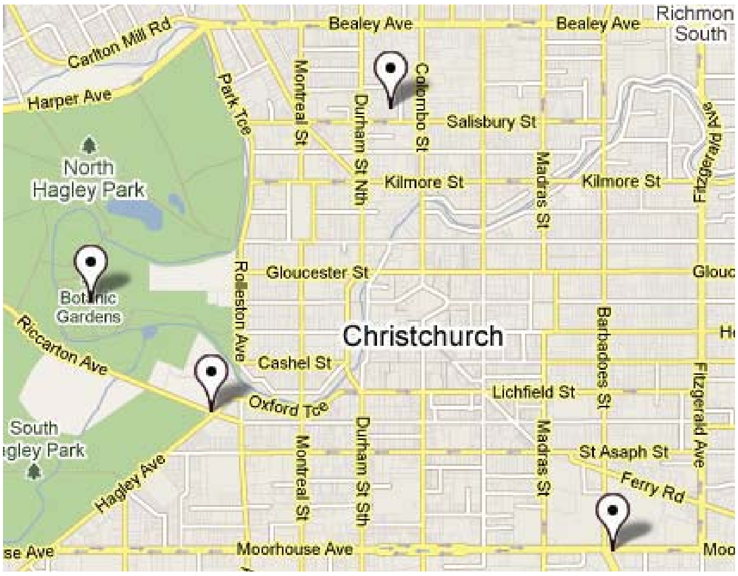


Figure 4-1: Location of Nearby Monitoring Stations

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.

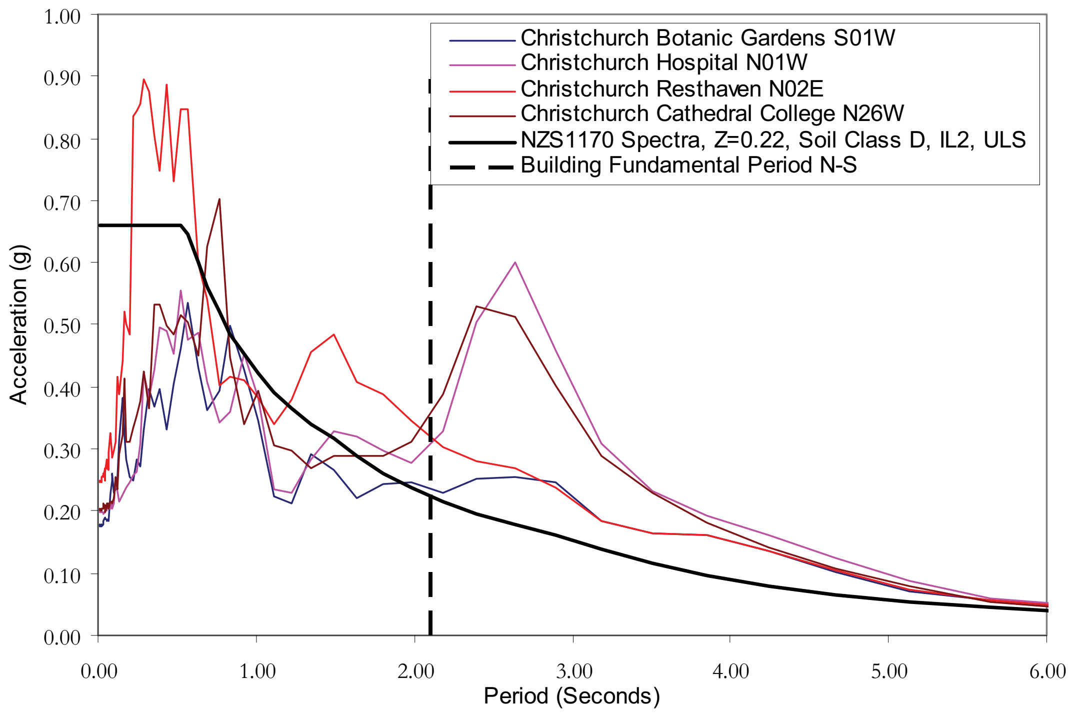


Figure 4-2: 5% Damped Spectra – North-South

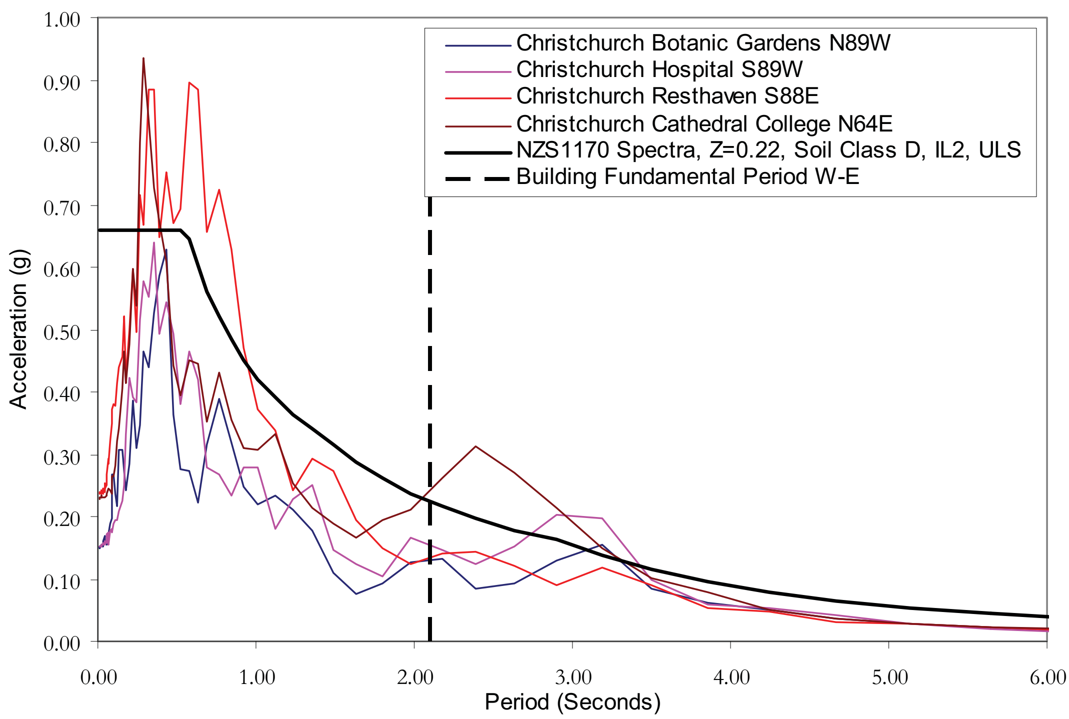


Figure 4-3: 5% Damped Spectra – East-West

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.

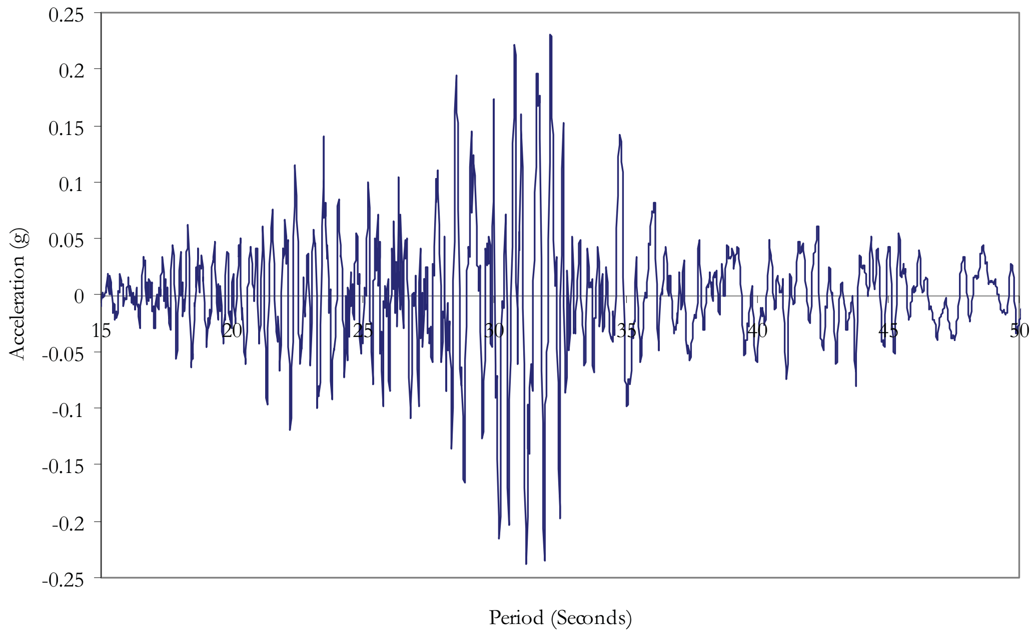


Figure 4-4: Earthquake Record from Christchurch Resthaven Site

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 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*, 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

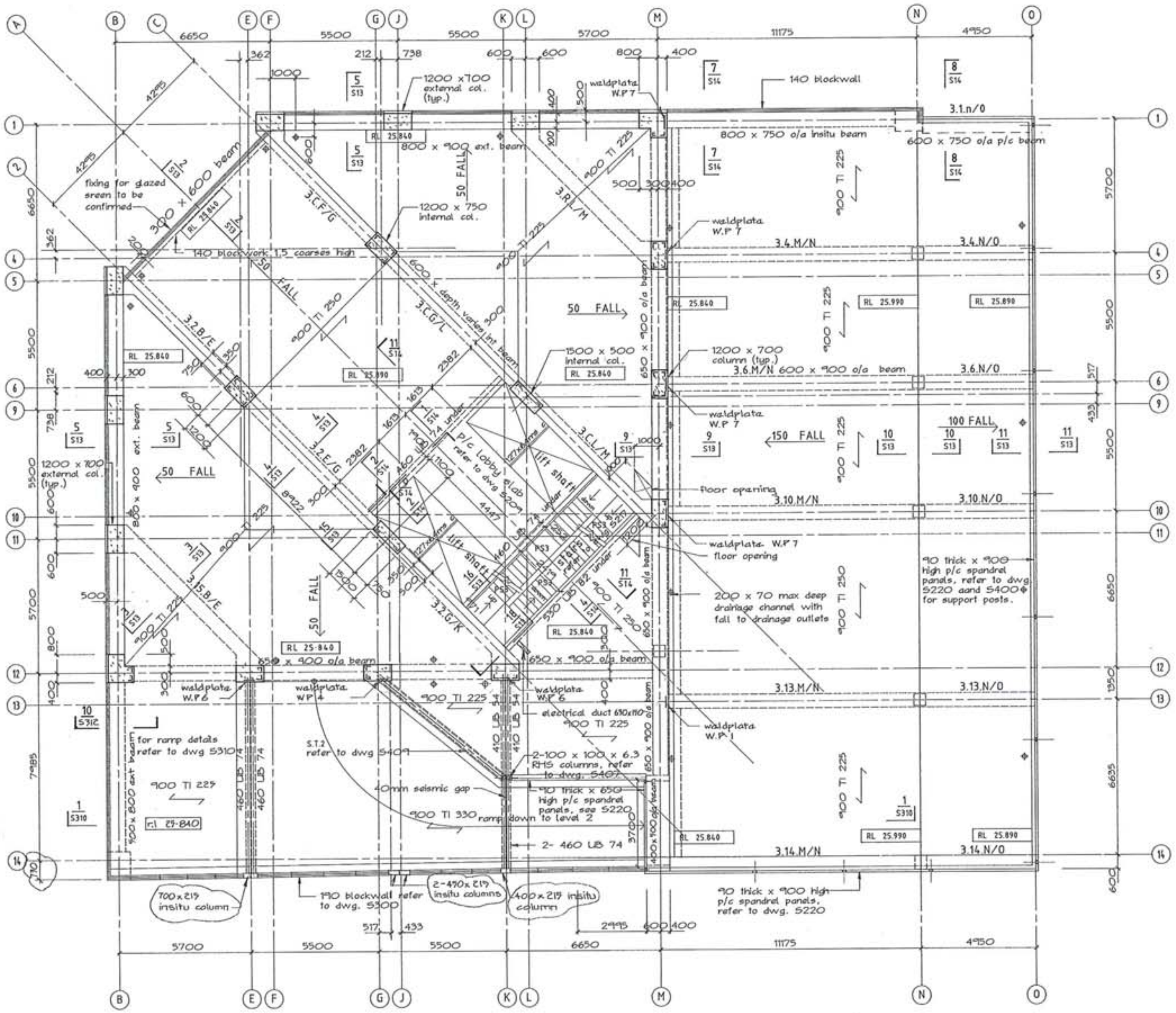
DATE REC: 28 SEP 2011 FINAL BUI.COL764.0003A.63

ALL DIMENSIONS TO BE VERIFIED ON SITE BEFORE MAKING ANY SHOP DRAWINGS OR COMMENCING ANY WORK

For package number

4	6	7	8	9	10	11	12	13	14	15	17	31	45

Rev.	Date	Ammendment
1	22/2/88	foundation permit
2	4/3/88	permit
A	16/3/88	construction
B	19.5.88	electrical duct relocated
C	30.9.88	sections 9 and 11 added
D	22.7.88	podium hold removed



Project Name: FORSYTH BARR DETAILED BQ REVIEW

Project #: 105448.01

Sketch #: C01

Title: LOCATION LABELLING PLAN

Date: 18.11.10 Drawn:

ROBERT JONES HOUSE
CHRISTCHURCH

DRAWN: H.A.O SCALE: 1:100
APPROVED:

SHEET TITLE:
FLOOR PLANS
LEVEL 3
CARCASS

JOB NO:	SHEET NO:	REV:
2281	S7	D

APPENDIX D

Project Name: FORSYTH BARR EQ REVIEW

Project No: 105448.01

Calcs By:

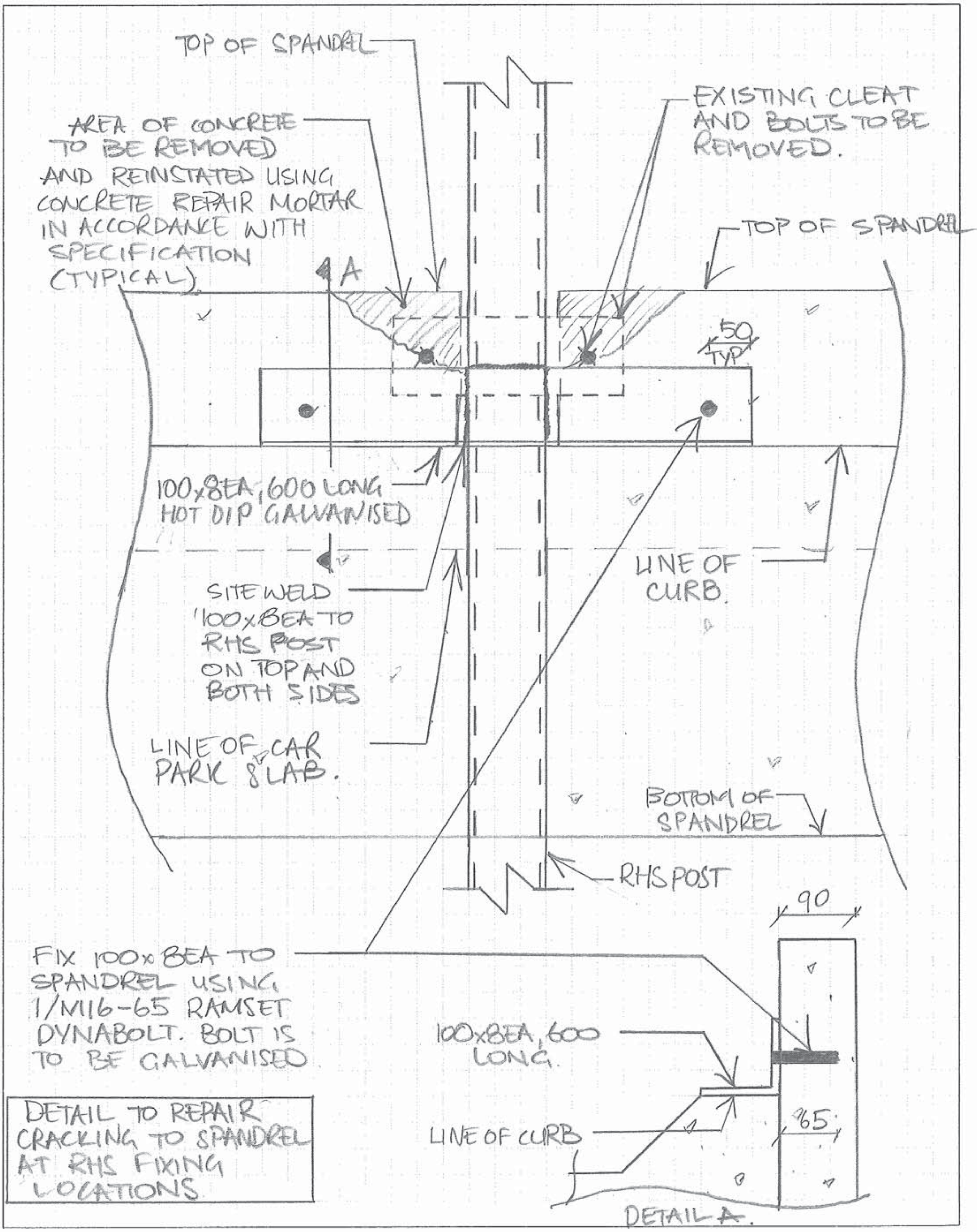
CALCS/SKETCHES

Date: 22.11.10

Page No: 1

Sketch No: DETAIL DO1

Revision: A



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 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

1.6.2 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

Appendix A2.3

Level 2 Rapid Assessment April 2011

Christchurch Eq RAPID Assessment Form - LEVEL 2

Inspector Initials
Territorial Authority: Christchurch City
Date: 30 - 31 MARCH.
Time:
Final Posting (e.g. UNSAFE): Y2 YELLOW

Building Name: FORSYTH BARR.
Short Name:
Address: 764 COLOMBO ST.
GPS Co-ordinates: S° E°
Contact Name:
Contact Phone:
Storeys at and above ground level: 18
Below ground level: —
Total gross floor area (m2): —
Year built: 1988
No of residential Units: 0
Photo Taken: Yes / No (circled: No)

Type of Construction

- ☐ Timber frame
- ☐ Steel frame
- ☐ Tilt-up concrete
- ☑ Concrete frame
- ☐ RC frame with masonry infill
- ☐ Concrete shear wall
- ☐ Unreinforced masonry
- ☐ Reinforced masonry
- ☐ Confined masonry
- ☐ Other:

Primary Occupancy

- ☐ Dwelling
- ☐ Other residential
- ☐ Public assembly
- ☐ School
- ☐ Religious
- ☑ Commercial/ Offices
- ☐ Industrial
- ☐ Government
- ☐ Heritage Listed
- ☐ Other

Investigate the building for the conditions listed on page 1 and 2, and check the appropriate column. A sketch may be added on page 3

Overall Hazards / Damage	Minor/None	Moderate	Severe	Comments
Collapse, partial collapse, off foundation	☑	☐	☐	
Building or storey leaning	☑	☐	☐	
Wall or other structural damage	☑	☐	☐	
Overhead falling hazard	☐	☐	☑	INTERNAL FALL HAZARD FROM REMAINING STAIRS.
Ground movement, settlement, slips	☐	☑	☐	LIQUIFACTION DAMAGE TO GROUND FLOOR SLAB ON GRADE.
Neighbouring building hazard	☑	☐	☐	
~~Electrical, gas, sewerage, water, hazmats~~	☐	☐	☐	

Record any existing placard on this building:

Existing Placard Type (e.g. UNSAFE): RED [L1]

Choose a new posting based on the new evaluation and team judgement. Severe conditions affecting the whole building are grounds for an UNSAFE posting. Localised Severe and overall Moderate conditions may require a RESTRICTED USE. Place INSPECTED placard at main entrance. Post all other placards at every significant entrance. Transfer the chosen posting to the top of this page.

INSPECTED *GREEN* G1 | G2
RESTRICTED USE *YELLOW* Y1 | Y2 (circled: Y2)
UNSAFE *RED* R1 | R2 | R3

Record any restriction on use or entry:

NO ACCESS TO STAIRWELL AREA.

Further Action Recommended:

Tick the boxes below only if further actions are recommended

- ☐ Barricades are needed (state location):
- ☑ Detailed engineering evaluation recommended
  - ☑ Structural
  - ☐ Geotechnical
  - ☐ Other:
- ☐ Other recommendations:

Estimated Overall Building Damage (Exclude Contents)

None	☐		
0-1 %	☐	31-60 %	☐
2-10 %	☑	61-99 %	☐
11-30 %	☐	100 %	☐

Inspection ID: ________ *(Office Use Only)*

Structural Hazards/ Damage	Minor/None	Moderate	Severe	Comments
Foundations	☑	☐	☐	
Roofs, floors (vertical load)	☑	☐	☐	
Columns, pilasters, corbels	☐	☑	☐	CAR PARK GROUND FLOOR
Diaphragms, horizontal bracing	☐	☐	☐	COLUMN FAILURE (NOW
Pre-cast connections	☑	☐	☐	PROPPED)
Beam	☐	☑	☐	
Non-structural Hazards / Damage				
Parapets, ornamentation	☑	☐	☐	GLAZING BROKEN TO ALLOW
Cladding, glazing	☐	☑	☐	USAR ACESS, SINGLE BROKEN
Ceilings, light fixtures	☐	☑	☐	WINDOW - LOW LEVEL WEST
Interior walls, partitions	☑	☐	☐	ELEVATION
~~Elevators~~	☐	☐	☐	
Stairs/ Exits	☐	☐	☑	STAIR COLLAPSE UP TO LEVEL 15
~~Utilities (eg. gas, electricity, water)~~	☐	☐	☐	
Other	☐	☐	☐	
Geotechnical Hazards / Damage				
Slope failure, debris	☑	☐	☐	
Ground movement, fissures	☑	☐	☐	
Soil bulging, liquefaction	☑	☐	☐	

General Comment BUILDING APPEARS TO HAVE PREFORMED WELL APART FROM STAIR COLLAPSE. CRACKS IN BEAMS <1mm INDICATE YEILDING BEGAN DURING EQ. FAILED CAR-PARK COLUMN HAS BEEN PROPPED BY USAR. NO ACCESS TO STAIRWELL AREA.

Usability Category

Damage Intensity	Posting	Usability Category	Remarks
Light damage *Low risk*	Inspected (Green)	G1. Occupiable, no immediate further investigation required	
		G2. Occupiable, repairs required	
Medium damage *Medium risk*	Restricted Use (Yellow)	Y1. Short term entry	
		Y2. No entry to parts until repaired or demolished	
Heavy damage *High risk*	Unsafe (Red)	R1. Significant damage: repairs, strengthening possible	
		R2. Severe damage: demolition likely	
		R3. At risk from adjacent premises or from ground failure	

 Inspection ID: __________ (Office Use Only)

Appendix A2.4

Site Report
31st March 2011

STRUCTURAL AND CIVIL ENGINEERS

SITE REPORT

Project Name	Forsyth Barr
Project No:	
S.R. No:	
Date:	30-31st March 11:30am
Reviewed By:	

Work Reviewed:

Rapid Structural Assessment, external walk around, access to levels .16,15,13,11,9,8,7,6,5,4 via crane through external windows or balconies. Access to levels 3,2,1 and ground floor via car-park or ground floor entrances.

<u>YELLOW PLACARD – Y2</u> – No access to stairwell area – no-one to be with-in 3 metres of stairwell doors unless for assessment or making safe with appropriate fall protection.

