Detailed Seismic Assessment Report

Industrial Units
37 Washbournes Road

Prepared for:

MPROP Ltd

By:

Engenium Ltd

Date:

11 July 2023

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Detailed Seismic Assessment Report

Industrial Units

37 Washbournes Road

Prepared by:

Thomas Ryan (CPEng)

On behalf of Engenium Ltd

Reviewed and Approved for Issue by:

Jeremy Mitchell (CPEng)

On behalf of Engenium Ltd

DOCUMENT HISTORY

REVISION	DATE	NOTES	DISTRIBUTION
1	11/07/2023	Client Issue	MPROP

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This report shall not be relied upon until a peer review has been completed on this Detailed Seismic Assessment.

EXECUTIVE SUMMARY

Engenium has been engaged by the MPROP Ltd to undertake a Detailed Seismic Assessment of the buildings at 37 Washbournes Road. The buildings are a complex of single level 100m^2 industrial units, designed in 2008, and are considered to be Importance Level 2 in accordance with the New Zealand Building Code. This report has been prepared to summarise the findings of this assessment. The basis of assessment was the document The Seismic Assessment of Existing Buildings [1] which is a joint document of MBIE, EQC, NZSEE, SESOC and NZGS, and on-site observations on 5 May 2023.

The building structures are comprised of concrete tilt-panels on all four sides of each unit and connected at the corners. The roofs are lightweight metal cladding on steel purlins spanning between concrete walls. The floor is a 120mm thick concrete slab on grade. The walls are supported on shallow concrete posthole footings.

A geotechnical assessment report carried out by Geotech Consulting Ltd, including a liquefaction hazard assessment, has concluded that the buildings may be considered 'structurally dominated' for the purposes of a DSA.

A qualitative assessment has been completed in the form of an Initial Seismic Assessment in accordance with the guidance document [1]. This initial assessment resulted in a seismic lateral load resisting capacity of approximately 75%NBS (IL2).

A quantitative assessment of the building has been completed based on the guidance document. Based on this assessment we have determined that the building has a seismic lateral load resisting capacity of 75%NBS (IL2). This building is therefore rated as Grade B under the NZSEE building grading scheme.

Readers of this report should familiarise themselves with the limitations of its use as described within the report body.

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1 INTRODUCTION

Engenium Limited has been engaged by MPROP Ltd to conduct a Detailed Seismic Assessment (DSA) for the buildings at 37 Washbournes Road, in accordance with the Ministry for Business, Innovation and Employment document *The Seismic Assessment of Existing Buildings* (July 2017) ('*MBIE Guidelines*') [1].

1.1 Scope

The scope of work includes the following:

- Review relevant building information regarding the building's construction.
- Undertake visual site observations.
- Undertake a DSA in accordance with the 2017 MBIE guidelines.
- Prepare a report including the assessed %NBS rating for the building in accordance with the 2017 MBIE guidelines. The guidelines advised for this review includes Section C5 (The Yellow Book) dated 30 November 2018. This section is not to be used for EBP purposes.

1.2 Available Information

The following information relating to the original design has been obtained from Christchurch City Council records:

- Architectural drawings dated 17 December 2008 by Alan Reay Consultants Ltd.
- Structural drawings dated 17 December 2008 by Alan Reay Consultants Ltd.

The following documentation relating to the building following the Canterbury Earthquake Sequence has been made available for this DSA.

- Geotechnical Assessment Report dated 21 June 2023 by Geotech Consulting Ltd.

A selection of the above documentation is included in the Appendices of this report.

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1.3 Limitations

This report has been prepared for the benefit of MPROP Ltd as our client, with respect to the brief given to us. Data and opinions contained in this report may not be used in other contexts or for any other purpose without our prior review and agreement. Reliance on this report by any other parties is entirely at their own risk.

The review of documentation and observations of the building have been limited to structural aspects only. Our observations did not include an assessment of any other elements of the building or services. Items such as fire safety systems, the glazing systems, racking, finishes, suspended ceilings, partitions, tenant fit-out, power, water, sewerage, mechanical services and architectural elements have not been reviewed as part of this evaluation, except where specifically required as part of the DSA procedure. Where obvious damage has occurred or any damage to these elements may be material to the DSA, these may have been commented on.

Our visual observations of the building were limited to a high-level examination of the building where safe and ready access existed at the time. No intrusive examinations, testing, surveying, removal, or destruction of any building elements were undertaken as part of these observations other than as noted in this report. No geotechnical investigations were carried out other than as noted in this report. Seismic events subsequent to our observations may change the assessment contained in this report.

The focus of the DSA procedures, as outlined in the *MBIE Guidelines* is on life safety issues as the primary objective. This means that earthquake scores or ratings are based primarily on life safety considerations rather than damage to the building or its contents. The earthquake rating assigned is, therefore, not reflective of serviceability performance.

Our assessment of the building was based on existing building documentation as noted above and contained in the appendices to this report. Calculations and analyses were carried out as required in accordance with the guidance document [1]. Only seismic load cases have been reviewed.

Reports and documents relevant to the building (as noted above) that have been reviewed as part of this assessment were produced by outside parties. Engenium is not responsible for the information contained within those documents. Where appropriate, Engenium has summarised the information within these documents for inclusion within this report.

This report is necessarily limited in respect of the above and does not address any matter that is not discoverable from such an examination, including any damage or defect in inaccessible places and latent defects. Engenium is not able to give any warranty or guarantee that all possible damage, defects, conditions, or qualities have been identified during the course of this assessment.

The basis of Engenium's advice and our responsibility to our client is set out in the terms of engagement with our client, being the ACENZ Short Form Agreement for Consultant Engagement. The professional services conducted by Engenium were performed in accordance with the standards generally exercised by other professional consultants acting under similar circumstances.

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2 BACKGROUND

This section describes the seismic assessment basis and guidance that has been used in the preparation of this report.

2.1 Detailed Seismic Assessment (DSA) of Existing Buildings

The Seismic Assessment of Existing Buildings (MBIE Guidelines) provides a technical basis for engineers to carry out seismic assessments of existing buildings in New Zealand. The MBIE Guidelines support seismic assessments for a range of purposes, including whether or not a building is earthquake prone in terms of the Building Act 2004 and for property risk identification more generally.

Specifically, the MBIE Guidelines provide the assessment component of the earthquake prone building regulations and EPB Methodology that came into force on 1 July 2017. They must be used for all engineering assessments that territorial authorities use to determine whether or not a building is earthquake prone.

The MBIE Guidelines provide methods for two levels of assessment: Initial Seismic Assessment, for the broad indication of the likely level of seismic performance of a building, and Detailed Seismic Assessment, for a more comprehensive assessment. Both seismic assessment methods rate a building as a percentage of the new building standard applied to an equivalent new building on the same site. For seismic assessment purposes, new building standard refers to the minimum life safety performance requirements of the New Zealand Building Code Clause B1 – Structure.

Further advice for building owners regarding the Earthquake Assessment process is available to view on the EQ-Assess website: *www.eq-assess.org.nz.*

2.2 Earthquake Scores and Ratings

Earthquake scores and ratings are expressed as a percentage of New Building Standard to provide a comparison of the assessed building performance relative to the current New Zealand Building Code (NZBC) requirements for seismic design for a new building. Earthquake scores are assigned to individual aspects of the building, and the earthquake score of the lowest scoring element is the earthquake rating for the building.

Due to design detailing and technological improvements, buildings designed to older codes are unlikely to comply with current codes and standards in every respect. However, an existing building can potentially achieve 100%NBS without being fully compliant with the current codes and standards in every respect if it can be demonstrated that the building is likely to have similar or equivalent seismic capacity / strength to that of a new building.

2.3 Regulatory Requirements

The Building Act 2004 [2] has specific provisions relating to earthquake prone buildings. The primary purpose of this legislation is to reduce earthquake risk in the community. In broad terms, the definition of an 'earthquake prone' building (under the Building Act 2004 as revised in 2016) is a building or part of a building that is likely to collapse in a moderate earthquake (defined as 34%NBS of the current seismic loading standard in the Regulations)

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causing injury, death, or damage to other property. The Building Act stipulates that Territorial Authorities are required to identify, assess, and take action on EPBs.

MBIE has set out the EPB methodology to establish procedures to undertake the TA's role. Seismic assessments must be carried out in accordance with the MBIE Guidelines to fulfil the requirements of the EPB methodology. It is important to note that the TA determines whether the building is earthquake prone in accordance with the Building Act 2004.

The MBIE Guidelines recommend upgrading of buildings as near as reasonably practicable to new building standard and provide an overview of the improvement process and establishment of performance objectives. Existing buildings that can be shown to have an earthquake rating ≥67%NBS can be considered to represent a low to medium risk in regard to life safety.

The following table taken from the guidance document [1] indicates the relative seismic risk of a building based on the assessed %NBS.

Table 2.1: Relative Risk Comparison.

%NBS	Grade	Relative risk	Description
>100	A+	<1 time	Low risk
80-100	A	1-2 times	Low risk
67-80	B	2-5 times	Low risk
33-67	C	5-10 times	Moderate risk
20-33	D	10-25 times	High risk
<20	E	>25 times	High risk

The relative risk comparison is the ratio of probabilities that the ultimate strength will be exceeded in any given period of time.

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3 BUILDING DESCRIPTION

This section describes the structural features of the building based on visual observations of the building and the available documentation.

3.1 Site Description

The site is located at 37 Washbournes Road, Christchurch.

Figure 3.1: Aerial photo of the site indicating the boundary (red), building 1 (yellow), and building 2 (blue).

3.2 Building Form

The buildings are comprised of several single-level industrial units designed by Alan Reay Consultants in 2008. Building 1 (yellow) is comprised of six units each approximately 100m^2. Building 2 (blue) is comprised of nine units each approximately 100m^2. Units are separated by full-height concrete walls. Each unit is rectangular in plan, measuring approximately 14m long by 7.5m wide.

The building would be classified as an Importance Level 2 (IL2) structure in accordance with AS/NZS1170.0 [3].

The buildings are made of lightweight metal roofing, solid concrete walls on all sides of each unit, and concrete floors. The units have a single 3m wide roller door, man door, and two windows in the front.

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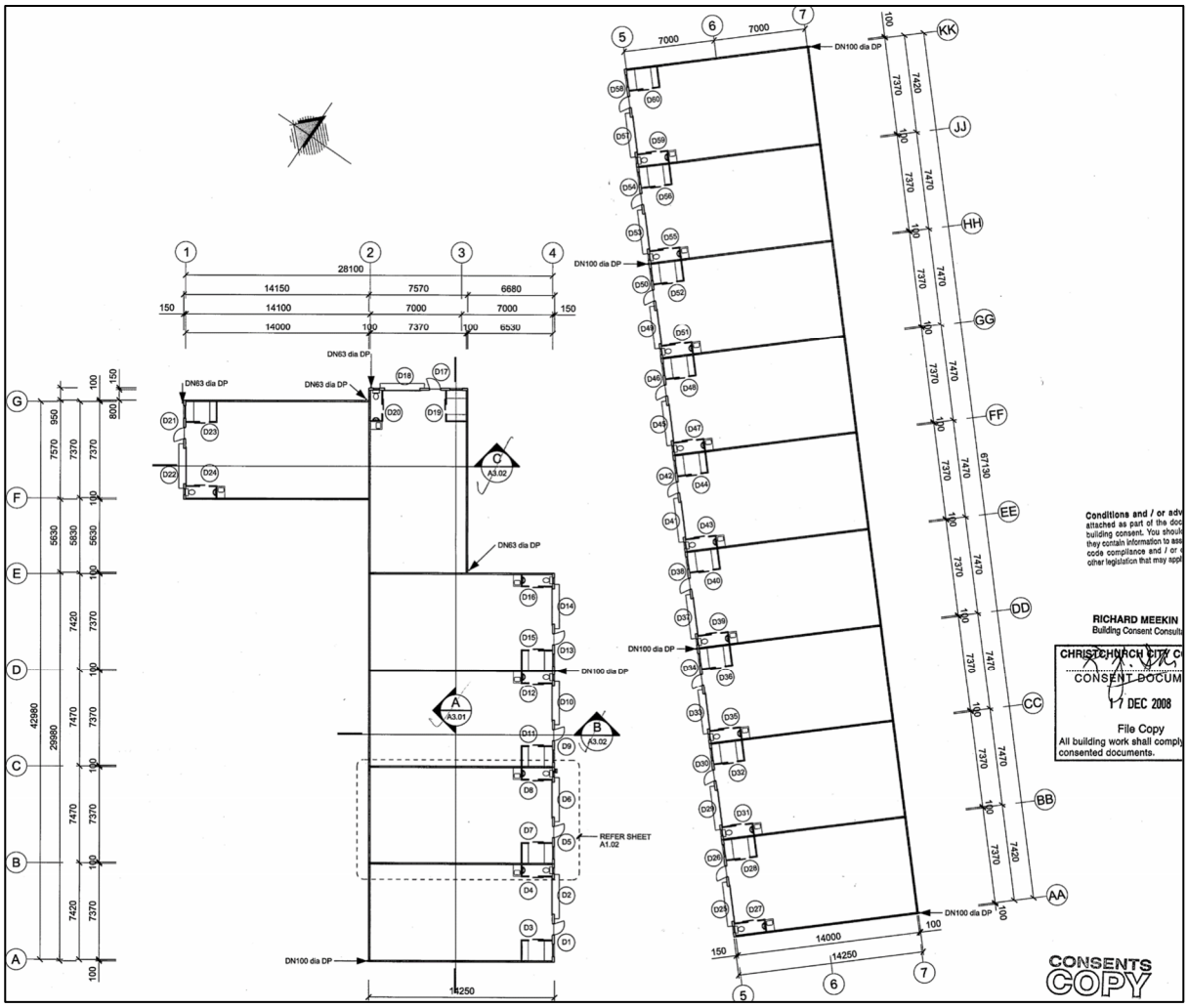

Figure 3.2a: Architectural Ground Floor Plan.

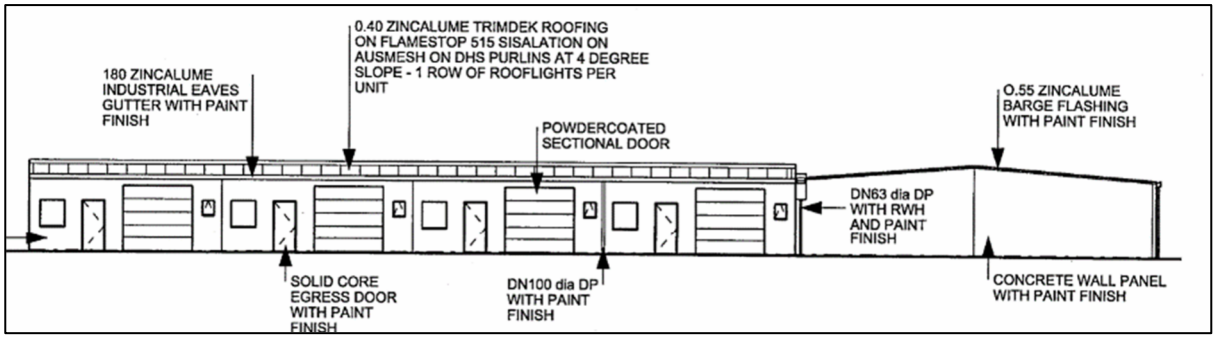

Figure 3.2b: Architectural Elevation of the Building.

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3.3 Gravity Load System

Roof Structure:

- Lightweight metal roofing on steel purlins spanning between precast concrete walls.

Walls:

- Full-height precast concrete tilt-panel walls to the front, sides, and rear of the building and dividing each unit.
- Full-height precast concrete tilt-panel dividing walls between units.

Foundations:

- Unreinforced mass-concrete pad and posthole foundations, shown as 1.0m deep on the drawings, supporting concrete walls and bear onto the underlying soils. The drawings note that the foundations are to extend to solid bearing.

3.4 Primary Lateral Load Resisting System

The primary lateral load resisting systems are similar in both the North-South and East-West directions in the building and are summarised below:

Roof Level:

- There is no bracing in the roof plane, therefore seismic mass is supported on a tributary area basis by the adjacent face loaded concrete walls that support the purlins.

Concrete Walls:

- The concrete walls forming each unit are laterally supported on two or three sides by the concrete floor slab and adjacent return panel at each corner of the building.

Foundations:

- Unreinforced concrete pad and posthole foundations resist lateral loads by passive pressure and ground floor slab friction.

3.5 Secondary Structure and Non-Structural Elements

Timber Framing:

- Several lightweight timber partitions constructed in the units are laterally supported by the concrete walls.

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3.6 Geotechnical Investigations

We have engaged Geotech Consulting Ltd to provide a geotechnical assessment as part of this DSA. The report includes a liquefaction assessment and bearing capacity assessment for the site, based on shallow borehole excavations to a depth of 2.6 m and nearby geotechnical testing.

The report provides probable soil properties and bearing capacities for the foundations in the shallow silts near the ground surface, and notes that the foundations are expected to be sufficiently far above the groundwater level to avoid punching shear failure in the event of liquefaction of an underlying soil layer.

Table 3.6: Foundation Probable Bearing Capacities

Foundation Type	Probable Bearing Capacity
Postholes A, B, & C (0.6 m diameter x 1 m deep)	600 kPa
Postholes D (0.6 m diameter x 0.5 m deep)	300 kPa

The report states that the site has been subject to repeated shaking from the Canterbury Earthquake Sequence and that the site is considered to have been 'sufficiently tested' by two of these events, meaning that the estimated peak ground accelerations exceeded the threshold for Serviceability Limit State shaking. The report notes that there are no reports of ground damage in the area from EQC records and no visible surface liquefaction or significant ground settlement as a result of the previous ground shaking.

On that basis the report has assessed the risk of damaging liquefaction as 'low', and that the buildings are considered 'structurally dominated' for the purposes of a DSA.

3.7 Alterations to the Building Post-Construction

There are several alterations to the building post-construction, including the following:

- Lightweight timber framed partition walls have been constructed in some of the units.
- Lightweight timber framed ceilings over timber framed partition areas.

3.8 Previous Earthquake Assessments

We are not aware of any previous earthquake assessments to these buildings.

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4 STRUCTURAL PERFORMANCE ASSESSMENT

This section describes the seismic assessment carried out by Engenium on the Industrial Units at 37 Washbournes Road.

4.1 Qualitative Assessment Methodology

A qualitative desktop assessment of the building has been undertaken based on the available documentation, using the Initial Seismic Assessment set out in Part B of the Guidelines. This assessment is summarised using the Initial Evaluation Procedure (IEP) template.

Table 4.1: IEP Assessment Assumptions

IEP Item	*Assumption*	*Justification*
Date of Building Design	2008	Drawings
Soil Type	D – Shallow soil	Engineering Judgement
Building Importance Level	Importance Level 2	In accordance with NZS1170.0:2002
Ductility of Structure	Concrete walls & frames μ=1.00	Engineering Judgement, based on details plus MBIE guidance. Non-ductile connections between panels are assumed to govern.
Plan Irregularity Factor, A	Factor A = 1.0	Single level structure, no plan irregularity.
Vertical Irregularity Factor, B	Factor B = 1.0	Single level building.
Short Columns Factor, C	Factor C = 1.0	No short columns.
Pounding Factor, D	Factor D = 1.0	No immediately adjacent buildings.
Site Characteristics Factor, E	Factor E = 1.0	Low liquefaction hazard area, no previous liquefaction observed at or near the site.
F Factor	Factor F = 1.0	Engineering Judgement, based on details plus MBIE guidance.

A copy of the full IEP assessment is included in the appendices to this report. The assessment indicates that the building can achieve a seismic lateral load capacity of approximately 75%NBS (IL2) in both directions. This corresponds to a seismic grade of 'B' as defined by the New Zealand Society for Earthquake Engineering (NZSEE) building grading scheme.

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4.2 Quantitative Assessment Objectives

The objectives of the DSA procedure as set out in the MBIE Guidelines are as follows:

- Assess the level of earthquake shaking at which the ultimate seismic capacity is reached for an existing building and to identify the point at which conditions are met for a significant life safety hazard.
- Provide a procedure that allows different engineers to produce consistent assessments for the same information and briefing.
- Determine an earthquake rating for the building that is more reliable than the rating available from an ISA.
- Provide details on the expected mode of failure (where possible) to assist territorial authorities to make decisions on life safety issues to determine earthquake prone building status.
- Determine whether or not a building is an earthquake risk building (ERB) as defined in the MBIE Guidelines (>67%NBS).

4.3 Quantitative Assessment Methodology

A detailed desktop assessment of the building was undertaken based on available documentation. The assessment was based on the MBIE Guidelines.

The following procedures and assumptions have been used in assessing the building:

- Capacities for primary structural elements have been calculated based on the MBIE Guidelines, using probable material strengths and a strength reduction factor of 1.0.
- Seismic demands were calculated in accordance with NZS1170.5:2004.
- Seismic demands were based on site Subsoil Class D, Importance Level 2, Return Period Factor of 1.
- Seismic weights were assessed based on the structural drawings.
- The primary structural element capacities were compared to the design actions to establish the hierarchy of failure mechanisms and to determine the probable strength of the building at elastic load levels.
- The primary lateral and gravity structure was analysed using a force-based assessment basis as outlined in the MBIE Guidelines, based on elastic or nominally ductile response of the primary building elements, as appropriate. Ductility has been limited to a maximum of 1.25 on the basis that the structural elements of the building are not detailed to sustain higher levels of ductility.
- The capacity of the primary lateral load-resisting systems was factored in each direction to account for the accidental loading eccentricity.
- The maximum displacements of the gravity structure were assessed.
- The roof system was assessed based on an elastic force-based analysis using the Parts and Components provisions of NZS1170.5:2004.

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- Foundation bearing and lateral load-resisting capacities were determined based on typical geotechnical bearing pressures and soil properties of the soil types underlying the site.

4.4 Visual Observations

Visual observations of the building were carried out by Engenium on 5 May 2023. The purpose of these observations was to:

- Verify where possible the extent to which the building is consistent with available documentation.
- Verify where possible the assumptions made in assessing the building's earthquake scores and rating.

Further to the limitations outlined in Section 1.3, the scope of the observations was as follows:

- Visual review of exposed external structural elements from ground level, where not concealed by linings.

A selection of photographs representative of our site observations is presented in Table 4.1 below. Plan locations of these items are shown in sketch form in the appendices to this report.

Our observations on site are summarised below:

- The building appears to have generally been constructed in accordance with the structural drawings.
- There are several additions to the building as described in section 3.7.
- There does not appear to be any existing earthquake damage to the building. No significant cracking to concrete elements (greater than hairline cracking) was observed in the building, other than those identified in the photos in Table 4.4.

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Table 4.4: Engenium Site Observations.

Element and Description	Photographs
Photo 1: Typical Front Façade Wall	
Photo 2: Typical Side / Rear External Walls	
Photo 3: Typical Warehouse Interior	

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Photo 4: Typical Timber Framed Office	
Photo 5: Cracking Observed at Re-Entrant Corner of Doors.	
Photo 6: Typical Timber Framing Extension Including Plywood Ceiling.	

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Photo 7: Typical Full-Height Timber Partition.	
Photo 8: Typical Single Panel Connector	
Photo 9: Typical Double Panel Connector	

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5 DETAILED SEISMIC ASSESSMENT RESULTS

5.1 Primary Lateral Structure

The primary lateral load resisting system of the building, consisting of reinforced concrete walls, has been assessed as having an earthquake score of 75%NBS (IL2), based on a maximum ductility of μ=1.

The primary lateral load resisting system was assessed for earthquake loading in each horizontal direction using a force-based approach based on the structure remaining at low levels of ductility as outlined in the MBIE Guidelines.

In both directions the building lateral load resisting capacity is limited by the connectors between concrete panels which are inherently non-ductile, and by the capacity of wall panels to sustain face loads.

5.2 Primary Gravity Structure

There are no primary gravity structural elements fully independent of the primary lateral structure. Concrete wall panels are supported directly on concrete posthole foundations and are stabilised by panel-to-panel connections at the corners of the units.

5.3 Foundations

The foundations below the primary lateral load resisting system have been assessed using an elastic force-based analysis. Shallow concrete posthole foundation capacities have been assessed in accordance with section C4 of the MBIE Guidance. The foundations have been assessed as having an earthquake score of 100%NBS (IL2), based on probable geotechnical bearing strength calculations.

5.4 Roof Structure

The roof structure has been assessed in accordance with the Parts and Components provisions of NZS1170.5:2004 using an elastic force-based analysis. It was determined that the earthquake score of the roof structure 80%NBS (IL2), limited by the capacity of the side walls of the building to support the roof structure out-of-plane.

5.5 Secondary and Non-Structural Elements

Under section C10 of the MBIE Guidelines, building elements which do not form part of the primary lateral or gravity load-resisting systems are to be assessed in accordance with the MBIE Guidelines and assigned an earthquake score. These elements are referred to as Secondary Structure and Non-Structural (SSNS) elements. The objective is to determine whether these elements represent a significant life safety hazard or may damage adjacent property, should their capacity be exceeded. It is not intended to assess the risk of damage to the elements themselves or their effects on building operation.

The secondary and non-structural elements that have been identified in these buildings are:

- Lightweight timber framing partition walls
- Lightweight timber framed ceilings

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- Lightweight shelving installed by building occupants.

None of the above secondary and non-structural building elements are considered to represent a significant life safety hazard and have not been specifically assessed as part of this report.

5.6 Assessment Findings Summary

Key findings of the detailed seismic assessment of the building are as follows:

- The building has been assessed as having an earthquake score of 75%NBS (IL2).
- The primary lateral load-resisting system in both directions has been assessed as having an earthquake score of 75%NBS (IL2). The assessment limited the ductility of the primary lateral load resisting system to 1.
- The foundations have been assessed as having an earthquake score of 100%NBS (IL2).

A detailed breakdown of the components is given in the following tables.

Table 5.6a Detailed %NBS Score by Building Element.

System	Element	%NBS (IL2)
Concrete walls	Front walls out-of-plane	95%
	Side walls out-of-plane	80%
	Rear walls out-of-plane	100%
	External corner panel to slab connectors	75%
	External corner panel to panel connectors	80%
Foundations	Bearing Capacity	100%
Primary Lateral Load Resisting Structure	**Lowest Scoring Element**	**75%**

Table 5.6b Summary of SSNS Element Assessment

Elements Included	Mode of Failure	%NBS (IL2)
None	N/A	N/A

Table 5.6c: SSNS Elements Excluded from Assessment.

Elements Excluded	Justification for Excluding
Glazing system	Toughened glass – not a significant life safety hazard.
Suspended ceiling	Lightweight ceilings above Class II or III space, as defined in Table A4.1 and A4.2 of MBIE Guidelines.
Mechanical services	Lightweight units located on the roof are not a significant life safety hazard.
Lightweight internal partitions	<25kg/m^2 walls – not a significant life safety hazard.

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6 CONCLUSIONS

Engenium has carried out a seismic assessment of the building at 37 Washbournes Road in accordance with the MBIE Guidelines (July 2017). The conclusions of this assessment are as follows:

- A quantitative Detailed Seismic Assessment of the building identified an earthquake rating for the building of 75*%NBS* (IL2).
- Based on this assessment, the building is not categorised as an Earthquake Risk Building, as its earthquake score exceeds the criteria that could classify it as an Earthquake Prone Building. It is important to note that only the Territorial Authority can determine whether the building is earthquake prone in accordance with the Building Act 2004.
- The earthquake rating of 75*%NBS* (IL2) corresponds to a grade of 'B', "Low risk Building" as defined by the NZSEE grading scheme.
- The primary lateral load-resisting system has been assessed as having an earthquake score of 75*%NBS* (IL2) in both horizontal directions.
- The foundations were assessed as having an earthquake score of 100%NBS (IL2) in lateral load-resisting capacity.
- A geotechnical assessment report carried out as part of this DSA concludes that the buildings are 'structurally dominated' in accordance with the NZSEE guidelines.
- There are no secondary and non-structural building elements which are considered to represent a significant life safety hazard which are required to be assessed under the MBIE Guidelines.

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7 REFERENCES

1. Ministry of Business, Innovation and Employment, the Earthquake Commission, the New Zealand Society for Earthquake Engineering, the Structural Engineering Society, and the New Zealand Geotechnical Society: *The Seismic Assessment of Existing Buildings,* July 2017

2. New Zealand Legislation, *Building Act 2004,* March 2012

3. Standards New Zealand; *NZS1170.1-5 Structural Design Actions Set*

4. Ministry of Business, Innovation and Employment, the Earthquake Commission, the New Zealand Society for Earthquake Engineering, the Structural Engineering Society, and the New Zealand Geotechnical Society: *Concrete Buildings – Technical Proposal to Revise the Engineering Assessment Guidelines,* November 2018 (*The Yellow Book*).