Observations & Comments:

Building (from the original drawings)

19 storey concrete frame building with three storey podium carpark structure.
The building was constructed in 1988 as a ductility 6 concrete frame on a 1600mm concrete raft foundation. The seismic load resisting system consists of two 'L' shaped concrete moment resisting frames in the north and south corners. There are two gravity frames running parallel in the north-west to southeast direction. The floors are typically 225 o/a, precast concrete beams at 900crs with timber infill. 75mm topping and 665 mesh reinforcing with saddle bars o

Existing placard – Red.

- The existing L1 assessment (external only) was conducted by on the 28 February, the Red placard was issued due to the collapsed stairs and the possibility of loose/dangerous partitions and fittings.

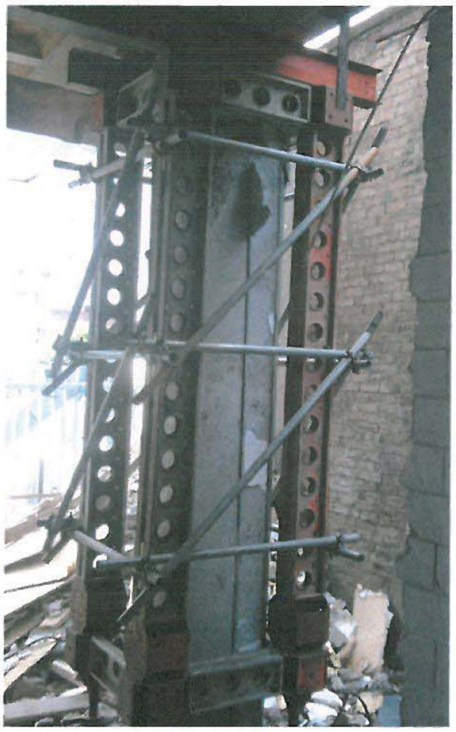
Anecdotal observations from USAR team.

The stairs have collapsed or been removed from the ground floor until the top three flights. The remaining flights at the top have reduced seating and were temporally secured by welding the stair flight seating angles to the supporting channels.

A car-park column was propped, as well as a car-park lower ramp. The ramp was propped or shimmed because of rocking as the forklift used to remove the stairs passed over.

Observations

Carpark
The east elevation levels four to eight has had glazing sections removed to allow USAR access. Otherwise it appears undamaged.
The north elevation appears undamaged.
The damage to the east elevation consists of cladding removed by USAR to remove the collapsed stairs, car-park barrier on the top car-park level section removed by USAR for stair



removal, South east ground floor car-park column has failed at the underside of the cantilever. Propping for the failed has been installed by USAR.

The car-park columns have cracked in general in similar locations to the September 4th earthquake. The injection repairs for these were being carried out around the time of the February 22nd earthquake.

Only the lower half and underside of the forth floor tower beams is visible from the third floor car-park deck. Limited cracks less than 1mm are visible at the beam column interface.

Tower

General

There is minor damage to the seismic frame beams and gravity frame beams cracks typically less than .5mm. The cracks were most prolific on levels four to seven. There are up to 4mm cracks in the floor slabs, these were localised around the corner columns. Very little diaphragm damage was evident generally.

Stairs

The stairs have collapsed up to level 15, from level 15 to 16 the north stair remains. This stair has approximately 20-25mm seating remaining. The stairs above have 40mm plus seating remaining.

The remaining stairs represent a severe fall hazard. There should be no access to the stairwell area on any level until the stairs are secured.

Access to the remaining areas of the building is permissible following the appropriate access health and safety requirements.

<u>YELLOW PLACARD – Y2</u> – No access to stairwell area – no-one to be with-in 3 metres of stairwell doors unless for assessment or making safe with appropriate fall protection.

Appendix A2.5

Preliminary Structural Review 13th April 2011

STRUCTURAL AND CIVIL ENGINEERS

REPORT

Christchurch

Preliminary Structural Review

Forsyth Barr Building

64 Colombo Street

Revision 1 - 13 April 2011

Executive Summary

Following the earthquake on the 22nd of February, has been engaged by to conduct a Level 2 assessment and preliminary structural review of the building at 64 Colombo Street.

The building is essentially a 20 story reinforced concrete framed building with a three story podium car park. The building was constructed in 1988 on a 1600mm concrete raft foundation. The building was built to predecessor standards. Basic code comparison shows that, excluding the performance of the floors and sliding stairs, the building is considered to have capacity in excess of current code level.

Based on strong motion data gathered from Geonet, the building likely experienced shaking well in excess of a current code level earthquake. However the strong shaking that was experienced was relatively short in comparison to what is expected of a design level earthquake.

The main structural damage observed was: inelastic displacement of the main frame structure, cracking of the floor around columns, collapse of both stairs up to level 15 and possible damage to the connections between the podium and the main structure.

It should also be noted that additional strengthening works may be required, depending on any regulatory changes resulting in the coming months. This has not been addressed in any way in this report.

Introduction

Following the earthquake on the 22nd of February, has been engaged by to conduct Level 2 assessment and preliminary structural review of the building at 64 Colombo street.

On the 30th and 31st of March, along with builders and safety teams entered the building via crane to conduct preliminary structural observations of the damage. Key structural locations and floors were identified before entering the building and the damage was documented at these locations. Structural components on the stairwell were not observed due to fall hazard from the remaining stairs in the upper levels.

This report is considered a live document and will be updated as more information is obtained.

Limitations

Findings presented as a part of this project are for the sole use of the owner and their insurer(s) in their 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 neither calculations nor other analyses have been performed. Our inspections have been restricted to structural aspects only. Waterproofing elements, electrical and mechanical equipment, 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.

Building Description – Pre September 4th

The building is a 20 storey reinforced concrete framed building with a three story podium car park that relies on the main structure for seismic resistance. The building was constructed in 1988 as a reinforced concrete frame on a 1600mm concrete raft foundation with thickenings under the columns and walls.

The structural form consists of two 'L' shaped seismic resisting frames in the north-east and south-west sides of the building. Diagonal gravity frames run from the north-west to the south-east of the building. The floors are typically 225 o/a precast beams spanning north-east to south-west at 900crs with timber infill. 75mm topping and 665 mesh reinforcing with saddle bars over gravity beams. This form of construction (using mesh in suspended slabs) is now not recommended due to recent research from the University of Canterbury showing that mesh is unable to sustain large crack widths without fracturing.

Lift and stair penetrations exist in the floor in the south-east corner with a pair of steel beams tying the building together at this location. Steel channels support the stairs and cantilever out the rear of the building to support the toilet area.

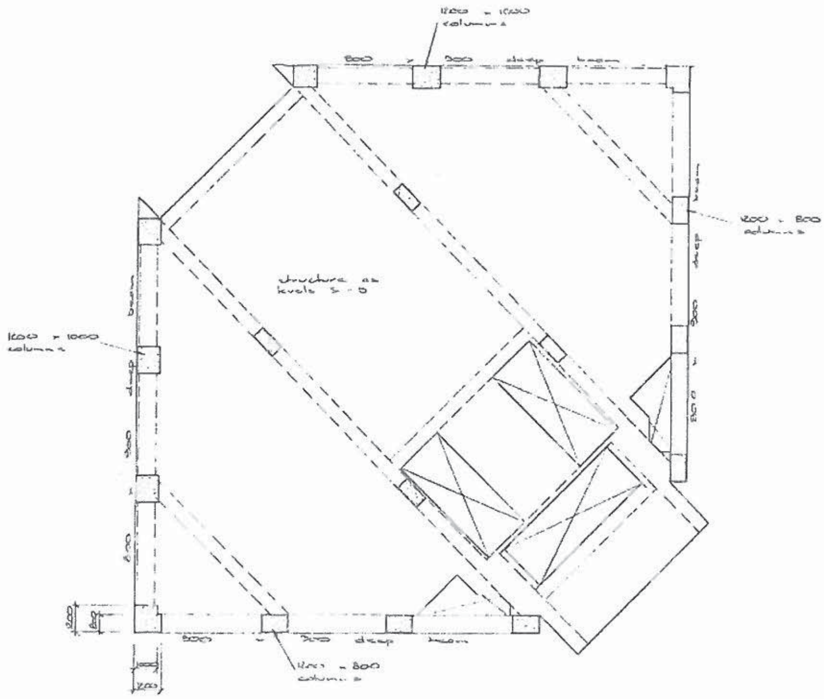


Figure 1: General Structural System

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

The loadings standard NZS4203:1984 has now been replaced by NZS1170.5:2004, with the concrete code being replaced with NZS3101:2006. The building was originally designed as a ductility 6 concrete frame, however to current codes this ductility would be likely limited to somewhere around 4. A comparison of the load levels represented by these two standards is plotted below. The building has a fundamental period of approximately 2.1 seconds in the north-west south-east direction and 2.3 seconds in the north-east south-west direction.

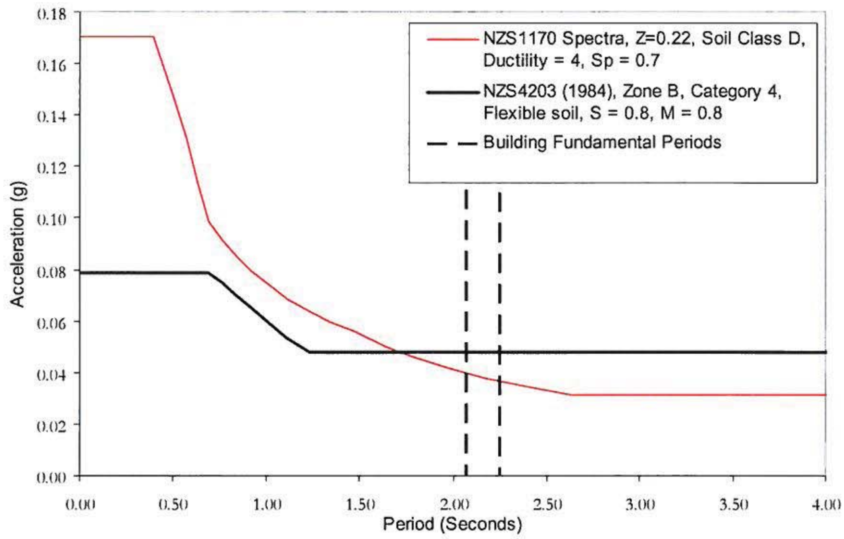


Figure 2: Comparison of Design Codes

It should be noted that although modern ductile buildings are designed to resist 100% of the code, they are detailed in such a way as to keep the floor apart under earthquake levels up to 180% of code (known as a Maximum Credible Event (MCE)). Until recent revisions of the code (NZS1170.5:2004 and NZS3101:2006), MCE events were not required for specific design and have only been infrerred from the detailing requirements. A consequence of this is that certain detailing such as seismic gaps, sliding stairs and brittle structures (such as unreinforced masonry buildings) are not able to sustain the displacements induced on them by earthquakes above design level.

Based on this, excluding the performance of the floors and sliding stairs noted above, the building is considered to have capacity in excess of current code levels.

It should be noted that discussions are underway in relation to the proposed seismic design levels for Christchurch going forward. The outcome of these discussions will most likely result in an increase in design load levels. Current consensus is that seismic loads may increase by about 35% for some buildings. Note that it is also possible that regulatory changes at national or territorial authority level may occur over the coming months with relation to building performance requirements. This has not been addressed in anyway in this report.

Earthquake Description

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 strong motion shaking data resulting from the initial main shock on the 22nd of February 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 figure below shows the shaking experienced by several sites around Christchurch with the black line representing the current code design level for a new building.

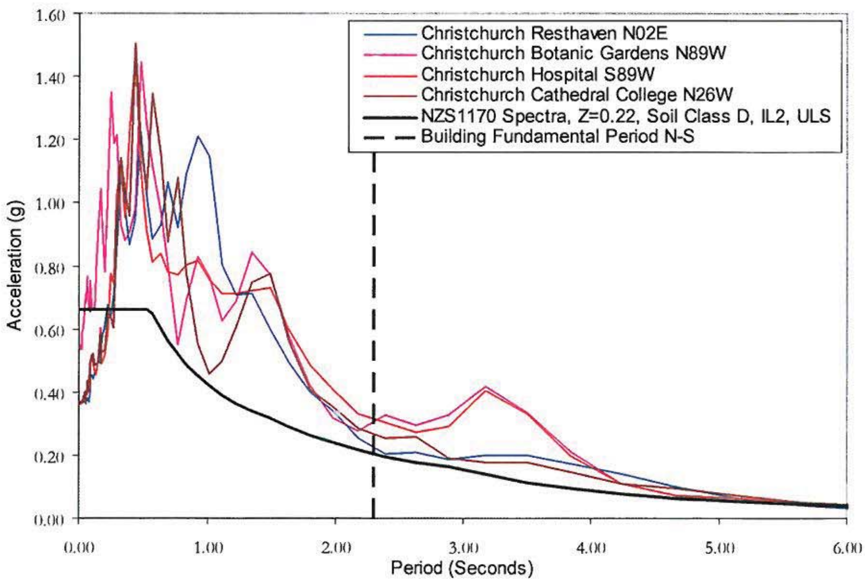


Figure 3: 5% Damped Spectra (North South Direction)

It is apparent that the shaking experienced by the Forsyth Barr Building was most likely well above the current design code level for this building.

However it should also be noted that this earthquake was relatively short in terms of the strong shaking produced (5-10 seconds). Damage is likely to be limited because the building has only gone through a limited number of 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.

Assessment of Damage

In general, excluding the stairs and podium structure, the building performed relatively well during the February 22nd earthquake.

The moment resisting frame has undergone limited inelastic displacement, causing some tearing to the floors around corner columns. The inelastic displacement has been most prolific on levels 4 to 7, causing cracks up to 0.5mm in the beams and up to 4.0mm in the floors. A vertical study has been conducted by critical buildings, suggesting an overall displacement from ground to top of about 80-120mm north. Due to allowable building tolerances of 10mm per floor, this could mean anything from 0 to 300mm or permanent inelastic displacement.

The stairs have collapsed up to level 15; from level 15 to 16 the north stair remains. This stair has approximately 20-25mm seating remaining. The stairs above have a 40mm plus seating remaining. The collapsed stairs have subsequently been removed by USAR teams and placed out the building. The remaining stair on level 15 still poses a risk of collapse in a significant aftershock.

The tie between the podium structure and the main building consists of steel RHS subs cast into the support beams for the podium structure and welded to cast in weld plates on the main structure. There exists significant cracks and spalling in some cases around these locations suggesting possible damage to these connections. Further investigation is required to confirm this. A gravity column in the south-east corner of the podium spalled significantly and subsequently has been propped up by USAR.

There has been some liquefaction to the ground floor tenancies, causing possible failure of the slab on grade in these locations. This is unlikely to affect the seismic performance of the building.

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

Table 1: Observed Damage

Damaged Item	Example
Up to 0.5mm cracks to moment resisting frames	
Spalling of concrete to base of corner columns	

Damaged Item	Example
Up to 4.0mm cracks to floors around corner columns	
Up to 2.0mm cracks around other columns	

Damaged Item	Example
Collapsed stairs	
Damage to tie between podium and main structure	

Seismic Assessment

The building has sustained minor to moderate damage from the strong shaking experienced from the Darfield and Lyttelton earthquakes. From our limited investigations, the building could be considered to still have the majority of its strength.

The seismic resisting system has undergone minor inelastic displacement, which will have likely resulted in a minor reduction of stiffness. It should be noted that as the structure has yielded the reinforcing bars, they are likely to have lost a certain amount of their low cycle fatigue resistance. At this stage no investigation has been undertaken to determine this.

Minor cracks exist to the floor diaphragms around columns. This will have reduced the shear strength of the diaphragm in these locations. Where the cracks are large enough, they may have also fractured the mesh.

The podium structure could be insufficiently tied into the main structure. This would result in the podium structure having minimal seismic resistance. Further investigation is required to verify this.

Recommendations

Going forward we recommend the following:

- Break out podium beams to expose podium to tower connection, so that the damage to this connection can be observed.

- Removal of remaining stairs so that the damage to the structural components within the stairwell can be observed. We will investigate different options to achieve this. Preliminary discussions indicate that lowering the stair units down the stairwell via a lifting beam is likely to be the best option.

Appendix A2.6

Site Visit
14th September 2011

A2.6 Site Visit 14th September 2011

On 14th September 2011, we were able to gain access by external crane to the upper floors of the Forsyth Barr building. We were able to visit the stair landings on Levels 14, 15 and 16, and observe the seismic gaps at the lower landings of the stair units remaining.

Refer to Figure A2.6.8 for stair nomenclature.

We were able to confirm that the following units were still in place:

- North-west side : from Level 14 upwards
- South-east side : from Level 15 upwards

At each floor, there were measurements of the gaps written on the floor by USAR over a number of days after the 22nd February earthquake, and these were also recorded at each floor on a sheet taped to a wall.

At all seismic gaps there was evidence of movement sufficient to split the floor covering (which may have also been damaged by USAR investigations).

We undertook further measurements of the gaps and of relative movement between the stair units and the side walls close to the landings (limited by safety considerations). Where practical, we exposed the gaps to observe their contents. We were looking for evidence that the gaps had been formed as specified on the drawings, and whether there might have been any other obstructions potentially reducing the effective width of the gaps.

We observed the following:

- Level 14, north-west unit: the seismic gap in the south-west landing was measured to be 57 mm. The movement of the flight, relative to the north-west stairwell wall, was measured to be 30 mm vertically and 22 mm horizontal. There was rubble in the gap.
- Level 15, north-west unit, south-west landing. There was a large diagonal crack along the lower landing (this had been reported by others so was not a surprise) (Figure A2.6.1). The crack width was measured to be 40 mm wide at its widest part. The gap was measured to be 100 mm wide at the slab surface, and the drop of the stair flight at the location of the lower stair tread was measured to be 40 mm vertically. 23 mm of the back, top edge of the landing appeared to have been removed during construction.
- Level 15, south-east unit, north-east landing: The seismic gap was measured to be 55 mm. The drop in the stair flight was measured to be a maximum of 40 mm vertically.
- Level 16, north-west unit: the unit appeared to have been seated on two small steel spacer plates sitting on the steel channel. We could slide these plates horizontally – indicating that the unit was now being supported elsewhere – possibly on the thin (approximately 2 mm) layer of latence/mortar present on the top flange of the channel. Approximately 27 mm of the upper edge of the unit's bottom landing above the seat was not present to a depth of 120 mm. The concrete vertical face of the edge had a finish consistent with having been formed by a rotating saw/grinder. The top of the steel-angle toe was exposed, and the current seismic gap at the toe was approximately 30 mm. The current displacement of the first step relative to the north-west wall was 19 mm horizontally away from the gap, and 30 mm downwards. Some nails, a nail-gun cartridge, wood shavings and mortar fragments were present in the gap. No flexible gap-filling rod or similar was present in place or nearby, but this may have been removed and discarded by USAR. There was a significant horizontal crack (> 2 mm) visible along the exposed end of the

unit's landing. See photographs in Figures A2.6.4 and A2.6.5, and Figure A2.6.6 (measurements).

- Level 16, south-east unit: The gap in the north-west landing was filled with a full-depth piece of polystyrene sheet which was measured on removal to be 20 mm thick. See Figures A2.6.2 and A2.6.3. The stair-unit side of the sheet had a sheet of black polythene stuck to it. When removed, the polystyrene had a loose fit in the gap. We took the polystyrene away for possible testing. The current displacement of the first step relative to the south-east wall was 8 mm horizontally away from the gap, and 10 mm downwards. We measured the gap as 33 mm, compared to measurements recorded by others of 1¼" (32 mm) and 35 mm on 1st March and 19th March, respectively.
- Level 17, north-west unit: Looking at the underside from the Level 16 south-west landing, we could see significant full-width cracks corresponding to where the landing meets the first step (see Figure A2.6.7).
- Balance of landings: There was evidence that a 30 mm diameter compressible foam rod had been present in the upper region of the seismic gaps, and covered by a thin layer of mortar to give a smooth surface for the vinyl-type floor covering. The mortar had generally fragmented and fallen into the gap. A sample of the foam rod was taken away for possible testing. Pieces of a smaller diameter foam rod were also present – possibly used to seal the sides of the landing. On one upper landing, a 1 mm crack was observed extending from the seismic gap of the adjacent unit across the construction joint between the unit and the floor.

We did not gain access to floor levels above Level 16 or below Level 14.

We note that some of the current stair positions appear to vary from those recorded on 14th March 2011 by Hyland.

On the accessed floors, we were able to have an only cursory inspection of the building structure as much of it is hidden behind furnishings. We were therefore unable to confirm whether or not the building structure has been damaged to the extent predicted by our analyses.

In our opinion:

- The seismic gaps in the stairs could have been as little as 11 mm prior to the 4th September 2010 earthquake (compared to the 30 mm specified).
- The construction of the gaps on Levels 14, 15 and 16 was not carried out in a consistent manner from floor to floor.
- The steel spacers used between the underside of the stair unit and the lower seat were not detrimental to the stair performance.
- Some lower landings of the stair units were partially shortened during installation in order to express the specified gap at the floor surface.
- Construction debris was present in some of the gaps, and would have lessened the effective gap to some, but indeterminate, extent.

We conclude that this site visit has confirmed our previous opinions as to why the stairs were both damaged in the 4th September 2010 earthquake and collapsed in the 22nd February earthquake.

Our observations reinforce our recommendations in the body of our report.

Figure A2.6.1: Level 15 North-West Unit, South-West Landing, Gap with 23 mm Removed off Unit

Figure A2.6.2: Level 16, North-East Landing, South-East Unit, Seismic Gap with 20 mm Polystyrene Sheet Infill

Figure A2.6.3: Level 16, North-East Landing, South-East Unit, Seismic Gap with 20 mm Polystyrene Sheet Infill Removed

Figure A2.6.4: Level 16, South-West Lobby, North-West Unit, Seismic Gap with Stepped Landing

Figure A2.6.5: Level 16 : North-West Unit, South-West Landing – Relative Movement between Stair and North-West Wall (19 mm horizontally, 30 mm vertically)

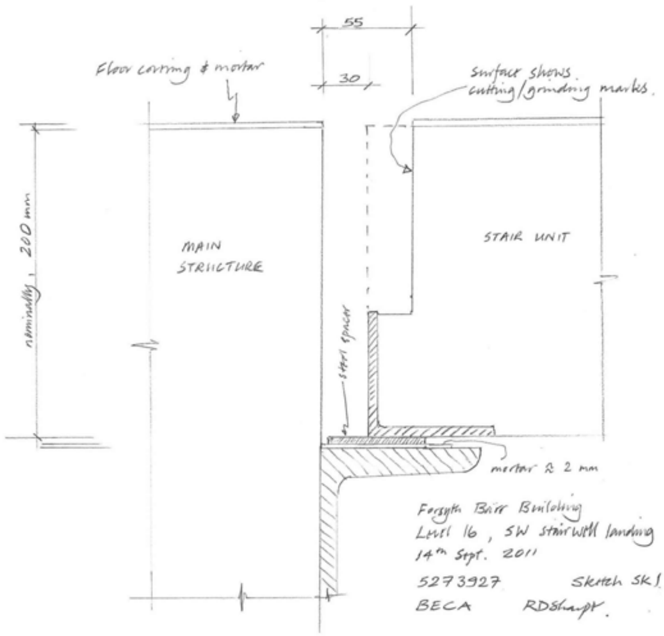


Figure A2.6.6: Level 16, South-West Lobby, North-West Unit, Seismic Gap Measurements

Figure A2.6.7: Level 16, South West Lobby, Looking up at Underside of North-West Unit Showing Cracking

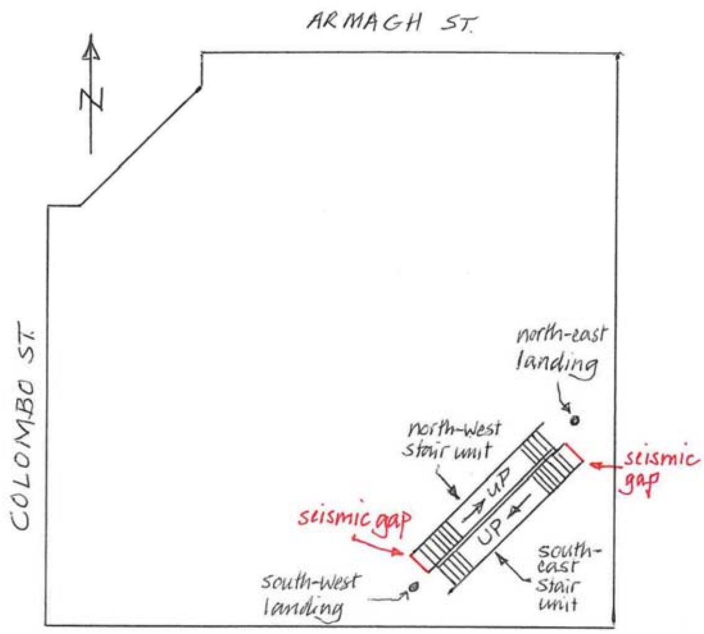


Figure A2.6.8: Stairs, Landings and Seismic Gaps Locations

Appendix A3

Structural Analyses

A3 Str uctural Analyses

A3.1 Introduction

This Appendix summarises the structural analyses that have been carried out.

Included are:

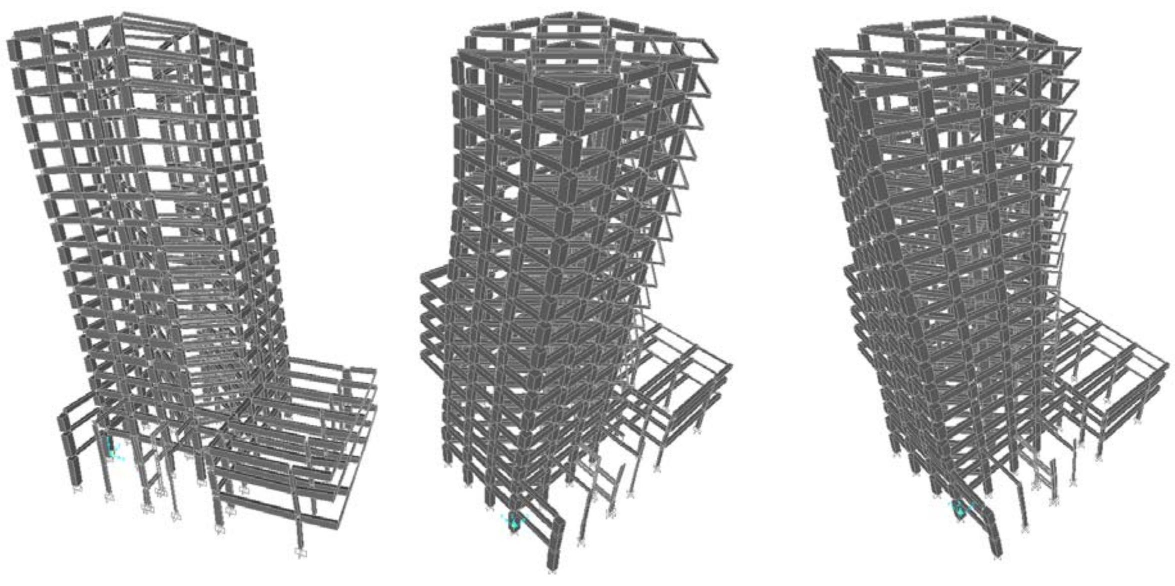
- Modal analyses of the building model.
- Time-history analyses of the building model to determine the inter-storey drifts.
- Elastic analyses of the stair flights.
- Non-linear analyses of the stair flights.

The analyses reported below have been carried out on the proprietary computer software package SAP2000 V14.2.3.

A3.2 3D Building Model – Elastic Modal Analysis

A 3D elastic model of the Forsyth Barr Building was prepared.

Results of a modal analysis of this model are presented in Figure A3.1 and Tables A3.1 (fixed base) and A3.2 (soil springs included).



Mode 1 : T=1.91 s **Mode 2 : T=1.69 s** **Mode 3 : T=1.16 s**

Figure A3.1 : First Three Mode Shapes of the Building Model

Table A3.1 : Mode Shapes and Mass Distribution – Fixed Base

		Effective Mass			Cumulative Mass		
Mode	Period (s)	X	Y	RZ	X	Y	RZ
1	1.91	30%	35%	0%	30%	35%	0%
2	1.69	35%	30%	37%	64%	66%	38%
3	1.16	0%	0%	23%	64%	66%	61%
4	0.70	9%	12%	1%	73%	78%	62%
5	0.63	11%	8%	12%	84%	85%	73%
6	0.51	0%	0%	11%	84%	86%	84%

Table A3.2: Mode Shapes and Mass Distribution – Vertical Soil Springs

		Effective Mass			Cumulative Mass		
Mode	Period (s)	X	Y	RZ	X	Y	RZ
1	2.38	30%	36%	0%	30%	36%	0%
2	1.96	35%	31%	38%	66%	67%	39%
3	1.25	0%	0%	24%	66%	67%	63%
4	0.72	9%	12%	1%	75%	79%	64%
5	0.63	10%	8%	12%	85%	87%	76%
6	0.53	0%	0%	10%	85%	87%	86%

A3.3 3D Building Model – Elastic Time-History Analyses

A3.3.1 Introduction

In order to estimate the inter-storey drifts that occurred during the 4th September 2010 and 22nd February 2011 earthquakes, time-history analyses were carried out using the 3D model of the structure described above and the strong-motion record obtained from the REHS instrument. A review of the reported damage to the primary elements of the building structure after the February earthquake indicated minor to moderate damage, and it was considered that an elastic analysis with allowance for cracking was a reasonable approach.

The REHS records were applied in north-south and east-west directions, and also on a ± 45 degrees angle. The vertical record was applied simultaneously.

A3.3.2 Assumptions

The main assumptions made for the 3D model include:

- 0.4 and 0.55 stiffness modifiers assumed for beams and columns, respectively.
- A concrete strength of 45 MPa from testing reports.
- Diaphragms assigned at all floor levels.

A3.3.3 Assessment of Inter-storey Drifts

The inter-storey drifts assessed for the building are shown in Table A3.3. These have been converted to a longitudinal and transverse displacement occurring along and across the stair units. The stair units have been divided in to a front and back units, as these run in opposite directions. The figure below shows the location of these units within the building and the points used to measure the inter-storey displacement (upper floor minus lower).

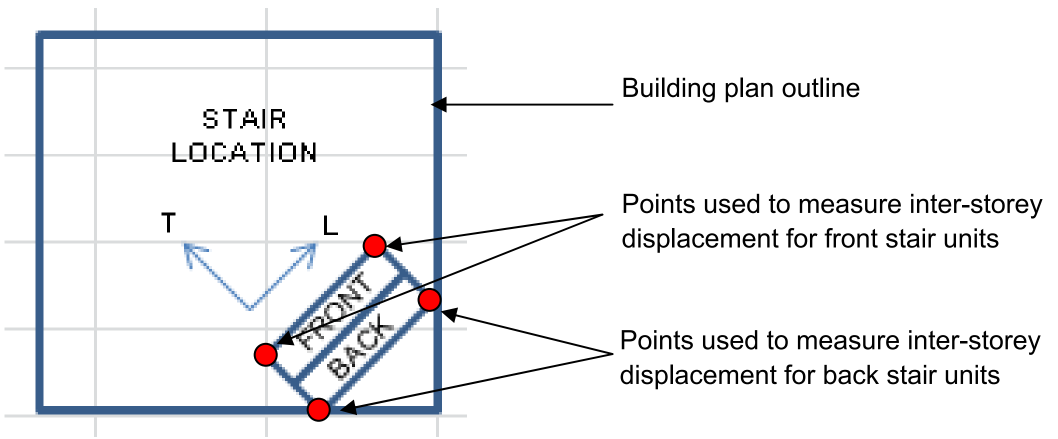


Figure A3.2 : Locations of Reference Points for Inter-Storey Drifts

The inter-storey displacements for Levels 13 to 16, are shown in Tables A3.4 and A3.5. Compression indicates that the supposed 30 mm seismic gap at the lower landing of the stair is decreasing, and extension indicates this is increasing. The displacements in the transverse direction are absolute values only. The apparent discrepancy between the drifts reported in Table

A3.3 and Tables A3.4 and A3.5 is due to the effect of rotation in plan of the building and where the displacements are measured (refer to Figure A3.2).

Drawings indicate there is a 90 mm gap in the transverse direction between the front and back stair flights and a further 75 mm gap on either side of the stair units to the building.

Table A3.3 : Inter-Storey Displacements (mm) for Back Stair Unit, REHS Record with Vertical Springs

Level	Longitudinal Extension	Longitudinal Compression	Transverse
18-17	38	38	46
17-16	40	44	48
16-15	43	51	49
15-14	46	58	49
14-13	49	62	49
13-12	51	65	47
12-11	53	66	44
11-10	53	62	39
10-9	50	54	33
9-8	48	45	30
8-7	47	42	28
7-6	47	39	27
6-5	48	38	27
5-4	50	40	27

Table A3.4 : Inter-Storey Longitudinal Stair Displacements (mm)

Level	Earthquake record: REHS, 22nd February 2011				Earthquake record: REHS 22nd February @ 45 degrees				Earthquake record: REHS 22nd February @ -45 degrees				Earthquake record: REHS, 4th September 2010			
	Extension		Compression		Extension		Compression		Extension		Compression		Extension		Compression	
	Front	Back	Front	Back	Front	Back	Front	Back	Front	Back	Front	Back	Front	Back	Front	Back
Fixed-Base Model																
16-15	46	42	29	47	40	32	19	41	32	29	29	33	22	17	12	22
15-14	28	49	28	56	47	48	27	49	38	34	37	39	25	21	18	26
14-13	61	46	54	63	54	42	33	53	41	39	42	44	29	23	22	29
Vertical Soil Springs																
16-15	39	43	51	51	-	-	-	-	-	-	-	-	19	33	33	26
15-14	43	46	57	58	-	-	-	-	-	-	-	-	21	35	35	28
14-13	47	49	62	62	-	-	-	-	-	-	-	-	23	38	38	30

Table A3.5 : Inter-Storey Transverse Stair Displacements (mm)

Level	Earthquake record: REHS, 22nd February 2011		Earthquake record: REHS 22nd February @ 45 degrees		Earthquake record: REHS 22nd February @ -45 degrees		Earthquake record: REHS, 4th September 2010	
	Front	Back	Front	Back	Front	Back	Front	Back
Fixed-Base Model								
16-15	95	93	110	66	64	75	56	47
15-14	35	89	110	58	67	75	59	47
14-13	84	86	109	49	68	77	61	47
Vertical Soil Springs								
16-15	31	49	-	-	-	-	24	39
15-14	33	49	-	-	-	-	28	41
14-13	33	49	-	-	-	-	30	42

A3.4 Elastic Stair Unit Analyses

A3.4.1 Model Overview

Figure A2.1 shows the basic model in SAP2000 of a typical stair unit from Levels 4 to 17 in the main Forsyth Barr stairwell. The stair is modelled along the centreline of members. As the top landing appears to have some fixity back to the structure, both pinned and fixed supports have been tested to give a range of possible design actions.

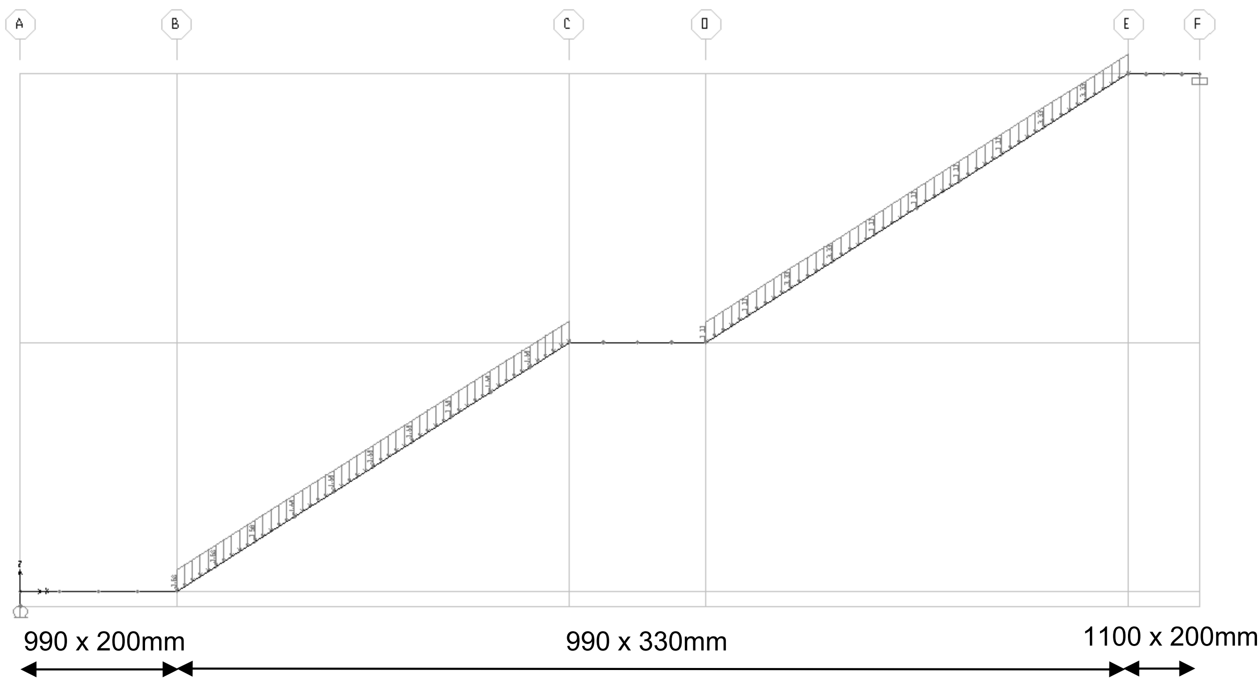


Figure A3.3 : Model Representation for SAP Model

The Site Examination Report indicated that at the lower landing of the stair the concrete above the steel bearing angle had been cut away on at least some units to increase the seismic gap. When the stair landing collides with the structure, the impacts will occur over the lower section of the landing. An extra member was added to represent the height of the landing from the centreline member to the lower edge. This allowed the earthquake force/displacement to be conservatively applied at the bottom edge of the landing, accounting for any bending effects this creates. A weight modifier of 0.001 was applied to this member to ensure that it would not contribute to self-weight, and a bending modifier of 1000 was applied to ensure a rigid member.

A3.4.2 Assumptions

The following assumptions were made:

- Concrete strength of 30 MPa.
- Self-weight accounted for automatically with section size. In addition, a uniformly distributed load (UDL) has been applied to represent the stair treads which can be seen in Figure A3.3.
- A stiffness modifier of 0.5 has been used for effective section properties.

- The lower landing support is modelled as simply-supported. As displacement loads cannot be applied to unrestrained degrees of freedom, an equivalent force was used for earthquake loading.
- 5 % damping assumed for modal case.
- Analyses have been run in the X-Z plane only. No out-of-plane rotation has been considered.

A3.4.3 Modal Analyses

A modal analysis was run to compare the fundamental vertical modes with vertical accelerations recorded at these frequencies from four central city locations. The locations of the recordings used are Christchurch Hospital, Botanic Gardens, Catholic Cathedral College, and Resthaven Rest Home. The corresponding maximum and average accelerations are shown in Table A3.6 below for each fundamental time period.

Table A3.6 : Modal Analysis Results for Stair Unit Model

Mode	Time Period (s)	Vertical Mass Participation	Maximum Acceleration (g)	Average Acceleration (g)
1	0.210	64%	1.09	0.77
4	0.025	23%	1.62*	0.98*
5	0.014	5%	1.62*	0.98*

*Accelerations taken from 0.04 s time period, as this is the smallest value available from the modal analysis.

A3.4.4 Linear Static Analysis

A linear static analysis was undertaken for self-weight loads and combined self-weight and imposed displacement earthquake loads. Each case was run twice, once with a pinned support, and secondly with a fixed support at the top landing.

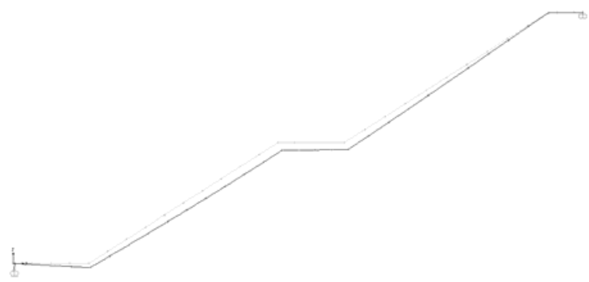
Analyses were completed for inter-storey drifts that varied from 5 to 30 mm applied at the lower landing.

The figures below show exaggerated deflected shapes and moment diagrams, while the following tables list the values of these moments and deflections obtained for each node.

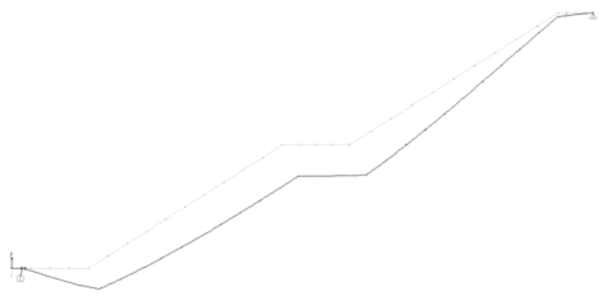
A3.4.5 Deflected Shapes

The deflected shapes are shown in Figure A3.4.

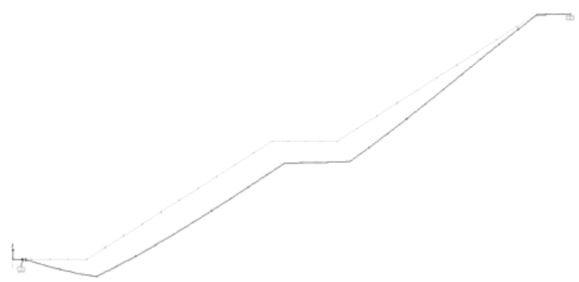
(a) Pinned support, self weight (SW)



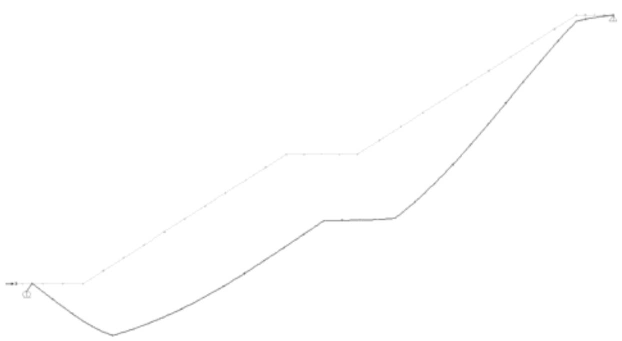
(b) Fixed upper support, self weight (SW)



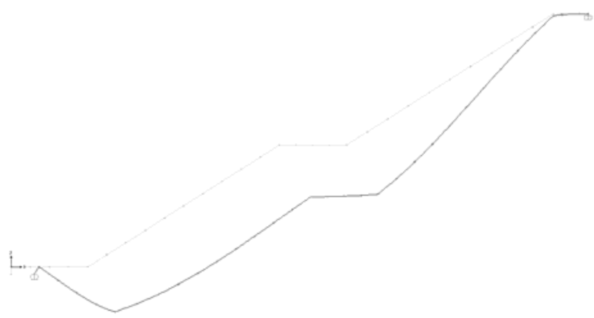
(c) Pinned support, 10 mm differential horizontal drift between adjacent floors (10 mm EQ) + SW



(d) Fixed upper support, 10 mm EQ + SW



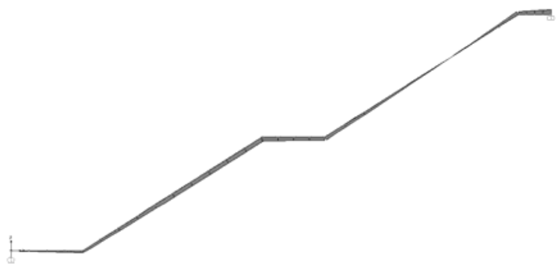
(e) Pinned support, 30 mm EQ + SW



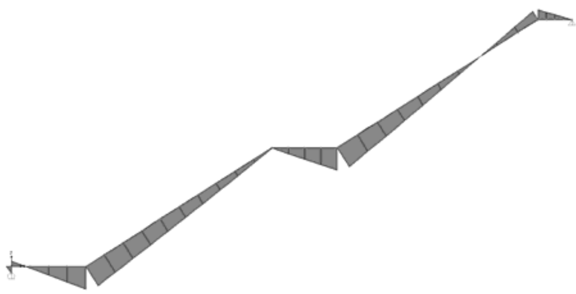
(f) Fixed upper support, 30 mm EQ + SW

Figure A3.4 : Deflected Shapes from the Forced Displacement Analyses (

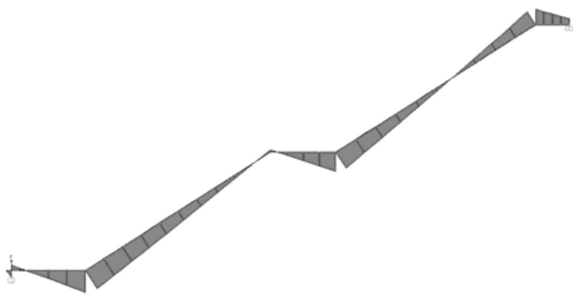
(a) Pinned support, self weight (SW)



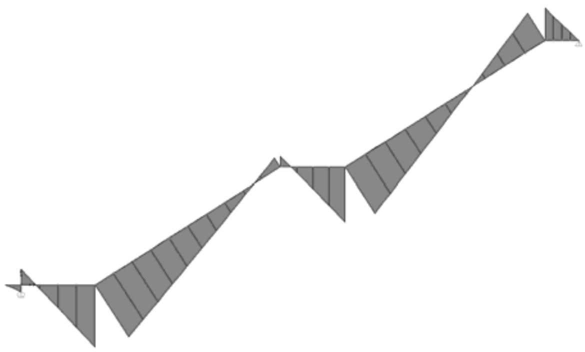
(b) Fixed upper support, self weight (SW)