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APPENDIX A
Existing Drawings

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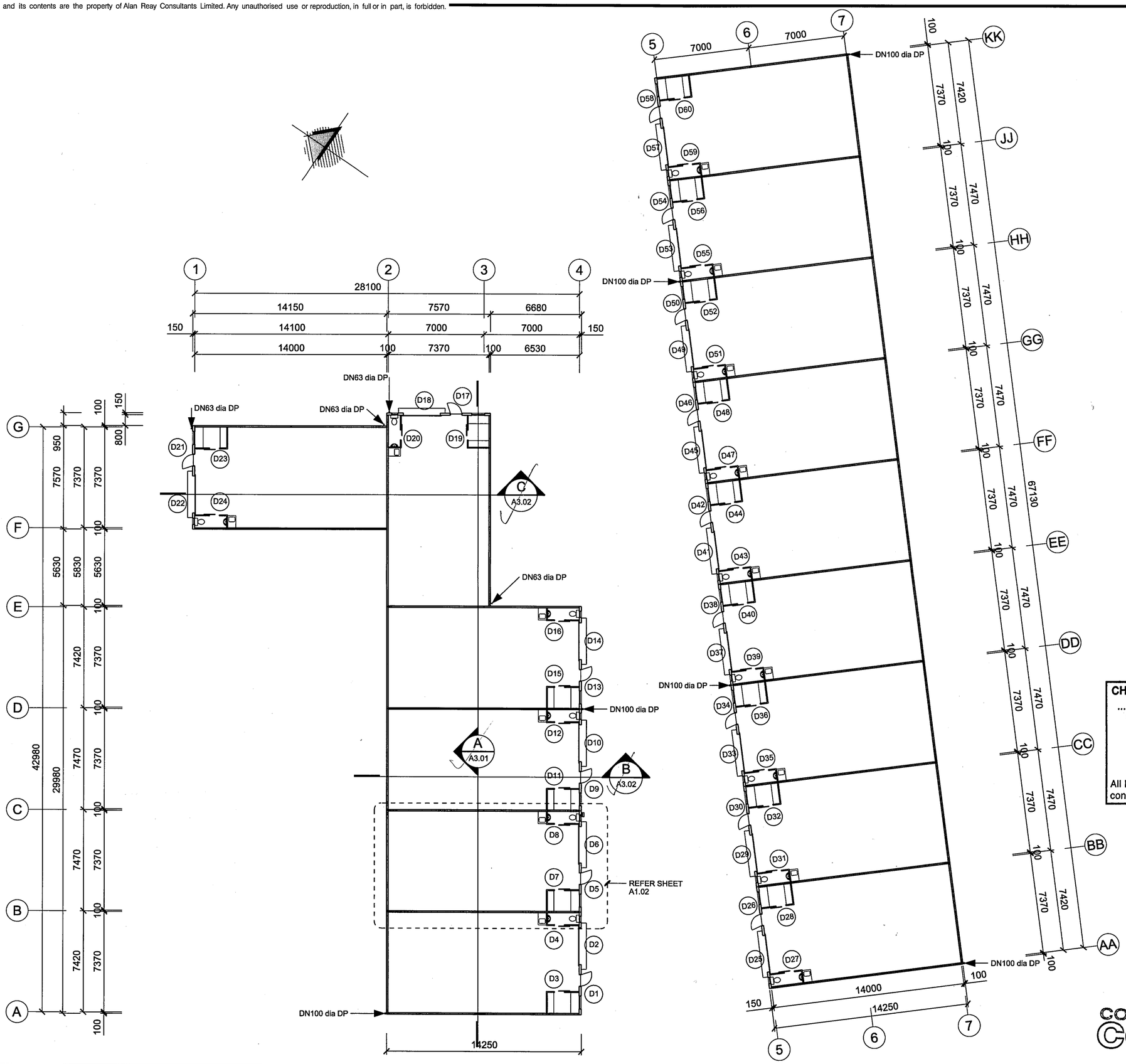

Alan Reay Consultants

395 MADRAS STREET
P.O. BOX 3911
CHRISTCHURCH

TEL: (03) 366 0434
FAX: (03) 379 3981
EMAIL: eng@arcl.co.nz

Conditions and / or advice notes are attached as part of the documents for this building consent. You should read these as they contain information to assist with achieving code compliance and / or compliance with other legislation that may apply

BCP

RECEIVED
26 SEP 2008
10089011
EPA & IE UNITS

RICHARD MEEKIN
Building Consent Consultant

CHRISTCHURCH CITY COUNCIL
CONSENT DOCUMENT
17 DEC 2008
File Copy
All building work shall comply with the consented documents.

ISSUE	DESCRIPTION
A	CONSENT ISSUE

PROJECT NAME:
DEVELOPMENT WASHBOURNES ROAD

DRAWING NAME:
OVERALL FLOOR PLAN

SCALES AT A2	DRAWING No.	ISSUE	FILE
1:200	A1.01	A	8740

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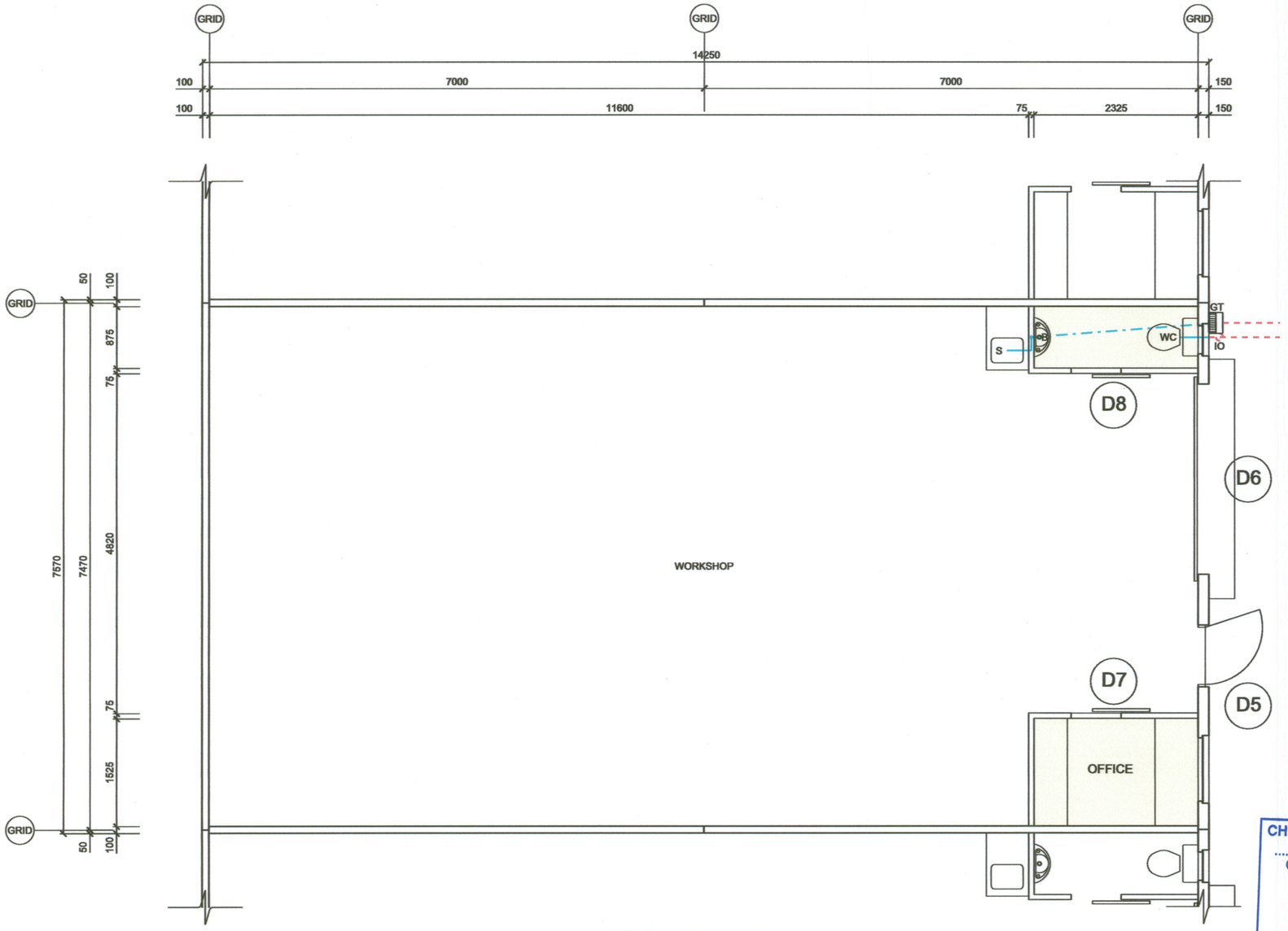
TEL: (03) 366 0434
FAX: (03) 379 3981
EMAIL: eng@arcl.co.nz



TYPICAL FLOOR PLAN

PLUMBING LEGEND:

- DRAIN BY DRAINLAYER
- SANITARY PLUMBING UNDER FLOOR BY PLUMBER
- SANITARY PLUMBING ABOVE FLOOR BY PLUMBER

ALL MATERIALS, WORKMANSHIP & DESIGN OF SANITARY PLUMBING & DRAINAGE TO AS/NZS 3500 2.2: 2003 UNLESS NOTED OTHERWISE ON THIS DRAWING.

		PIPE SIZE	MIN. FALL
B	BASIN	DN40	1:40
S	SINK	DN50	1:40
WC	TOILET - S TRAP	DN100	1:60
GT	GULLY TRAP		
IO	INSPECTION OPENING		
HWC		DN20	DISCHARGE TO BASIN ABOVE P-TRAP

CEILING LEGEND:

13mm GIB LINING - 2420 AFL

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25 SEP 2008
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ISSUE	DESCRIPTION
A	CONSENT ISSUE

PROJECT NAME:
DEVELOPMENT WASHBOURNES ROAD

DRAWING NAME:
TYPICAL FLOOR PLAN

SCALES AT A2	DRAWING No.	ISSUE	FILE
1:50	A1.02	A	8740

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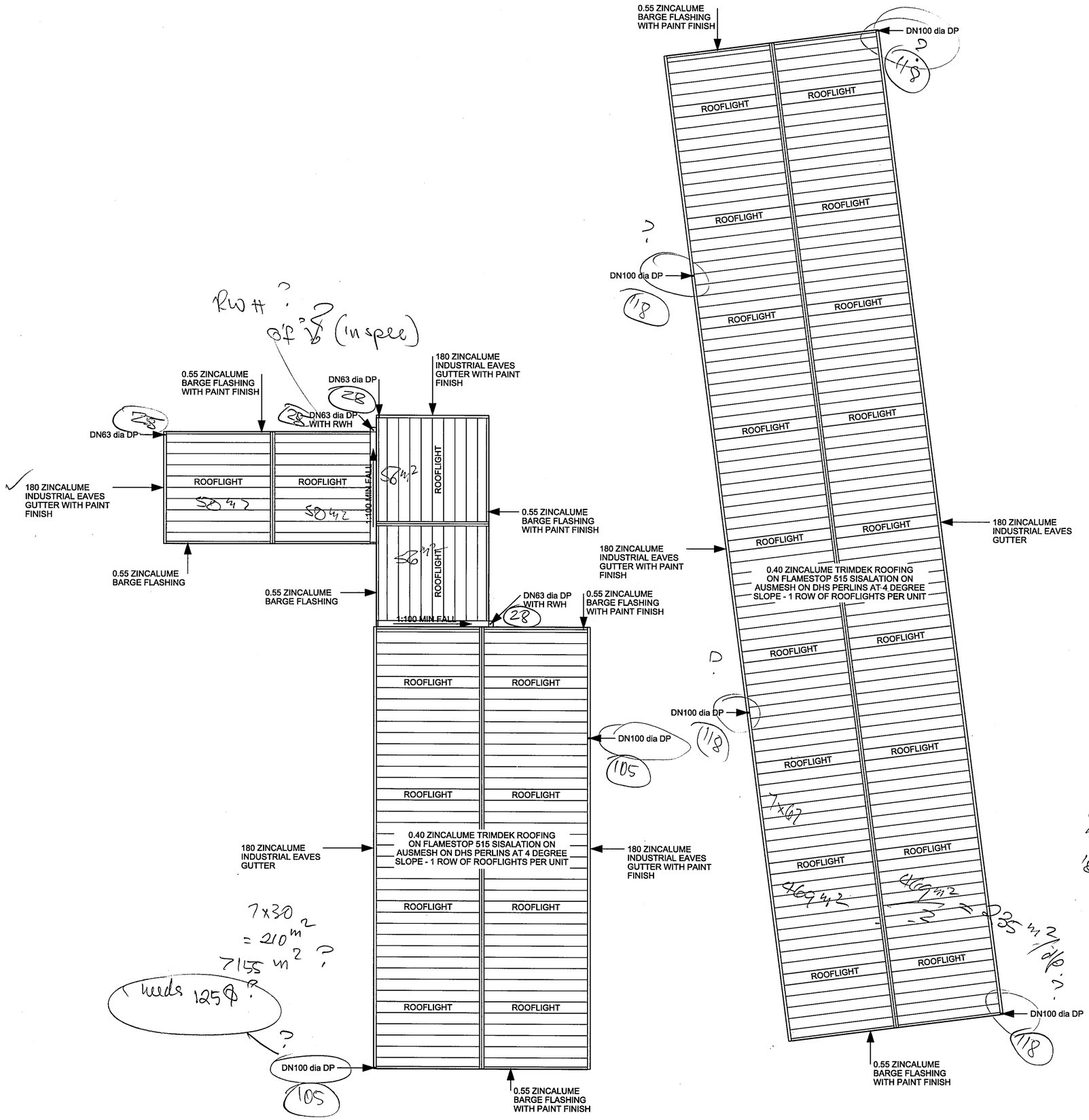
395 MADRAS STREET
P.O. BOX 3911
CHRISTCHURCH

TEL: (03) 366 0434
FAX: (03) 379 3981
EMAIL: eng@arcl.co.nz



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gutter 235 m2
needs 20,050 mm2
180w = 111 mm depth
<125mm OK
needs - 135Ø ?
seen 150 ?

ISSUE	DESCRIPTION
A	CONSENT ISSUE

PROJECT NAME:
DEVELOPMENT WASHBOURNES ROAD

DRAWING NAME:
ROOF PLAN

SCALES AT A2	DRAWING No.	ISSUE	FILE
1:200	A1.03	A	8740

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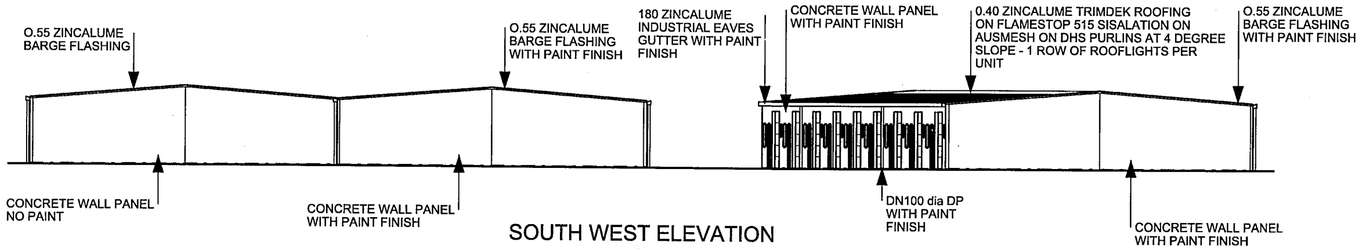

SOUTH WEST ELEVATION

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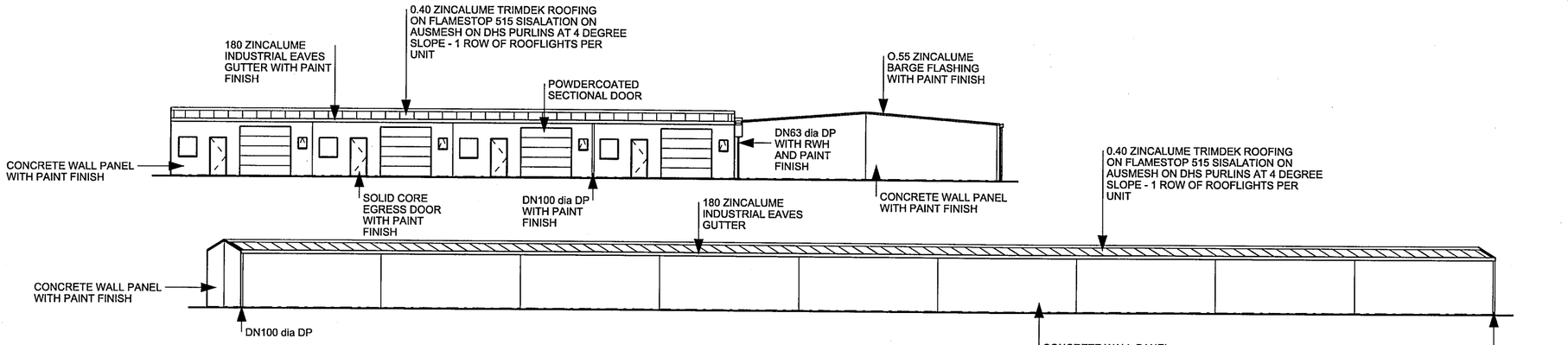
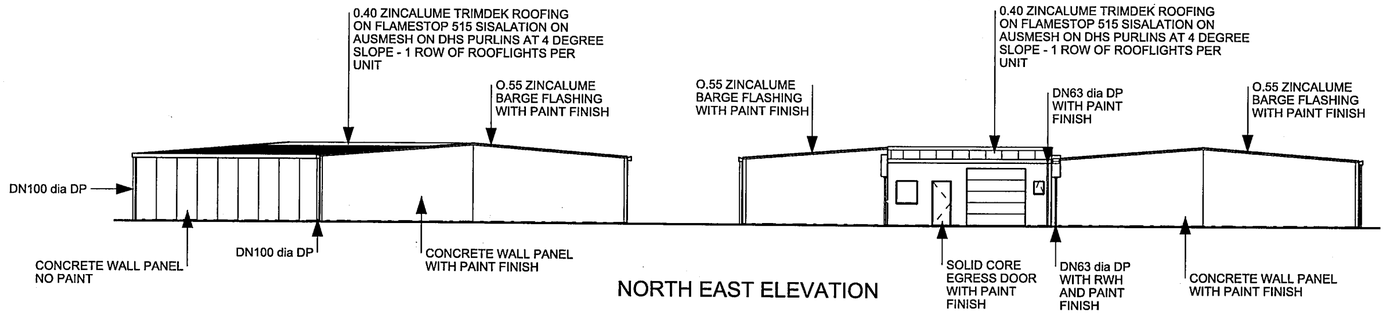
FAX: (03) 379 3981
EMAIL: eng@arcl.co.nz



SOUTH EAST ELEVATIONS



NORTH EAST ELEVATION

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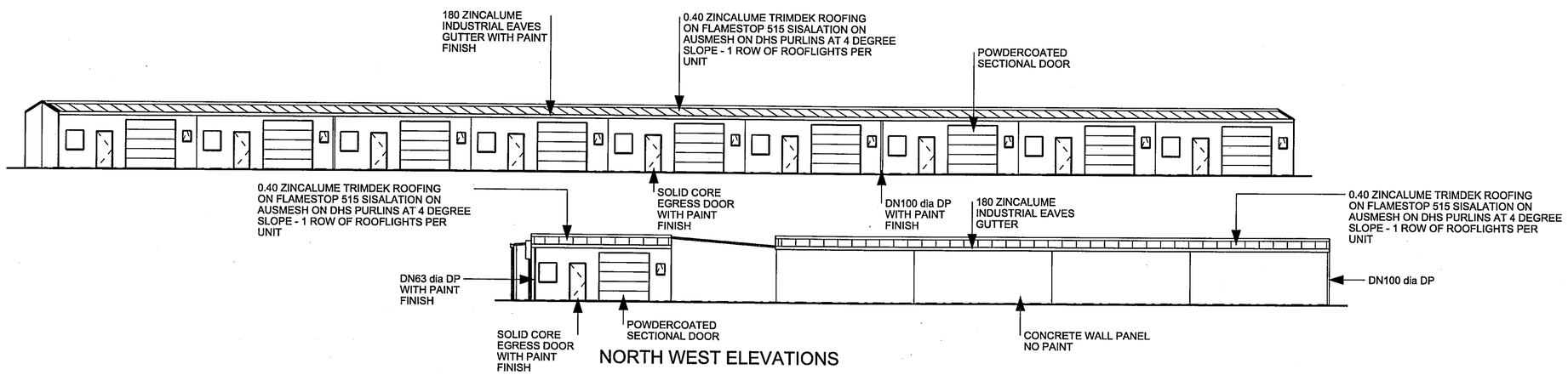

NORTH WEST ELEVATIONS

ISSUE	DESCRIPTION
B	REFER ENGINEERS DRAWINGS DELETED FROM PANEL NOTES - 11.12.08
A	CONSENT ISSUE

PROJECT NAME:

DEVELOPMENT WASHBOURNES ROAD

DRAWING NAME:

ELEVATIONS

SCALES AT A2	DRAWING No.	ISSUE	FILE
1:200	A2.01	B	8740

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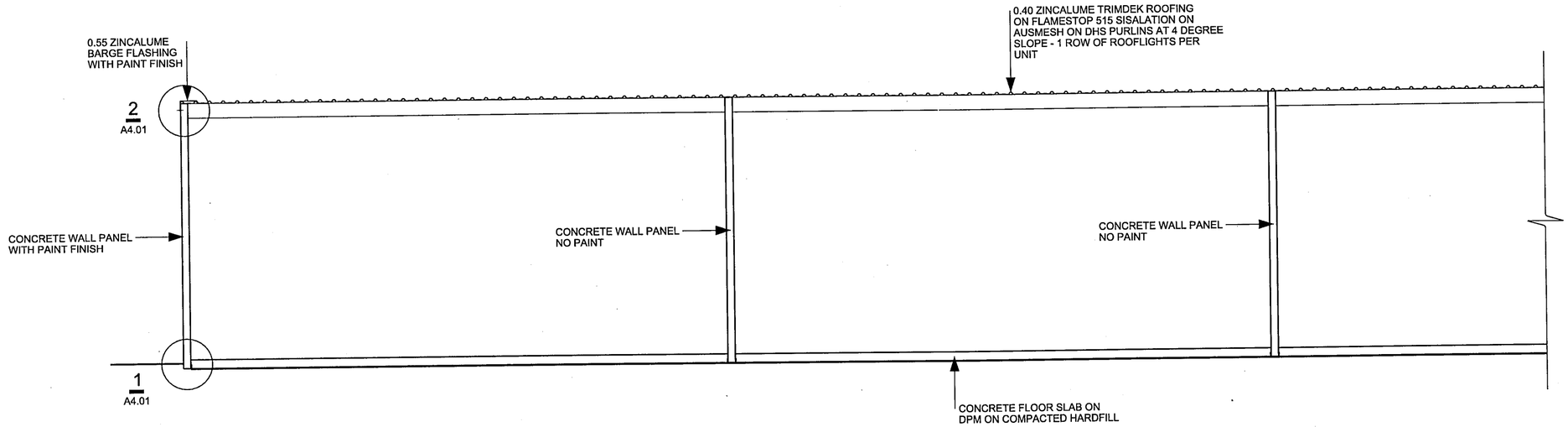
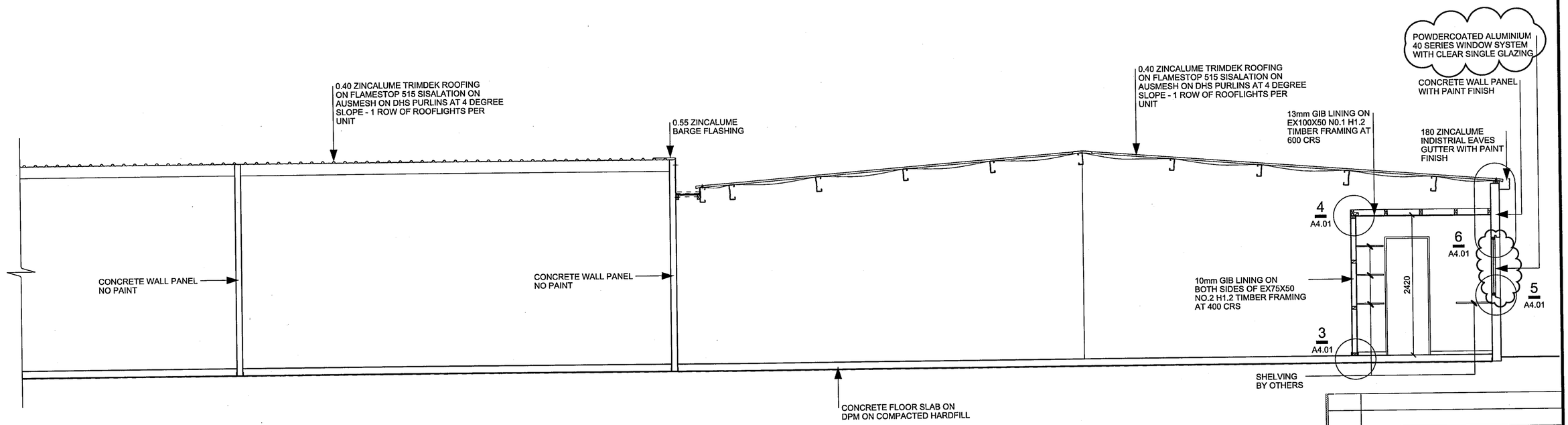
FAX: (03) 379 3981
EMAIL: eng@arcl.co.nz





A / A1.01 SECTION

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ISSUE	DESCRIPTION
B	ALUMINIUM JOINERY ADDED, REFER ENGINEERS DRAWINGS DELETED FROM PANEL NOTE -11.12.08
A	CONSENT ISSUE

PROJECT NAME:
DEVELOPMENT WASHBOURNES ROAD

DRAWING NAME:
CROSS SECTION A

SCALES AT A2	DRAWING No.	ISSUE	FILE
1:50	A3.01	B	8740

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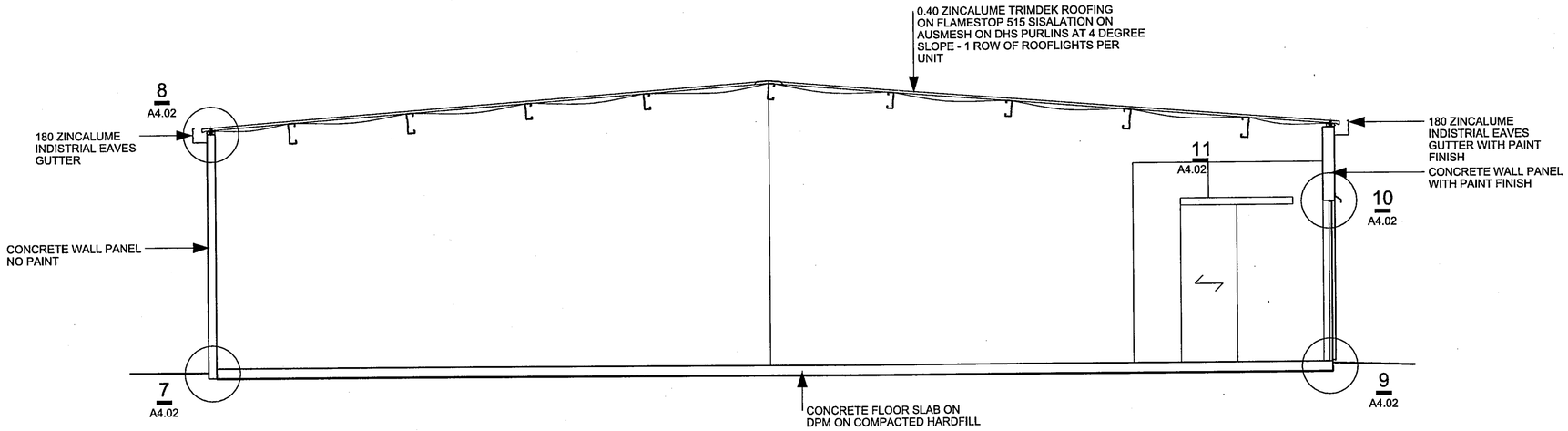
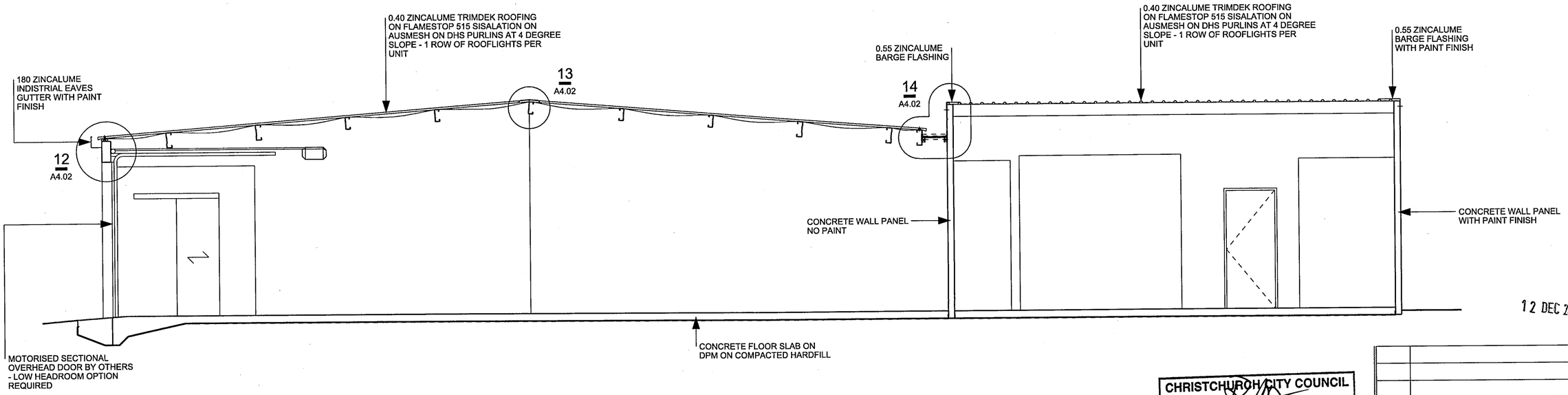


B / A1.01 SECTION



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C / A1.01 SECTION

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ISSUE	DESCRIPTION
B	REFER ENGINEERS DRAWINGS DELETED FROM PANEL NOTES - 11.12.08
A	CONSENT ISSUE

PROJECT NAME:
DEVELOPMENT WASHBOURNES ROAD

DRAWING NAME:
CROSS SECTIONS B & C

SCALES AT A2	DRAWING No.	ISSUE	FILE
1:50	A3.02	B	8740

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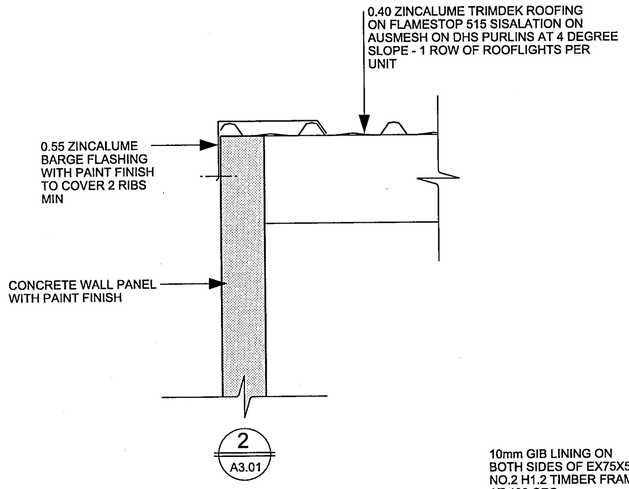
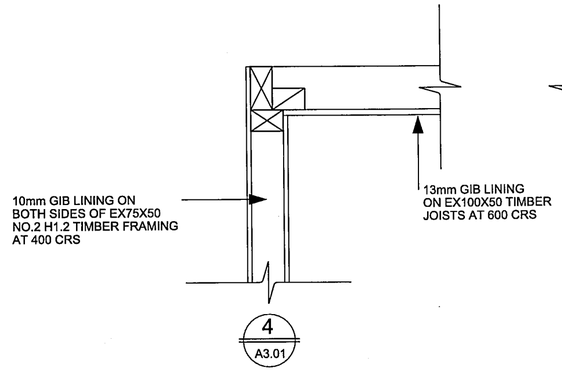
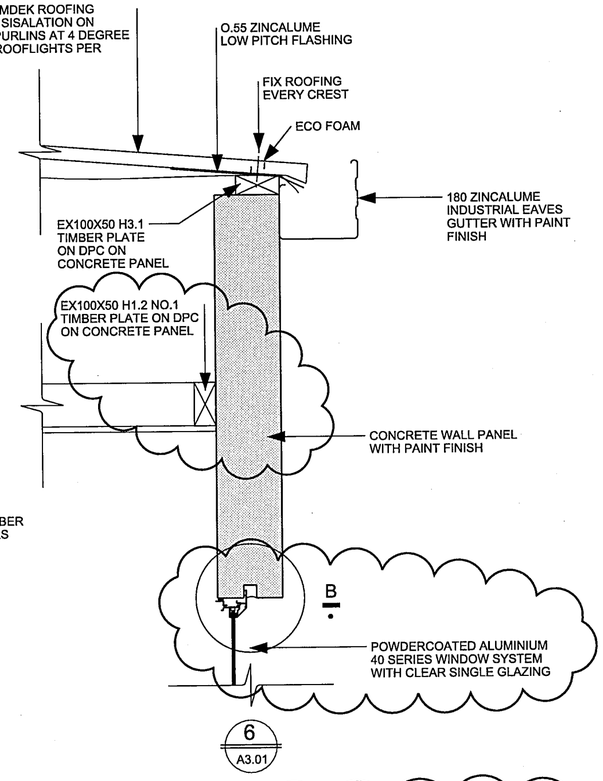
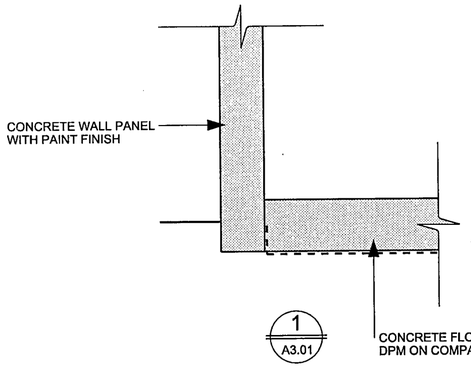
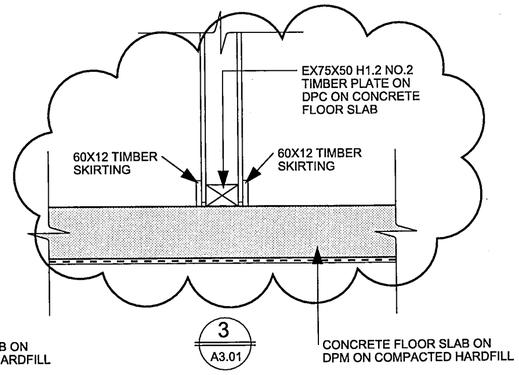
EMAIL: eng@arcl.co.nz

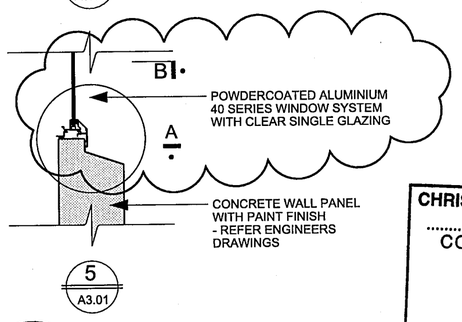


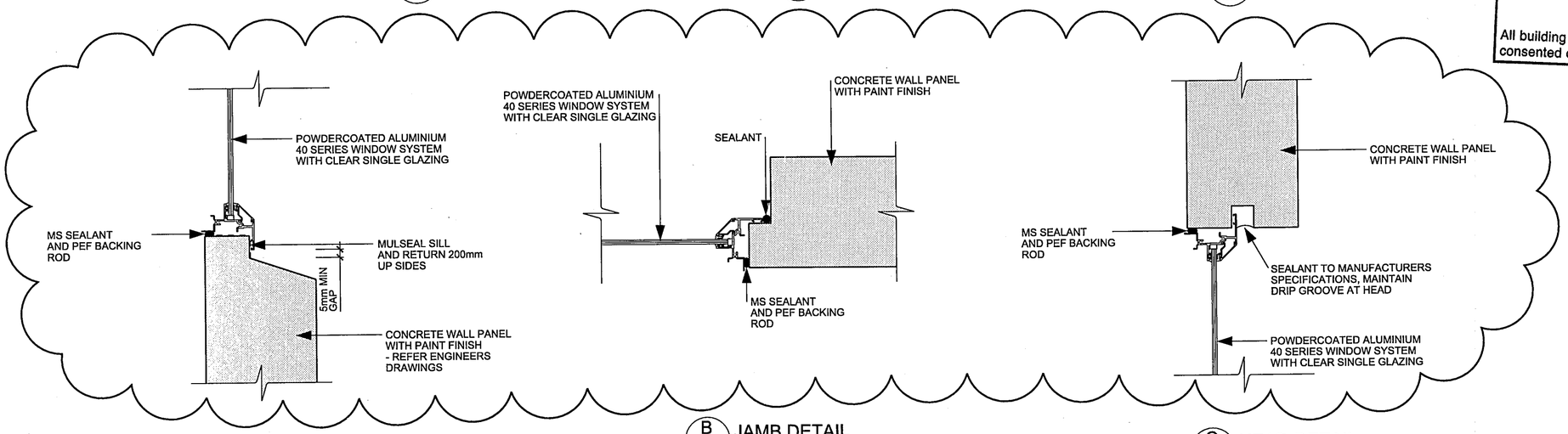












A SILL DETAIL
SCALE 1:5

B JAMB DETAIL
SCALE 1:5

C HEAD DETAIL
SCALE 1:5

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ISSUE	DESCRIPTION
B	DPC NOTED, ALUMINIUM JOINERY AND LOW PITCH FLASHING ADDED - 11.12.08
A	CONSENT ISSUE

PROJECT NAME:
DEVELOPMENT WASHBOURNES ROAD

DRAWING NAME:
DETAILS

SCALES AT A2	DRAWING No.	ISSUE	FILE
1:10 1:5	A4.01	B	8740

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P.O. BOX 3911
CHRISTCHURCH

TEL: (03) 366 0434
FAX: (03) 379 3981
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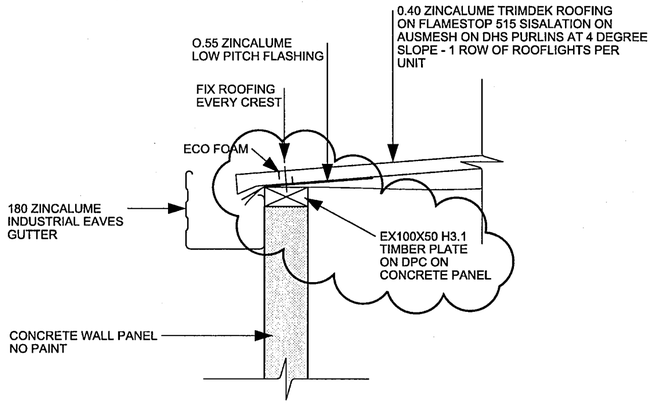
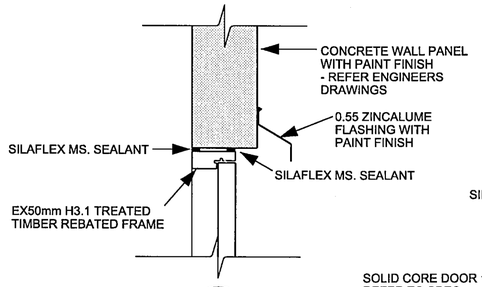


8
A3.02



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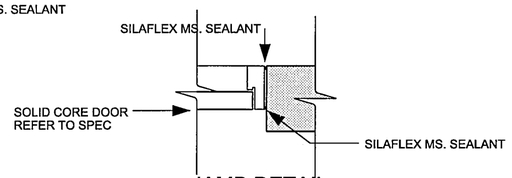
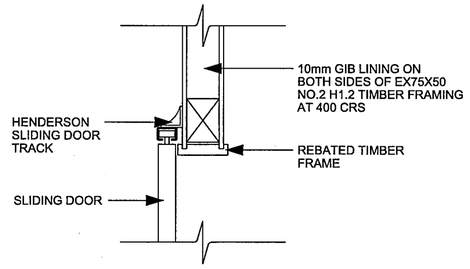


JAMB DETAIL

A



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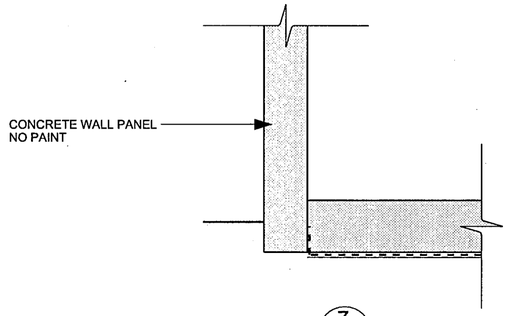
A3.02



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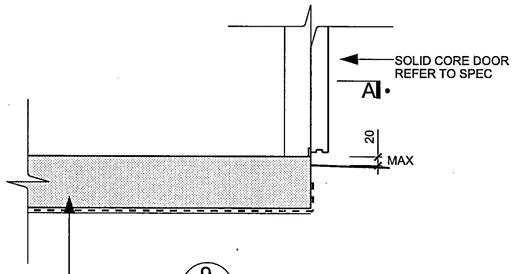

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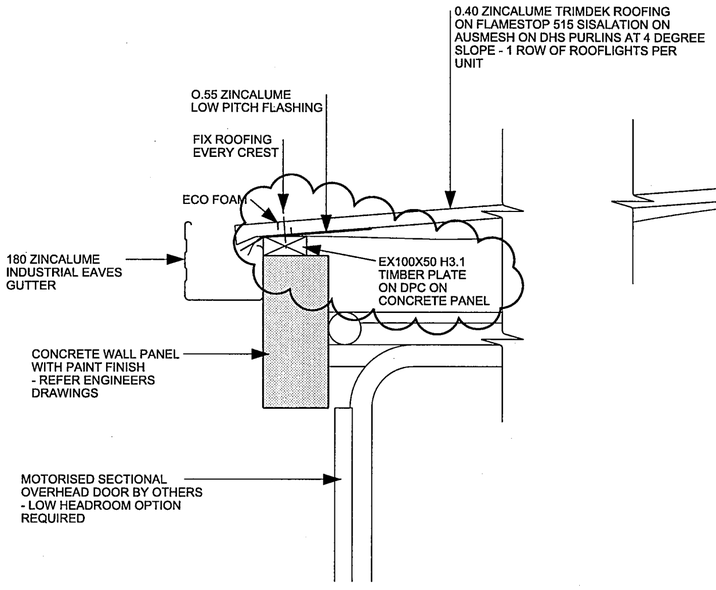

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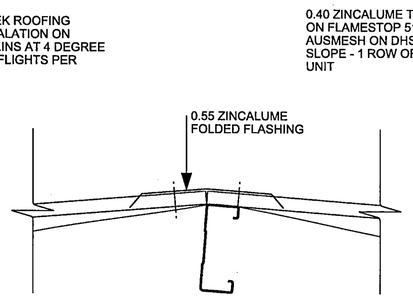

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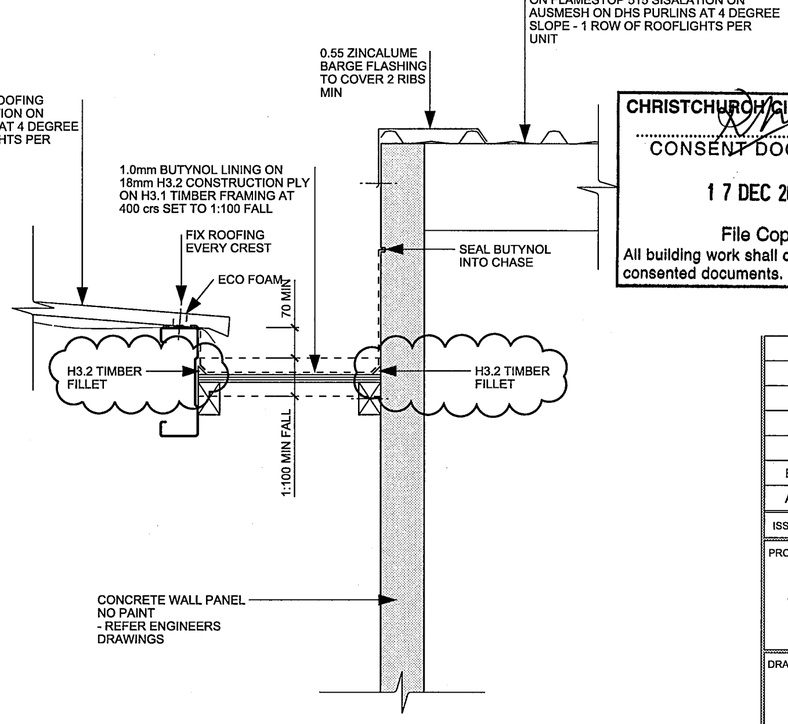

14
A3.02

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ISSUE	DESCRIPTION
B	TIMBER FILLETS AND DPC NOTED, LOW PITCH FLASHING ADDED - 11.12.08
A	CONSENT ISSUE

PROJECT NAME:

DEVELOPMENT WASHBOURNES ROAD

DRAWING NAME:

DETAILS

SCALES AT A2	DRAWING No.	ISSUE	FILE
1:10	A4.02	B	8740

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395 MADRAS STREET
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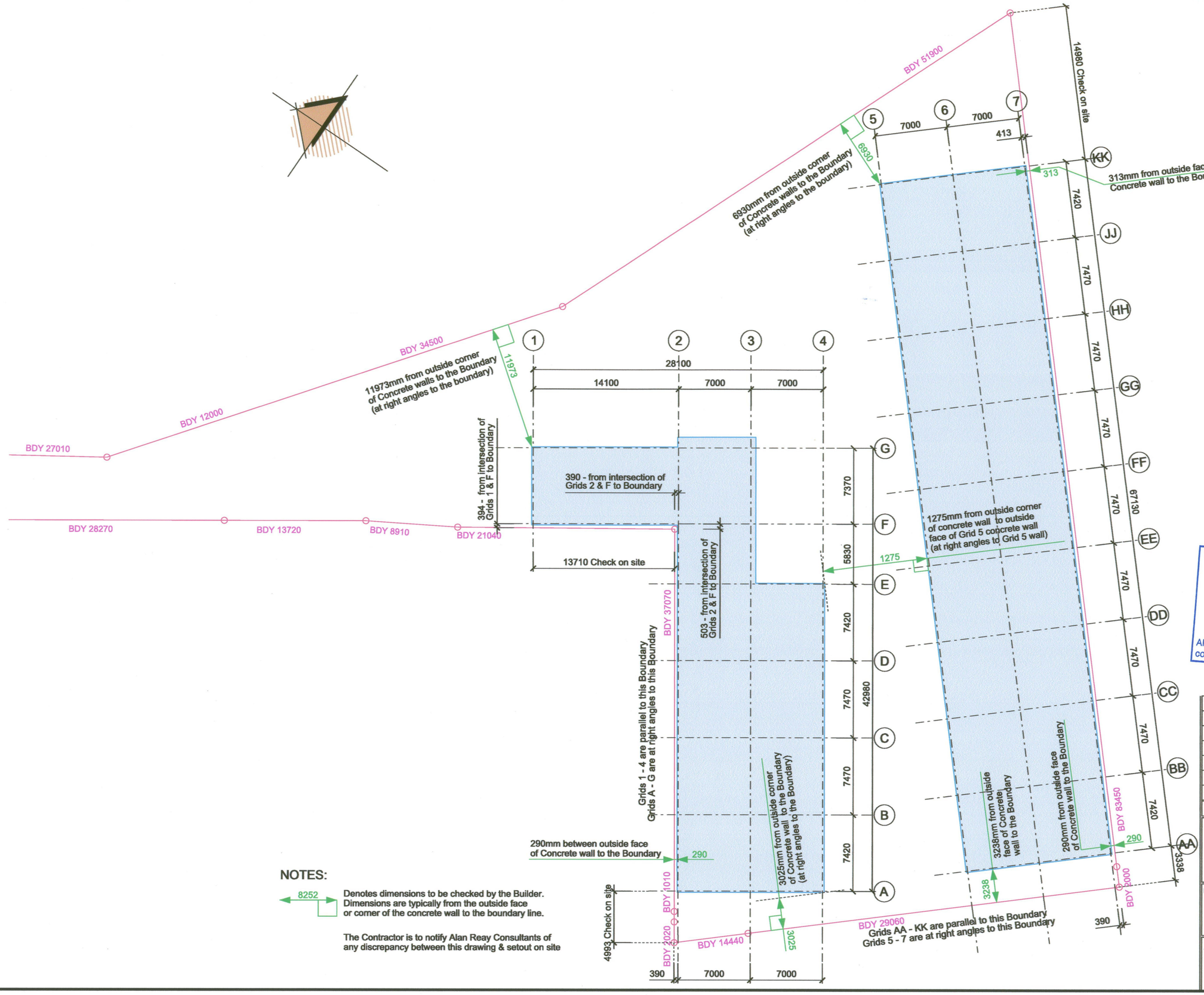
CHRISTCHURCH
TEL: (03) 366 0434
FAX: (03) 379 3981
EMAIL: eng@arcl.co.nz



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NOTES:

8252 — Denotes dimensions to be checked by the Builder. Dimensions are typically from the outside face or corner of the concrete wall to the boundary line.

The Contractor is to notify Alan Reay Consultants of any discrepancy between this drawing & setout on site

ISSUE	DESCRIPTION
A	CONSENT ISSUE

PROJECT NAME:
DEVELOPMENT WASHBOURNES ROAD

DRAWING NAME:
GRID SETOUT PLAN

SCALES AT A2	DRAWING No.	ISSUE	FILE
1:250	S0.01	A	8740

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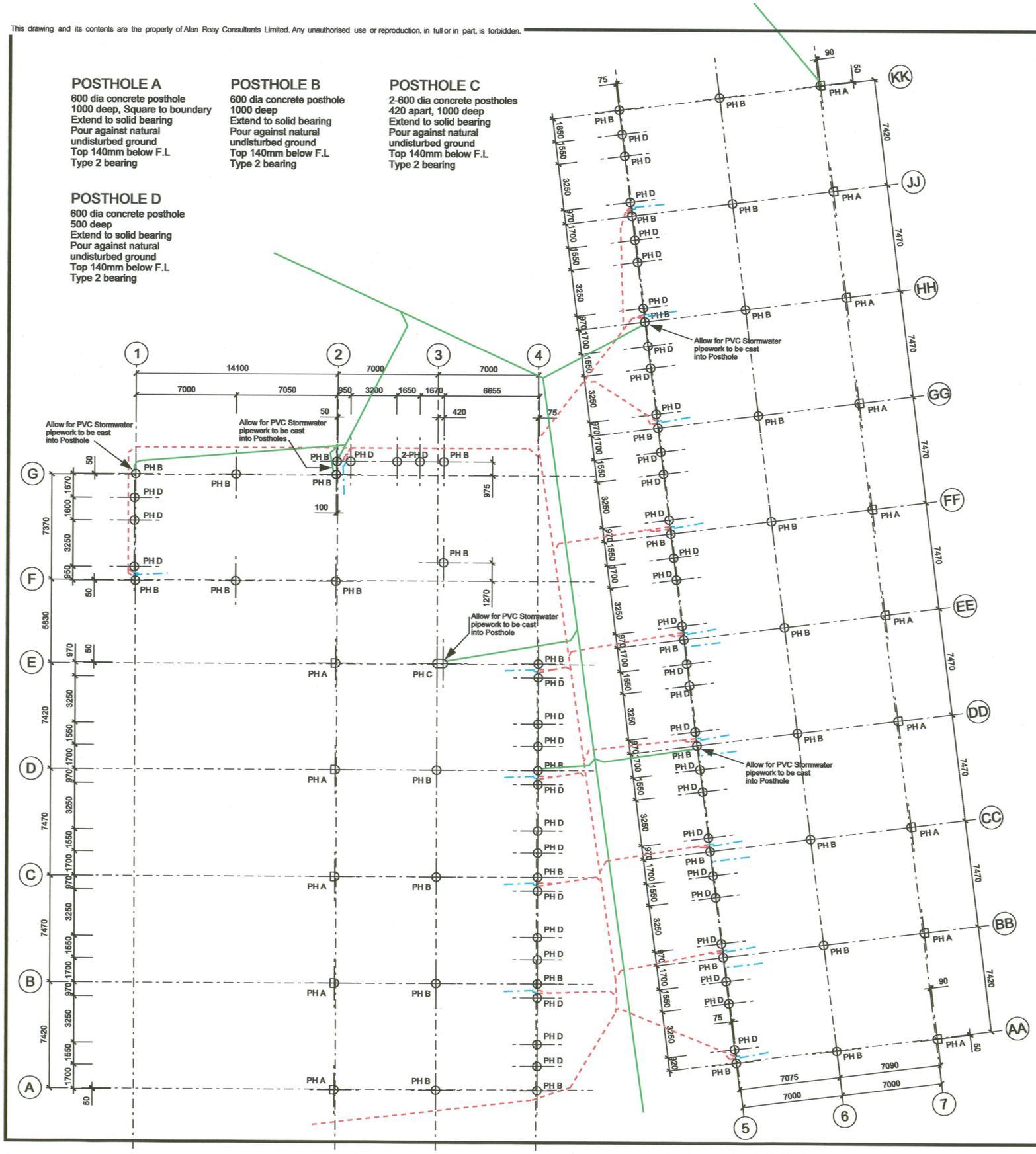
POSTHOLE A
600 dia concrete posthole
1000 deep, Square to boundary
Extend to solid bearing
Pour against natural undisturbed ground
Top 140mm below F.L
Type 2 bearing

POSTHOLE B
600 dia concrete posthole
1000 deep
Extend to solid bearing
Pour against natural undisturbed ground
Top 140mm below F.L
Type 2 bearing

POSTHOLE C
2-600 dia concrete postholes
420 apart, 1000 deep
Extend to solid bearing
Pour against natural undisturbed ground
Top 140mm below F.L
Type 2 bearing

POSTHOLE D
600 dia concrete posthole
500 deep
Extend to solid bearing
Pour against natural undisturbed ground
Top 140mm below F.L
Type 2 bearing



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Contractor to verify all dimensions on site BEFORE commencing works.

Contractor to determine positions of all building services i.e. sewer, stormwater, power, telephone etc., BEFORE pouring foundations.

Refer to Architectural drawings for all nibs, holes, rebates etc..

Allow to build drainage into foundations.

Carry all foundations to solid bearing

Pour all foundations against natural undisturbed ground.

This drawing is to be read in conjunction with the specification.

Reinforce ground floor slab with 665 mesh throughout, 50 top cover, Cut alternate bars crossing shrinkage control joints.

Lap all mesh a minimum of 2-cross wires, but no less than 225mm.

All reinforcing to be continuous around corners

Bearings between panels and postholes shall be one of the following types in accordance with the specification.

Type 2 bearing Mortar pack fully between base and posthole after the panel is erected.
Do NOT provide seating shims.

Minimum covers for foundation beam reinforcing are
75mm - when concrete is in contact with ground
50mm - when concrete is poured over DPC

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SERVICE PIPE KEY
This Drawing is to be read in conjunction with Siteworks, Architectural and Electrical drawings

- Proposed Stormwater
- Proposed Stormwater Overflow
- Proposed Sewer
- Proposed Electrical Services
- Proposed Sanitary Plumbing

ISSUE	DESCRIPTION
A	CONSENT ISSUE

PROJECT NAME:
DEVELOPMENT WASHBOURNES ROAD

DRAWING NAME:
FOUNDATION PLAN

SCALES AT A2	DRAWING No.	ISSUE	FILE
1:200	S1.01	A	8740

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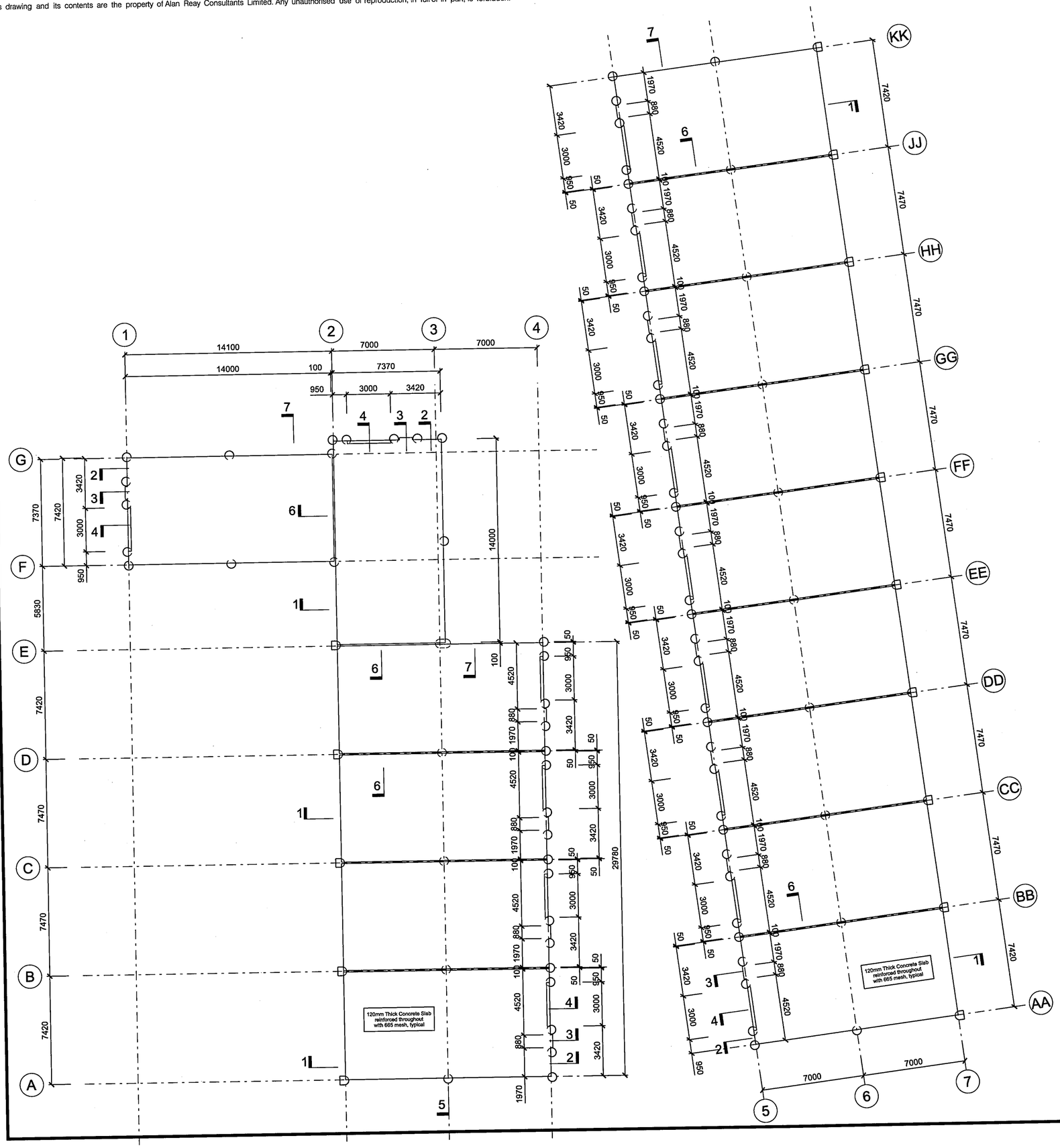

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Refer to S1.05 for details
Refer to S1.01 for notes

ISSUE	DESCRIPTION
A	CONSENT ISSUE

PROJECT NAME:
DEVELOPMENT WASHBOURNES ROAD

DRAWING NAME:
SLAB PLAN

SCALES AT A2	DRAWING No.	ISSUE	FILE
1:200	S1.02	A	8740

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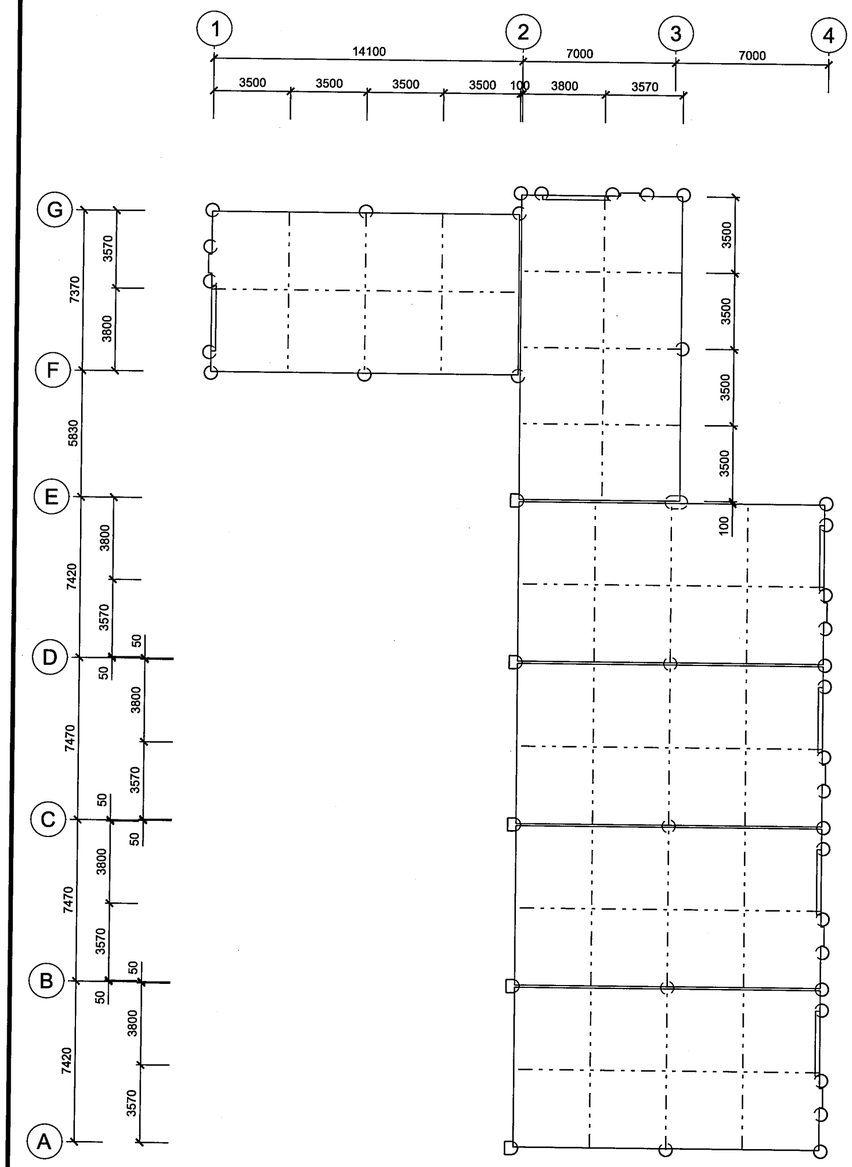