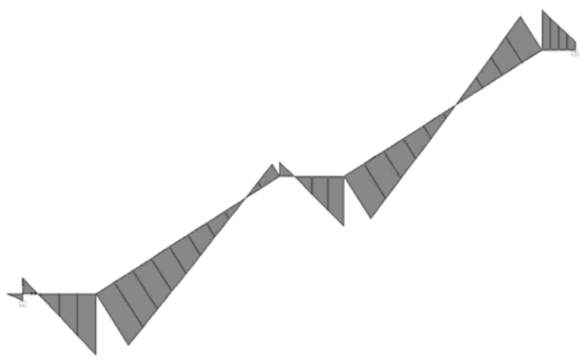
(c) Pinned support, 10 mm EQ + SW



(d) Fixed upper support, 10 mm EQ + SW



(e) Pinned support, 30 mm EQ + SW



(f) Fixed upper support, 30 mm EQ + SW

Figure A3.5 Bending Moments from the Forced Displacement Analyses

Table A3.7 : Vertical Deflections from the Forced Displacement Analyses

Top Restraint	Fixed	Pinned	Fixed	Pinned	Fixed	Pinned	Fixed	Pinned	Fixed	Pinned	Fixed	Pinned
Case	Node 1 Deflection (mm)		Node 2 Deflection (mm)		Node 3 Deflection (mm)		Node 4 Deflection (mm)		Node 5 Deflection (mm)		Node 6 Deflection (mm)	
Self-weight (SW)	0	0	-6	-9	-11	-20	-10	-19	-1	-4	0	0
5 mm EQ + SW	0	0	-15	-19	-21	-31	-19	-30	-1	-5	0	0
10 mm EQ + SW	0	0	-25	-29	-31	-42	-29	-40	-2	-5	0	0
20 mm EQ + SW	0	0	-43	-48	-51	-64	-49	-62	-2	-7	0	0
30 mm EQ + SW	0	0	-62	-68	-72	-86	-68	-83	-3	-8	0	0

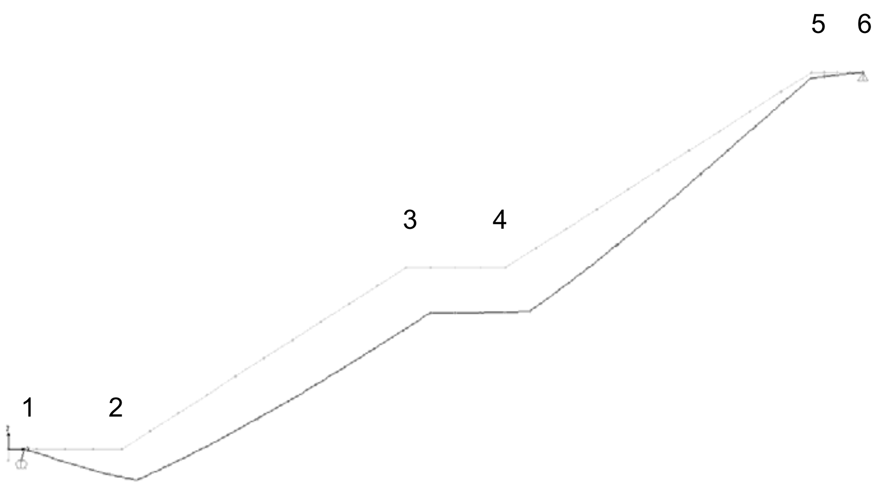


Table A3.8 : Horizontal Deflections from the Forced Displacement Analyses

Top Restraint	Fixed	Pinned	Fixed	Pinned	Fixed	Pinned	Fixed	Pinned	Fixed	Pinned	Fixed	Pinned
Case	Node 1 Deflection (mm)		Node 2 Deflection (mm)		Node 3 Deflection (mm)		Node 4 Deflection (mm)		Node 5 Deflection (mm)		Node 6 Deflection (mm)	
Self-weight (SW)	2	2	2	3	6	10	6	10	0	0	0	0
5 mm EQ + SW	7	7	8	9	12	16	12	16	0	0	0	0
10 mm EQ + SW	12	12	14	15	18	23	18	23	0	0	0	0
20 mm EQ + SW	22	22	26	27	31	36	30	36	0	0	0	0
30 mm EQ + SW	32	32	38	39	43	50	43	49	0	0	0	0

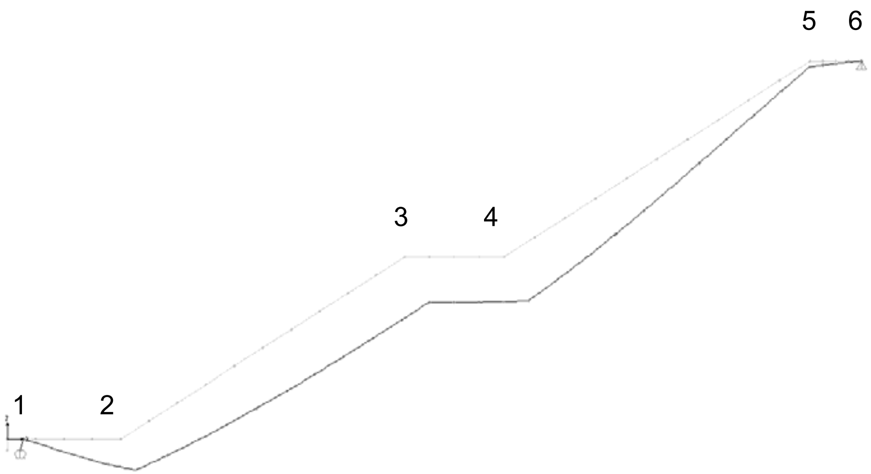
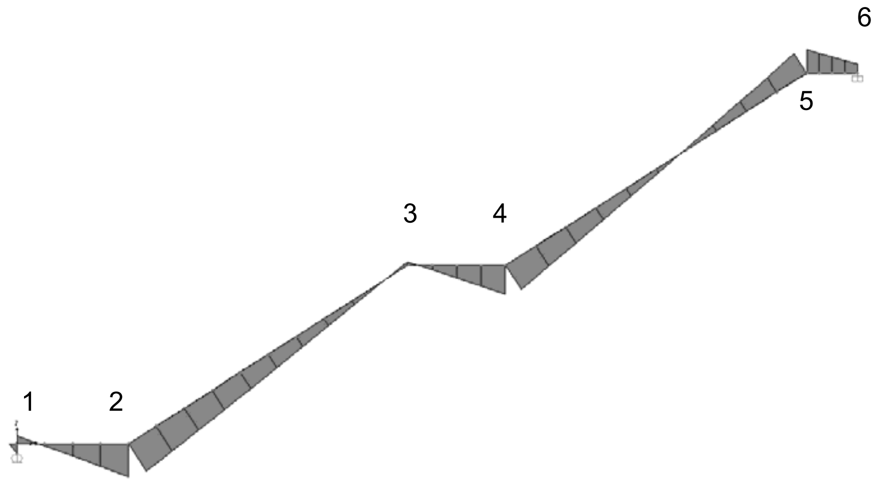


Table A3.9 : Moment Demands from the Forced Displacement Analyses

Top Restraint	Fixed	Pinned	Fixed	Pinned	Fixed	Pinned	Fixed	Pinned	Fixed	Pinned	Fixed	Pinned
Case	Node 1 Moment (kNm)		Node 2 Moment (kNm)		Node 3 Moment (kNm)		Node 4 Moment (kNm)		Node 5 Moment (kNm)		Node 6 Moment (kNm)	
Self-weight (SW)	0	0	32	42	59	92	49	90	-46	21	-71	0
5 mm EQ + SW	-38	-38	173	183	14	51	160	206	-135	-60	-79	0
10 mm EQ + SW	-75	-75	314	325	-31	9	272	322	-223	-142	-86	0
20 mm EQ + SW	-150	-150	596	609	-121	-74	495	554	-400	-306	-101	0
30 mm EQ + SW	-226	-226	878	893	-210	-157	719	786	-577	-469	-115	0



A3.5 Secti on Capacities

Section capacities of the stair sections were calculated using the computer program RESPONSE. The steel yield strength was specified as 300 MPa, and ultimate steel strength as 450 MPa.

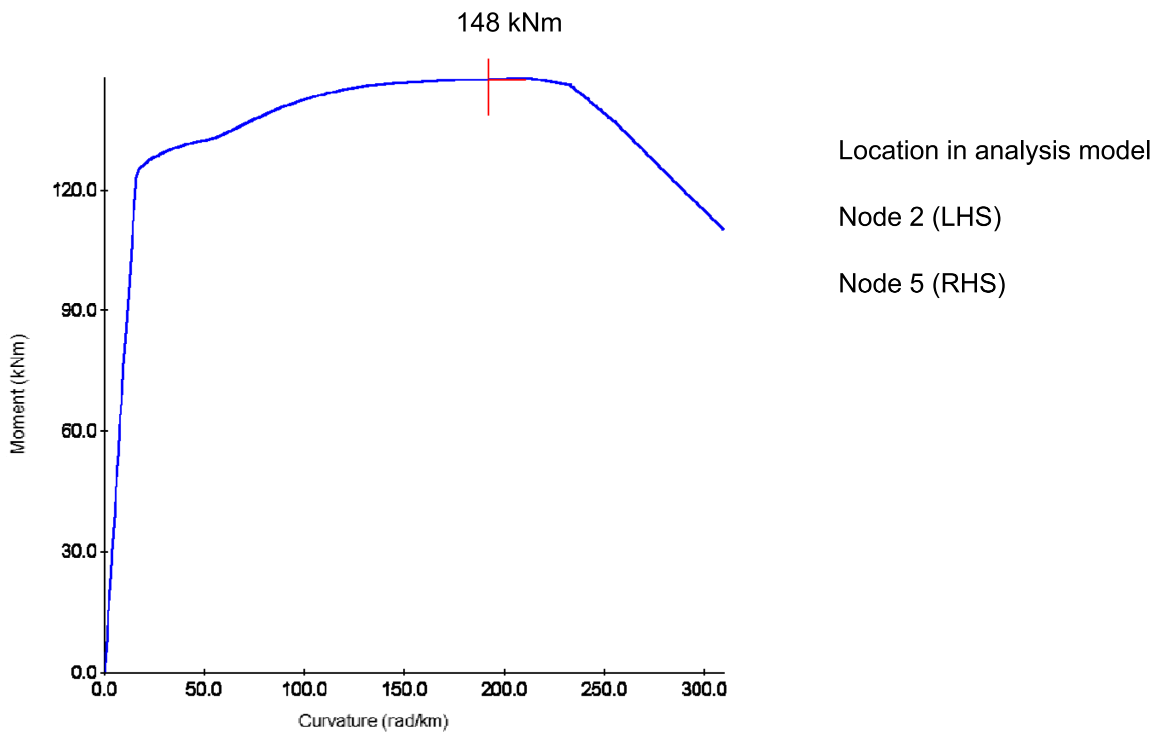


Figure A3.6 : Moment-Curvature for 990 x 200 mm Section, Positive Bending

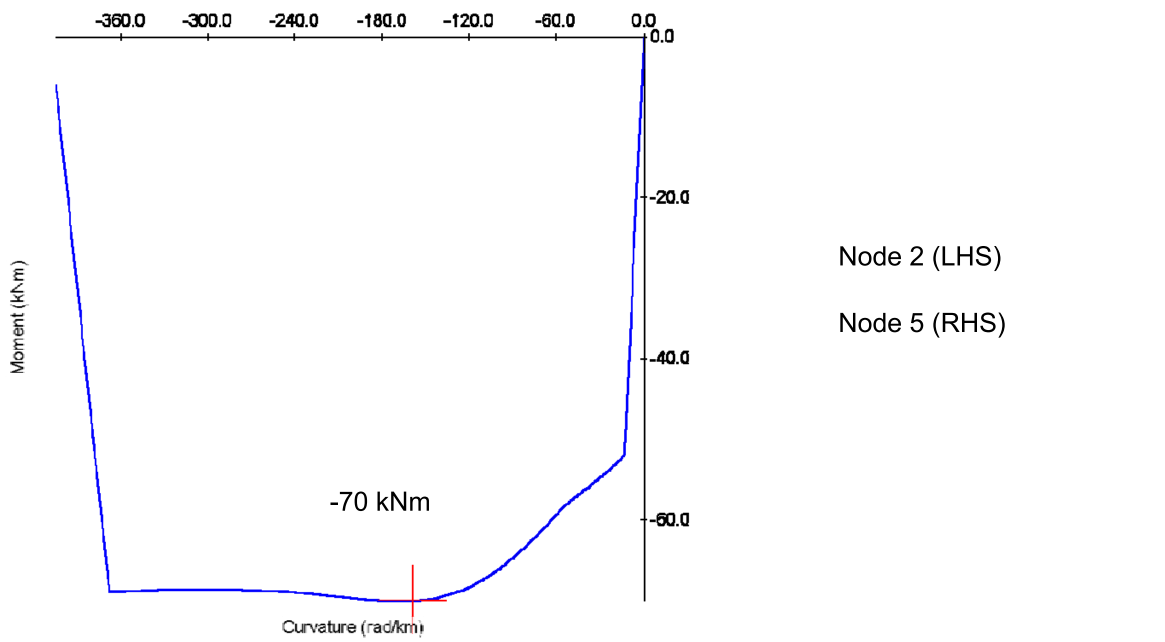


Figure A3.7 Moment-Curvature for 990 x 200 mm Section, Negative Bending

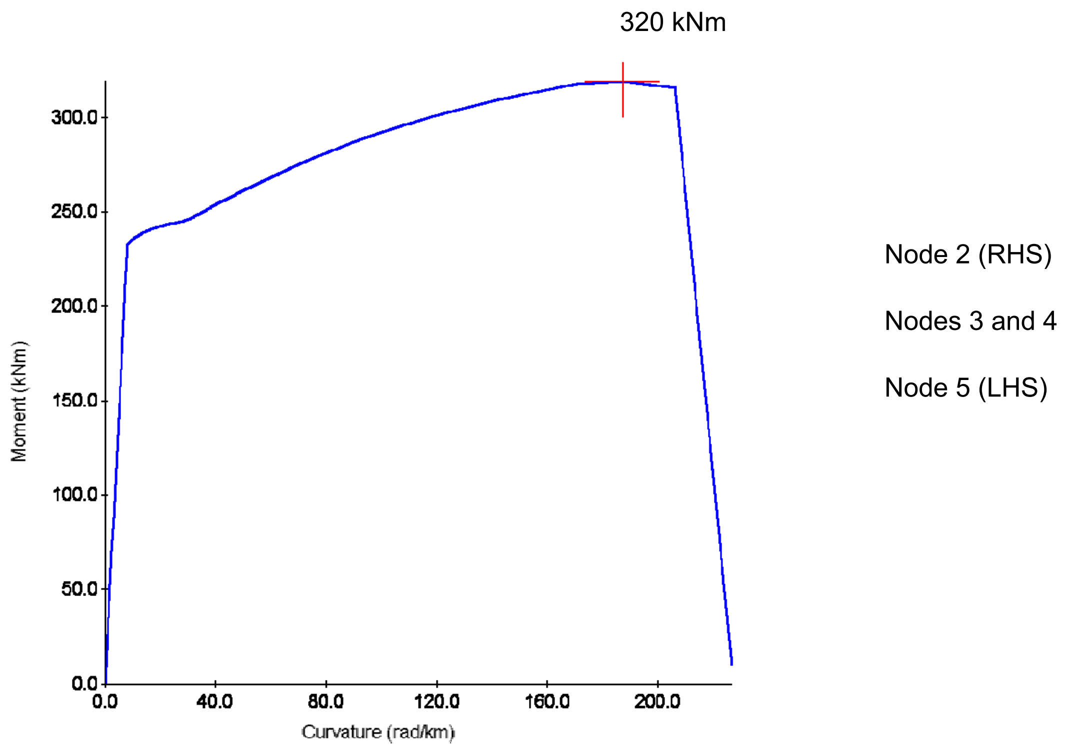


Figure A3.8 : Moment-Curvature for 990 x 330 mm Section, Positive Bending

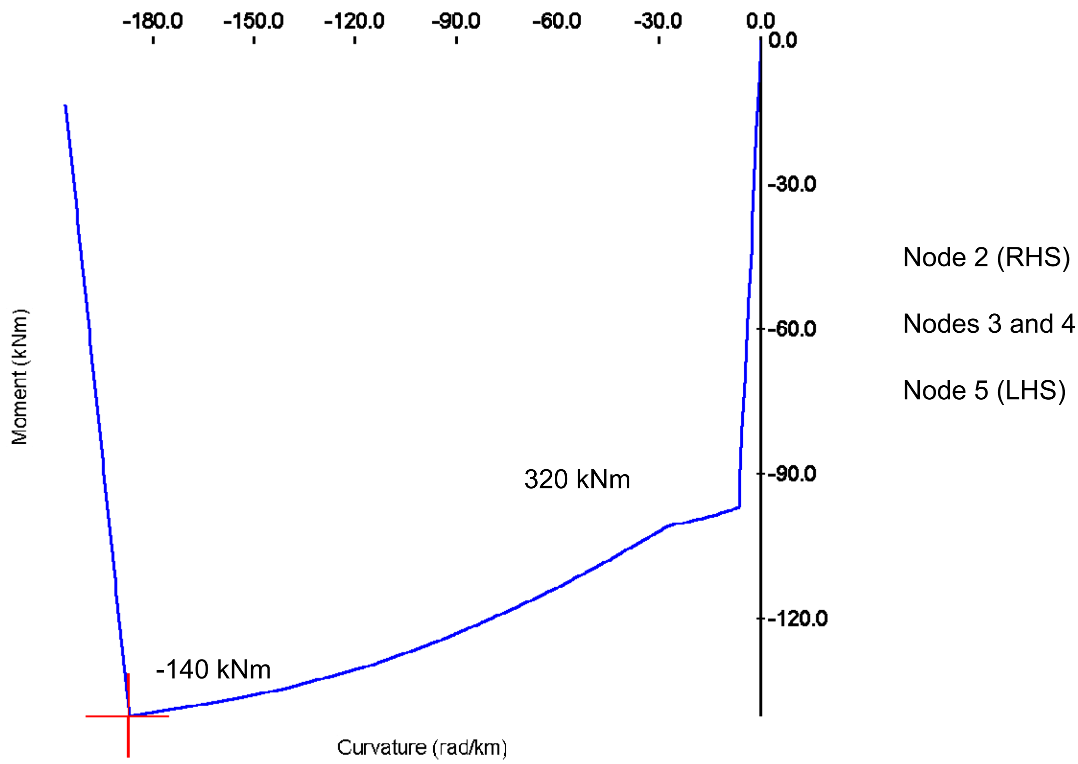


Figure A3.9 : Moment-Curvature for 990 x 330 mm Section, Negative Bending

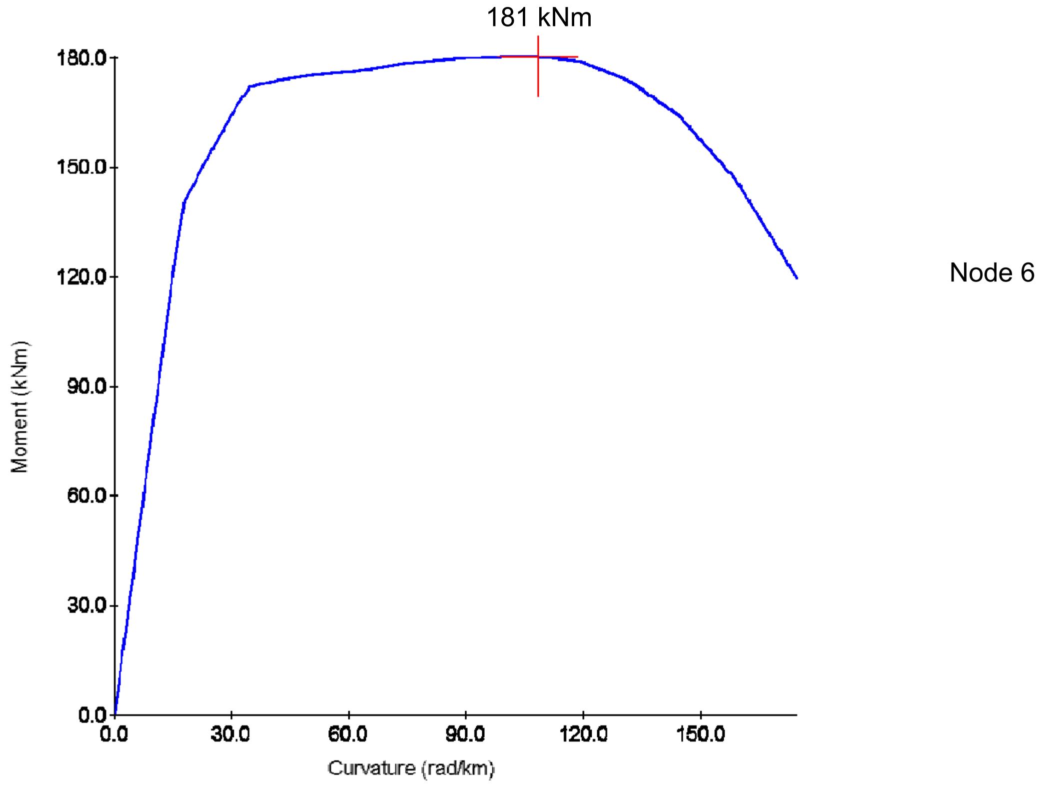


Figure A3.10 : Moment-Curvature for 990 x 200 mm Section + Dowels, Positive

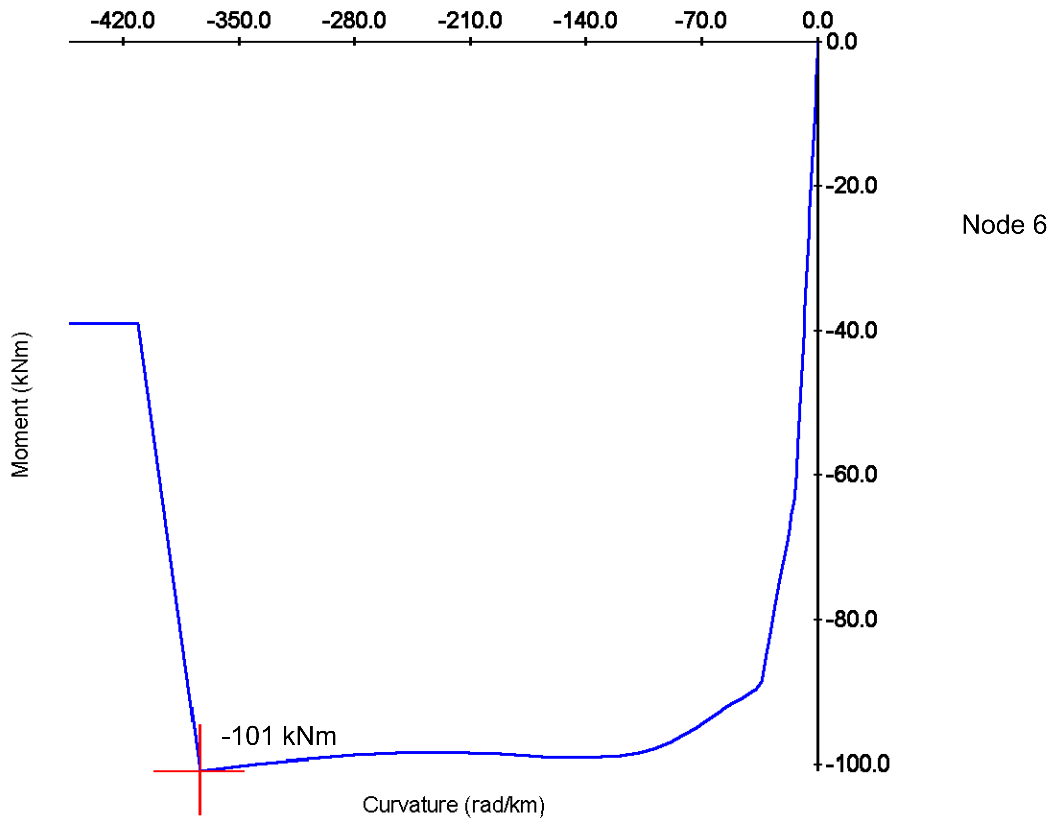


Figure A3.11 : Moment-Curvature for 990 x 200 mm Section + Dowels, Negative Bending

A3.6 Non-linear Pushover Stair Unit Analysis

A non-linear pushover analysis was carried out on the stair unit to confirm the potential permanent horizontal shortening and vertical displacement of the stair unit. Moment-curvature hinges were added adjacent to joints 2 and 5, in the 900 mm x 200 mm stair section as noted below.