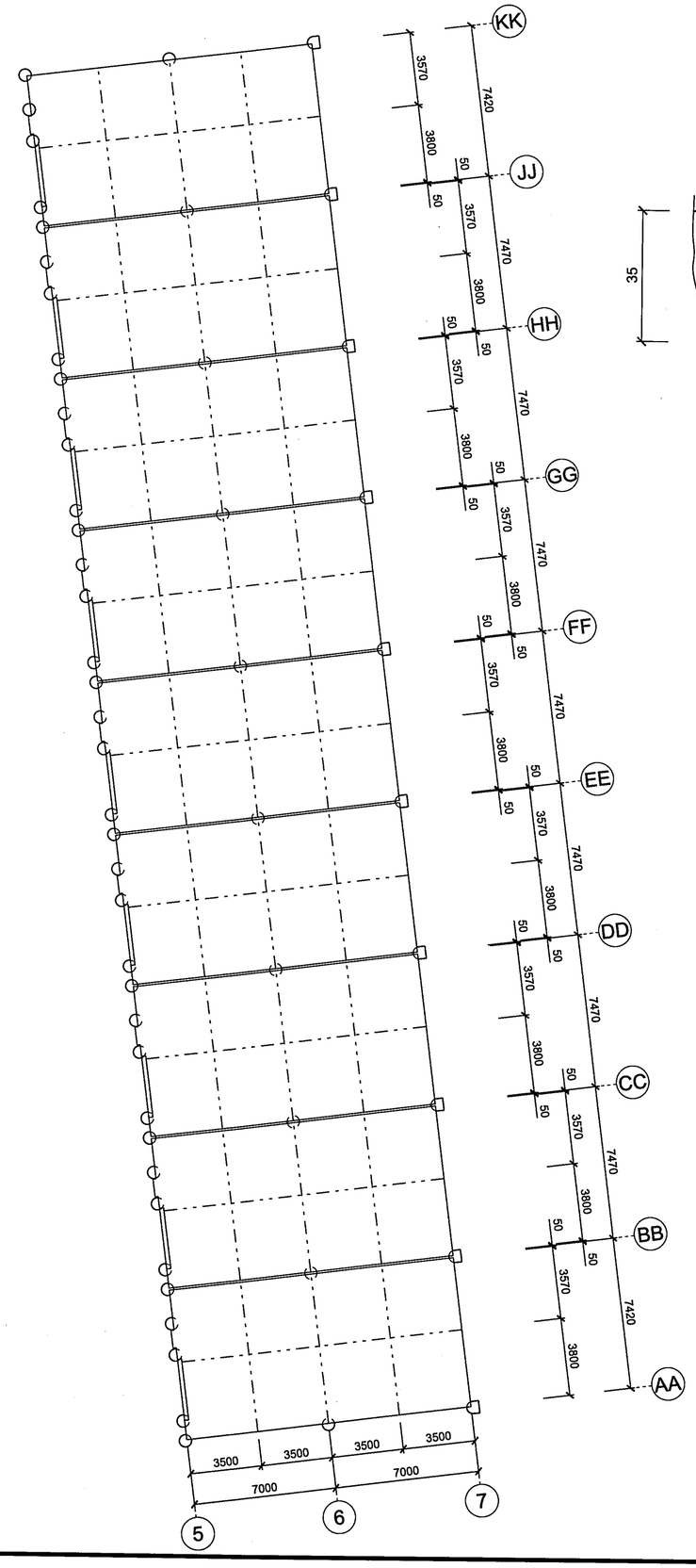

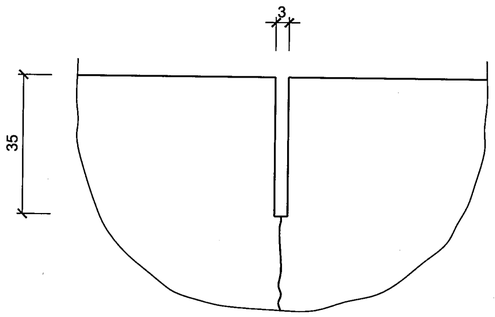

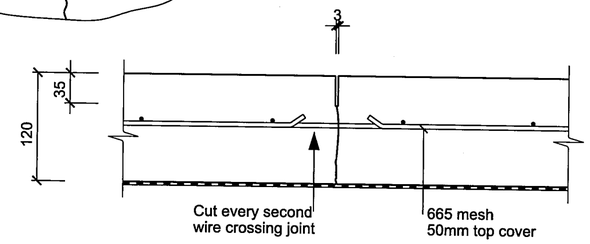

TYPICAL SAWCUT JOINT

NOTES:

This drawing is to be read in conjunction with Foundation drawings.

Reinforce ground floor slab with 665 mesh throughout (NOT DEFORMED). Cut alternate wires crossing shrinkage control joints.

Lap all mesh a minimum of 2-cross wires, but no less than 225mm. DO NOT coincide mesh laps with shrinkage control joints. (Mesh size on hold pending slab thickness confirmation)

- - —Denotes sawcut or formed joints in slab. Refer to details below for sawcut and joint details. Install a temporary filler to cut joints if required for precast concrete construction and remove to provide detailed filler at job completion.

- - - -Denotes construction joints to slab. Do not lap mesh across this joint except immediately infront of roller door opening.

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ISSUE	DESCRIPTION
A	CONSENT ISSUE

PROJECT NAME:
DEVELOPMENT WASHBOURNES ROAD

DRAWING NAME:
SAWCUT PLAN

SCALES AT A2	DRAWING No.	ISSUE	FILE
1:200	S1.03	A	8740

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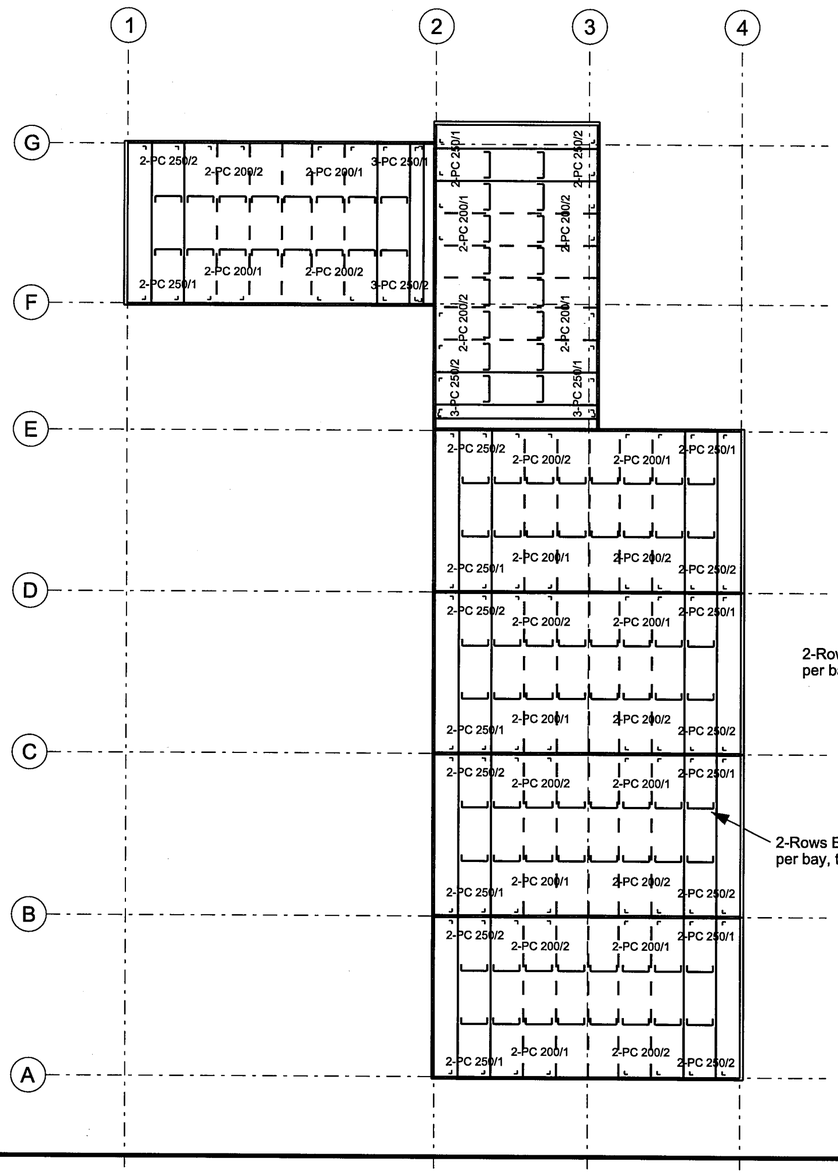
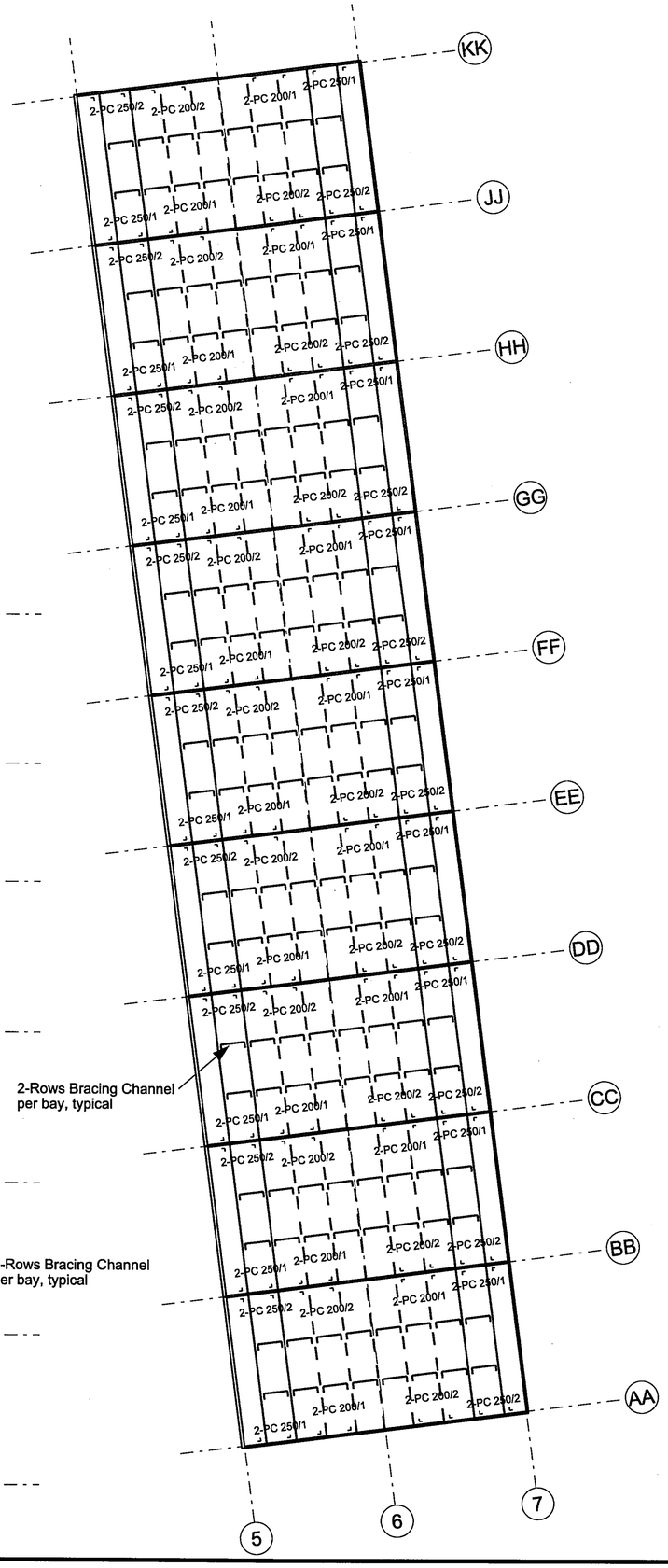
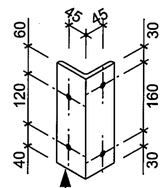
P.O. BOX 3911
CHRISTCHURCH

TEL: (03) 366 0434
FAX: (03) 379 3981
EMAIL: eng@arcl.co.nz

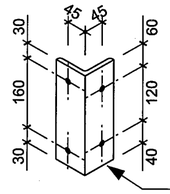






PURLIN CLEAT PC 250/1

76x76x5.0 ms angle
220 long
4-14 dia holes for
4-M12 HDG Bolts



PURLIN CLEAT PC 250/2

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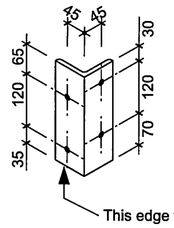
220 long
4-14 dia holes for
4-M12 HDG Bolts



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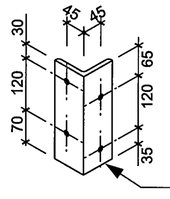
76x76x5.0 ms angle
220 long
4-14 dia holes for
4-M12 HDG Bolts



PURLIN CLEAT PC 200/2

76x76x5.0 ms angle
220 long
4-14 dia holes for
4-M12 HDG Bolts

— — Denotes DHS 200/12 Purlins
—— Denotes DHS 250/13 Purlins

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ISSUE	DESCRIPTION
A	CONSENT ISSUE

PROJECT NAME:
DEVELOPMENT WASHBOURNES ROAD

DRAWING NAME:
ROOF PURLIN PLAN

SCALES AT A2	DRAWING No.	ISSUE	FILE
1:200	S1.04	A	8740

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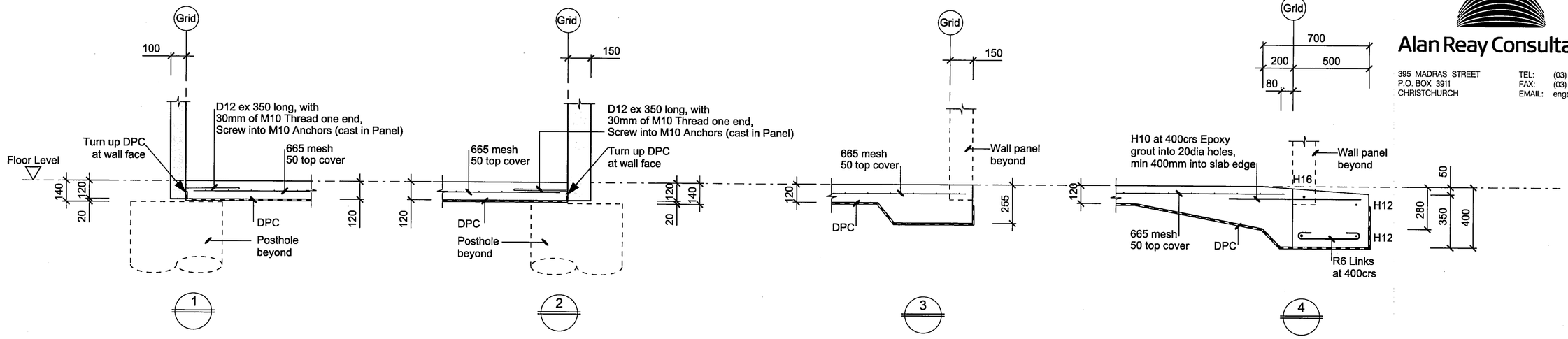

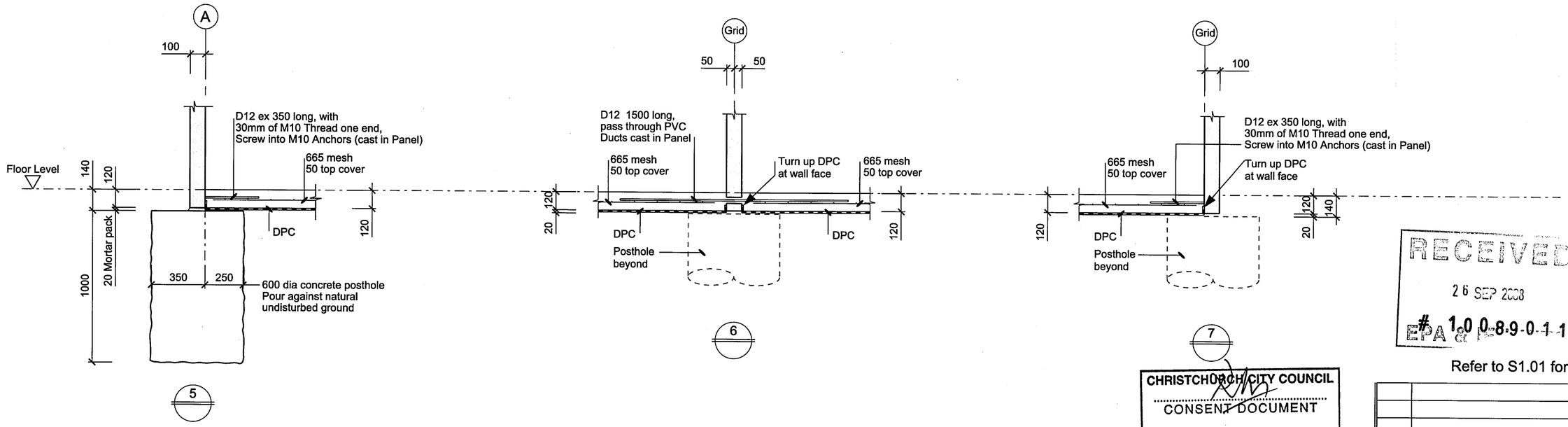

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ISSUE	DESCRIPTION
A	CONSENT ISSUE

PROJECT NAME:
DEVELOPMENT
WASHBOURNES ROAD

DRAWING NAME:
FOUNDATION DETAILS

SCALES AT A2	DRAWING No.	ISSUE	FILE
1:20	S1.05	A	8740

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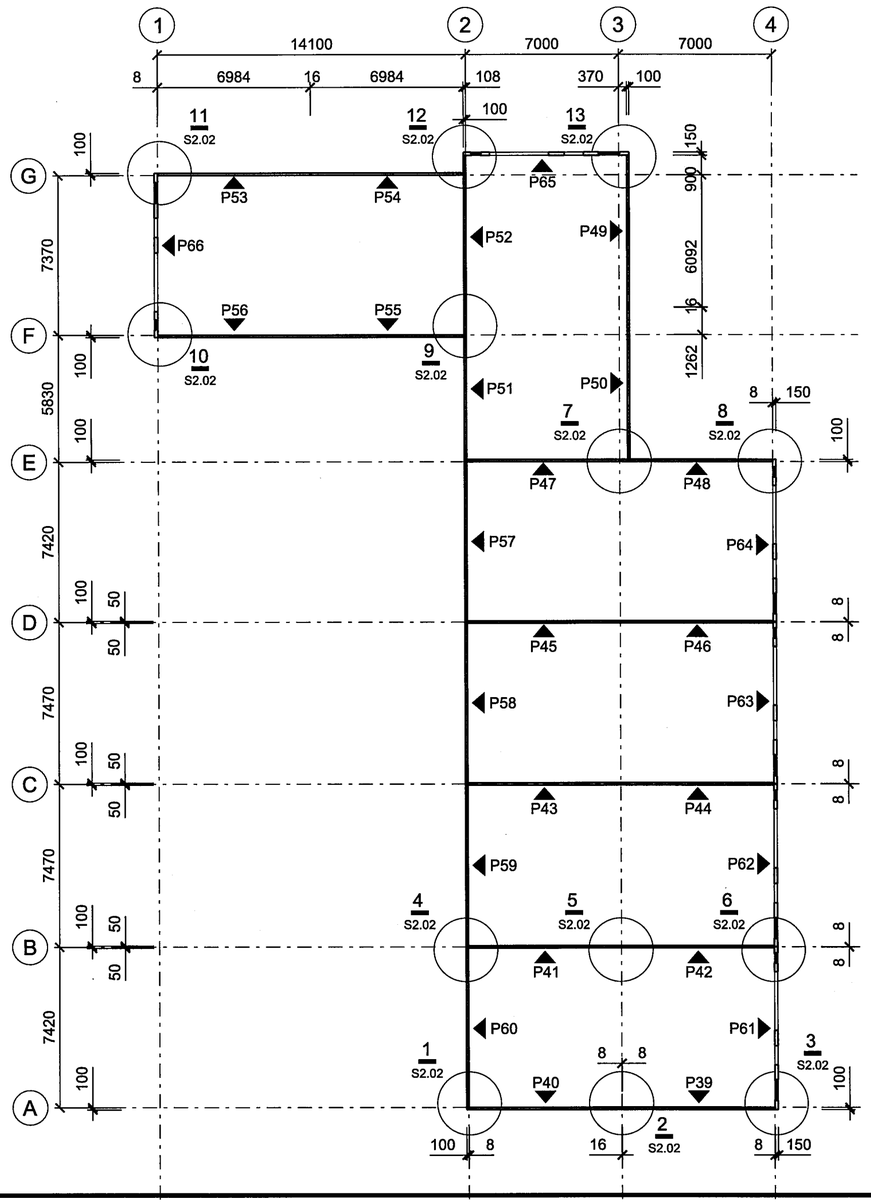
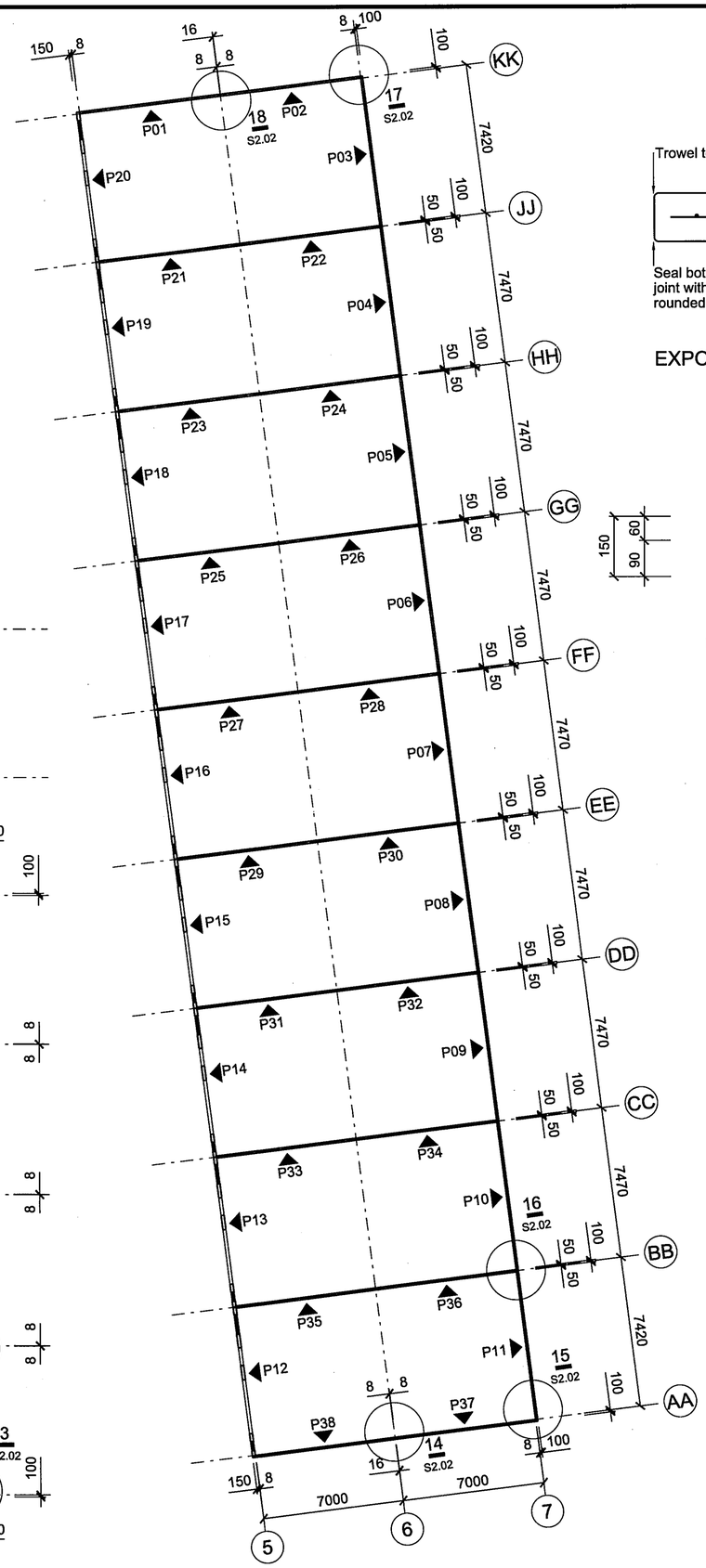
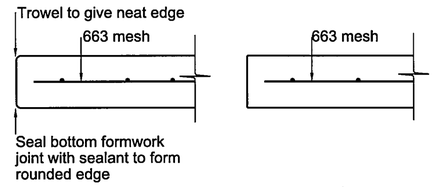
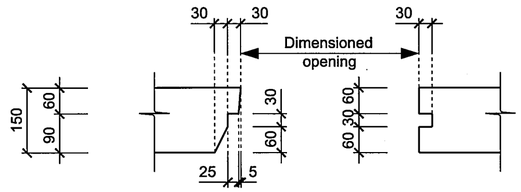




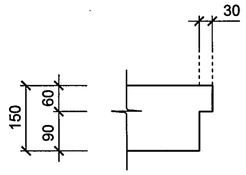


EXPOSED EDGE HIDDEN EDGE



NOTE: Refer to Panel Elevations for all reinforcing around Windows & Doors

SILL & HEAD, TYPICAL



JAMB, TYPICAL

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CHRISTCHURCH
TEL: (03) 366 0434
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NOTES

Contractor to check all dimensions on site BEFORE commencing works

All materials and workmanship to comply with the New Zealand Building Code.

All Panels/Columns viewed from inside of building and drawn in the casting position with inside face up.
▲ Denotes the direction that panels have been elevated from on panel elevation sheets.
Any changes from this casting pattern must be approved by the Engineer in writing.
All panels to have 1 layer of 663 or D212 mesh in centre of panel. Other reinforcing noted on panels to have covers as noted below.

Lap all mesh minimum of 2 cross wires but no less than 225mm

Refer to the Specification for concrete strengths. Strength to be 20MPa at time of lift unless nominated otherwise on the drawing.

Tolerances Height +0mm, less 6mm
Width +0mm, less 6mm
Thickness +0mm, less 3mm

Legend

- Denotes Type A fixing on top face unless stated otherwise on panel
- Denotes 5 Tonne Ramset face lifting eye fixing on top face
- Denotes 2.5 Tonne Ramset erection anchor to be used in accordance with manufacturers specifications. Supply with 1 Rod type 4 & 5

Suffix (T) Denotes top steel with 25 cover
(t) Denotes top steel with 37 cover
(b) Denotes bottom steel with 52 cover
(B) Denotes bottom steel with 40 cover

Cover to be maintained past recesses in panels.

Contractor to ensure panel frees from recess formers during panel erection.

Contractor is to check that opening edge details match joinery requirements before forming edges.

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ISSUE	DESCRIPTION
A	CONSENT ISSUE

PROJECT NAME:
DEVELOPMENT WASHBOURNES ROAD

DRAWING NAME:
PRECAST CONCRETE PANEL LOCATION PLAN

SCALES AT A2	DRAWING No.	ISSUE	FILE
1:200	S2.01	A	8740

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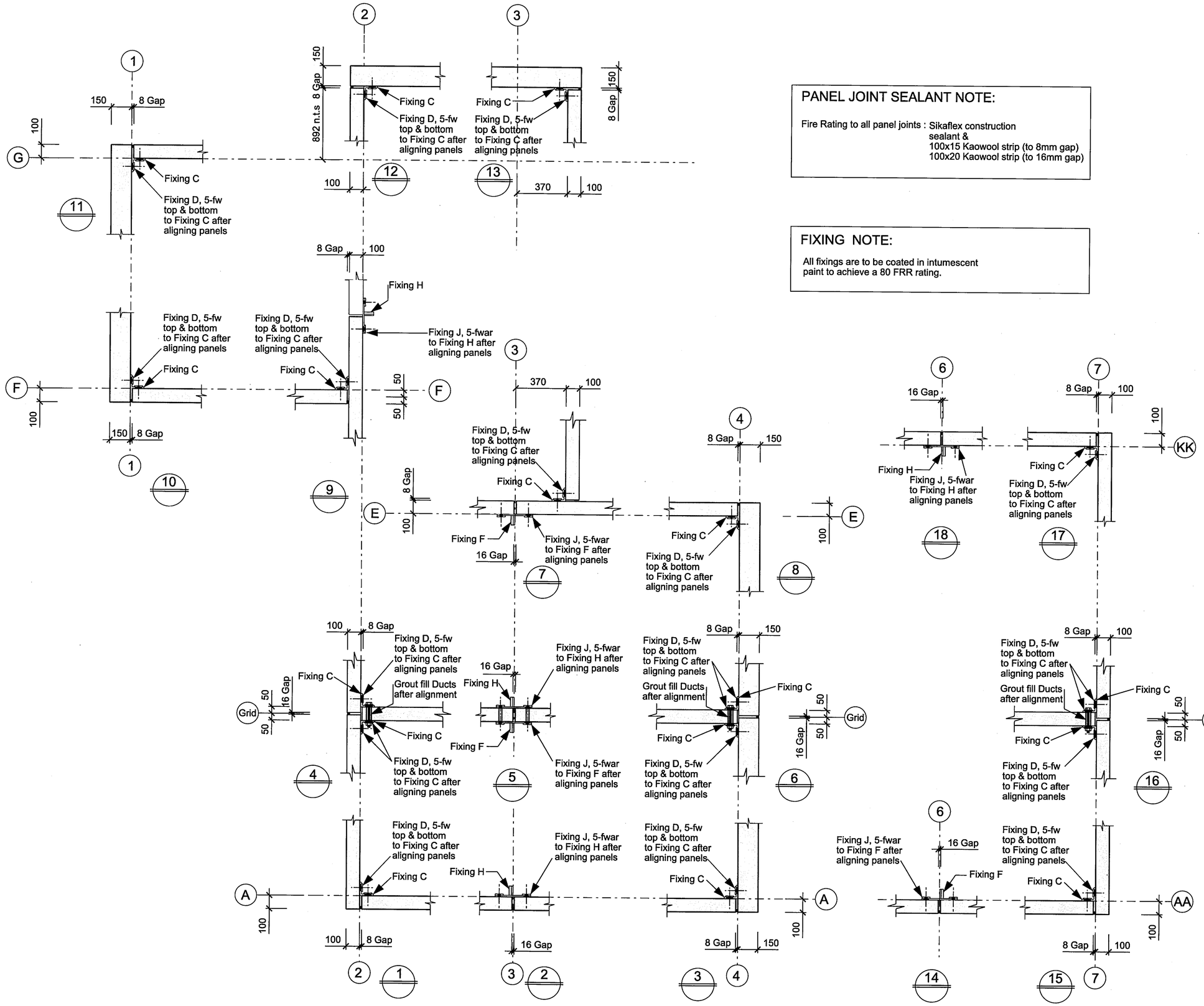
PANEL JOINT SEALANT NOTE:

Fire Rating to all panel joints : Sikaflex construction sealant &
100x15 Kaowool strip (to 8mm gap)
100x20 Kaowool strip (to 16mm gap)

FIXING NOTE:

All fixings are to be coated in intumescent paint to achieve a 80 FRR rating.



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ISSUE	DESCRIPTION
A	CONSENT ISSUE

PROJECT NAME:
DEVELOPMENT WASHBOURNES ROAD

DRAWING NAME:
PRECAST CONCRETE PANEL CONNECTION DETAILS

SCALES AT A2	DRAWING No.	ISSUE	FILE
1:20	S2.02	A	8740

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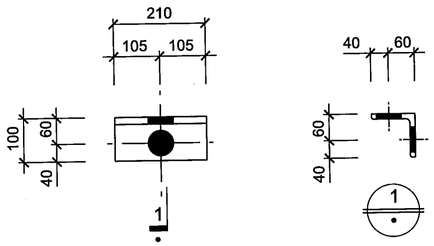
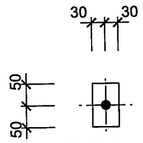


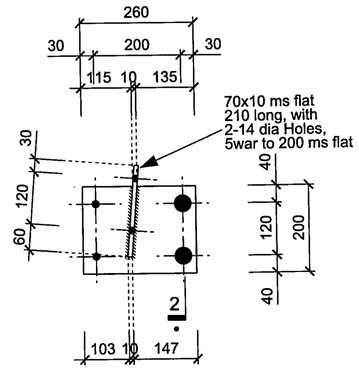
100x100x12 ms angle
210 long
2-60 dia holes

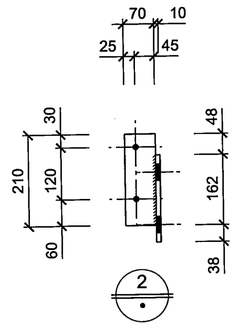
FIXING C



100x60x6 ms flat
1-22 dia hole
Supply with M20 bolt
55 long

FIXING D

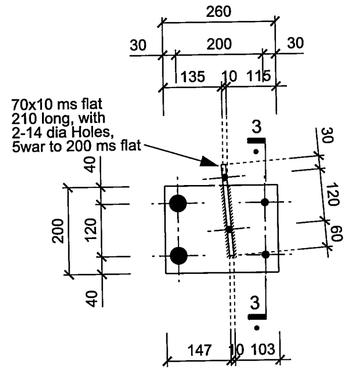


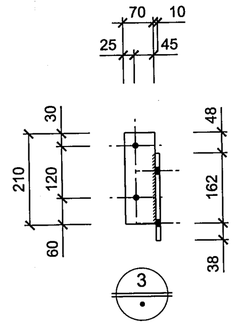


200x10 ms flat
260 long, with 2-18 dia Holes
and 2-40 dia holes

HOT DIP GALVANISED
AFTER FABRICATION

FIXING F

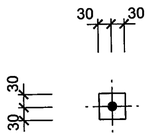




200x10 ms flat
260 long, with 2-18 dia Holes
and 2-40 dia holes

HOT DIP GALVANISED
AFTER FABRICATION

FIXING H



60x60x10 ms flat
1-18 dia hole,
Supply with M16 Bolt
180 long

FIXING J

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ISSUE	DESCRIPTION
A	CONSENT ISSUE

PROJECT NAME:
DEVELOPMENT
WASHBOURNES ROAD

DRAWING NAME:
FIXINGS

SCALES AT A2	DRAWING No.	ISSUE	FILE
1:10	S3.01	A	8740

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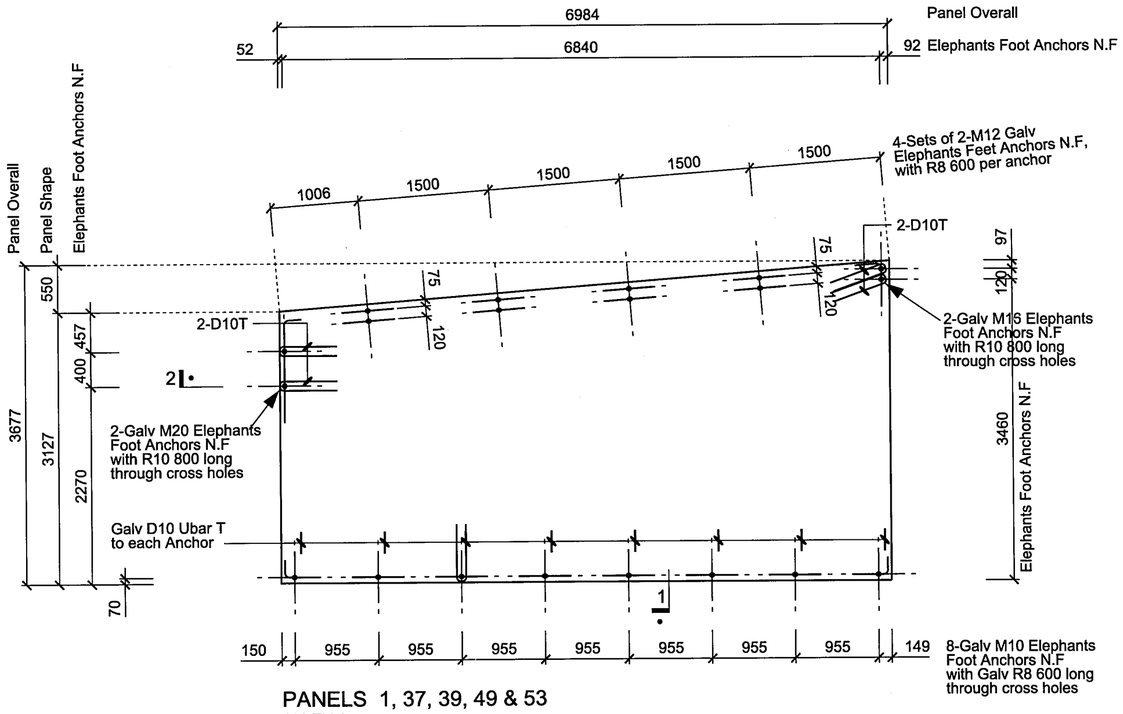
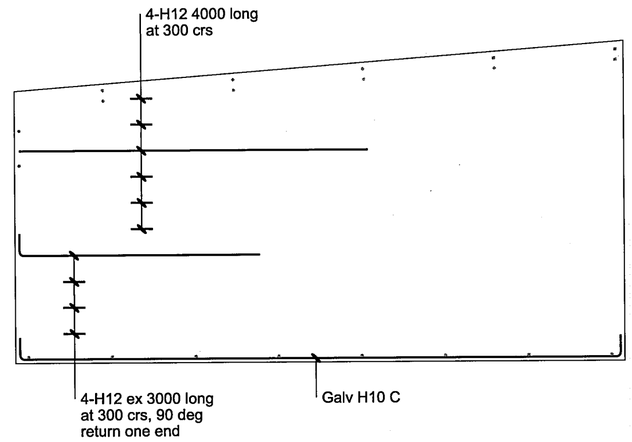
395 MADRAS STREET
P.O. BOX 3911
CHRISTCHURCH
TEL: (03) 366 0434
FAX: (03) 379 3981
EMAIL: eng@arcl.co.nz



PANELS 1, 37, 39, 49 & 53
100 Thick
5.83 Tonne
2.38 Cubic Metres



PANEL 1 - REINFORCING ONLY

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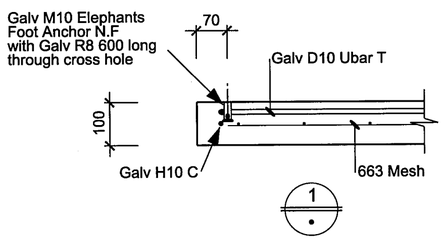

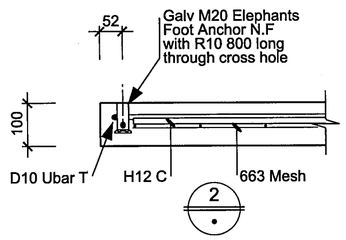

Refer to Panel Location Plan for notes

ISSUE	DESCRIPTION
A	CONSENT ISSUE

PROJECT NAME:
DEVELOPMENT
WASHBOURNES ROAD

DRAWING NAME:
PRECAST CONCRETE
PANELS 1, 37, 39, 49 & 53

SCALES AT A2	DRAWING No.	ISSUE	FILE
1:50 1:10	S4.01	A	8740

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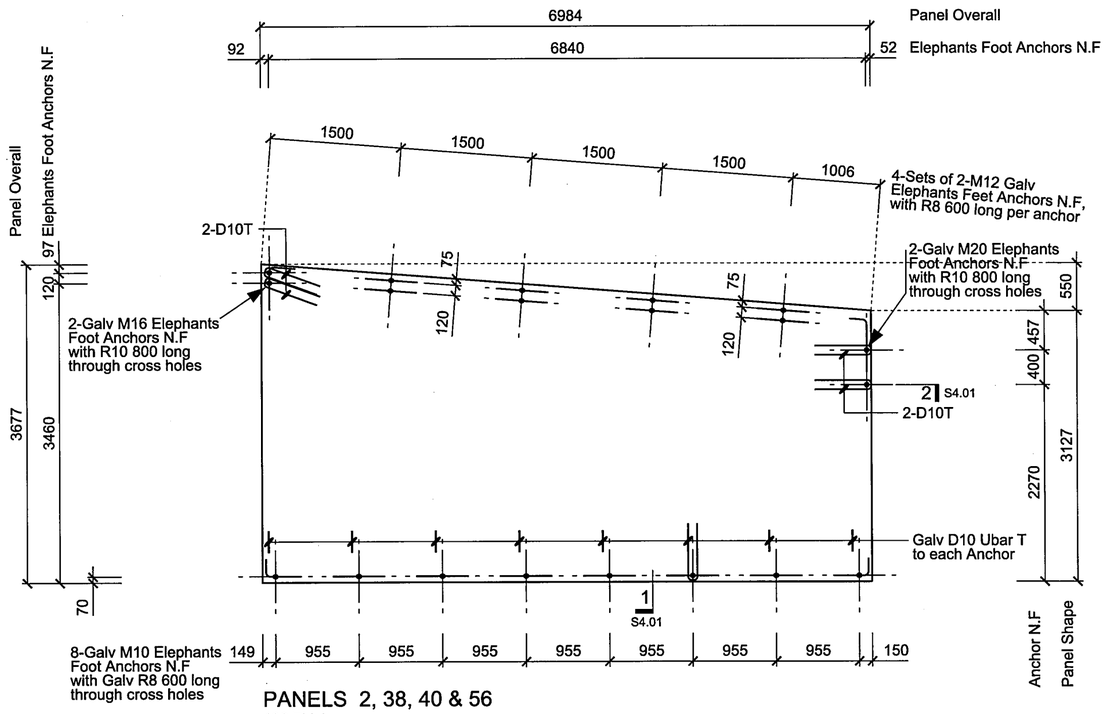
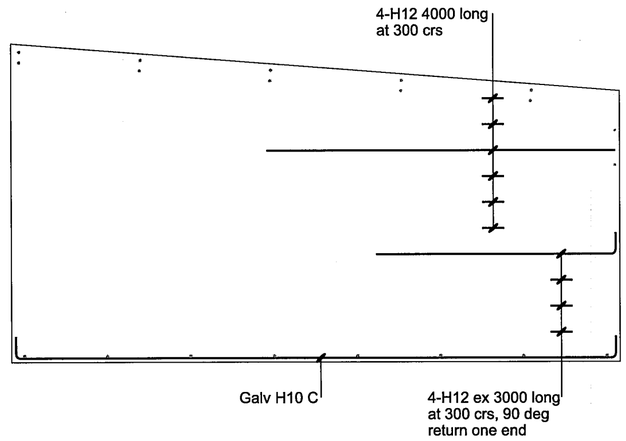


PANELS 2, 38, 40 & 56
100 Thick
5.83 Tonne
2.38 Cubic Metres



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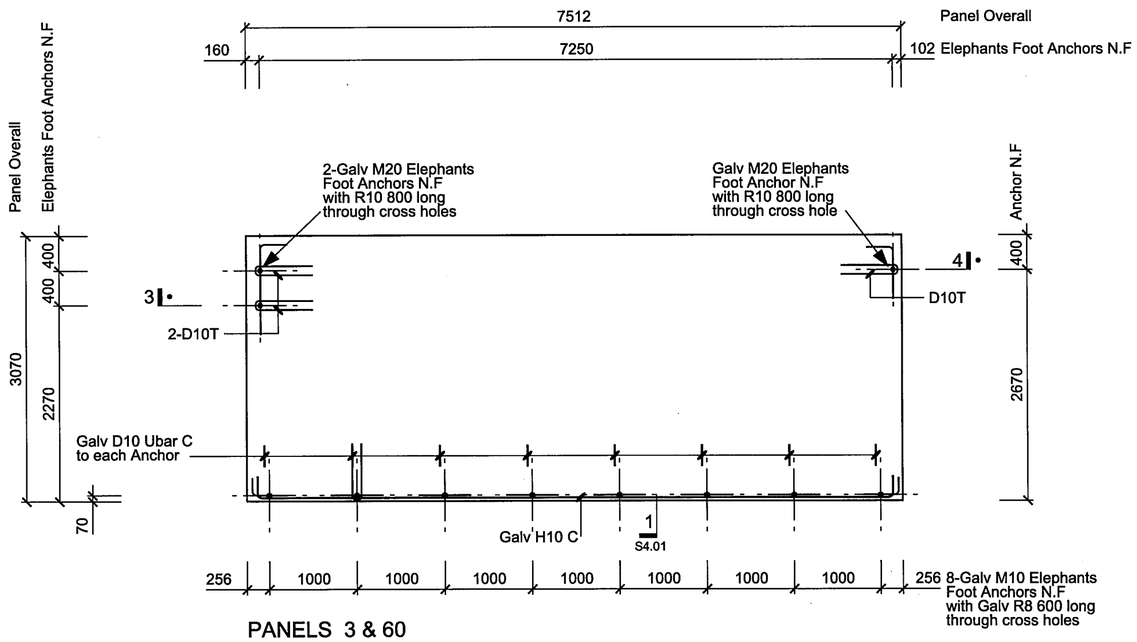
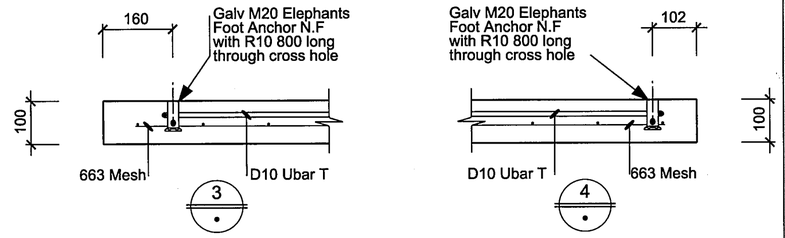


PANELS 3 & 60
100 Thick
5.66 Tonne
2.31 Cubic Metres



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ISSUE	DESCRIPTION
A	CONSENT ISSUE

PROJECT NAME:
DEVELOPMENT WASHBOURNES ROAD

DRAWING NAME:
PRECAST CONCRETE PANELS 2, 3, 38, 40, 56 & 60

SCALES AT A2	DRAWING No.	ISSUE	FILE
1:50 1:10	S4.02	A	8740

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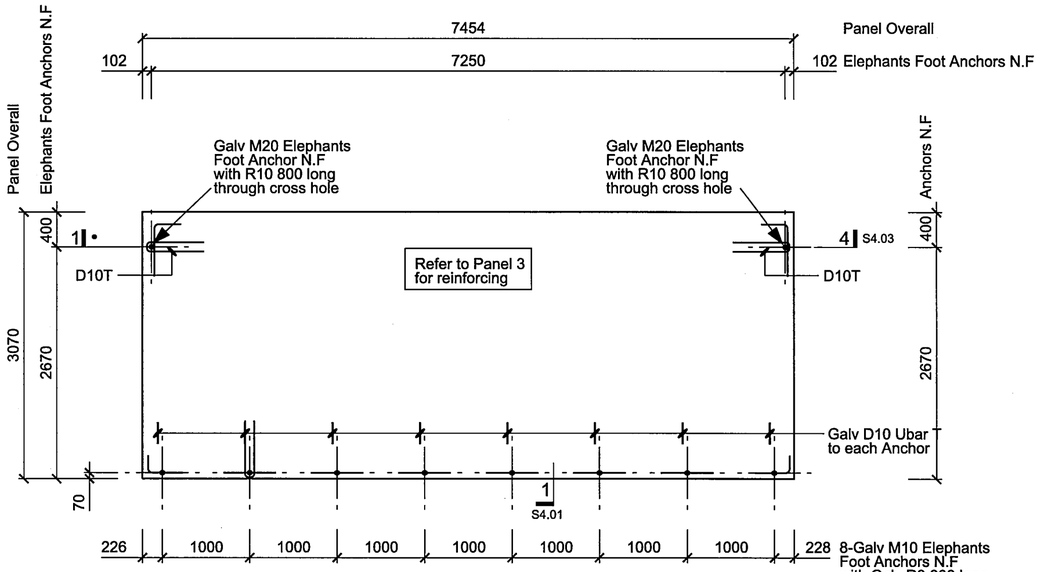
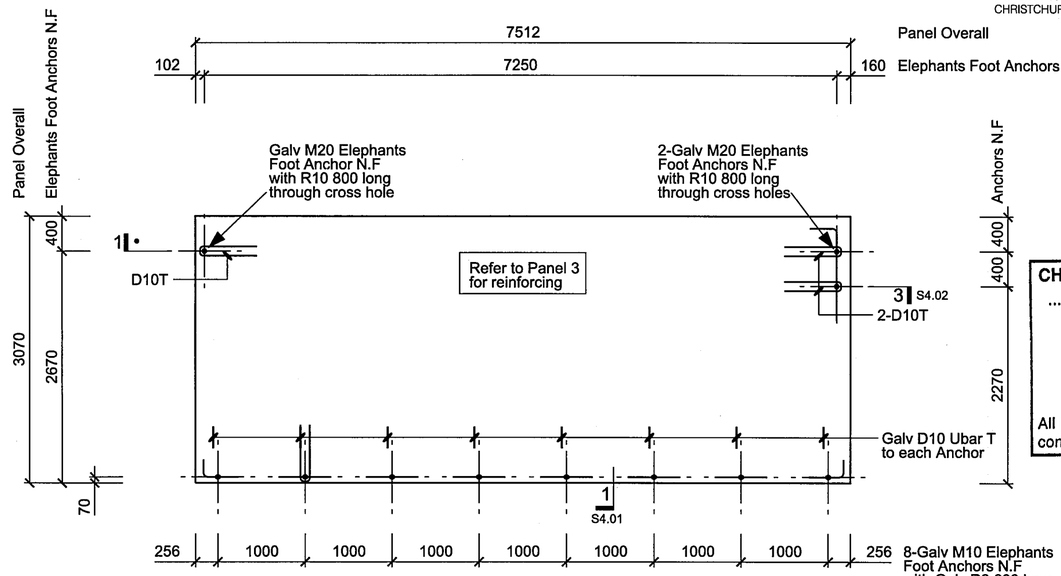
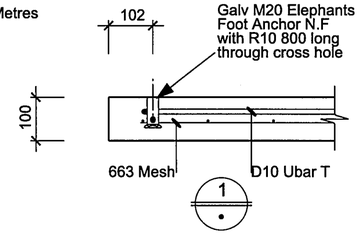
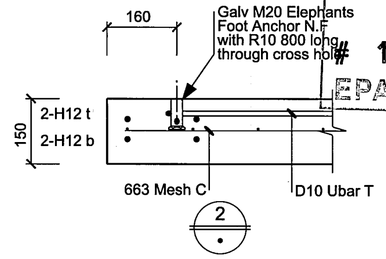


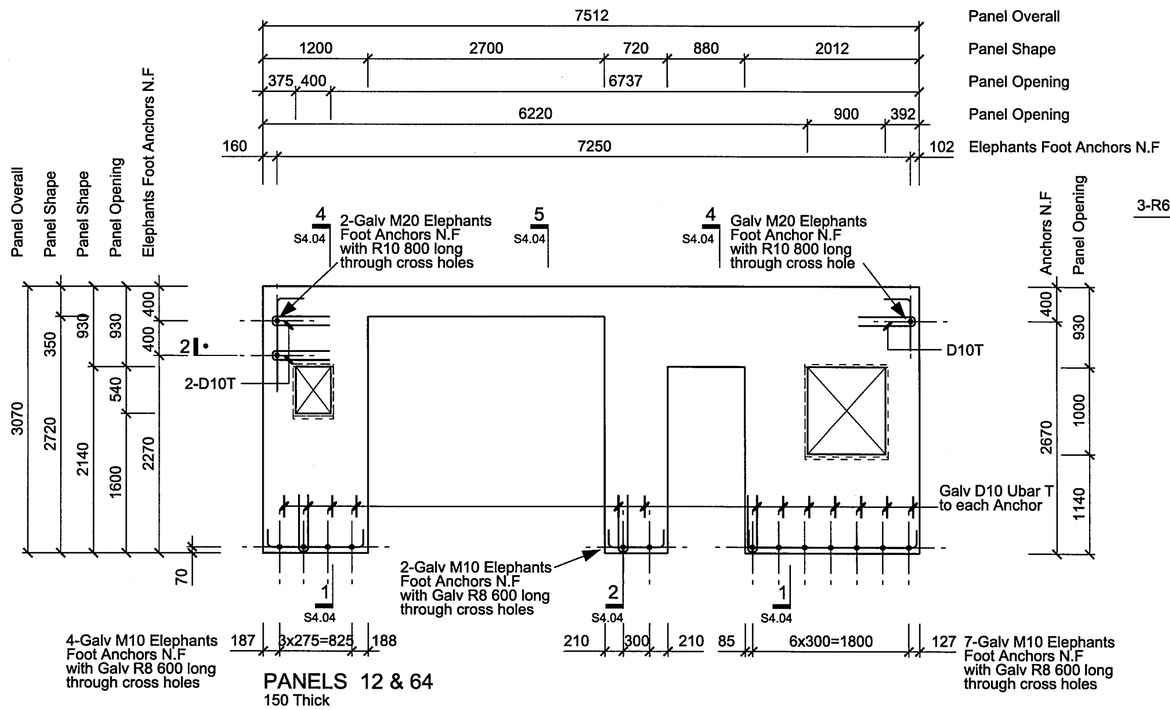
PANELS 4, 5, 6, 7, 8, 9, 10, 57, 58 & 59
100 Thick
5.61 Tonne
2.29 Cubic Metres



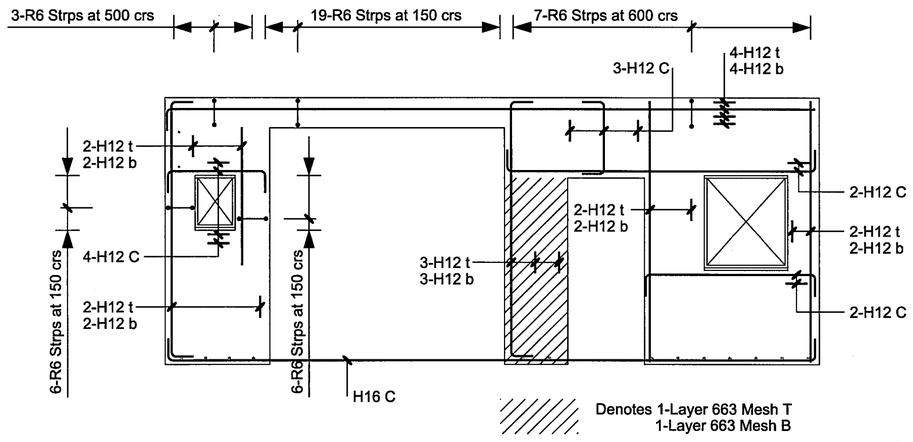
PANEL 11
100 Thick
5.66 Tonne
2.31 Cubic Metres







PANELS 12 & 64
150 Thick
4.68 Tonne
1.91 Cubic Metres



PANELS 12 & 64 - REINFORCING ONLY

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Refer to Panel Location Plan for notes

ISSUE	DESCRIPTION
A	CONSENT ISSUE

PROJECT NAME:
DEVELOPMENT
WASHBOURNES ROAD

DRAWING NAME:
PRECAST CONCRETE
PANELS 4 - 10, 12,
57, 58, 59 & 64

SCALES AT A2	DRAWING No.	ISSUE	FILE
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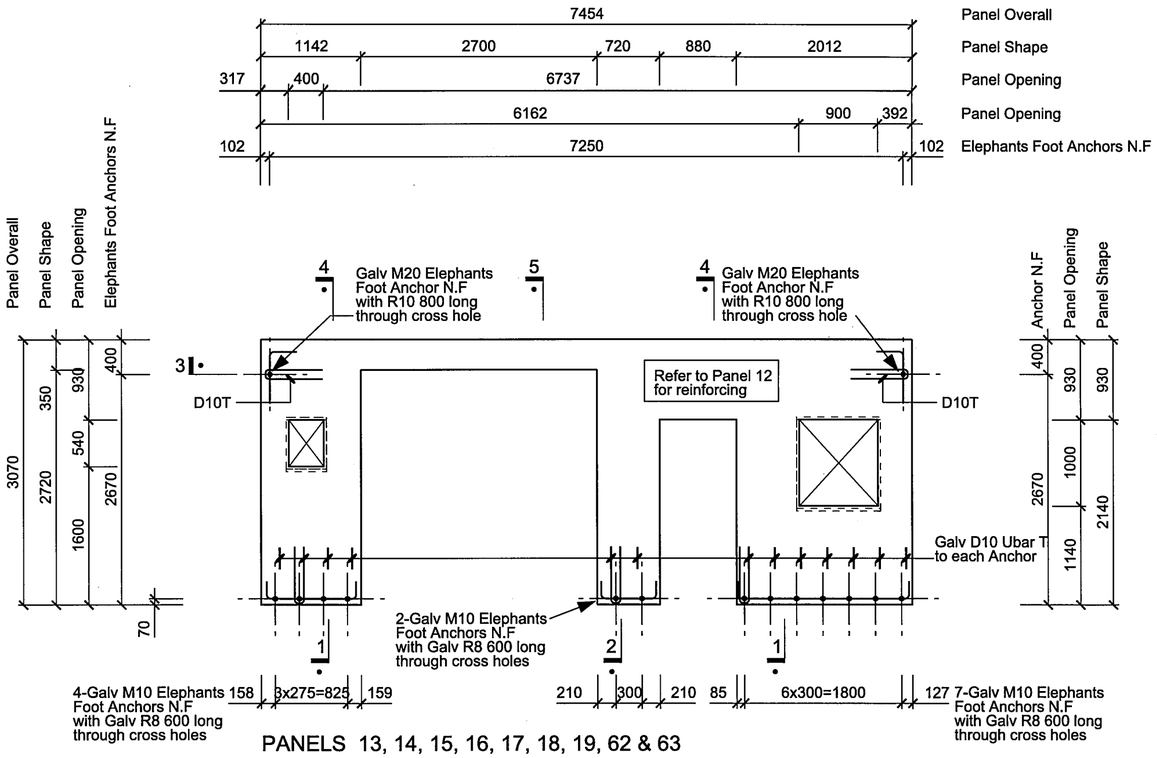
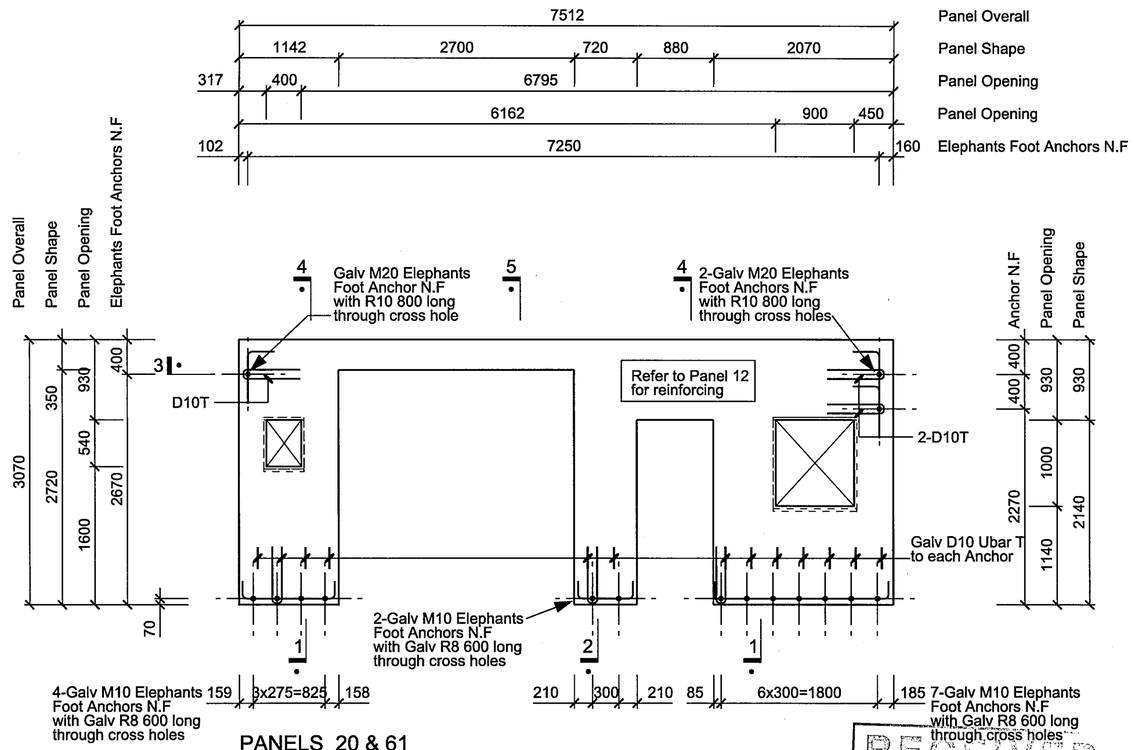