Figure A3.12 : Location of Hinges in the Non-linear Stair Flight Analysis

A displacement was applied to the lower landing joint until the first hinge failure.

As well as displacement of this lower joint in the X direction (as shown in Figure 20 below), displacements were also applied in the Y direction. No hinges were considered in this direction. Moment and torsion demands can be found in Table A3.13.

The assumptions made were similar to those for the linear case. Other assumptions include hinge lengths assumed as 0.2 m (the same as the depth of the section), and the location of the hinge located at a distance of 130 mm from the end of the member. Flexural stiffness modifiers of 0.5 were assumed.

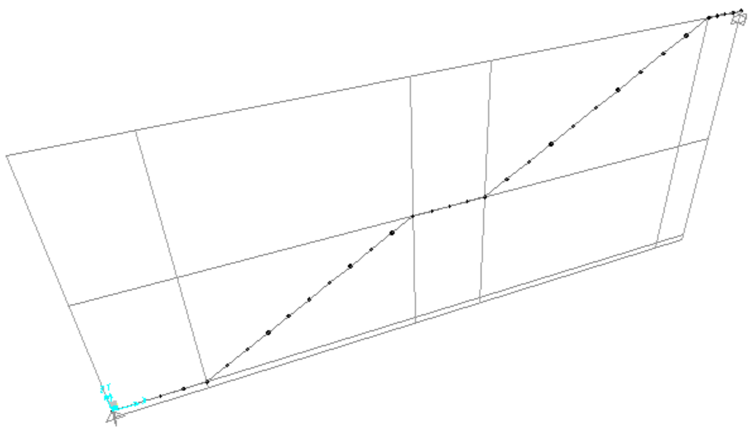


Figure A3.13 : Stair Model Showing Axes

Table A3.10 : Vertical Deflections from Inelastic Stair Unit Analyses

Top Restraint	Fixed	Pinned	Fixed	Pinned	Fixed	Pinned	Fixed	Pinned	Fixed	Pinned	Fixed	Pinned
Case	Node 1 Deflection (mm)		Node 2 Deflection (mm)		Node 3 Deflection (mm)		Node 4 Deflection (mm)		Node 5 Deflection (mm)		Node 6 Deflection (mm)	
Self-weight (SW)	0	0	-5	-8	-9	-17	-8	-16	-1	-3	0	0
2.5 mm EQ + SW	0	0	-9	-13	-15	-22	-14	-22	-1	-4	0	0
5 mm EQ + SW	0	0	-14	-18	-22	-27	-21	-26	-1	-4	0	0
10 mm EQ + SW	0	0	-25	-30	-29	-34	-26	-32	-1	-5	0	0
Hinge Failure	0	N/A	-36	N/A	-35	N/A	-31	N/A	-1	N/A	0	N/A

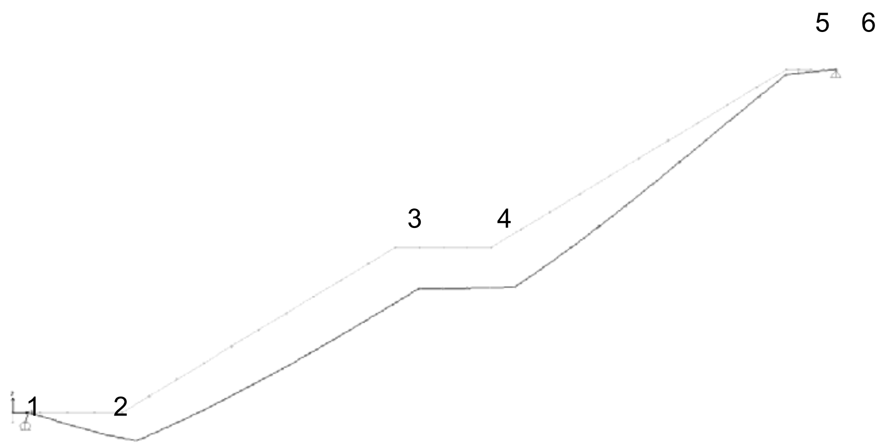


Table A3.11 : Horizontal Deflections from Non-Linear Stair Unit Analyses

Top Restraint	Fixed	Pinned	Fixed	Pinned	Fixed	Pinned	Fixed	Pinned	Fixed	Pinned	Fixed	Pinned
Case	Node 1 Deflection (mm)		Node 2 Deflection (mm)		Node 3 Deflection (mm)		Node 4 Deflection (mm)		Node 5 Deflection (mm)		Node 6 Deflection (mm)	
Self-weight (SW)	2	2	2	3	5	8	5	8	0	0	0	0
2.5 mm EQ + SW	4	4	5	6	9	12	9	12	0	0	0	0
5 mm EQ + SW	7	7	8	9	13	14	13	14	0	0	0	0
10 mm EQ + SW	12	11	14	14	17	17	17	17	0	0	0	0
Hinge failure	16	N/A	20	N/A	20	N/A	20	N/A	0	N/A	0	N/A

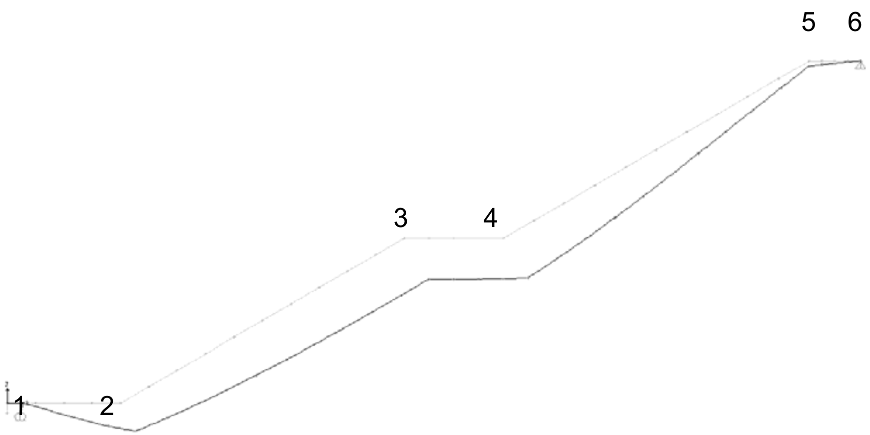


Table A3.12 : X-Direction Displacement and Moment Demands from the Non-Linear Stair Unit Analyses

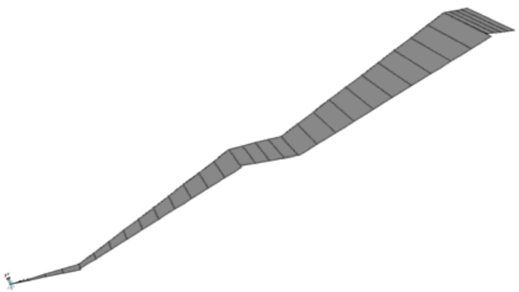
Top Restraint	Fixed	Pinned	Fixed	Pinned	Fixed	Pinned	Fixed	Pinned	Fixed	Pinned	Fixed	Pinned
Case	Node 1 Moment (kNm)		Node 2 Moment (kNm)		Node 3 Moment (kNm)		Node 4 Moment (kNm)		Node 5 Moment (kNm)		Node 6 Moment (kNm)	
Self-weight (SW)	0	0	32	42	59	92	49	90	-46	21	-72	0
5 mm EQ + SW	-32	-32	160	163	48	57	178	190	-66	-49	-19	0
10 mm EQ + SW	-35	-34	170	172	47	54	188	196	-63	-54	-15	0
Hinge Failure	-35	N/A	173	N/A	47	N/A	191	N/A	-69	N/A	-15	N/A

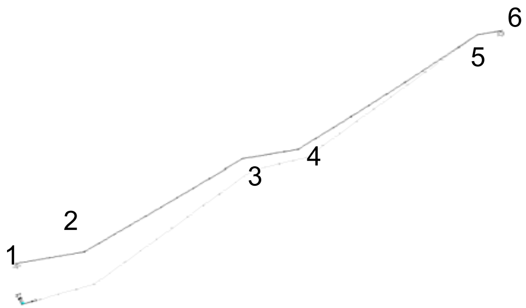
Note: the moments given above are at the intersections of the model centrelines. The plastic hinges are displaced from these intersection points and therefore the moments shown exceed the plastic hinge capacities given in A3.5.

Table A3.13. Y-Direction Displacement and Moment Demands from Stair Flight Analyses

	Node 2		Node 3		Node 4		Node 5		Node 6	
Displacement	Moment (kNm)	Torsion* (kNm)	Moment (kNm)	Torsion* (kNm)	Moment (kNm)	Torsion* (kNm)	Moment (kNm)	Torsion* (kNm)	Moment (kNm)	Torsion* (kNm)
20 mm	-27	0, 15	-113	15, -43	-108	-43, 27	-209	27, -89	-203	-89, N/A
40 mm	-54	0, 29	-206	29, -86	-237	-86, 54	-419	54, -179	-407	-179, N/A
60 mm	-81	0, 43	-309	43, -129	-355	-129, 82	-628	82, -268	-610	-268, N/A
80 mm	-108	0, 58	-412	58, -172	-473	-172, 109	-838	109, -358	-814	-358, N/A
100 mm	-135	0, 72	-515	72, -215	-591	-215, 136	-1047	136. -447	-1017	-447, N/A

*Note: Two values given for torsion indicate the values in the members either side of the node (left, right).





Moment Displacem **ent** **Torsion**

A3.7 Knee Joint

One area of possible concern under gravity load only is the knee joint (i.e., where the mid landing joins the top step of the bottom flight). These have been shown to perform well below their expected capacity if insufficient transverse steel is provided to transfer tension across the joint.

A Concrete Construction publication 1988[1] on this issue provides a tentative recommendation that whenever $2\ A_s\ f_y\ \sin(\Theta/2) > Kbdf_c^{0.5}$, steel cross ties linking top and bottom steel (anchored around both), with a total section area $A_v = 2\ A_s\ \sin(\Theta/2)$ should be provided.

It appears this requirement is not meet in these units, neither with respect to the area of transverse steel required, nor the requirement for anchorage.

Appendix A4

Site Data

Appendix A4.1

Geotechnical Report

Beca

www.beca.com

Factual Geotechnical Report

Investigation into the collapse of the Pyne Gould Corporation Building and the Forsyth Barr Building stairs on 22 February 2011

Prepared for Department of Building and Housing (DBH)

By Beca Carter Hollings & Ferner Ltd (Beca)

30 June 2011

Factual Geotechnical Report

Investigation into the collapse of the Pyne Gould Corporation Building and the Forsyth Barr Building stairs on 22 February 2011

Prepared for Department of Building and Housing (DBH)

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30 June 2011

Revision History

Revision Nº	Prepared By	Description	Date
A	Mark Hill		

Document Acceptance

Action	Name	Signed	Date
Prepared by	Mark Hill	Mark Hill	30.6.11
Reviewed by	Gavin Alexander	[signature]	30.6.11
Approved by	Rob Jury	[signature]	30.6.11
on behalf of	Beca Carter Hollings & Ferner Ltd		

Table of Contents

Appendices

1 Introduction

The New Zealand Department of Building and Housing (DBH) has commissioned Beca Carter Hollings & Ferner Ltd (Beca) to undertake an investigation into why the Pyne Gould Corporation (PGC) Building at 231-233 Cambridge Terrace and the stairs in the Forsyth Barr Building collapsed during the Magnitude 6.3 earthquake that struck Christchurch at 12.51pm on Tuesday 22nd February 2011.

This Geotechnical report presents the factual results of the geotechnical investigations at the PGC site and at the nearby strong motion recorder (REHS) site. The current investigations comprised of four machine boreholes, four cone penetration tests (CPTs) and five concrete cores. Historic ground investigation data for the Forsyth Barr Building is also presented.

2 Site Description

The sites are located at 231-233 Cambridge Terrace (PGC) and 764 Colombo Street (REHS), Christchurch.

The PGC building is 40m to the north of the Avon river. The ground profile is generally flat. The building was demolished to the ground floor level at the time of the investigation. The Forsyth Barr Building is 140m to the south west of the Avon river and the ground profile is generally flat.

Both sites are surrounded by other commercial buildings.

3 Previous Investigations

Soils and Foundations Ltd carried out a geotechnical investigation of the Forsyth Barr site in 1987 and 1988.

Five boreholes were drilled for the investigation program. The Borehole logs and site plan are included in Appendix E.

4 Scope of Investigations

The field works were carried out on 8th June to 13th June 2011 at the locations shown in Appendix A. The current investigation positions were determined using hand held GPS (Garmin Vista C Etrex) and summarised in Table 1 below. Co-ordinates are in terms of NZTM.

The field investigation work proceeded as directed by Beca staff.

Table 1 – Investigation Summary

Investigation No.	Site	Date	Coordinates		Investigation Method	Depth (mbgl)
			Easting (m)	Northing (m)		
BH 100	PGC	10/6/11	2480806	5742174	Machine Borehole (BH)	15
BH 101		8/6/11	2480810	5742159		15
BH 102		10/6/11	2480811	5742140		15
BH 103	REHS	13/6/11	2481311	5742669		14.45
CPT 100	PGC	9/6/11	2480806	5742174	Cone Penetration Test (CPT)	5.96
CPT 101		9/6/11	2480810	5742159		3.92
CPT 102		9/6/11	2481311	5742669		5.17
CPT 103	REHS	9/6/11	2481311	5742669		20.35

Note: 1. Accuracy of coordinates is ±5m

4.1 Concrete Cores at PGC

Table 2 – PGC Ground Floor Slab

Investigation No.	Concrete Thickness (mm)	Underlying Material	Gap
CC1	250	Medium to coarse GRAVEL, trace sand.	N
CC2	210	Coarse GRAVEL and cobbles	N
CC3	130	Fine to medium sandy fine to coarse GRAVEL.	N
CC4	120	10mm of SAND. Followed by medium to coarse GRAVEL.	N
CC5	130	Fine to coarse GRAVEL	N

Note: 1. Refer to Figure 1 for locations

5 Investigation Methodology

5.1 Machine Boreholes

Four HQ sized machine boreholes were drilled by Pro Drill (Auck) Ltd using a tractor mounted rotary drilling rig.

Standard Penetration Tests (SPTs) were generally undertaken at 1.5m intervals where appropriate, within each borehole, using the technique described in ASTM D1586 Rev A, 2008.

All samples were logged and photographed on site by a Geotechnical Engineer or an Engineering Geologist. The logs were generally described in accordance with the NZGS Guidelines for the Field Description of Soil and Rock (NZGS, 2005). The borehole logs and photographs are presented in Appendices B and C, along with a bore log key that clarifies the symbols used in the logs.

All samples were wrapped in plastic and are stored in labelled boxes on site. Some unavoidable drying out of the samples is to be expected with time.

Boreholes were backfilled with bentonite on completion of drilling.

5.2 Cone Penetration Test (CPT)

Cone Penetration Tests (CPTs) were conducted by Perry Drilling Ltd, to depths of between 3.92 m and 20.35 m, using a track mounted 100 kN capacity hydraulic rig fitted with a 10 cm^2 (end area) cone to measure cone resistance, sleeve friction and pore water pressure. Testing was conducted following test Standard ASTM D5778-95 (2000) Standard Test Method for performing Electronic Friction Cone and Piezocone Penetration Testing of Soils.

Cone resistance (qc) and Friction ratio (Rf) test records are presented in Appendix D.

6 Notice to Reader/User of this Document

This report is the property of our client, The New Zealand Department of Building and Housing and Beca Carter Hollings & Ferner Ltd.

This is a factual report of field investigations. The field investigations have been undertaken at discrete locations and no inferences about the nature and continuity of ground conditions away from the investigation locations are made. Furthermore logs are provided presenting description of the soils and geology based on our observation of the samples recovered in the fieldwork and may not be truly representative of the actual underlying conditions.

To the maximum extent permitted by law, Beca Carter Hollings & Ferner Ltd disclaims all liability and responsibility (in contract or tort, including negligence, or otherwise) for any loss or damage whatsoever which may be suffered as a result of any reliance by any third party on this report, whether that loss is caused by any fault or negligence on the part of Beca Carter Hollings & Ferner Ltd or otherwise.

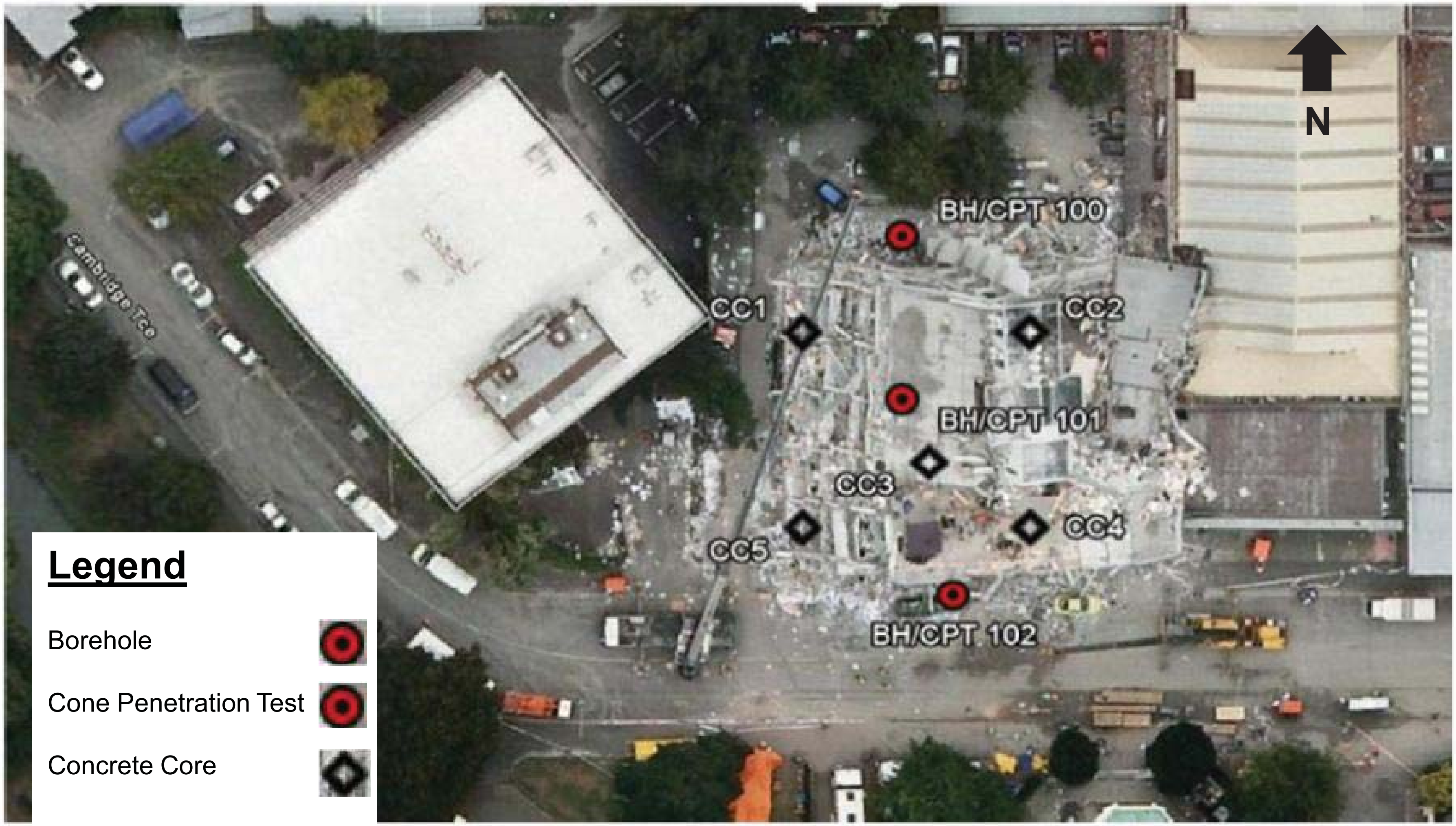
No interpretation of the investigation results has been made in this report. Should you be in any doubt as to the applicability of this report for the proposed development described herein, it is essential that you carry out independent investigations to satisfy your needs.

7 References

ASTM D5778 – 95 (2000): Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils.

Appendix A

Geotechnical Investigation Plans

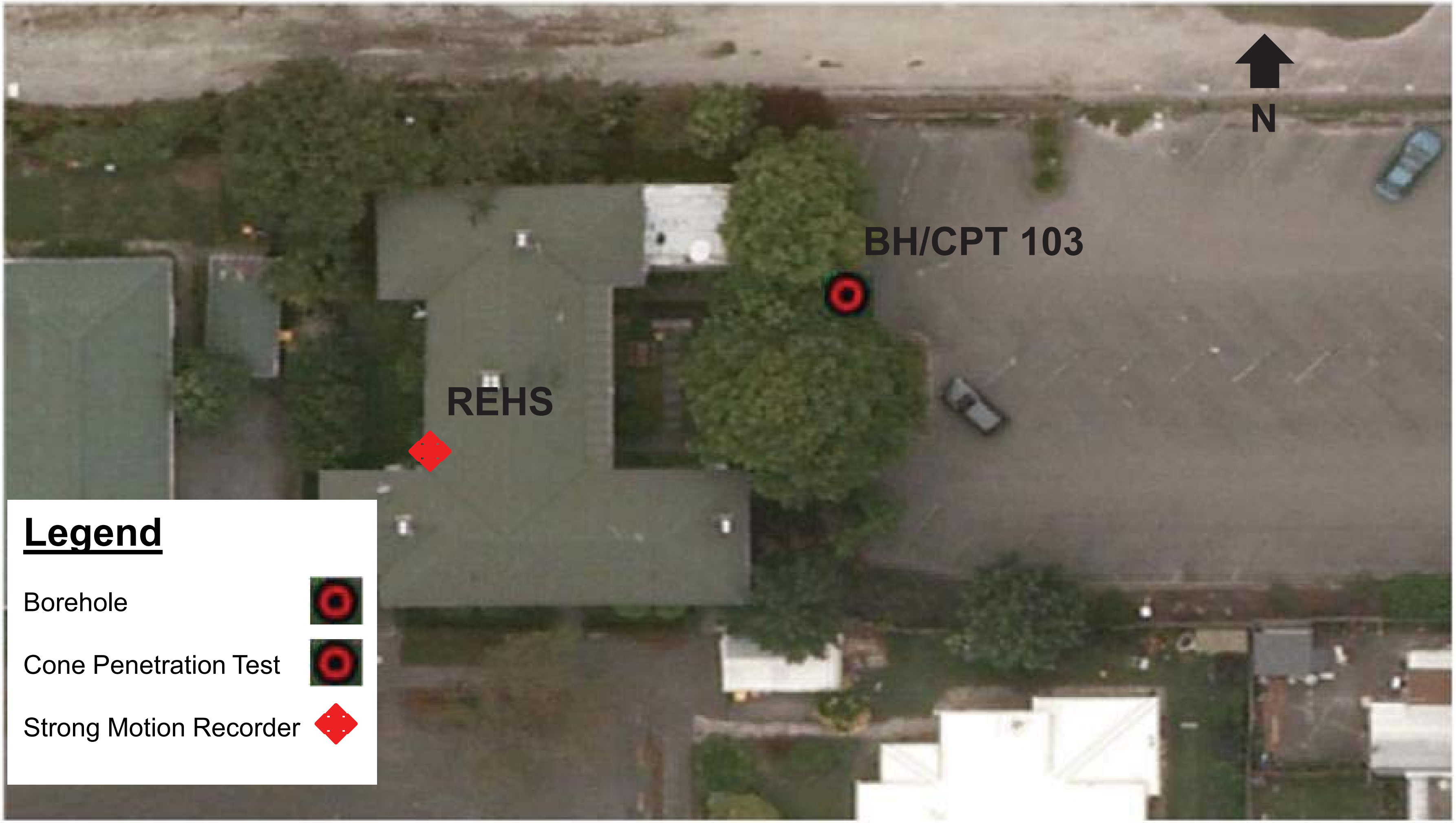


SCALE 1:356 AT A3

PGC Site Location Plan

Figure 1

Beca
5273927



SCALE 1:400 AT A3

REHS Site Location Plan

Figure 2

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5273927

Appendix B

Borehole Logs

Beca

BOREHOLE No: **BH 100**

MACHINE AUGER LOG

SHEET 1 of 2

PROJECT: DBH Christchurch Earthquake Building Investigation
JOB NUMBER: 5273927
SITE LOCATION: Christchurch
CLIENT: Dept of Building and Housing

CIRCUIT: NZMG
COORDINATES: N 5,742,175 m, E 2,480,806 m
BOREHOLE LOCATION: Pyne Gould Building
R L:
DATUM: Mean Sea Level

Depth (m)	Core recovery	Method	SPT 'N'	USCS	Moisture	Soil / Rock Description	Geological Unit
0.0	75 %	OB		SW	S	Loosely packed, fine to coarse SAND, some silt, minor fine to coarse gravel; grey brown; saturated. Gravel: sub rounded, SW, greywacke.	Springston Formation
				SP	M	Loosely packed, silty fine SAND; grey brown; moist, non plastic. Minor fine to coarse gravel and cobbles.	
				SP	M	Loosely packed, fine to medium SAND, minor silt; grey brown, moist, non plastic.	
				ML	M	Loose, SILT, some fine sand, trace rootlets; grey brown; moist, non plastic.	
1.0				GW	M	Medium dense, fine to coarse GRAVEL, some fine to coarse sand, trace silt; brown grey; moist. Gravel: rounded to subrounded, SW, greywacke. (Silt probably washed out during drilling).	
1.5	40 %	TT					
2.0	33 %	SPT	8, 8, 9, N=17			Medium dense.	
2.5	28 %	TT					
3.5	45 %	SPT	8, 13, 16, N=29				
4.0	38 %	TT					
5.0	0 %	SPT	12, 17, 18, N=35	GW	M	Dense.	
5.5				SP	W	Dense, fine to medium SAND; grey; wet.	
5.5	47 %	TT		GW	W	Dense, fine to coarse GRAVEL, trace cobble; grey; wet. Gravel: rounded to sub rounded, SW, greywacke.	
6.5	55 %	SPT	18, 19, 20, N=39	GW	W	Dense, fine to coarse GRAVEL, some fine to coarse sand; grey; wet. Gravel: rounded to sub rounded, SW, greywacke.	
7.0	43 %	TT					
8.0	100 %	SPT	9, 13, 20, N=33	SP	W	Dense, fine to coarse SAND; grey to brownish grey; wet.	
8.5	95 %	TT					
9.0						Some silt; non plastic.	
9.5	67 %	SPT	5, 12, 14, N=26				

DATE STARTED: 10/6/11
DATE FINISHED: 10/6/11
LOGGED BY: GJG
SHEAR VANE No:

DRILLED BY: Pro Drill (Auck) Ltd
EQUIPMENT: Edson Tractor Rig
DRILL METHOD: OB and TT
DRILL FLUID: Water and mud
DIAMETER/INCLINATION: - / 90°

COMMENTS:
0m = top of borehole not top of concrete pad. Co-ordinates from hand held GPS. Most core losses not shown for clarity of log. Core losses assumed to be a result of matrix being washed away during drilling.

FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS SEE KEY SHEET

Revision A

A4 Scale 1:50

Beca

BOREHOLE No: **BH 100**

MACHINE AUGER LOG

PROJECT: DBH Christchurch Earthquake Building Investigation
JOB NUMBER: 5273927
SITE LOCATION: Christchurch
CLIENT: Dept of Building and Housing

CIRCUIT: NZMG
COORDINATES: N 5,742,175 m
E 2,480,806 m
BOREHOLE LOCATION: Pyne Gould Building
R L:
DATUM: Mean Sea Level

CORE RECOVERY	METHOD	CASING	SPT 'N'	DEPTH (m)	USCS	MOISTURE	SOIL / ROCK DESCRIPTION	GEOLOGICAL UNIT
42 %	TT			10.0–10.5	SP	W	Dense, fine to coarse SAND, some silt, trace wood fragments; dark grey; wet; non plastic.	Springston Formation (*Contd.*)
45 %	SPT		14 19 21 N=40	11				
38 %	TT			11.45–12	SW	W	Dense, fine to coarse SAND, trace wood fragments; dark grey; wet.	
							Minor wood fragments and organic material. Organic odour.	
33 %	SPT		13 16 22 N=38	12.5–13	SP	W	Dense, fine to medium SAND; grey; wet.	
95 %	TT			13–14				
100 %	SPT		15 21 23 N=44	14			Grey mottled yellow grey.	
85 %	TT	15.0m @ 10/6/2011		14.5–15			Yellow brown banded grey.	
				15			END OF LOG @ 15 m	

DATE STARTED: 10/6/11
DATE FINISHED: 10/6/11
LOGGED BY: GJG
SHEAR VANE No:
DRILLED BY: Pro Drill (Auck) Ltd
EQUIPMENT: Edson Tractor Rig
DRILL METHOD: OB and TT
DRILL FLUID: Water and mud
DIAMETER/INCLINATION: - / 90°

COMMENTS:
0m = top of borehole not top of concrete pad. Co-ordinates from hand held GPS. Most core losses not shown for clarity of log. Core losses assumed to be a result of matrix being washed away during drilling.

Beca

BOREHOLE No: **BH 101**

MACHINE AUGER LOG

SHEET 1 of 2

PROJECT: DBH Christchurch Earthquake Building Investigation
JOB NUMBER: 5273927
SITE LOCATION: Christchurch
CLIENT: Dept of Building and Housing

CIRCUIT: NZMG
COORDINATES: N 5,742,159.3 m
E 2,480,810.9 m
BOREHOLE LOCATION: Pyne Gould Building
R L:
DATUM: Mean Sea Level

Depth (m)	Core Recovery	Method	SPT 'N'	USCS	Moisture	Soil / Rock Description	Geological Unit
0.0–0.55	100 %	OB				CONCRETE.	Concrete
0.55–1.45	44 %	OB		ML	M	Stiff, fine to coarse gravelly SILT, minor fine sand; brown; moist; low plasticity. Gravel: rounded to subrounded, SW, greywacke.	Fill
1.45–1.95	100 %	TT				CONCRETE.	Concrete
1.95–3.9	0 %	TT				Core loss: 1.95-3.9m (1.7m)	
~3.5–3.9	11 %	SPT	12, 9, 3, N=12				
3.9–4.45	60 %	TT		GW	M	Medium dense, fine to medium sandy fine to coarse GRAVEL, minor silt; dark brownish grey; wet. Gravel: sub angular to rounded, SW, greywacke.	Springston Formation
4.45–4.6	60 %	TT		SP	M	Medium dense, silty fine SAND, trace fine gravel; grey; moist.	Springston Formation
4.6–5.0				GP	W	Medium dense, medium to coarse GRAVEL; dark grey; wet. Gravel: rounded to subrounded, greywacke.	Springston Formation
5.0–5.45		SPT	17, 25, 20, N=45	GW	M	Medium dense, fine to coarse sandy GRAVEL; grey; moist. Gravel: angular to rounded, greywacke.	Springston Formation
5.45–5.6	100 %	TT					
5.6–6.95	30 %	TT					
6.95–7.1	0 %	SPT	50, N=50+ for 70mm				
7.1–8.0	50 %	TT					
8.0–8.45	0 %	SPT	12, 16, 14, N=30			Medium dense.	
8.45–9.5	47 %	TT					
9.5–9.95	0 %	SPT	12, 22, 29, N=50+				

DATE STARTED: 8/6/11
DATE FINISHED: 9/6/11
LOGGED BY: GJG/DG
SHEAR VANE No:

DRILLED BY: Pro Drill (Auck) Ltd
EQUIPMENT: Edson Tractor Rig
DRILL METHOD: OB and TT
DRILL FLUID: Water and mud
DIAMETER/INCLINATION: - / 90°

COMMENTS:
Co-ordinates from hand held GPS. Most core losses not shown for clarity of log. Core losses assumed to be a result of matrix being washed away during drilling.

Beca

BOREHOLE No: **BH 101**

MACHINE AUGER LOG

PROJECT: DBH Christchurch Earthquake Building Investigation
JOB NUMBER: 5273927
SITE LOCATION: Christchurch
CLIENT: Dept of Building and Housing

CIRCUIT: NZMG
COORDINATES: N 5,742,159.3 m, E 2,480,810.9 m
BOREHOLE LOCATION: Pyne Gould Building
R L:
DATUM: Mean Sea Level

Core recovery	Method	SPT 'N'	USCS	Moisture	Soil / Rock Description	Geological Unit
67 %	TT		GW / GW	M / M	Medium dense, fine to coarse sandy GRAVEL; grey; moist. Gravel: angular to rounded, greywacke.	Springston Formation *(Contd.)*
			SP	M	Very dense, fine to medium SAND, minor fine to coarse gravel; grey; wet. Gravel: subrounded, SW, greywacke.	
			SW	M	Very dense, fine to coarse SAND; grey; moist.	
44 %	SPT	6, 12, 13, N=25	ML	W	Very dense, SILT, minor fine sand; grey; wet; non plastic.	
			SP	W	Very dense, fine to medium SAND, trace silt; grey; wet.	
			SM	M	Medium dense fine to medium SAND; grey, moist. Interbedded with extremely closely spaced, very thin to thinly laminated sub horizontal beds of firm to stiff SILT.	
86 %	TT		SW	W	Medium dense, fine to coarse SAND, some wood fragments; grey; wet.	
			SP	W	Medium dense, fine to medium SAND, minor wood fragments; grey; wet. Organic odour.	
100 %	SPT	5, 8, 12, N=20	SW	W	Medium dense, fine to coarse SAND, trace wood fragments; grey; wet. Organic odour.	
43 %	TT					
			OL	M	Firm, organic SILT, trace clay; brown; moist, low plasticity. Organic odour.	
100 %	SPT	18, 10, N=28	SP	W	Medium dense, fine to medium SAND; grey; wet.	
100 %	TT					
					END OF LOG @ 15 m	

Depth scale (m): 11, 12, 13, 14, 15, 16, 17, 18, 19

DATE STARTED: 8/6/11	DRILLED BY: Pro Drill (Auck) Ltd	COMMENTS:	
DATE FINISHED: 9/6/11	EQUIPMENT: Edson Tractor Rig	Co-ordinates from hand held GPS. Most core losses not shown for clarity of log. Core losses assumed to be a result of matrix being washed away during drilling.	
LOGGED BY: GJG/DG	DRILL METHOD: OB and TT		
SHEAR VANE No:	DRILL FLUID: Water and mud		
	DIAMETER/INCLINATION: - / 90°		

Beca

BOREHOLE No: **BH 102**

MACHINE AUGER LOG

SHEET 1 of 2

PROJECT: DBH Christchurch Earthquake Building Investigation
JOB NUMBER: 5273927
SITE LOCATION: Christchurch
CLIENT: Dept of Building and Housing

CIRCUIT: NZMG
COORDINATES: N 5,742,140 m
E 2,480,811 m
BOREHOLE LOCATION: Pyne Gould Building
R L:
DATUM: Mean Sea Level

Depth (m)	Core Recovery	Method	SPT 'N'	USCS	Moisture	Soil / Rock Description	Geological Unit
0–0.3	40 %	OB				ASPHALT	Fill
0.3–0.6				GM	W	Loosely packed, silty fine to coarse sandy fine to coarse GRAVEL, trace cobble; grey brown; wet. Gravel: subrounded, SW, greywacke.	Fill
0.6–1.0	100 %	OB		SM	S	Loosely packed, silty fine SAND, minor fine to coarse GRAVEL; brown; saturated. Gravel: subangular to subrounded, SW, greywacke.	Fill
1.0–1.2	50 %	OB				CONCRETE; grey with fine to coarse subrounded greywacke gravel.	
1.2–1.5				SW	W	Loose, fine to coarse SAND, trace fine gravel; grey; wet.	
1.5–2.3						Core loss: 1.5-2.25m (0.75m)	
~2.0	22 %	SPT	12, 13, 18, N=31				
2.3–3.3	71 %	TT		SW	W	Medium dense, fine to coarse SAND, some organics; grey; wet. Organics: wood fragments and fibres.	Springston Formation
3.3–3.5				GW	M	Medium dense, fine to medium sandy fine to coarse GRAVEL, trace cobble; grey; moist. Gravel: subrounded, SW, greywacke.	Springston Formation
3.5–4.0	0 %	SN	15, 20, 17, N=37	SP	M	Medium dense, fine SAND, trace silt, trace wood fragments and fibres, trace fine to coarse gravel; grey; moist. Odourous.	Springston Formation
4.0–6.5	57 % / 0 % / 47 %	TT / SN / TT	17, 18, 18, N=36	GW	M	Dense, fine to coarse GRAVEL; grey; moist. Gravel: subangular to subrounded, SW, greywacke. Likely that fines have been washed away during drilling.	Springston Formation
6.5–9.1	0 % / 0 % / 0 % / 42 %	SN / TT / SN / TT	18, 19, 16, N=35; 20, 16, 17, N=33			Core loss: 6.5-9.1m (2.6m)	Springston Formation
9.1–9.35				SW	W	Dense, fine to coarse SAND; grey; wet.	Springston Formation
9.35–10.0	22 %	SPT	18, 21, 17, N=38	GW	M	Dense, fine to coarse GRAVEL, trace cobble; grey; moist. Gravel: subrounded to rounded, SW, greywacke. Likely fines have washed away during drilling.	Springston Formation

DATE STARTED: 10/6/11
DATE FINISHED: 12/6/11
LOGGED BY: GJG
SHEAR VANE No:

DRILLED BY: Pro Drill (Auck) Ltd
EQUIPMENT: Edson Tractor Rig
DRILL METHOD: OB and TT
DRILL FLUID: Water and mud
DIAMETER/INCLINATION: - / 90°

COMMENTS:
0m = top of borehole not top of concrete pad. Co-ordinates from hand held GPS. SN = Solid Nose SPT. Most core losses not shown for clarity of log. Core losses assumed to be a result of matrix being washed away during drilling.

FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS SEE KEY SHEET
Revision A

A4 Scale 1:50

MACHINE_AUGER P:\527\5273927\TGE\FIELD LOGS\LOGS.GPJ BECA.GDT 23/8/11

Beca

BOREHOLE No: **BH 102**

MACHINE AUGER LOG

PROJECT: DBH Christchurch Earthquake Building Investigation
JOB NUMBER: 5273927
SITE LOCATION: Christchurch
CLIENT: Dept of Building and Housing

CIRCUIT: NZMG
COORDINATES: N 5,742,140 m E 2,480,811 m
BOREHOLE LOCATION: Pyne Gould Building
R L:
DATUM: Mean Sea Level

Core recovery	Method	SPT 'N'	Depth (m)	USCS	Moisture	Soil / Rock Description	Geological Unit
42 %	TT		10.0	SW	W	Dense, fine to coarse SAND, trace fine to coarse gravel, trace cobble; grey; wet. Gravel: subrounded to rounded, SW, greywacke.	Springston Formation (*Contd.*)
0 %	SN	12, 15, 18, N=33	11				
42 %	TT		12				
22 %	SPT	12, 13, 18, N=31	12.5	SP	W	Dense, fine to medium SAND; grey; wet.	
0 %	TT		12.7			Core loss: 12.7-14.5m (1.8m)	
22 %	SPT	18, 11, N=11	14				
44 %	TT		14.5	SP	W	Dense, fine to medium SAND; grey; wet.	
			15			END OF LOG @ 15 m	

Depth scale: 11, 12, 13, 14, 15, 16, 17, 18, 19

DATE STARTED: 10/6/11
DATE FINISHED: 12/6/11
LOGGED BY: GJG
SHEAR VANE No:
DRILLED BY: Pro Drill (Auck) Ltd
EQUIPMENT: Edson Tractor Rig
DRILL METHOD: OB and TT
DRILL FLUID: Water and mud
DIAMETER/INCLINATION: - / 90°

COMMENTS:
0m = top of borehole not top of concrete pad. Co-ordinates from hand held GPS. SN = Solid Nose SPT. Most core losses not shown for clarity of log. Core losses assumed to be a result of matrix being washed away during drilling.

Beca

BOREHOLE No: **BH 103**

MACHINE AUGER LOG

PROJECT: DBH Christchurch Earthquake Building Investigation
JOB NUMBER: 5273927
SITE LOCATION: Christchurch
CLIENT: Dept of Building and Housing

CIRCUIT: NZMG
COORDINATES: N 5,742,669 m
E 2,481,312 m
BOREHOLE LOCATION: REHS
R L:
DATUM: Mean Sea Level

Depth (m)	Core Recovery	Method	SPT 'N'	USCS	Moisture	Soil / Rock Description	Geological Unit
0.0						ASPHALT.	
0.2	73 %	OB		GW	M	Loosely packed, fine to coarse GRAVEL, minor sand, trace cobble; grey; moist. Gravel: rounded to subrounded, SW, greywacke.	
1.5				ML	M	Stiff, fine to coarse gravelly SILT; grey brown; moist, non plastic.	
	22 %	SPT	2, 5, 8 N=13	ML	M	Stiff, SILT; greyish brown; moist, non plastic.	
2.0	43 %	OB		ML	M	Firm, fine sandy SILT; grey mottled orange; moist, non plastic.	
2.75				SP	M	Loose, fine to medium SAND, some silt; grey mottled orange; moist; non plastic.	
	67 %	SPT	2, 2, 3 N=5	SP	M	Loose, fine to medium SAND, trace silt, trace gravel; grey; moist.	
3.4	76 %	TT		OL	M	Very soft, organic SILT, minor wood fragments; dark brown; moist, non plastic.	
4.05				OH	M	Soft, SILT, some clay, some organics; grey mottled brown; moist, high plasticity. Organic odour. Organics: leaves, wood fragments.	
				OL	M	Soft, organic SILT; dark brown; moist, non plastic. Organic odour.	
	100 %	SPT	0, 1, 2 N=3	OH	M	Very soft, organic SILT, some clay; brown mottled grey; moist, high plasticity. Organic odour.	
5.0	67 %	TT		OH	M	Very soft, organic SILT, some clay; grey mottled brown; moist, high plasticity. Organic odour. Organics: roots, leaves, wood fragments.	Springston Formation
5.5				PT	M	Very soft, PEAT, some silt; dark brown; moist, non plastic. Organic odour. Peat: wood, leaves, roots.	
				OH	M	Very soft, organic SILT, some clay; grey mottled brown; moist, high plasticity. Organic odour. Organics: roots, leaves, wood fragments.	
	45 %	SPT	0, 0, 0 N=0	MH	M	Firm, SILT, some clay, minor organics; grey mottled brown; moist, high plasticity. Organic odour.	
6.35	90 %	TT				No core: 6.35-7.15m (0.8m)	
7.15				MH	M	Firm SILT, some clay; with pockets (15mm) of fine to medium sand; grey; moist, high plasticity. Organic odour.	
	100 %	SPT	2, 6, 8 N=14	SM	M	Medium dense, silty fine SAND; grey; moist.	
8.0	60 %	TT		ML	M	Firm, fine sandy SILT, trace organics; grey; moist, non plastic. Organic odour.	
8.9	100 %	SPT	2, 6, 10 N=16	SP	M	Medium dense, fine to medium SAND, with pockets (20mm) of silt, trace silt; grey; moist.	
	59 %	TT					

CASING: 3.0m @ 12/6/2011

DATE STARTED: 12/6/11
DATE FINISHED: 13/6/11
LOGGED BY: GJG
SHEAR VANE No:

DRILLED BY: Pro Drill (Auck) Ltd
EQUIPMENT: Edson Tractor Rig
DRILL METHOD: OB and TT
DRILL FLUID: Water and mud
DIAMETER/INCLINATION: - / 90°

COMMENTS:
0m = top of borehole not top of concrete pad. Co-ordinates from hand held GPS. SN = Solid Nose SPT. Most core losses not shown for clarity of log. Core losses assumed to be a result of matrix being washed away during drilling.

Beca

BOREHOLE No: **BH 103**

MACHINE AUGER LOG

PROJECT: DBH Christchurch Earthquake Building Investigation
JOB NUMBER: 5273927
SITE LOCATION: Christchurch
CLIENT: Dept of Building and Housing

CIRCUIT: NZMG
COORDINATES: N 5,742,669 m
E 2,481,312 m
BOREHOLE LOCATION: REHS
R L:
DATUM: Mean Sea Level

Core Recovery	Method	SPT 'N'	Depth (m)	USCS	Moisture	Soil / Rock Description	Geological Unit
59 %	TT			SP	M	Medium dense, fine to medium SAND, with pockets (20mm) of silt, trace silt; grey; moist.	Springston Formation (*Contd.*)
0 %	SN	13, 19, 18, N=37	11				
35 %	TT			GW	M	Medium dense, fine to coarse GRAVEL; grey; moist. fines washed away.	
			12			No core: 11.5-11.8m (0.3m)	
90 %	TT		13	SW	M	Medium dense, fine to coarse SAND, minor fine to coarse gravel; grey brown; moist. Gravel: rounded to subrounded, SW, greywacke.	
100 %	SPT	20, 31, 19, N=50+ for 55mm	14				
						END OF LOG @ 14.45 m	

DATE STARTED: 12/6/11
DATE FINISHED: 13/6/11
LOGGED BY: GJG
SHEAR VANE No:

DRILLED BY: Pro Drill (Auck) Ltd
EQUIPMENT: Edson Tractor Rig
DRILL METHOD: OB and TT
DRILL FLUID: Water and mud
DIAMETER/INCLINATION: - / 90°

COMMENTS:
0m = top of borehole not top of concrete pad. Co-ordinates from hand held GPS. SN = Solid Nose SPT. Most core losses not shown for clarity of log. Core losses assumed to be a result of matrix being washed away during drilling.