PANELS 13, 14, 15, 16, 17, 18, 19, 62 & 63
150 Thick
4.61 Tonne
1.88 Cubic Metres



PANELS 20 & 61
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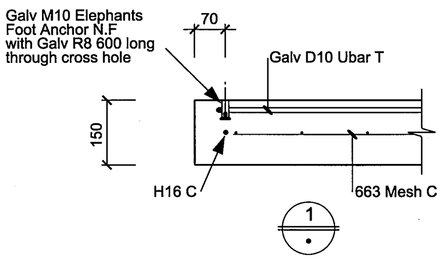
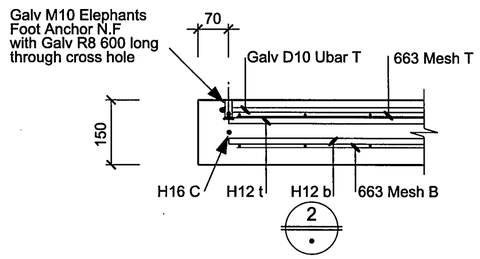
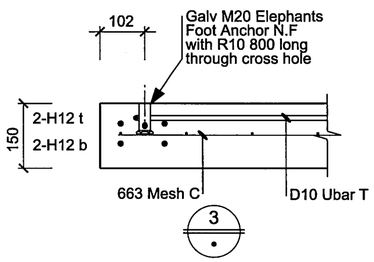
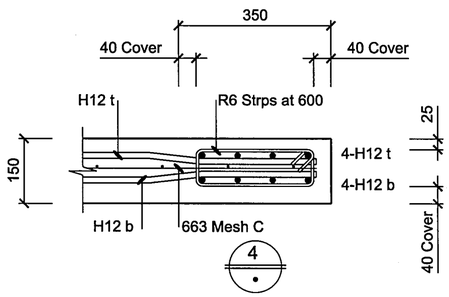
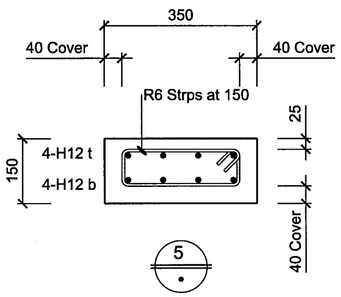
4.68 Tonne
1.91 Cubic Metres











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Alan Reay Consultants
395 MADRAS STREET
P.O. BOX 3911
CHRISTCHURCH
TEL: (03) 366 0434
FAX: (03) 379 3981
EMAIL: eng@arcl.co.nz

Refer to Panel Location Plan for notes

ISSUE	DESCRIPTION
A	CONSENT ISSUE

PROJECT NAME:
DEVELOPMENT
WASHBOURNES ROAD

DRAWING NAME:
PRECAST CONCRETE
PANELS 13 - 20, 61, 62 & 63

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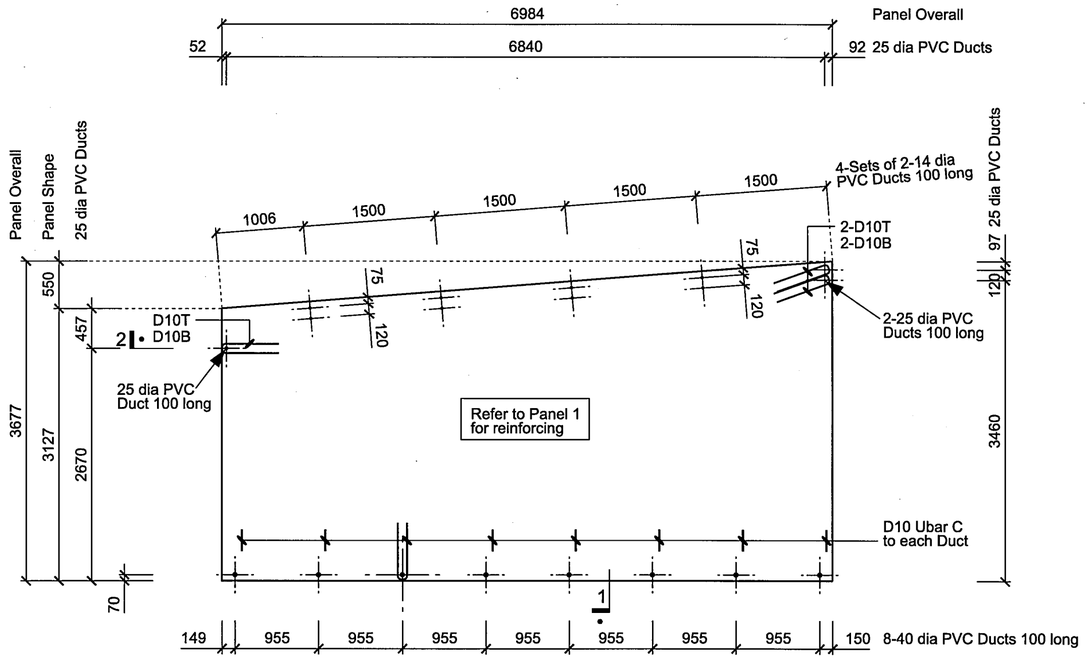

PANELS 21, 23, 25, 27, 29, 31, 33, 35, 41, 43 & 45

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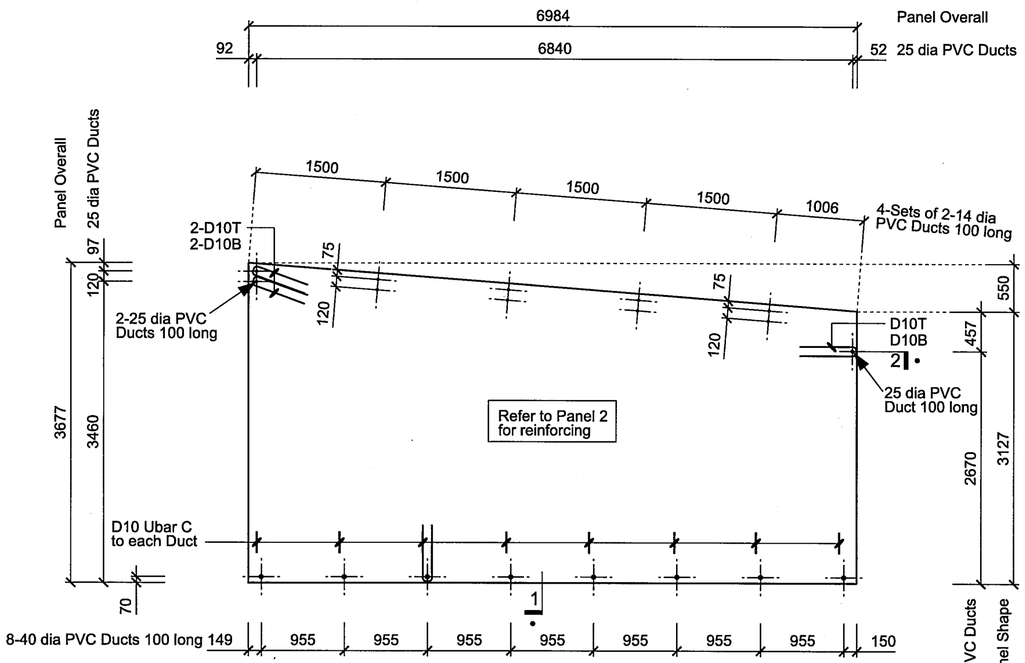
5.83 Tonne
2.38 Cubic Metres



PANELS 22, 24, 26, 28, 30, 32, 34, 36, 42, 44, & 46

100 Thick
5.83 Tonne
2.38 Cubic Metres

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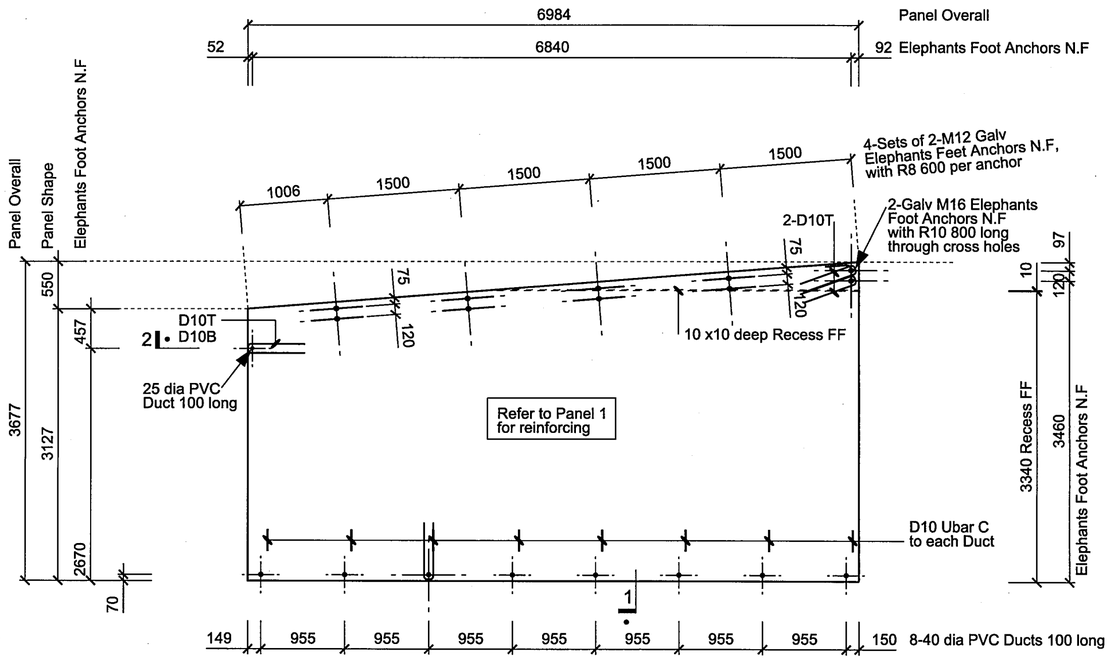
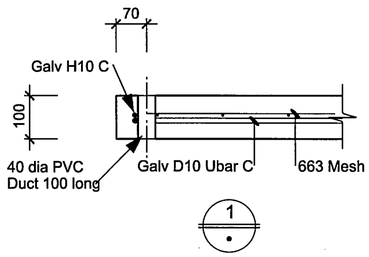
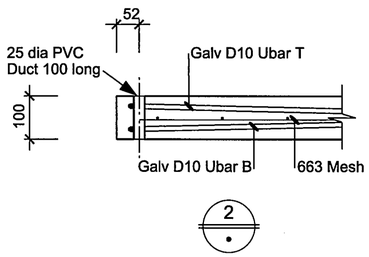


PANEL 47

100 Thick
5.83 Tonne
2.38 Cubic Metres





Alan Reay Consultants
395 MADRAS STREET
P.O. BOX 3911
CHRISTCHURCH
TEL: (03) 366 0434
FAX: (03) 379 3981
EMAIL: eng@arcl.co.nz

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ISSUE	DESCRIPTION
A	CONSENT ISSUE

PROJECT NAME:
DEVELOPMENT WASHBOURNES ROAD

DRAWING NAME:
PRECAST CONCRETE PANELS 21 - 36 & 41 - 47

SCALES AT A2	DRAWING No.	ISSUE	FILE
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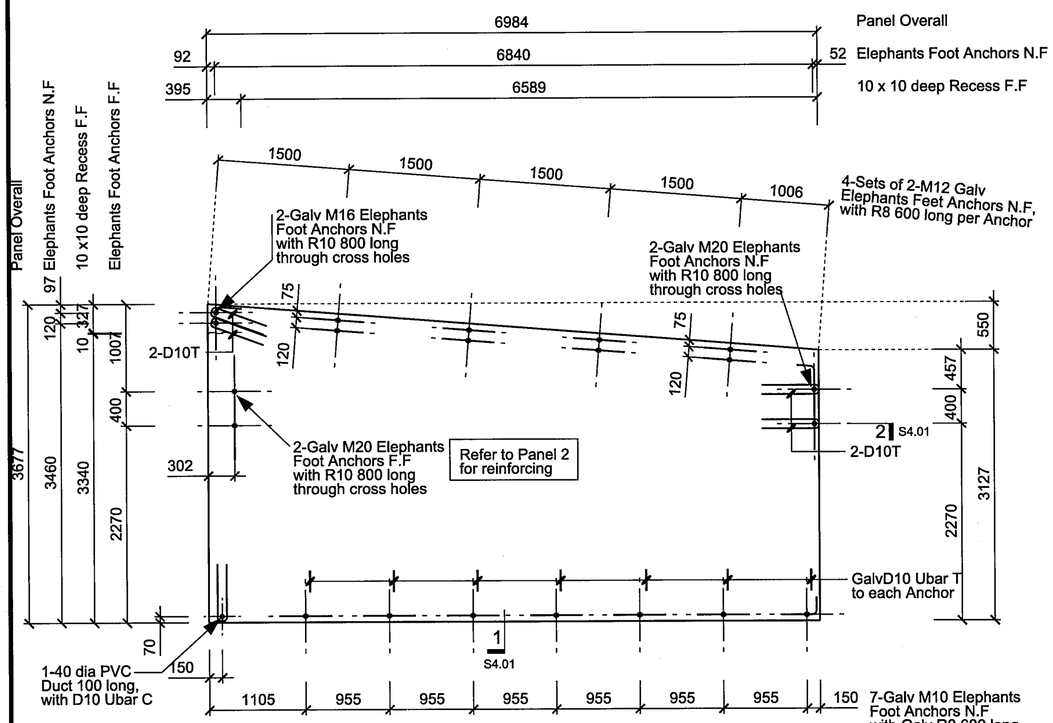
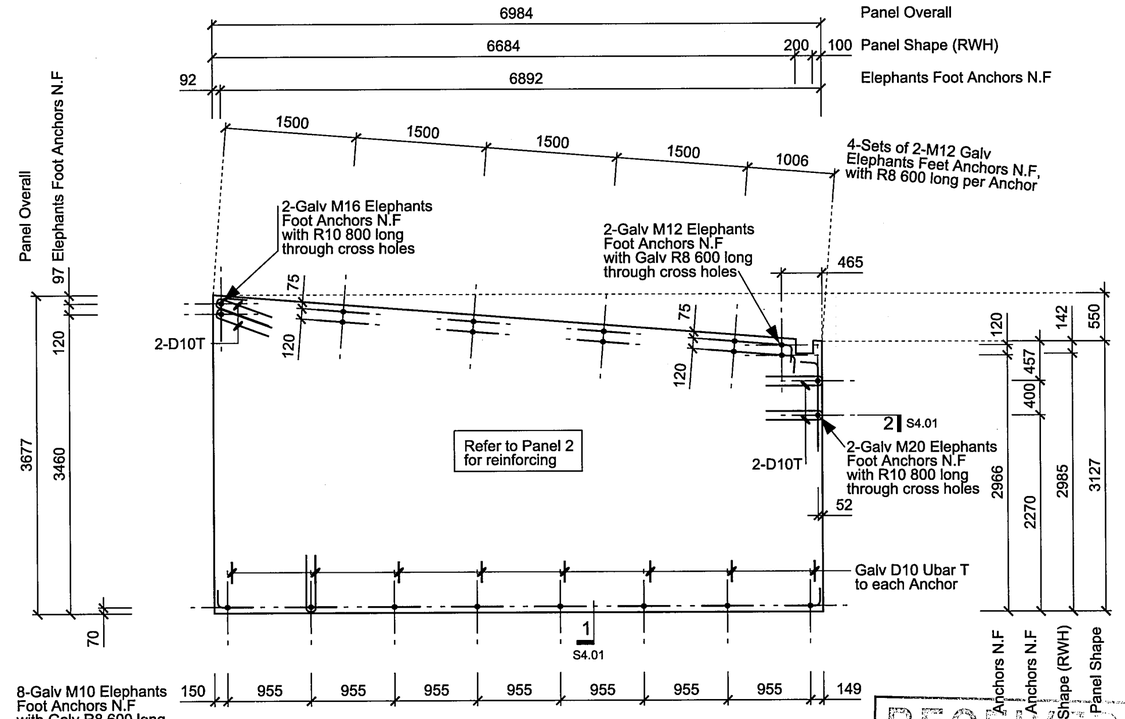


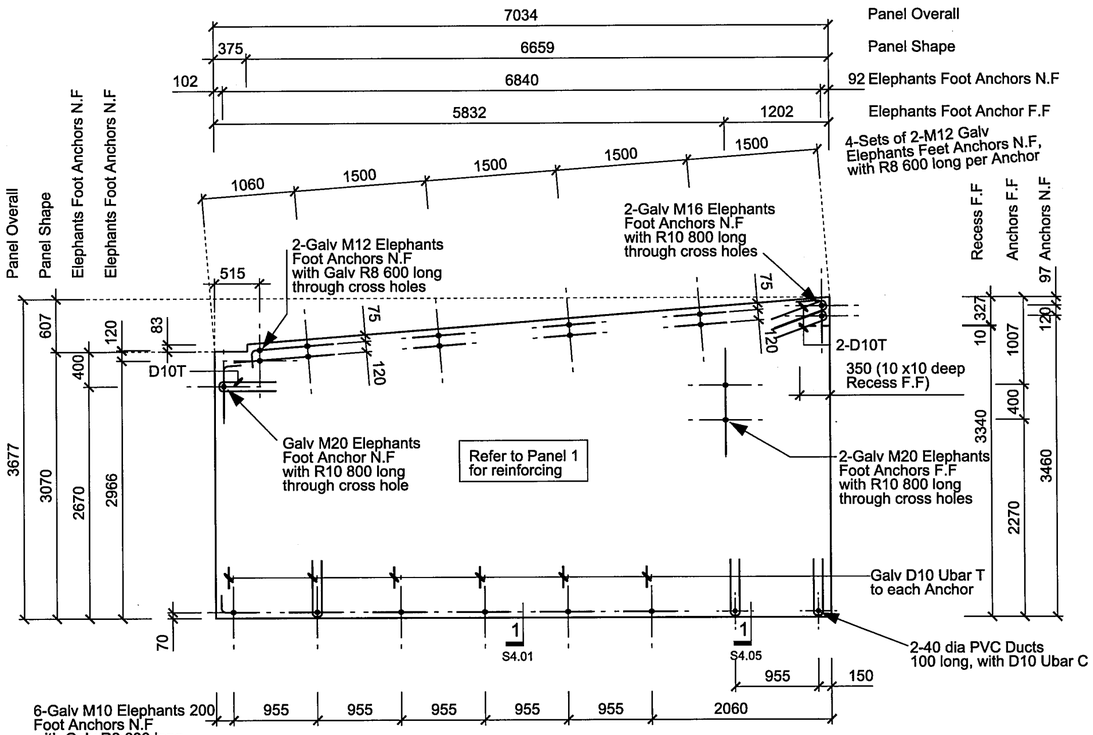
PANEL 48

100 Thick
5.83 Tonne
2.38 Cubic Metres



PANELS 50 & 54

100 Thick
5.81 Tonne
2.37 Cubic Metres



PANEL 51

100 Thick
5.83 Tonne
2.38 Cubic Metres

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FAX: (03) 379 3981
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ISSUE	DESCRIPTION
A	CONSENT ISSUE

PROJECT NAME:
DEVELOPMENT WASHBOURNES ROAD

DRAWING NAME:
PRECAST CONCRETE PANELS 48, 50, 51 & 54

SCALES AT A2	DRAWING No.	ISSUE	FILE
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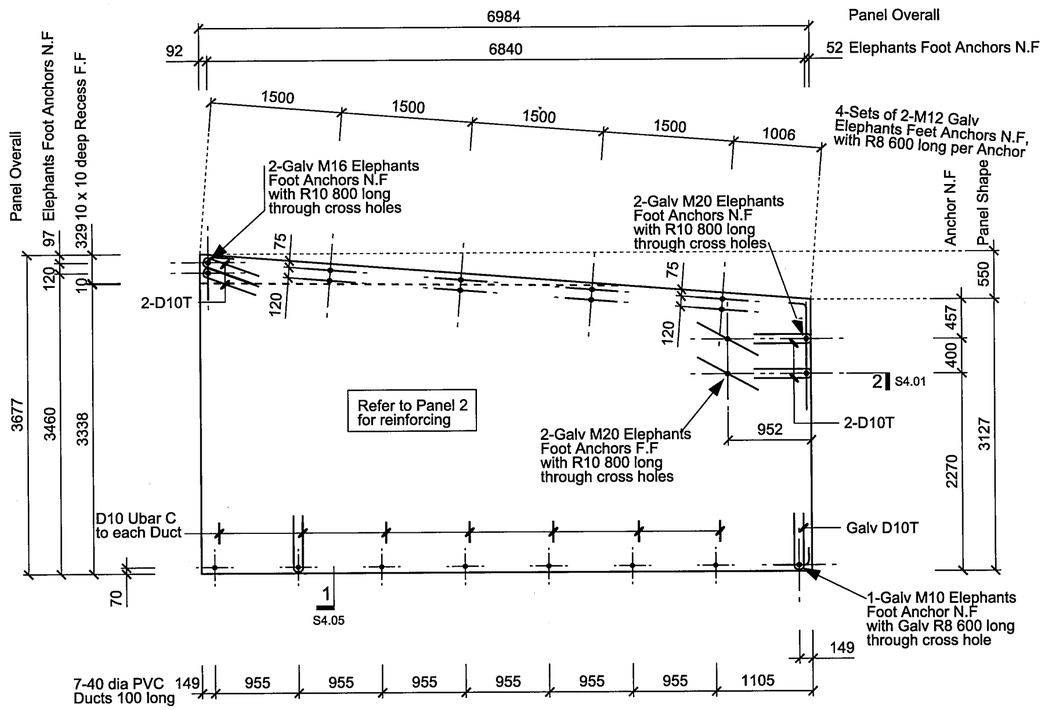
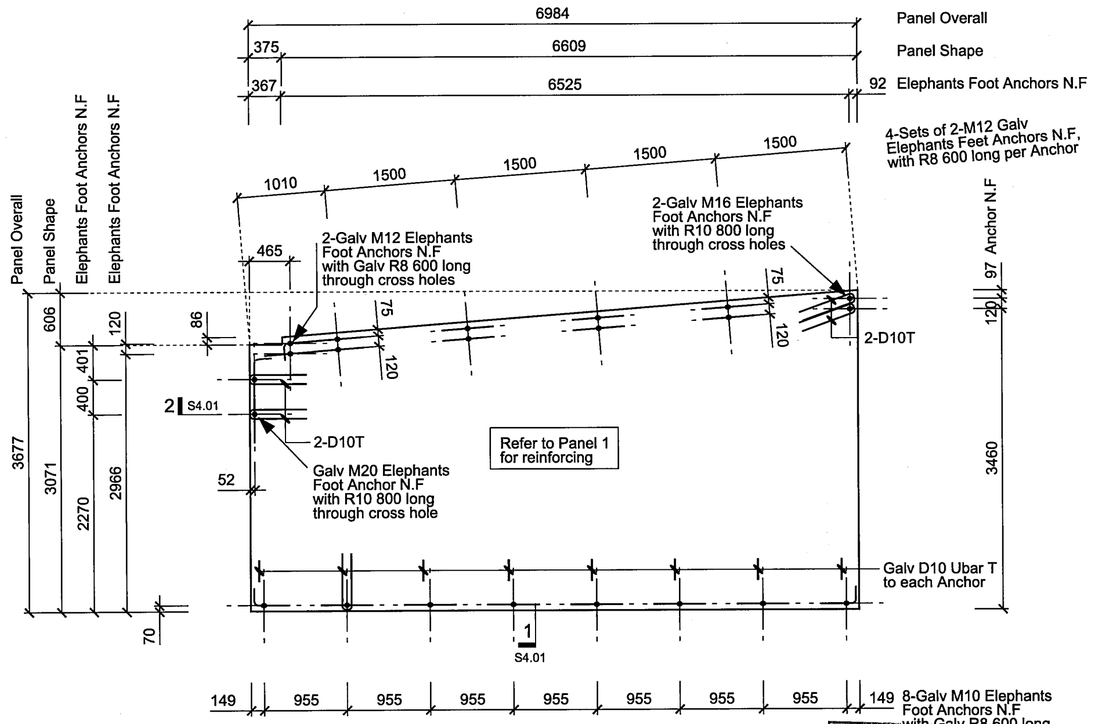


PANEL 52
100 Thick
5.83 Tonne
2.38 Cubic Metres



PANEL 55
100 Thick
5.83 Tonne
2.38 Cubic Metres

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P.O. BOX 3911
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FAX: (03) 379 3981
EMAIL: eng@arcl.co.nz

Refer to Panel Location Plan for notes

ISSUE	DESCRIPTION
A	CONSENT ISSUE

PROJECT NAME:
DEVELOPMENT
WASHBOURNES ROAD

DRAWING NAME:
PRECAST CONCRETE
PANELS 52 & 55

SCALES AT A2	DRAWING No.	ISSUE	FILE
1:50	S4.07	A	8740

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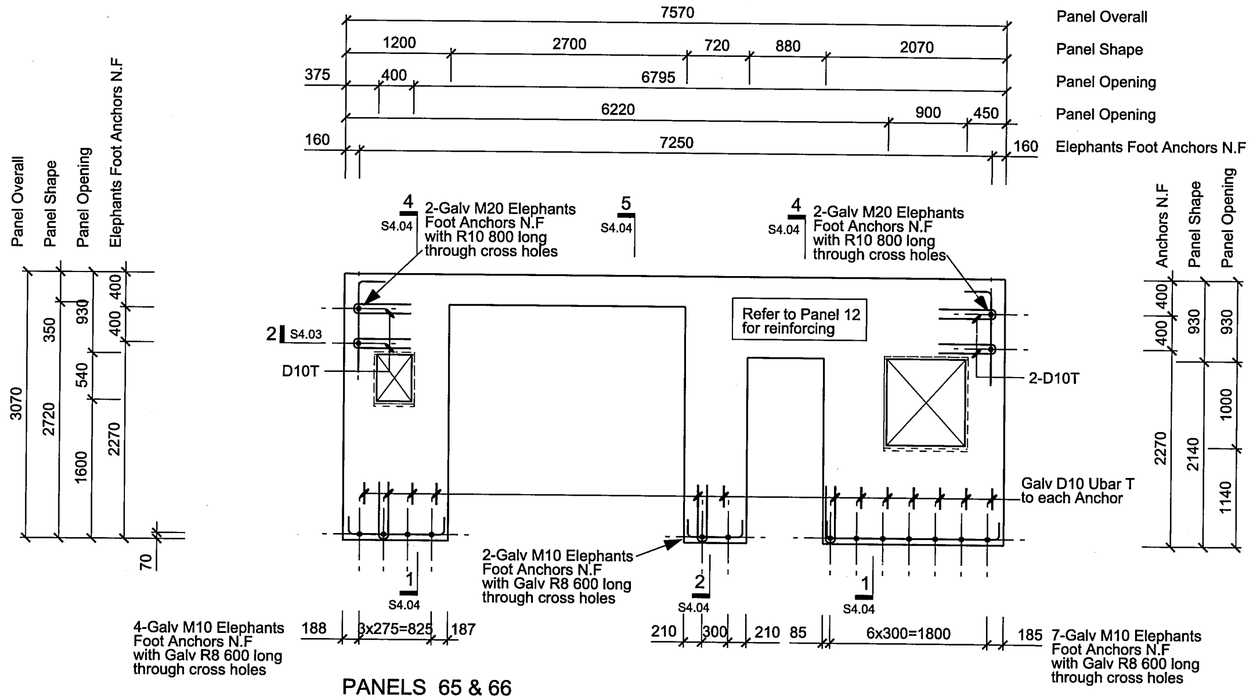
FAX: (03) 379 3981
EMAIL: eng@arcl.co.nz



PANELS 65 & 66
150 Thick
4.73 Tonne
1.93 Cubic Metres

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CONSENT DOCUMENT
17 DEC 2008
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ISSUE	DESCRIPTION
A	CONSENT ISSUE

PROJECT NAME:
DEVELOPMENT WASHBOURNES ROAD

DRAWING NAME:
PRECAST CONCRETE PANELS 65 & 66

SCALES AT A2	DRAWING No.	ISSUE	FILE
1:50	S4.08	A	8740

APPENDIX B
Geotechnical Assessment Report

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5814

21 June 2023

GEOTECH
consulting ltd

Engenium Ltd
PO Box 3911
Christchurch

Attention Thomas Ryan
thomas.ryan@engenium.co.nz

Dear Thomas

Geotechnical assessment for 37 Washbournes Road, Wigram (Lot 2 DP 370841)

1. Introduction

37 Washbournes Road is a 4021 m^2 commercial/industrial site, set back from the south-east side of Washbournes Road, 140 m from the Vickery Street intersection. The terrain is flat, however the Hayton Stream runs just outside the north-east boundary in a 2 m deep channel.

The property was developed in 2009 with two blocks of light industrial buildings, being, single storey precast concrete construction with 3 m high walls and lightweight, low pitch, roofs. Each unit is around 7.4 m by 14.2 m. The smaller block (yellow highlight in Fig 1) comprises six standard units with a larger block of nine units (purple highlight in Fig 1) aligned to the south-east boundary.

We understand that Engenium are completing a Detailed Seismic Assessment of the buildings and you have requested a geotechnical opinion on:

a) the probable shallow bearing capacity and

b) liquefaction hazard to the buildings.

In undertaking this assessment we have reviewed:

- Engenium Soil Investigation logs, file E1942, dated 8 June 2023
- Structural drawings by Alan Reay Consultants, File 8740, Consent Issue, 2008.
- Existing data from the NZ Geotechnical Database (NZGD), ECan wells database and Geotech Consulting records.

We have been advised that the buildings are in good condition and were in continuous use throughout and since the Canterbury earthquakes, with only minor structural damage.

This assessment is for the purpose of the DSA only. If new foundations or new buildings are proposed a more comprehensive assessment and additional investigation is recommended.

Nick Traylen **email** ntraylen@geotech.co.nz
Ian McCahon **email** mccahon@geotech.co.nz
Andrew Hurley **email** ahurley@geotech.co.nz

Geotech Consulting (NZ) Ltd
PO Box 130 122
Christchurch
New Zealand

2. Ground Conditions

Existing records

We are not aware of any soils testing on the site prior to construction. Geotech Consulting hold records from 11 Washbournes Road (100 m to the north-east) which show shallow gravels from the near ground surface to 11m depth. The NZGD holds records from 47 Washbournes Road (100 m to the west) which shows a sequence of soft silts and sands with gravel only struck at 8 m depth. The gravels are continuous to 14 m depth where there is a layer of sand and then more gravels from 16 m to 20 m.

The regional geological model[1] predicts the Riccarton Gravels to be at 19 m depth and about 21 m thick in this location. The borehole from No 47 erroneously records Riccarton Gravels from 8 m depth.

The Riccarton gravel is underlain with further layers of silt, sand and gravel for another ≈450 m before volcanic rock from the Lyttelton volcano is encountered.

Site Investigation

Engenium have carried out 3 machine auger boreholes on the site, as shown on Figure 1. Findings from these tests can be generalized as silts over gravels with gravels found at varying depths from 0.8 m (BH2 – north east of site), 1.3 m (BH1 – north-west of site) and not encountered within the 2.6 m depth of BH3 (south-west of site). No field testing was carried out with these boreholes.

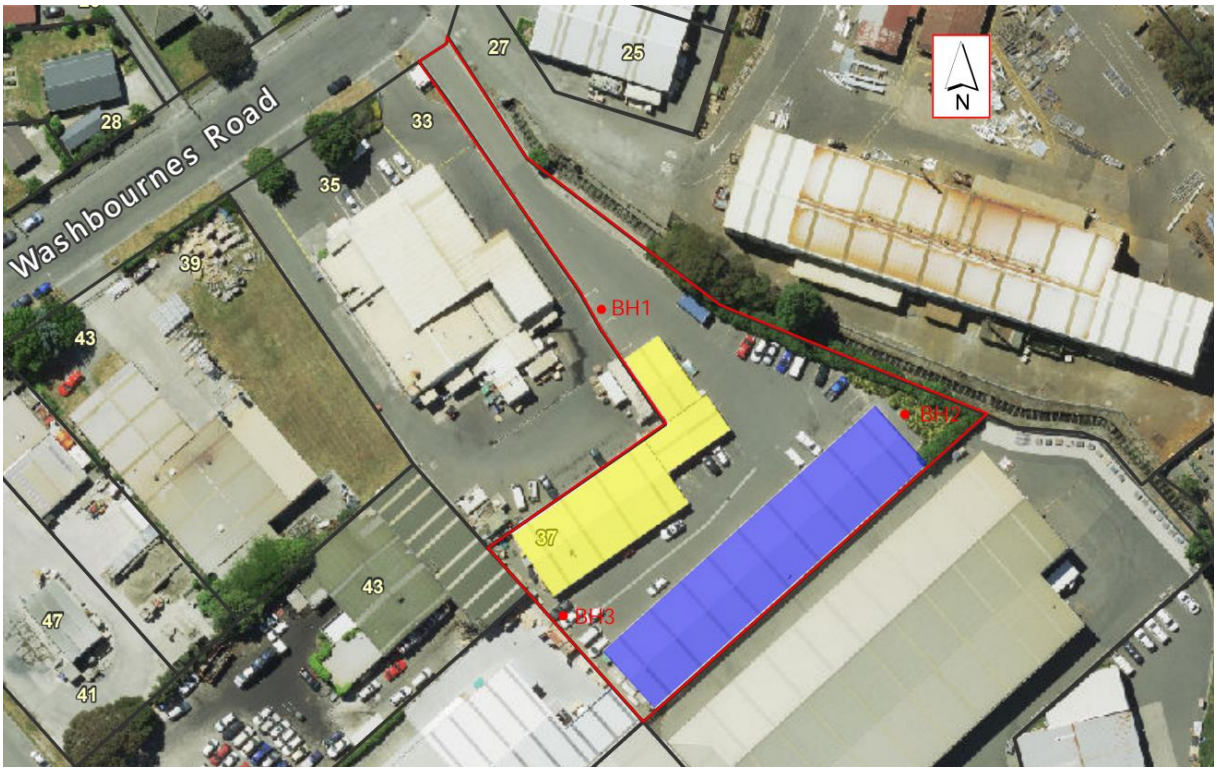


Figure 1 Site location with borehole test sites

For the purpose of this assessment we have assumed the soil profile is similar to that seen at 47 Washbournes Road. This is consistent with the BH3 and is more conservative than the ground profile indicated by BH1 and BH2.

[1] Begg, Jones & Barrell (2015). *Geology and geomorphology of urban Christchurch and north-eastern Canterbury. GNS Science Geological Map 3*

Groundwater

The GNS Science median water table elevation study[2] indicates a groundwater depth of around 5.2 m, rising to 4.2 m, at the 85%ile. Measurements from previous investigation are 7.2 m deep at 11 Washbournes Road and 6 m deep at 47 Washbournes Road. Groundwater was observed at 2.3 m depth in BH3 in the Engenium investigation but this is noted as being within a plastic silt so may be 'perched' and not representative of the actual groundwater level.

For this assessment we prefer the GNS model which has the benefit of measurements taken over many months.

3. Seismicity

Seismic Category

The deep alluvial soils that underlie this part of Christchurch make this a Class D, deep or soft soil site, in terms of the seismic design requirements of NZS 1170.5:2004.

Seismic Hazard

Peak ground accelerations (PGA) for liquefaction assessment, are shown in Table 3-1. These PGA's are appropriate for Class D sites and Importance Level 2 (IL2) structures, that is, normal occupancy buildings.

Design Case	PGA	Magnitude	Return period
SLS_A[3]	0.13g	M7.5	25 yr
SLS_B	0.19g	M6	25 yr
ULS[4]	0.35g	M7.5	500 yr

Table 3-1 Seismic design cases for liquefaction assessment (MBIE 2012, 2014)

Recent Earthquakes

The site has been subject to repeated shaking from the Canterbury earthquakes. Estimates[5] of peak ground acceleration are shown in Table 3-2.

Earthquake	**Mag.**	**Peak Ground Acceleration, PGA**		
		Mean	**Equivalent M7.5**	**$PGA_{10_7.5}$**
4 Sep 2010	7.1	0.26	0.23	0.15
22 February 2011	6.2	0.29	0.21	0.13
13 June 2011	6.0	0.16	0.10	0.06
23 Dec 2011	5.9	0.14	0.09	0.05

Table 3-2 Estimated PGA for the main Canterbury earthquakes
(green background indicates 'sufficiently tested')

[2] "GNS Science Median Groundwater Surface Elevations", Map Layer CGD5160 - 10 June 2014, retrieved June 2023 from the New Zealand Geotechnical Database

[3] SLS = serviceability limit state, the building should perform such that damage is minimal, easily repairable and doesn't affect the function of the structure.

[4] ULS = ultimate limit state, the building may suffer significant damage but should not collapse.

[5] By Bradley and Hughes 2012 as provided in the New Zealand Geotechnical Database

To establish whether the site can be considered 'sufficiently tested' at the 1 in 25 year, 'Serviceability Limit State' (SLS), the estimated mean PGA for each earthquake has been converted to an equivalent PGA for a magnitude M7.5 earthquake plus the PGA with 90% probability of being exceeded ($PGA_{10_7.5}$).

A 90% exceedance PGA of 0.13g is the level at which the MBIE guidance accepts a site as being 'sufficiently tested'. This site has almost certainly been tested in excess of SLS levels in the September 2010 and February 2011 earthquakes.

4. Liquefaction Assessment

Technical Category

MBIE Technical Categories[6] are not directly applicable to commercial/industrial sites but they can be used to give an indication of the likely scale of effects. The nearest residential properties are on the opposite side of Washbournes Road and are classified as TC1.

Liquefaction Vulnerability

The 2020 Christchurch Liquefaction Vulnerability Study, by Tonkin & Taylor, shows the site as being within an extensive area of "Low" vulnerability[7].

Site Performance

There are no reports of ground damage for this area in EQC records[8], including in the residential properties across the road.

Our own examination of aerial photographs taken after the February 2011 earthquake confirms that there is no surface ejecta on the site or in the immediate vicinity.

Interpretation of LiDAR surveys[9] suggests a total liquefaction induced settlement as less than 0.1 m between September 2010 and June 2011 (including the three main earthquakes in that period). 0.1 m is within the margin of error for these LiDAR surveys.

Liquefaction potential

The risk of damaging liquefaction is assessed as being low, given the groundwater depth and the observed building performance in the Canterbury Earthquakes. The site has been 'sufficiently tested' in terms of the MBIE Guidelines, such that we can rely on these observations for future performance at SLS.

Indicative liquefaction settlements can be obtained (using the liquefaction evaluation layer in the NZGD[10]) for the two CPT's on 47 Washbournes Road where the soil profile is silts to around 8 m depth over gravels. These tests indicate 10-20 mm liquefaction settlement at SLS and 40-60 mm at ULS. Both with the 85%ile groundwater level and standard deterministic parameters of P_L = 15% and C_{FC} = 0.0.

These analytical results are on the boundary of TC1 & TC2 behaviour (with 15 mm (SLS) & 50 mm (ULS) as the 'index' limits). However, given the building and site performance and the nearby residential site classification we assess this site as being equivalent to TC1.

---

[6] MBIE (2012). Guidance on repairing and rebuilding houses affected by the Canterbury earthquakes.

[7] https://apps.canterburymaps.govt.nz/ChristchurchLiquefactionViewer/

[8] NZ Geotechnical Database (2013), Map Layers CGD0200 & CGD0300, retrieved June 2023.

[9] NZ Geotechnical Database (2015) "LiDAR and Digital Elevation Models", Map Layer CGD0500 - 20 July 2015, retrieved June 2023.

[10] NZ Geotechnical Database (2015) "Liquefaction Evaluation of CPT Investigations", Map Layer CGD0050 - 20 July 2015, retrieved June 2023.

5. Foundations

The existing foundations are a series of post hole footings as shown on the 2008 structural drawings by Alan Reay Consultants, File 8740, Drawing numbers S1.01 to S1.05.

To derive %NBS under earthquake conditions, a 'probable' capacity has been assessed using standard bearing capacity formulae with assumed soil properties of ϕ = 29° and γ = 17 kN/m^3 in the shallow silts.

Postholes A, B & C (600 diameter, 1000 deep)	600 kPa
Postholes D (600 diameter, 500 deep)	300 kPa

This probable capacity is a 'geotechnical ultimate' capacity and can be used, with a G & 0.3Q & Eu load combination, without applying a capacity reduction factor.

The foundations are expected to be sufficiently far above the groundwater level to avoid punching shear failure, and as such, the provided bearing capacities assume that liquefaction does not affect the capacity.

6. Geotechnical Hazards

A Detailed Seismic Assessment requires an assessment of geotechnical hazards. Potential geohazards are outlined in Table 6-1.

Hazard	Assessment	Category
Inundation	The site is not within the District Plan Flood Management Area	Hazard originates outside the building footprint so does not influence %NBS rating
Erosion	There are no significant waterways nearby and hence no significant risk of erosion	n/a
Slope stability	The built area of the site is flat so there is no risk of slope failure	n/a
Falling debris	The site is flat so there is no risk from falling debris	n/a
Active Faulting	There are no known active faults within the site	Hazard originates outside the building footprint so does not influence %NBS rating
Earthquake	The site is subject to earthquake shaking, in common with the rest of Christchurch	Hazard originates outside the building footprint so does not influence %NBS rating
Liquefaction	There is a low risk of subsidence in the event of seismic liquefaction.	Specifically assessed (ref Section 4)

Table 6-1 Geohazard summary

An initial review of geotechnical hazards, in collaboration with Engenium, and considering building performance, and the likely effects of liquefaction, has led to the conclusion that the building is "structurally dominated" for the purposes of a DSA.

Outstanding risks and uncertainties relating to these hazards are considered in Table 6-2. The possible response should be considered if these geotechnical risks are considered material for this "structurally dominated" building.

Key uncertainty	Possible response	Mitigating factors
Groundwater level is higher than expected	Install a standpipe and monitor variations until an estimate of long-term median level is obtained	Observed performance of the site and structure to date, indicates that shallow liquefaction is unlikely to have occurred.
Ground profile is assumed from a combination of shallow tests and deep tests that are 100 m away.	Site specific deep testing.	Observed performance of the structure to date. Further testing would be needed if a repeat of this settlement is structurally not tolerable.

Table 6-2 Summary of key uncertainties

The risk posed by each of these uncertainties is that the foundations will not perform as well as expected, that is, may undergo excessive settlement in a future earthquake. However, the good performance of the foundations, in the September 2010 and February 2011 earthquakes, gives confidence that this is unlikely.

7. Conclusions

- Ground conditions include a hardfill surface layer, underlain by alluvial deposits of interbedded silts, sands and silt/sand mixtures. Shallow gravels are assumed to be at around 8 m depth with the Riccarton gravels expected at 19 m deep.
- Groundwater is assessed from the GNS Shallow Groundwater model, at 5.2 m median depth, rising to 4.2 m at the 85%ile.
- Liquefaction is assessed as being a low risk based on recorded site and building performance. The site was 'sufficiently tested' and future performance can be expected to be similar to what was experienced during the Canterbury earthquakes.
- 'Probable' bearing capacities for posthole foundations are assessed at 300 kPa, for 0.5 m deep and 600 kPa for 1m deep.
- The buildings are assumed to be 'structurally dominated' for assessment of %NBS.
- This assessment is for the purpose of a detailed seismic assessment of existing buildings. Any new foundation work will require additional testing and assessment.

8. Limitations

This report has been prepared for the benefit of Engenium Ltd as our client, with respect to the brief. The reliance by other parties on the information or opinions contained in the report shall be at such parties' sole risk.

Recommendations and opinions (not to be construed as guarantees) in this report are based on a discrete number of test pits, boreholes, and data derived from other sources. The nature and continuity of subsoil conditions away from the test locations are inferred and it must be appreciated that actual conditions could vary from the assumed model.

During any excavation works, the site should be examined by an Engineer or Engineering Geologist competent to judge whether the exposed subsoils are compatible with the inferred conditions on which the report has been based. It is possible that the nature of the exposed subsoils may require further investigation, and the subsequent modification of design work.

NZGD Important notice

Information in this report that references the NZGD was created from maps and/or data extracted from the NZ Geotechnical Database (https://www.nzgd.org.nz/), which were prepared and/or compiled for the Earthquake Commission (EQC) to assist in assessing insurance claims made under the Earthquake Commission Act 1993. The source maps and data were not intended for any other purpose. EQC and its engineers, Tonkin & Taylor, have no liability for any use of the maps and data or for the consequences of any person relying on them in any way.

Yours faithfully
Geotech Consulting Limited

Andrew Hurley
Principal Geotechnical Engineer
CMEngNZ, CPEng
p. 03 982 2838

NZGD ID: BH_23577

104 m west of site

AECOM

LOG OF DRILLHOLE

HOLE IDENTIFICATION	BH01

Client	Mimico Limited
Project	Geotechnical Assessment
Project number	60271402

Co-ordinates	2473797mE		5740266mN
Orientation	-90°	Elevation	28m (Approx)
Location	45 & 47 Washbournes Road		
Feature	New building site		

GEOLOGICAL DESCRIPTION	Test Records (Shear Vane residual - peak 0 - 200 kPa; N Values 0 - 50)	Drilling Method / Casing remarks	Core Loss/Lift (0 - 100%)	Depth	Graphic Log	MATERIAL DESCRIPTION (Subordinate MAJOR minor; colour; structure. Strength; moisture condition; grading; bedding; plasticity; sensitivity; major fraction description; subordinate fraction description; minor fraction description etc)	Instrumentation
							N/A
FILL						Concrete	
		Sonic				Coarse GRAVEL and COBBLES; grey. Loose; dry. Gravel and cobbles are rounded to sub-angular; unweathered; Greywacke.	
TOP SOIL						Medium to coarse GRAVEL; grey. Loose; dry. Gravel is sub-rounded to sub-angular; unweathered; Greywacke.	
SPRINGSTON FORMATION - Alluvial channel and overbank deposits				1		Organic clayey fine SAND; dark brown. Very loose to loose; moist.	
	ss 3,2,2,2,2,2 N=8	SPT				Silty SAND; orange brown. Very loose to loose; moist.	
		Sonic		2			
	ss 0,0,0,1,1,1 N=3	SPT		3			
		Sonic		4			
	ss 0,0,0,1,1,2 N=4	SPT				Clayey SILT; orange-brown. Very soft to soft; moist; low plasticity.	
		Sonic		5		Fine SAND; orange-brown. Very loose to loose; moist.	1
	ss 0,0,1,0,0,1 N=2	SPT		6		Clayey SILT with some wood fragments; grey. Soft; moist; moderate plasticity. Organic fragments are black; <20mm long.	
		Sonic		7		SILT with some sand; grey. Soft to firm; moist; low plasticity.	
	ss 2,0,0,1,0,1 N=2	SPT				Fine SAND; orange. Very loose; moist.	
RICCARTON FORMATION - Alluvial channel and overbank deposits		Sonic		8		Sandy fine to coarse GRAVEL with minor COBBLES; grey. Dense to very dense; moist; well graded. Sand is fine to coarse. Gravel is rounded to subangular; unweathered; greywacke.	
	ss 6,13,15,16,14,14 N=56	SPT		9			
		Sonic		10			

SPRINGSTON FORMATION

RICCARTON FORMATION

GROUNDWATER OBSERVATIONS			
Depth	Piezometer	Reading	Date
6m	1		22/08/2012

Date logged	22/08/2012
Logged	JWW
Checked	MPN

Casing Details
Depth Diameter

Remarks

Hand held Shear Vane

vane shear strength per NZGD guideline

Driller	Started
McMillan	22/08/2012
Drill Rig	Finished
Sonic	22/08/2012
Core Boxes	10

Date Printed: 4/09/2012

DRILLHOLE LOG SOIL 45-47 WASHBOURNES ROAD BOREHOLE.GPJ BASE.GDT 04/09/12

NZGD ID: BH_23577

AECOM

LOG OF DRILLHOLE

HOLE IDENTIFICATION	BH01

Client	Mimico Limited
Project	Geotechnical Assessment
Project number	60271402

Co-ordinates	2473797mE		5740266mN
Orientation	-90°	Elevation	28m (Approx)
Location	45 & 47 Washbournes Road		
Feature	New building site		

GEOLOGICAL DESCRIPTION	Test Records (Shear Vane residual - peak 0 - 200 kPa; N Values 0 - 50)	Drilling Method / Casing remarks	Core Loss/Lift (0 - 100%)	Depth	Graphic Log	MATERIAL DESCRIPTION (Subordinate MAJOR minor; colour; structure. Strength; moisture condition; grading; bedding; plasticity; sensitivity; major fraction description; subordinate fraction description; minor fraction description etc)	Instrumentation
RICCARTON FORMATION - Alluvial channel and overbank deposits	sc 6,6,5,5,5,5 N=20	SPT				Sandy fine to coarse GRAVEL with minor COBBLES; grey. Dense to very dense; moist; well graded. Sand is fine to coarse. Gravel is rounded to subangular; unweathered; greywacke. 10.5m: Becoming medium dense.	N/A
		Sonic		11			
	sc 9,6,7,6,6,6 N=25	SPT		12			
		Sonic		13			
	sc 6,7,7,10,10,9 N=36	SPT		14		Silty fine SAND; orange-brown. Dense; moist.	
		Sonic		15			
	sc 8,12,9,7,7,7 N=30	SPT					
		Sonic		16		Sandy coarse GRAVEL and COBBLES; grey. Dense; moist; well graded. Sand is fine to coarse. Gravel and cobbles are rounded to subangular; unweathered; greywacke.	
	sc 9,9,8,7,9,10 N=34	SPT					
		Sonic		17		Sandy fine to coarse GRAVEL with some COBBLES; grey. Dense; moist; well graded. Sand is fine to coarse. Gravel and cobbles are rounded to subangular; unweathered; greywacke. 18m: Becoming very dense.	
	sc 7,7,7,12,25,18 N=62	SPT		18			
	sc 9,7,6,5,7,8 N=26	SPT					
		Sonic		19			
	sc 6,9,14,18,13 for 0mm REFUSAL N>50	SPT		20			
						BH01 terminated at 20m Target Depth	

RICCARTON FORMATION

GROUNDWATER OBSERVATIONS			
Depth	Piezometer	Reading	Date

Date logged	22/08/2012
Logged	JWW
Checked	MPN

Casing Details
Depth Diameter

Remarks

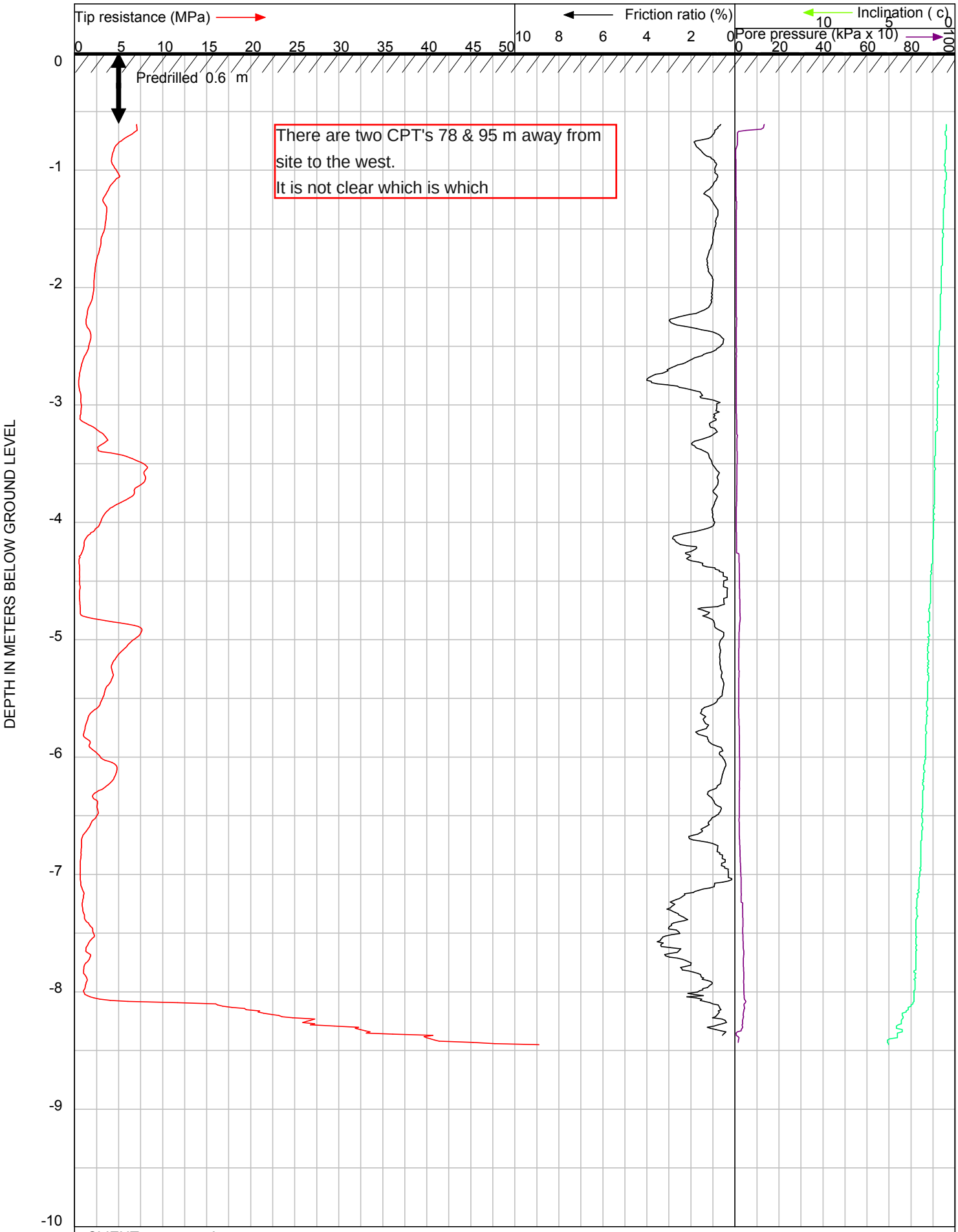
Hand held Shear Vane

vane shear strength per NZGD guideline

Driller	Started
McMillan	22/08/2012
Drill Rig	Finished
Sonic	22/08/2012
Core Boxes	10

Date Printed: 4/09/2012

DRILLHOLE LOG SOIL 45-47 WASHBOURNES ROAD BOREHOLE.GPJ BASE.GDT 04/09/12



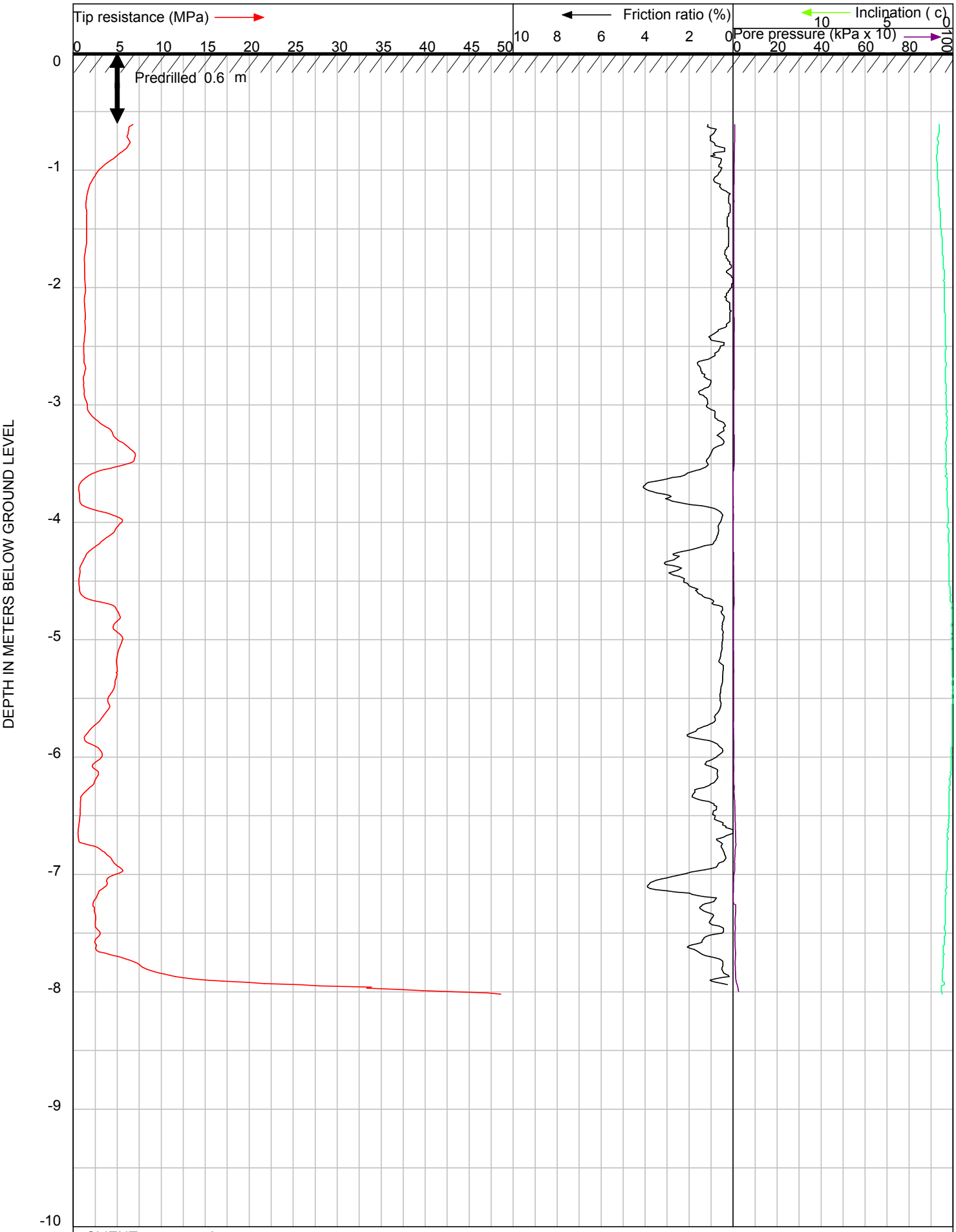
CLIENT	:	Aecom
LOCATION	:	45-47 Washbournes Road, Christchurch
DATE	:	13-7-2012
OPERATOR	:	S.Cardona
REMARK 1	:	CPTu001
REMARK 2	:	Effective Refusal

JOB # : 10857

TEST #: 1

CONE TYPE/SERIAL #: I-CFXYP20-10/ 100992T

McMILLAN DRILLING SERVICES
120 High St Southbridge CANTERBURY NZ
Ph +64 3 324 2571 Fax +64 3 324 2431
www.drilling.co.nz



CLIENT	:	Aecom
LOCATION	:	45-47 Washbournes Road, Christchurch
DATE	:	13-7-2012
OPERATOR	:	S.Cardona
REMARK 1	:	CPTu002
REMARK 2	:	Effective Refusal

JOB # : 10857

TEST #: 2

CONE TYPE/SERIAL #: I-CFXYP20-10/ 100992T

McMILLAN DRILLING SERVICES
120 High St Southbridge CANTERBURY NZ
Ph +64 3 324 2571 Fax +64 3 324 2431
www.drilling.co.nz

Borehole 100 m north-east of site

McMILLAN Drilling

Client: Geotech Consulting Ltd	**Bore No.:** BH001
Project: 11 Washbournes Road, Christchurch	**Job No.:** 20190

Site Location: 11 Washbournes Road, Christchurch **Date:** 2/9/2021 - 2/9/2021

Grid Reference: 1564021.58m E, 5178764.31m N (NZTM) - Map or aerial photograph **Rig Operator:** K. Morris