Appendix C

Borehole Photos

Christchurch Earthquake Building Investigation

BOX: 1 **DEPTH: 0.0 to 7.3m**

BOX: 2 **DEPTH: 7.3 to 12.65m**

BH100

[5273927]

Machine Borehole Photos

Christchurch Earthquake Building Investigation

BOX: 3 **DEPTH: 12.65 to 15m**

Christchurch Earthquake Building Investigation

BOX: 1 **DEPTH: 0.0 to 0.5m**

BOX: 2 **DEPTH: 0.6 to 5.6m**

Christchurch Earthquake Building Investigation

BOX: 3 **DEPTH: 5.6 to 10.8m**

BOX: 4 **DEPTH: 10.8 to 15m**

Christchurch Earthquake Building Investigation

BOX: 1 **DEPTH: 0.0 to 5m**

BOX: 2 **DEPTH: 5 to 15m**

Christchurch Earthquake Building Investigation

BOX: 1 **DEPTH: 0.0 to 3.95m**

BOX: 2 **DEPTH: 3.95 to 7.95m**

Christchurch Earthquake Building Investigation

BOX: 3 **DEPTH: 7.95 to 12.7m**

BOX: 4 **DEPTH: 12.7 to 14.45m**

BH103

[5273927]

Machine Borehole Photos

Christchurch Earthquake Building Investigation

BOX: 1 **CC1 - CC5**

BOX: 4 **DEPTH: 12.7 to 14.45m**

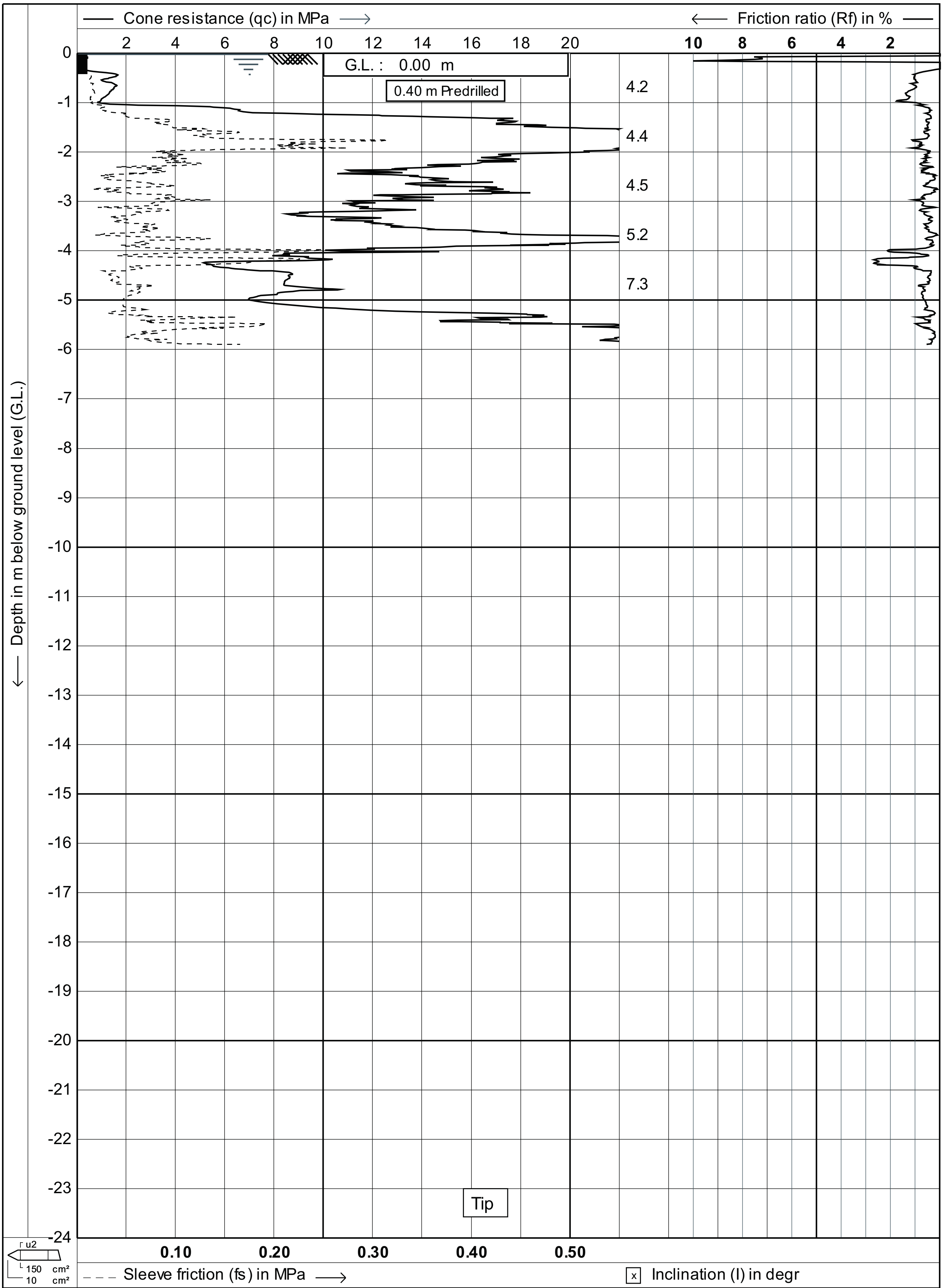
Christchurch Earthquake Building Investigation

BOX: 1 **Depth: 0 to 1.03m**

BOX: **DEPTH: 1.03 to 2.15m**

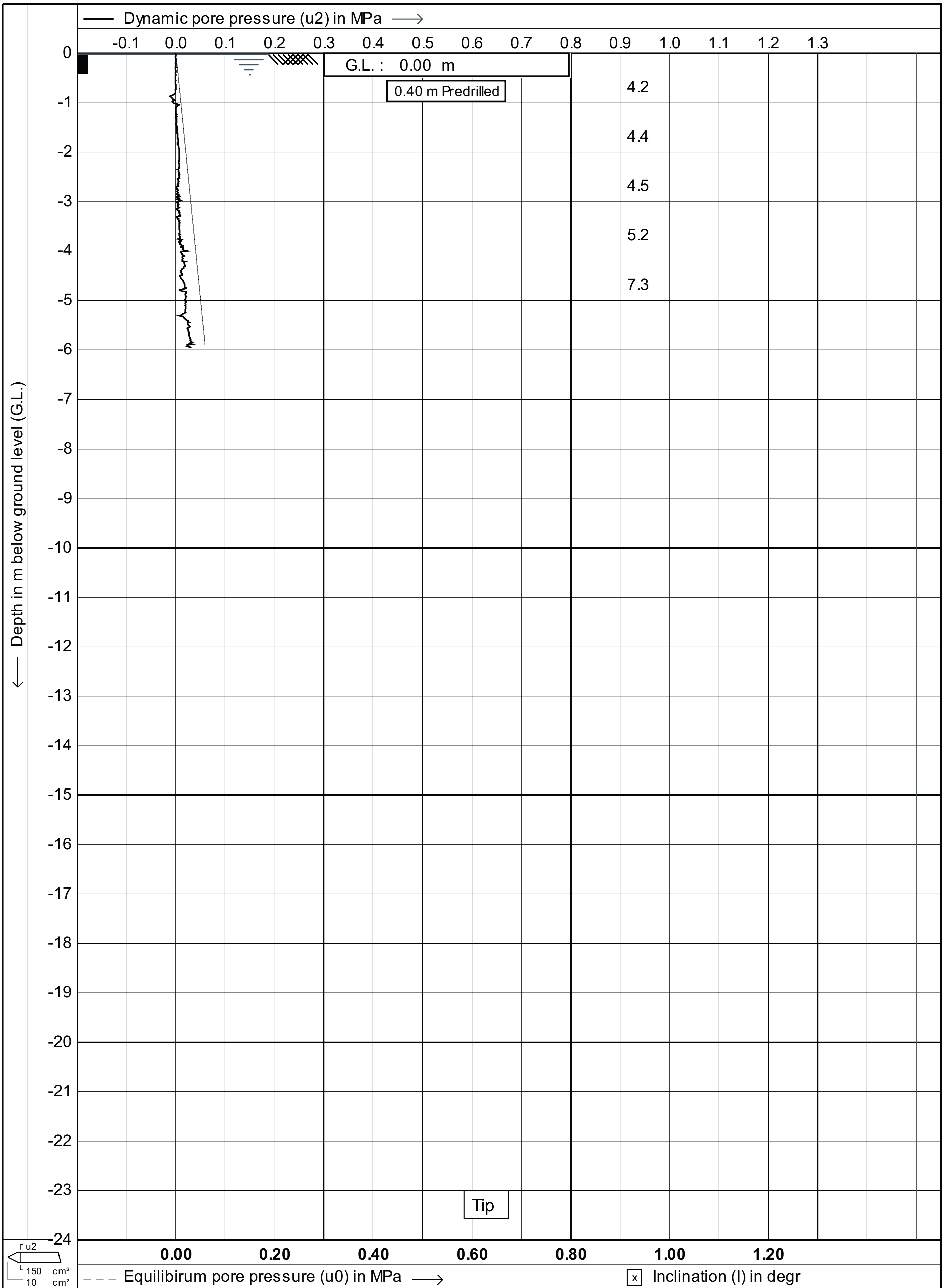
Appendix D

Cone Penetration Test Results



Perry Drilling GEO PROBE Ground Drilling and Environmental Specialists	Test according A.S.T.M. Standard D 5778-07	Date : **9-6-2011**
		Cone no. : **C10CFIIP.F14**
	Project : **Pyne Gould Corporation Investigations**	Project no. : **01BECA5**
	Location: **233 Cambridge Terrace - Christchurch City**	CPT no. : **100** 1/14

CPTask V1.25



CPTask V1.25

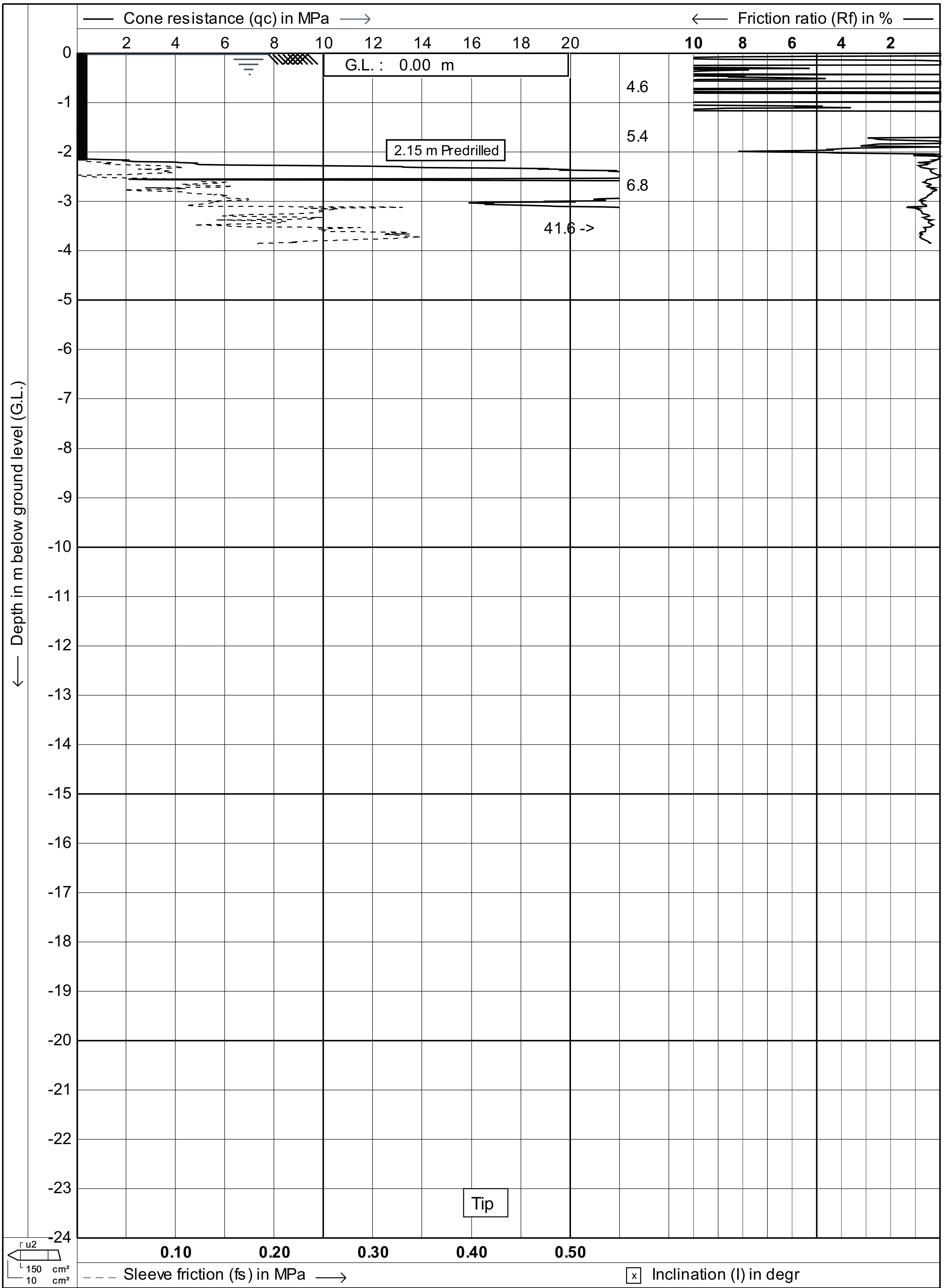
Perry Drilling GEO PROBE Ground Drilling and Environmental Specialists

Test according A.S.T.M. Standard D 5778-07

Project : **Pyne Gould Corporation Investigations**

Location: **233 Cambridge Terrace - Christchurch City**

Date	: **9-6-2011**
Cone no.	: **C10CFIIP.F14**
Project no. :	**01BECA5**
CPT no.	: **100**

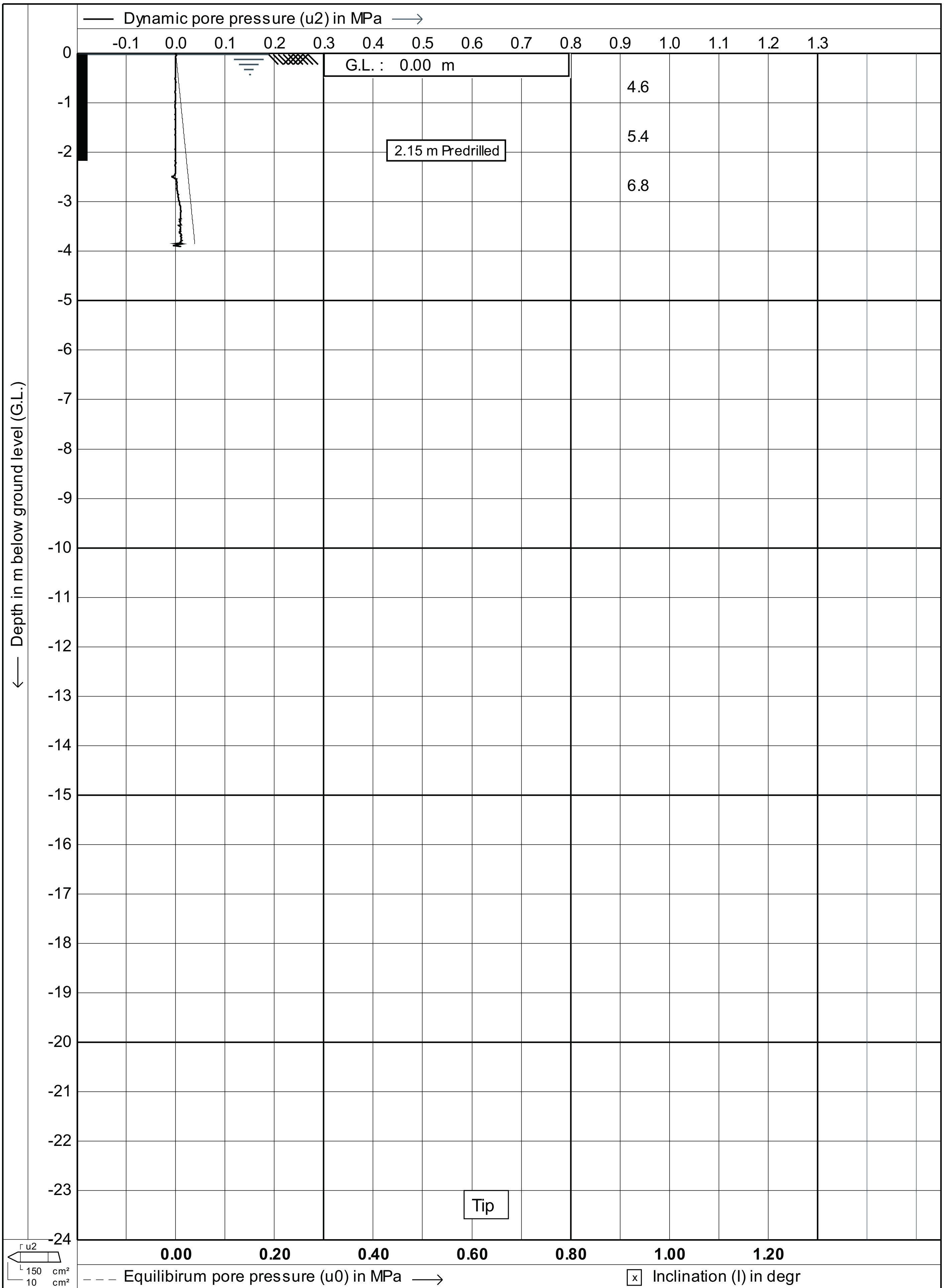


Test according A.S.T.M. Standard D 5778-07

Project :	Pyne Gould Corporation Investigations
Location:	233 Cambridge Terrace - Christchurch City
Date	: 9-6-2011
Cone no.	: C10CFIIP.F14
Project no. :	01BECA5
CPT no.	: 101

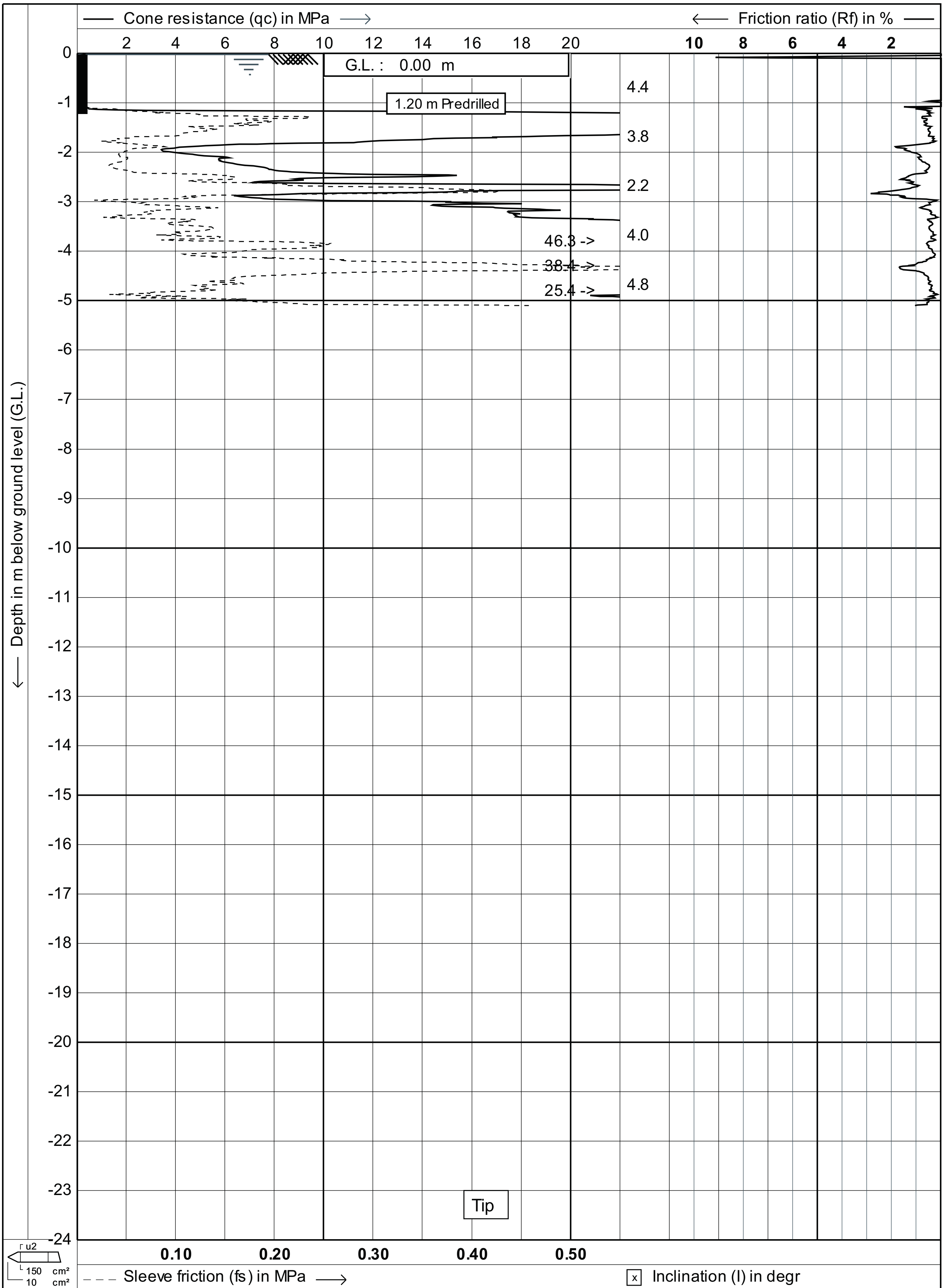
1/14

CPTask V1.25



Perry Drilling / Geo Probe — Ground Drilling and Environmental Specialists	Test according A.S.T.M. Standard D 5778-07	Date : **9-6-2011**
	Project : **Pyne Gould Corporation Investigations**	Cone no. : **C10CFIIP.F14**
		Project no. : **01BECA5**
	Location: **233 Cambridge Terrace - Christchurch City**	CPT no. : **101** — **2/14**

CPTask V1.25

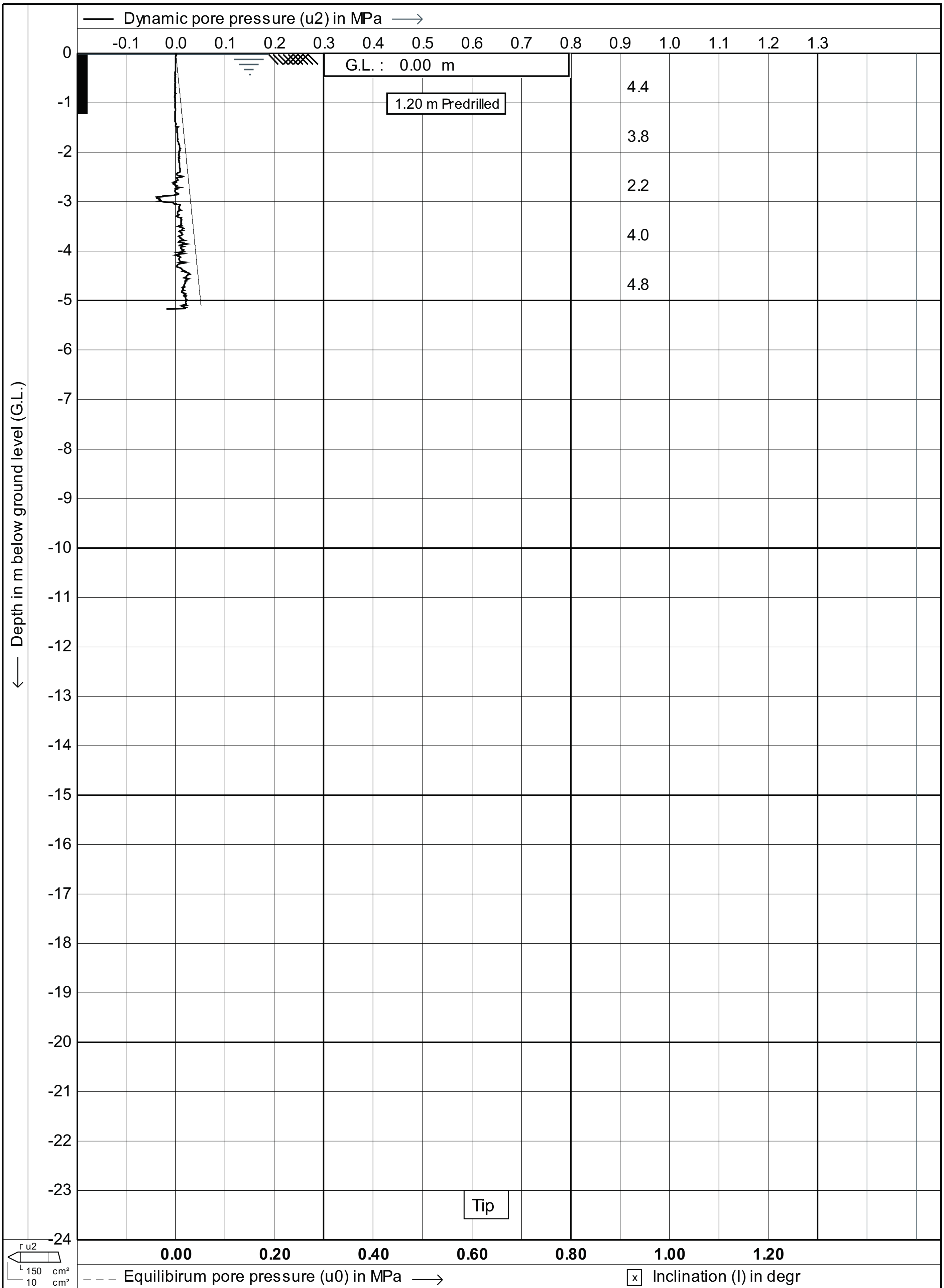


CPTask V1.25

Test according A.S.T.M. Standard D 5778-07

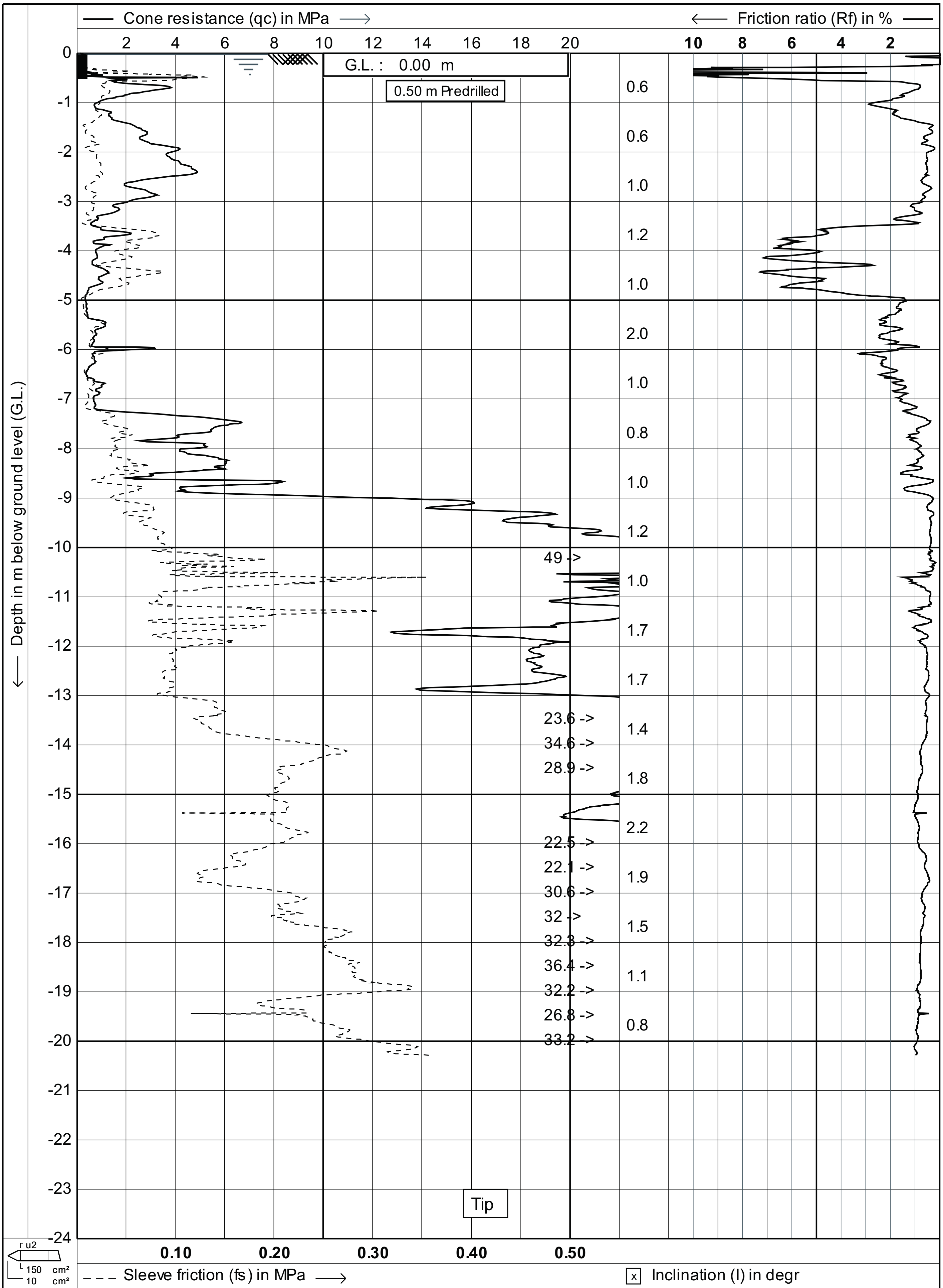
Project :	Pyne Gould Corporation Investigations
Location:	233 Cambridge Terrace - Christchurch City
Date	: 9-6-2011
Cone no.	: C10CFIIP.F14
Project no. :	01BECA5
CPT no.	: 102

1/14



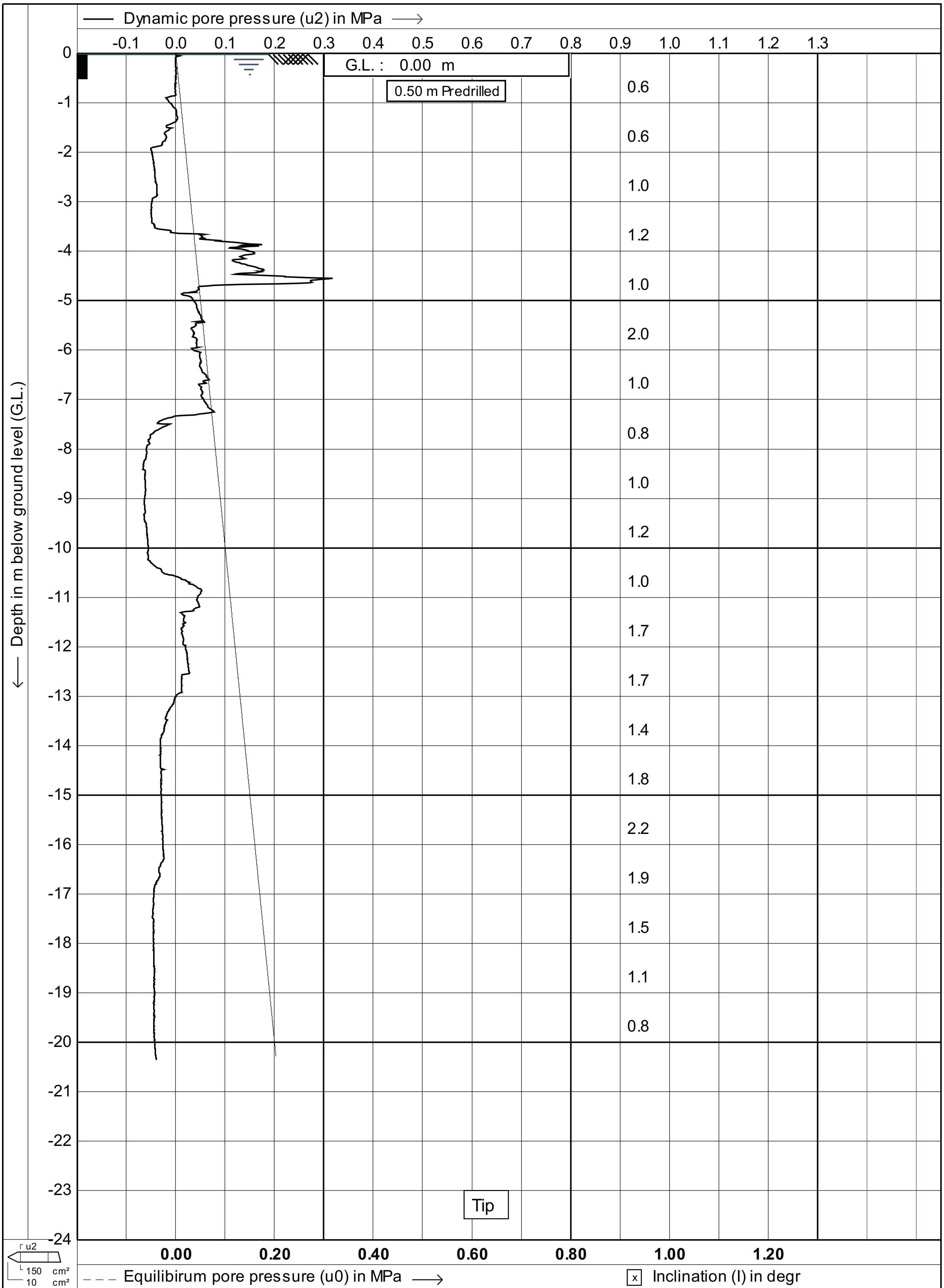
Test according A.S.T.M. Standard D 5778-07	Date : **9-6-2011**	
Project : **Pyne Gould Corporation Investigations**	Cone no. : **C10CFIIP.F14**	
	Project no. : **01BECA5**	
Location: **233 Cambridge Terrace - Christchurch City**	CPT no. : **102**	**2/14**

CPTask V1.25



Test according A.S.T.M. Standard D 5778-07	
Project :	**Site Investigations**
Location:	**REHS Colombo St - Christchurch City**

Date	: **9-6-2011**
Cone no.	: **C10CFIIP.G23**
Project no. :	**01BECA6**
CPT no.	: **103**



CPTask V1.25

Perry Drilling GEO PROBE Ground Drilling and Environmental Specialists

Test according A.S.T.M. Standard D 5778-07

Project : **Site Investigations**

Location: **REHS Colombo St - Christchurch City**

Date	: **9-6-2011**
Cone no.	: **C10CFIIP.G23**
Project no. :	**01BECA6**
CPT no.	: **103**

2/14

Appendix E

Existing Boreholes

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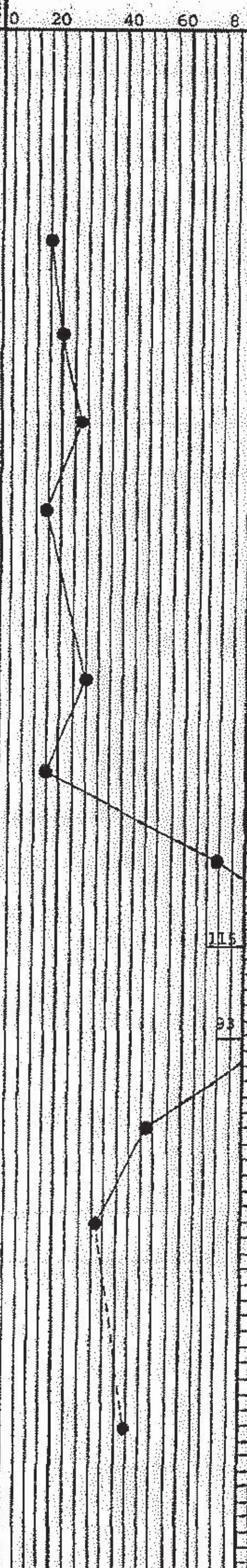
TELEPHONE 798-431 798-432

FEATURE: LANDMARK TOWERS
LOCATION: COLOMBO STREET
ATTITUDE/DIRECTION: VERTICAL

618/CT1
R.L. GROUND(m)
MACHINE: SULLIVAN

GEOLOGICAL UNIT DESCRIPTION	DEPTH (m) below GL	USC SYMBOL	DESCRIPTION OF MATERIAL RECOVERED	DRILL METHOD	DATE/DEPTH m	DATE	WATER LEVEL	CASING (mm)
RECENT	0–2.75		Greyish brown SILTY fine to medium SAND -wet	Cable Tool	3/12/87	5/12/87	▽	150
	2.75–9.75	GW	Grey SANDY fine to coarse GRAVEL -saturated -medium dense -sand-fine to medium -stianed from 2.75m to 3.25m -GRAVELLY fine to coarse SAND from 7.25m to 7.75m -fine gravel -some wood from 8.75m to 9.25m					
	9.75–11	SW	Grey fine to medium SAND -saturated -medium dense -very rare organics @ 10.0m -rare gravel @ 10.0m					
	11–12.25	PT/SW	Greyish brown SANDY PEAT with some silt -saturated -medium dense -sand-fine to coarse		4/12/87			
ALLUVIUM	12.25–20	SW	Grey fine to medium SAND -saturated -dense to very dense -lenses of silt from 11.75m to 12.75m -fine sand from 13.50m to 20.0m -some shells from 17.25m to 20.0m -trace of silt from17.75m to18.0m and 19.25m to 19.50m		5/12/87			
	20–21.75	ML	Grey SANDY SILT -saturated -dense -sand-fine to medium -trace of shells		6/12/87			
	21.75–22.25	GW	Grey SANDY fine to coarse GRAVEL -saturated -dense -sand-fine to medium -lenses of silt @ 22.0m		7/12/87			

PENETRATION (SPT) (uncorrected for overburden)



MOISTURE CONTENT (%)

COMMENTS All gravel subrounded to subangular greywacke

LOGGED MDA
DATE 11/12/87
LENGTH 22.25m
DRILLER CANT. DRILL.
STARTED 3/12/87
FINISHED 7/12/87

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TELEPHONE 798-431, 798-432

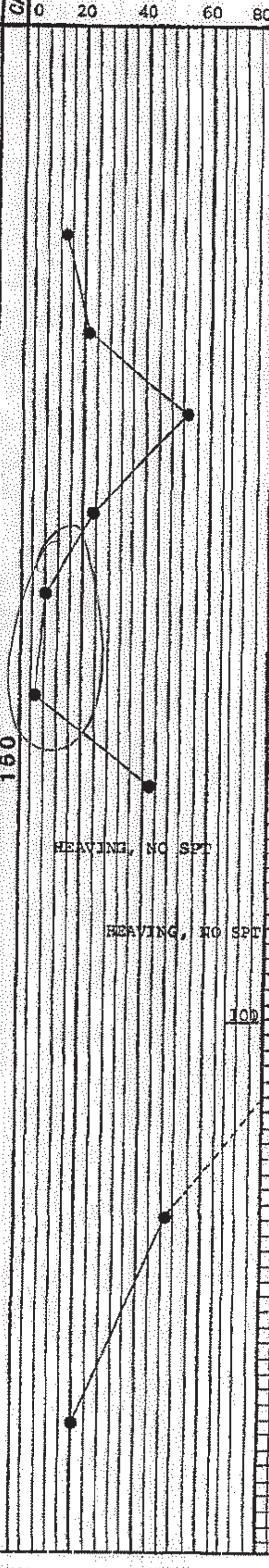
FEATURE. LANDMARK TOWER
LOCATION. ARMAGH STREET
ATTITUDE/DIRECTION; VERTICAL

618/C12
R.L. GROUND(m).
MACHINE; SULLIVAN

GEOLOGICAL UNIT DESCRIPTION	DEPTH (m) below GL	USC SYMBOL	DESCRIPTION OF MATERIAL RECOVERED.
	0–3		
RECENT	3–11	GW	Grey SANDY fine to coarse GRAVEL -saturated -medium dense to dense -sand-fine to medium -stained from 7.0m -GRAVELLY fine to medium SAND @ 8.0m -SAND with rare GRAVEL from 8.75m to 9.5m
	11–12	SW	Grey fine to medium SAND -saturated -dense -stained
	12–15	SW	Grey GRAVELLY fine to medium SAND -saturated -dense -gravel-fine to coarse -lenses of silt @14.0m and 14.50m -wood @ 14.25m
ALLUVIUM	15–20	SW	Grey fine SAND -saturated -dense to very dense -seashells from 18.5m -trace of silt @ 19.5m
	20–22.5	ML	Grey SILT with rare fine SAND -saturated - -medium dense -rare gravel @22.5m -wood @ 20.5m -rare organics from 20.25m to 22.5m

Column headings: CORE LOSS/LIFT % | DEPTH (m) below GL | GRAPHIC LOG | USC SYMBOL | DRILL METHOD | DATE/DEPTH m | SAMPLES AND TESTS | DATE | WATER LEVEL | CASING (mm) | PENETRATION (SPT) (uncorrected for overburden) | MOISTURE CONTENT (%)

DRILL METHOD: Cable Tool
WATER LEVEL: No Water Level Recorded
CASING (mm): 150



COMMENTS All gravel subrounded to subangular greywacke

LOGGED MDA
DATE 21/12/87
LENGTH 22.5m
DRILLER CANT DRILL
STARTED 11/12/87
FINISHED 17/12/87

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FEATURE: LANDMARK TOWERS — 618/CT3

LOCATION: R.O.W. SOUTHEAST CORNER — R.L. GROUND (m)

ATTITUDE/DIRECTION: VERTICAL — MACHINE: SULLIVAN

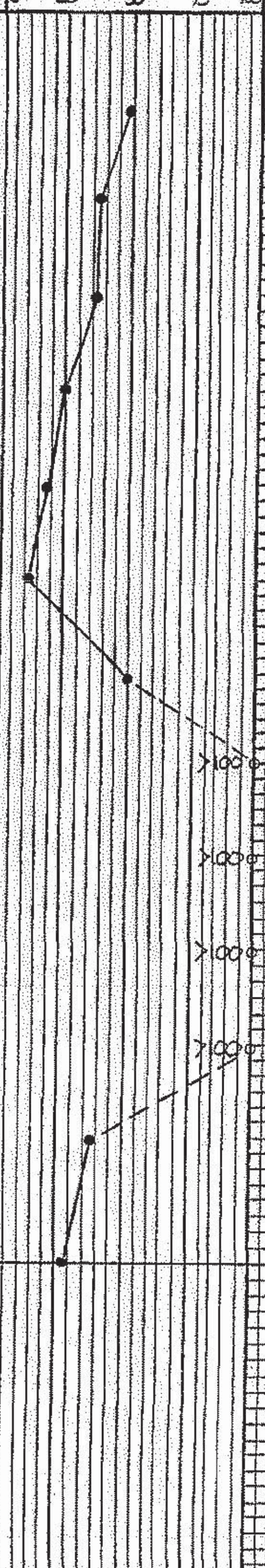
Geological Unit Description	Depth (m) below GL	USC Symbol	Description of Material Recovered
FILL	0 – 0.9		Seal over basecoarse
	0.9 – 1.5	SW	Yellow brown grey fine to medium SAND. - dense, moist
RECENT	1.5 – 10.0	GW	Grey brown sandy fine to coarse GRAVEL. - medium dense, - wet to saturated, - gravel - well graded sub-rounded to sub-angular greywack. - pea gravel 4.25 - 4.75m - wood @ 8.00m
ALLUVIUM	10.0 – 20.3	SW	Grey fine to medium SAND. - very dense, - saturated, - shells 17.25 - 20.00
	20.3 – 21.0		Grey SILT. - wet, - soft to firm

Drill method: Cable Tool. Date/Depth: 5/3/88 (to ~8 m); 6/3/88 (to ~15 m); 12/3/88 (to ~20 m).

Water level: 6/3/88 — 2.5m. Casing: 150mm.

Samples and tests: solid cone penetration tests at ~1.5–11 m; Raymond spoon penetration tests at ~12, 13.5, 15, 16.5, 18, 20.5 m.

PENETRATION (SPT) (uncorrected for overburden): 0, 25, 50, 75, 100



MOISTURE CONTENT (%)

COMMENTS: ⚲ Solid cone penetration test
⧗ Raymond spoon penetration test.

LOGGED M Smith — DRILLER Cant. Drill
DATE 14/03/88 — STARTED 05/03/88
LENGTH 20.0m — FINISHED 12/03/88

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618/CT4

FEATURE. LANDMARK TOWERS
LOCATION. COLOMBO STREET
ATTITUDE/DIRECTION; VERTICAL
R.L. GROUND(m)
MACHINE; SULLIVAN

GEOLOGICAL UNIT DESCRIPTION	DEPTH (m) below GL	USC SYMBOL	DESCRIPTION OF MATERIAL RECOVERED
	0		Excavated
	2	SM	Mottled grey brown silty fine to medium SAND. - wet, - firm.
RECENT	7	GW	Grey sandy fine to coarse GRAVEL. - wet to saturated, - medium dense, - Gravel - well graded fine to medium greywack.
	10	SW	Grey gravelly fine to medium SAND. - saturated, - compact.
	10.5	Pt	Dark brown sandy PEAT.
ALLUVIUM	11	SW	Grey silty fine to medium SAND with trace organics.
	11.5	Pt	Dark brown silty PEAT. - saturated, soft.
	13	SW	Grey silty fine to medium SAND - saturated, - dense, - wood 12.75 - 13.10m

Drill method: Cable Tool
Date/Depth: 7/04/88 8.3m; 8/04/88
Water level: 8/04/88 2.5m
Casing: 150 mm

Other columns: CORE LOSS/LIFT %; GRAPHIC LOG; SAMPLES AND TESTS; PENETRATION (SPT) (uncorrected for overburden); MOISTURE CONTENT (%)

COMMENTS

LOGGED M Smith
DATE 11/04/88
LENGTH 14.5m
DRILLER Canty Drill
STARTED 7/04/88
FINISHED 8/04/88

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618/CT5

FEATURE: LANDMARK TOWERS
LOCATION: COLOMBO STREET
ATTITUDE/DIRECTION: VERTICAL
R.L. GROUND(m):
MACHINE: HYDROMASTER

GEOLOGICAL UNIT DESCRIPTION	DEPTH (m) below GL	USC SYMBOL	DESCRIPTION OF MATERIAL RECOVERED	DRILL METHOD	DATE/DEPTH m	DATE	WATER LEVEL	CASING (mm)
	0		Excavated					
RECENT ESTUARINE SEDIMENTS	0–2.2	SM	Mottled yellow brown silty fine to medium SAND. – moist, – firm.	Highway Auger (to 4.00)	07/04/88 4.00	08/04/88	2.2m	150mm
RECENT	2.2–9	GW	Grey sandy fine to medium GRAVEL. – wet to saturated, – medium dense, – gravel-well graded subrounded to subangular greywack	Cable TOOL				
ALLUVIUM	9–11.8	SW	Grey gravelly fine to medium SAND. – dense to medium dense, – saturated. – timber at 9.75m					
	11.8–12.2	ML OL	Sandy SILT and ORGANIC material.					
	12.2–14	SM SW	Grey silty fine to medium SAND. – saturated, – dense		08/04/88			

Other columns: CORE LOSS/LIFT %; GRAPHIC LOG; SAMPLES AND TESTS; PENETRATION (SPT) (uncorrected for overburden); MOISTURE CONTENT (%)

COMMENTS

LOGGED M Smith
DATE 14/04/88
LENGTH 14.0m
DRILLER Canty Drill
STARTED 07/04/88
FINISHED 08/04/88