Elevation: 0.00m **Datum:** Ground **Equipment:** AMS VTR9570 - track

Description	Graphic Log	Depth	Method	Runs and recovery	Water at hole depth	SPT (NZS 4402) N-value Uncorrected	In-Situ Tests	Samples - Permeability tests	Installation & Resources
ASPHALT. 0.02m FILL. 0.60m		0.5	70mm OD Dual tube	100%					
Fine SAND; brown. 1.20m		1.0							
Sandy fine to coarse GRAVEL. Sand, fine to coarse. 1.50m		1.5					SPT (C) 1.50m 24, 17 / 33, 20, 7 375mm Effective Refusal N = **60+**		
Sandy fine to medium GRAVEL with trace of cobbles. Sand, fine to coarse.		2.0		80%					
		2.5							
		3.0		70%			SPT (C) 3.00m 1, 2 / 3, 3, 3, 4 450mm N = **13**		
		3.5							
		4.0							
		4.5		100%			SPT (C) 4.50m 11, 10 / 11, 13, 15, 13 450mm N = **52**		
		5.0							
Sandy fine to medium GRAVEL with trace of cobbles; grey. Loosely packed; sand, fine to coarse. 5.40m		5.5							
		6.0		30%			SPT (C) 6.00m 23, 20 / 18, 18, 14, 10 450mm N = **60**		
		6.5							
		7.0							
		7.5		20%			SPT (C) 7.50m 3, 4 / 4, 2, 5, 5 450mm N = **16**		
		8.0							
		8.5							
		9.0		70%			SPT (C) 9.00m 4, 3 / 4, 2, 6, 6 450mm N = **18**		
9.30 - 9.80m Trace of silt.		9.5							
		10.0							
	EOH: 10.5m				5.95m bgl		SPT (C) 10.50m 6, 7 / 9, 12, 13, 16 450mm N = **50**		

Remarks

Hammer energy ratio 77.9%

Additional Resources:

Core boxes: -
Liners/splits: 7m
Bentonite: 0.75 bag

APPENDIX C
Photograph Location Plan

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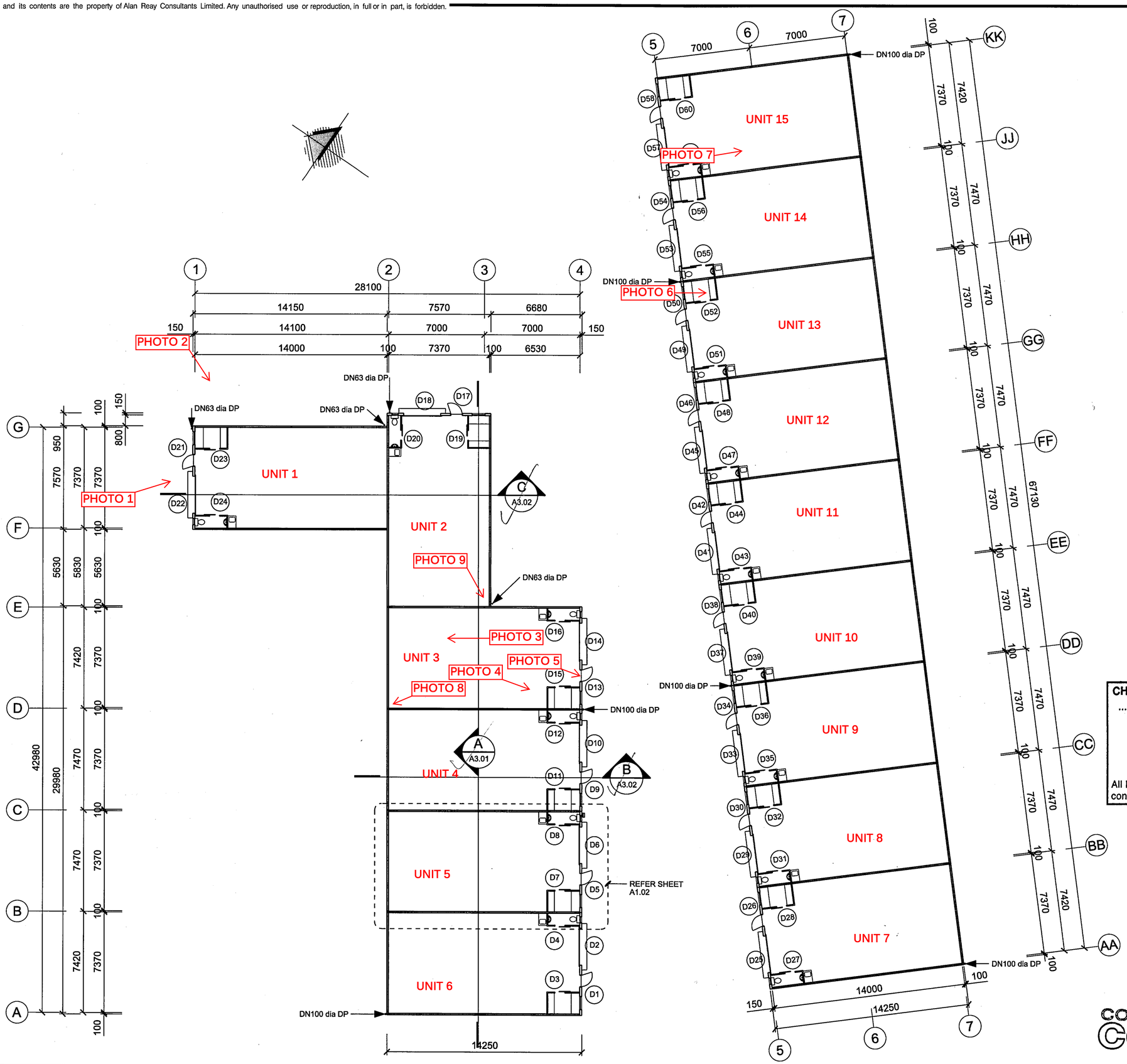


Alan Reay Consultants

395 MADRAS STREET
P.O. BOX 3911
CHRISTCHURCH

TEL: (03) 366 0434
FAX: (03) 379 3981
EMAIL: eng@arcl.co.nz

Conditions and / or advice notes are attached as part of the documents for this building consent. You should read these as they contain information to assist with achieving code compliance and / or compliance with other legislation that may apply

BCP

RECEIVED
26 SEP 2008
10089011
EPA & IE UNITS

RICHARD MEEKIN
Building Consent Consultant

CHRISTCHURCH CITY COUNCIL
CONSENT DOCUMENT
17 DEC 2008
File Copy
All building work shall comply with the consented documents.

ISSUE	DESCRIPTION
A	CONSENT ISSUE

PROJECT NAME:
DEVELOPMENT WASHBOURNES ROAD

DRAWING NAME:
OVERALL FLOOR PLAN

SCALES AT A2	DRAWING No.	ISSUE	FILE
1:200	A1.01	A	8740

CONSENTS COPY

APPENDIX D
Qualitative Assessment Report

© Engenium Consulting Engineers

Initial Evaluation Procedure (IEP) Assessment - Completed for MPROP Ltd

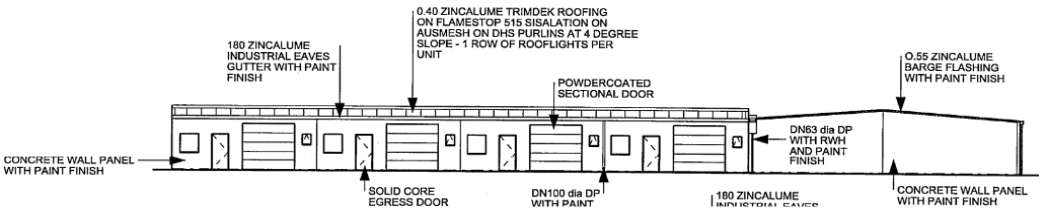
WARNING!! *This initial evaluation has been carried out solely as an initial seismic assessment of the building following the procedure set out in the "The Seismic Assessment of Existing Buildings" Technical Guidelines for Engineering Assessments, July 2017. This spreadsheet must be read in conjunction with the limitations set out in the accompanying report, and should not be relied on by any party for any other purpose. Detailed inspections and engineering calculations, or engineering judgements based on them, have not been undertaken, and these may lead to a different result or seismic grade.*

Street Number & Name:	**37 Washbournes Road**	**Job No.:**	**E1942**
AKA:		**By:**	**TR**
Name of building:		**Date:**	**2/03/2023**
City:	**Christchurch**	**Revision No.:**	**1**

Table IEP-1 Initial Evaluation Procedure Step 1

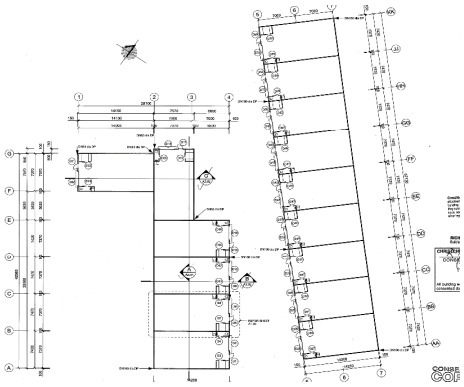
Step 1 - General Information

1.1 Photos (attach sufficient to describe building)



NOTE: THERE ARE MORE PHOTOS ON PAGE 1a ATTACHED

1.2 Sketches (plans etc, show items of interest)



NOTE: THERE ARE MORE SKETCHES ON PAGE 1a ATTACHED

1.3 List relevant features (Note: only 10 lines of text will print in this box. If further text required use Page 1a)

Single level small warehouse units with lightweight internal offices.
Precast concrete walls on all sides of each unit, 100mm to 150mm thick, two sides supported.
Lightweight roofing on steel purlins spanning between concrete panels. No roof bracing.
Panels clipped together with steel corner brackets bolted into M10 Elephant's foot ferrules.

1.4 Note information sources Tick as appropriate

Visual Inspection of Exterior	☐	**Specifications**	☐
Visual Inspection of Interior	☐	**Geotechnical Reports**	☐
Drawings (note type)	☑	**Other (list)**	☐

Council record of drawings only

Initial Evaluation Procedure (IEP) Assessment - Completed for MPROP Ltd

Street Number & Name:	**37 Washbournes Road**	**Job No.:**	**E1942**
AKA:		**By:**	**TR**
Name of building:		**Date:**	**2/03/2023**
City:	**Christchurch**	**Revision No.:**	**1**

Table IEP-2 Initial Evaluation Procedure Step 2

Step 2 - Determination of *(%NBS)* $_b$

(Baseline *(%NBS)* for particular building - refer Section B5)

2.1 Determine nominal *(%NBS)* = *(%NBS)* $_{nom}$

	Longitudinal	Transverse
a) Building Strengthening Data		
Tick if building is known to have been strengthened in this direction	☐	☐
If strengthened, enter percentage of code the building has been strengthened to	N/A	N/A
b) Year of Design/Strengthening, Building Type and Seismic Zone		
Pre 1935	○	○
1935-1965	○	○
1965-1976	○	○
1976-1984	○	○
1984-1992	○	○
1992-2004	○	○
2004-2011	◉	◉
Post Aug 2011	○	○
Building Type:	**Not applicable**	**Not applicable**
Seismic Zone:	**Not applicable**	**Not applicable**
c) Soil Type		
From NZS1170.5:2004, Cl 3.1.3 :	D Soft Soil	D Soft Soil
From NZS4203:1992, Cl 4.6.2.2 : **(for 1992 to 2004 and only if known)**	**Not applicable**	**Not applicable**
d) Estimate Period, *T*		
Comment:		
h_n =	4	4 m
A_c =	1.00	1.00 m^2
Moment Resisting Concrete Frames: $T = \max\{0.09h_n^{0.75}, 0.4\}$	○	○
Moment Resisting Steel Frames: $T = \max\{0.14h_n^{0.75}, 0.4\}$	○	○
Eccentrically Braced Steel Frames: $T = \max\{0.08h_n^{0.75}, 0.4\}$	○	○
All Other Frame Structures: $T = \max\{0.06h_n^{0.75}, 0.4\}$	○	○
Concrete Shear Walls $T = \max\{0.09h_n^{0.75}/A_c^{0.5}, 0.4\}$	○	○
Masonry Shear Walls: $T \leq 0.4$sec	○	○
User Defined (input Period):	◉	◉
Where h_n = height in metres from the base of the structure to the uppermost seismic weight or mass. **T:**	0.40	0.40
e) Factor A: Strengthening factor determined using result from (a) above (set to 1.0 if not strengthened) **Factor A:**	1.00	1.00
f) Factor B: Determined from NZSEE Guidelines Figure 3A.1 using results (a) to (e) above **Factor B:**	1.00	1.00
g) Factor C: For reinforced concrete buildings designed between 1976-84 Factor C = 1.2, otherwise take as 1.0. **Factor C:**	1.00	1.00
h) Factor D: For buildings designed prior to 1935 Factor D = 0.8 except for Wellington and Napier (1931-1935) where Factor D may be taken as 1.0, otherwise take as 1.0. **Factor D:**	1.00	1.00
***(%NBS)* $_{nom}$ = AxBxCxD** ***(%NBS)* $_{nom}$**	100%	100%

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Initial Evaluation Procedure (IEP) Assessment - Completed for MPROP Ltd

Street Number & Name:	37 Washbournes Road	Job No.:	E1942
AKA:		By:	TR
Name of building:		Date:	2/03/2023
City:	Christchurch	Revision No.:	1

Table IEP-2 Initial Evaluation Procedure Step 2 continued

2.2 Near Fault Scaling Factor, Factor E

If $T \leq 1.5$sec, Factor E = 1

		Longitudinal	Transverse
a) Near Fault Factor, *N(T,D)* (from NZS1170.5:2004, Cl 3.1.6)	N(T,D):	1	1
b) Factor E = 1/N(T,D)	Factor E:	1.00	1.00

2.3 Hazard Scaling Factor, Factor F

a) Hazard Factor, *Z*, for site

Location: Christchurch — Refer right for user-defined locations

Z =	0.3	(from NZS1170.5:2004, Table 3.3)
Z_{1992} =	0.8	(NZS4203:1992 Zone Factor from accompanying Figure 3.5(b))
Z_{2004} =	0.22	(from NZS1170.5:2004, Table 3.3)

b) Factor F

For pre 1992	=	$1/Z$
For 1992-2011	=	Z_{1992}/Z
For post 2011	=	Z_{2004}/Z

	Longitudinal	Transverse
Factor F:	0.73	0.73

2.4 Return Period Scaling Factor, Factor G

a) Design Importance Level, I

(Set to 1 if not known. For buildings designed prior to 1965 and known to be designed as a public building set to 1.25. For buildings designed 1965-1976 and known to be designed as a public building set to 1.33 for Zone A or 1.2 for Zone B. For 1976-1984 set I value.)

b) Design Risk Factor, R_o

(set to 1.0 if other than 1976-2004, or not known)

c) Return Period Factor, R

(from NZS1170.0:2004 Building Importance Level)

		Longitudinal	Transverse
I =		1	1
R_o =		1	1
Choose Importance Level		○ 1 ● 2 ○ 3 ○ 4	○ 1 ● 2 ○ 3 ○ 4
R =		1.0	1.0

d) Factor G = IR_o/R

	Longitudinal	Transverse
Factor G:	1.00	1.00

2.5 Ductility Scaling Factor, Factor H

a) Available Displacement Ductility Within Existing Structure

Comment:

		Longitudinal	Transverse
μ =		1.00	1.00
b) Factor H		k_μ	k_μ
For pre 1976 (maximum of 2)	=	1.00	1.00
For 1976 onwards	=	1	1
Factor H:		1.00	1.00

(where k_μ is NZS1170.5:2004 Inelastic Spectrum Scaling Factor, from accompanying Table 3.3)

2.6 Structural Performance Scaling Factor, Factor I

a) Structural Performance Factor, S_p

(from accompanying Figure 3.4)

Tick if light timber-framed construction in this direction

		Longitudinal	Transverse
		☐	☐
S_p =		1.00	1.00
b) Structural Performance Scaling Factor	= $1/S_p$		
Factor I:		1.00	1.00

Note Factor B values for 1992 to 2004 have been multiplied by 0.67 to account for Sp in this period

	Longitudinal	Transverse
2.7 Baseline *%NBS* for Building, $(\%NBS)_b$ (equals $(\%NBS)_{nom}$ x E x F x G x H x I)	73%	73%

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Initial Evaluation Procedure (IEP) Assessment - Completed for MPROP Ltd

Street Number & Name:	37 Washbournes Road	Job No.:	E1942
AKA:		By:	TR
Name of building:		Date:	2/03/2023
City:	Christchurch	Revision No.:	1

Table IEP-3 Initial Evaluation Procedure Step 3

Step 3 - Assessment of Performance Achievement Ratio (PAR)

(Refer Appendix B - Section B3.2)

a) Longitudinal Direction

potential CSWs — **Effect on Structural Performance** (Choose a value - Do not interpolate) — **Factors**

3.1 Plan Irregularity

Effect on Structural Performance ○ *Severe* ○ *Significant* ◉ *Insignificant* — **Factor A** 1.0

No irregularity, no rigid diaphragms.

3.2 Vertical Irregularity

Effect on Structural Performance ○ *Severe* ○ *Significant* ◉ *Insignificant* — **Factor B** 1.0

Single level building.

3.3 Short Columns

Effect on Structural Performance ○ *Severe* ○ *Significant* ◉ *Insignificant* — **Factor C** 1.0

No columns in building.

3.4 Pounding Potential

(Estimate D1 and D2 and set D = the lower of the two, or 1.0 if no potential for pounding, or consequences are considered to be minimal)

a) Factor D1: - Pounding Effect

Note:
Values given assume the building has a frame structure. For stiff buildings (eg shear walls), the effect of pounding may be reduced by taking the coefficient to the right of the value applicable to frame buildings.

Factor D1 For Longitudinal Direction: 1.0

Table for Selection of Factor D1	Severe	Significant	Insignificant
Separation	0<Sep<.005H	.005<Sep<.01H	Sep>.01H
Alignment of Floors within 20% of Storey Height	○ 1	◉ 1	○ 1
Alignment of Floors not within 20% of Storey Height	○ 0.4	○ 0.7	○ 0.8

Single level building.

b) Factor D2: - Height Difference Effect

Factor D2 For Longitudinal Direction: 1.0

Table for Selection of Factor D2	Severe	Significant	Insignificant
	0<Sep<.005H	.005<Sep<.01H	Sep>.01H
Height Difference > 4 Storeys	○ 0.4	○ 0.7	◉ 1
Height Difference 2 to 4 Storeys	○ 0.7	○ 0.9	○ 1
Height Difference < 2 Storeys	○ 1	○ 1	○ 1

Single level building.

Factor D 1.0

3.5 Site Characteristics - *Stability, landslide threat, liquefaction etc as it affects the structural performance from a life-safety perspective*

Effect on Structural Performance ○ *Severe* ○ *Significant* ◉ *Insignificant* — **Factor E** 1.0

Low liquefaction hazard in this area. (Source: Canterbury Maps).

3.6 Other Factors - *for allowance of all other relevant characterstics of the building*

For ≤ 3 storeys - Maximum value 2.5
otherwise - Maximum value 1.5.
No minimum.

Factor F 1.0

Record rationale for choice of Factor F:

Brittle panel to panel connections

3.7 Performance Achievement Ratio (PAR)
(equals **A x B x C x D x E x F**)

PAR

Longitudinal **1.00**

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Initial Evaluation Procedure (IEP) Assessment - Completed for MPROP Ltd

Street Number & Name:	37 Washbournes Road	Job No.:	E1942
AKA:		By:	TR
Name of building:		Date:	2/03/2023
City:	Christchurch	Revision No.:	1

Table IEP-3 Initial Evaluation Procedure Step 3

Step 3 - Assessment of Performance Achievement Ratio (PAR)

(Refer Appendix B - Section B3.2)

b) Transverse Direction

Factors

potential CSWs — **Effect on Structural Performance** (Choose a value - Do not interpolate)

3.1 Plan Irregularity

Effect on Structural Performance ○ *Severe* ○ *Significant* ○ *Insignificant* ◉ — **Factor A** 1.0

No irregularity, no rigid diaphragms.

3.2 Vertical Irregularity

Effect on Structural Performance ○ *Severe* ○ *Significant* ○ *Insignificant* ◉ — **Factor B** 1.0

Single level building.

3.3 Short Columns

Effect on Structural Performance ○ *Severe* ○ *Significant* ○ *Insignificant* ◉ — **Factor C** 1.0

No columns in building.

3.4 Pounding Potential

(Estimate D1 and D2 and set D = the lower of the two, or 1.0 if no potential for pounding, or consequences are considered to be minimal)

a) Factor D1: - Pounding Effect

Note:
Values given assume the building has a frame structure. For stiff buildings (eg shear walls), the effect of pounding may be reduced by taking the coefficient to the right of the value applicable to frame buildings.

Factor D1 For Transverse Direction: 1.0

Table for Selection of Factor D1	Severe	Significant	Insignificant
Separation	0<Sep<.005H	.005<Sep<.01H	Sep>.01H
Alignment of Floors within 20% of Storey Height	○ 1	○ 1	◉ 1
Alignment of Floors not within 20% of Storey Height	○ 0.4	○ 0.7	○ 0.8

Single level building.

b) Factor D2: - Height Difference Effect

Factor D2 For Transverse Direction: 1.0

Table for Selection of Factor D2	Severe	Significant	Insignificant
	0<Sep<.005H	.005<Sep<.01H	Sep>.01H
Height Difference > 4 Storeys	○ 0.4	○ 0.7	◉ 1
Height Difference 2 to 4 Storeys	○ 0.7	○ 0.9	○ 1
Height Difference < 2 Storeys	○ 1	○ 1	○ 1

Single level building.

Factor D 1.0

3.5 Site Characteristics - *Stability, landslide threat, liquefaction etc as it affects the structural performance from a life-safety perspective*

Effect on Structural Performance ○ *Severe* ○ *Significant* ○ *Insignificant* — **Factor E** 1.0

Low liquefaction hazard in this area. (Source: Canterbury Maps).

3.6 Other Factors - *for allowance of all other relevant characterstics of the building*

For ≤ 3 storeys - Maximum value 2.5
otherwise - Maximum value 1.5.
No minimum.

Factor F 1.00

Record rationale for choice of Factor F:

Brittle panel to panel connections

3.7 Performance Achievement Ratio (PAR)
(equals **A x B x C x D x E x F**)

PAR

Transverse **1.00**

WARNING!! *This initial evaluation has been carried out solely as an initial seismic assessment of the building following the procedure set out in "The Seismic Assessment of Existing Buildings" Technical Guidelines for Engineering Assessments, July 2017. This spreadsheet must be read in conjunction with the limitations set out in the accompanying report, and should not be relied on by any party for any other purpose. Detailed inspections and engineering calculations, or engineering judgements based on them, have not been undertaken, and these may lead to a different result or seismic grade.*

Initial Evaluation Procedure (IEP) Assessment - Completed for MPROP Ltd

Street Number & Name:	37 Washbournes Road	Job No.:	E1942
AKA:		By:	TR
Name of building:		Date:	2/03/2023
City:	Christchurch	Revision No.:	1

Table IEP-4 Initial Evaluation Procedure Steps 4, 5, 6 and 7

Step 4 - Percentage of New Building Standard *(%NBS)*

		Longitudinal	Transverse
4.1	**Assessed Baseline *%NBS* *(%NBS)*$_b$** (from Table IEP - 1)	73%	73%
4.2	**Performance Achievement Ratio (PAR)** (from Table IEP - 2)	1.00	1.00
4.3	**PAR x Baseline *(%NBS)*$_b$**	75%	75%
4.4	**Percentage New Building Standard *(%NBS)* - Seismic Rating** (Use lower of two values from Step 4.3)		75%

Step 5 - Is *%NBS* < 34? NO

Step 6 - Potentially Earthquake Risk (is *%NBS* < 67)? NO

Step 7 - Provisional Grading for Seismic Risk based on IEP

Seismic Grade B

Additional Comments (items of note affecting IEP based seismic rating)

Relationship between Grade and *%NBS*:

Grade:	A+	A	B	C	D	E
%NBS:	> 100	100 to 80	79 to 67	66 to 34	< 34 to 20	< 20

WARNING!! *This initial evaluation has been carried out solely as an initial seismic assessment of the building following the procedure set out in "The Seismic Assessment of Existing Buildings" Technical Guidelines for Engineering Assessments, July 2017. This spreadsheet must be read in conjunction with the limitations set out in the accompanying report, and should not be relied on by any party for any other purpose. Detailed inspections and engineering calculations, or engineering judgements based on them, have not been undertaken, and these may lead to a different result or seismic grade.*

Initial Evaluation Procedure (IEP) Assessment - Completed for MPROP Ltd

Street Number & Name:	37 Washbournes Road	Job No.:	E1942
AKA:		By:	TR
Name of building:		Date:	2/03/2023
City:	Christchurch	Revision No.:	1

Table IEP-5 Initial Evaluation Procedure Step 8

Step 8 - Identification of potential Severe Structural Weaknesses (SSWs) that could result in significant risk to a significant number of occupants

8.1	Number of storeys above ground level	1
8.2	Presence of heavy concrete floors and/or concrete roof? (Y/N)	N

Potential Severe Structural Weaknesses (SSWs):

Note: Options that are greyed out are not applicable and need not be considered.

Occupancy not considered to be significant - no further consideration required

Risk not considered to be significant - no further consideration required

The following potential Severe Structural Weaknesses (SSWs) have been identified in the building that could result in significant risk to a significant number of occupants:

1. None identified
2. Weak or soft storey (except top storey)
3. Brittle columns and/or beam-column joints the deformations of which are not constrained by other structural elements
4. Flat slab buildings with lateral capacity reliant on low ductility slab-to-column connections
5. No identifiable connection between primary structure and diaphragms
6. Ledge and gap stairs

IEP Assessment Confirmed by ..... **Signature**

..... **Name**

..... **CPEng. No**

WARNING!! *This initial evaluation has been carried out solely as an initial seismic assessment of the building following the procedure set out in "The Seismic Assessment of Existing Buildings" Technical Guidelines for Engineering Assessments, July 2017. This spreadsheet must be read in conjunction with the limitations set out in the accompanying report, and should not be relied on by any party for any other purpose. Detailed inspections and engineering calculations, or engineering judgements based on them, have not been undertaken, and these may lead to a different result or seismic grade.*

Initial Evaluation Procedure (IEP) Assessment - Completed for MPROP Ltd

Street Number & Name:	**37 Washbournes Road**	**Job No.:**	**E1942**
AKA:		**By:**	**TR**
Name of building:		**Date:**	**2/03/2023**
City:	**Christchurch**	**Revision No.:**	**1**

Table IEP-1a Additional Photos and Sketches

Add any additional photographs, notes or sketches required below:

Note: print this page separately

WARNING!! *This initial evaluation has been carried out solely as an initial seismic assessment of the building following the procedure set out "The Seismic Assessment of Existing Buildings" Technical Guidelines for Engineering Assessments, July 2017. This spreadsheet must be read in conjunction with the limitations set out in the accompanying report, and should not be relied on by any party for any other purpose. Detailed inspections and engineering calculations, or engineering judgements based on them, have not been undertaken, and these may lead to a different result or seismic grade.*

APPENDIX E
Quantitative Assessment Calculations

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STRUCTURAL CALCULATIONS

PROJECT	: 37 Washbournes Road - DSA	
PROJECT MANAGER	: Jeremy Mitchell	FILE: E1942
STRUCTURAL DESIGNER	: Thomas Ryan	DATE: 30/5/2023

SECTION	SECTION NO.	SIGN
Structural Description	1	
Geometry	2	
Loading	3	
Materials Information	4	
Walls In-Plane	5	
Walls Out-of-Plane	6	
Foundations	7	
Connections	8	
Miscellaneous	9	

CALCULATIONS	AUTHORISED FOR
COMPLETED BY: TR	ENGENIUM LTD
DATE: 12/7/2023	

CONFIDENTIAL
These calculations are confidential in terms of section 27 of the Building Act.

Calculations

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PROJECT	DATE	FILE
ENGINEER	SECT	PAGE

Assessment Summary.

		Page.	% NBS.
Panels in-plane	- Base connection shear friction	5-8	0.97
Panels out-of-plane	- Yieldline	6-3	0.96
Foundation	- Sliding	7-1	0.70 (not life safety)
Connection	- Base	8-5	0.76
	- Corner - outside	8-11	0.83
Misc.	- Amplified forces on connection	7-1	>1
	- Panel out of plane yielding at $\mu = 1.25$	9-3	0.79

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Tel 03 366 0434, Fax 03 379 3981, Email eng@engenium.co.nz, Internet www.engenium.co.nz

Calculations

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PROJECT 37 Washbourner Road	DATE	FILE E1942
ENGINEER TR	SECT	PAGE

Detailed Seismic Assessment

- 15 unit single level commercial building (8).
- Designed in 2008 by ARCL
- Located at 37 Washbourners Rd, Christchurch.
- Construction is concrete precast tilt-panels to form commercial units approx 7.5m wide x 14m deep, x 3.5m high at the apex.
- Purlins clear span across units supporting lightweight metal cladding
- Foundations are unreinforced concrete pad footings poured on firm gravels, nominal depth of 1m
- Floors are 120mm thick concrete with panel fixings built into the slabs.
- Each unit is designed with lightweight timber framed offices and toilet.
-

3.0 LOADS

Building importance level 2

Design working life 50 years

which gives annual probabilities of exceedance of

Ultimate Limit State	P	R
Wind	0.002	500
Snow	0.006667	150
Earthquake	0.002	500

Serviceability Limit State

SLS1	0.04	25
SLS2	-	

COMBINATIONS

Strength

$$1.35G$$
$$1.2G + 1.5Q$$
$$1.2G + 1.5\psi_l Q$$
$$1.2G + W_u + \psi_c Q$$
$$0.9G + W_u$$
$$G + E_u + \psi_E Q$$
$$1.2G + S_u + \psi_c Q$$

Serviceability

$$G$$
$$\Psi_s Q$$
$$\Psi_l Q$$
$$W_s$$
$$E_s$$
$$S_s$$

3.1 PERMANENT ACTIONS

Portal frame spacing (m) 7.5 *Spacing*

	(kPa)	(kN/m)	
Warehouse roof - general			
Cladding	0.060	0.45	*Topspan 0.55*
MSS purlins	0.052	0.39	*xx*
Steel frames	0.000	0.00	*No steel frames*
Solar panels	0.000	0.00	*No allowance for solar*
Insulation / netting / battens	0.010	0.08	*1kg/m2 allowance for insulation, netting, battens*
Sprinklers	0.000	0.00	*No allowance for sprinklers*
Services	0.020	0.15	*2kg/m2 allowance for other services*
SDL	0.000	0.00	*None*
Ceiling	0.000	0.00	*None*
On purlins	0.142	1.07	*ie weight excl steel frame*
On frames	0.142	1.07	*On portal frame / top chord*
Warehouse walls - precast concrete			
Precast concrete	2.40	18.00	*100mm thick*
Precast concrete	3.6	27.00	*150mm thick*
Office partition walls			
Linings	0.200	1.50	*10mm gib each side*
Framing	0.200	1.50	*Steel stud*
Services	0.010	0.00	*No steel frames*
	0.41		

3.5 EARTHQUAKE ACTIONS

Elastic Site Spectra $C(T) = C_h(T).Z.R.N(T,D)$

Site location	Christchurch	
Subsoil class	D	
Importance Level	2	
Design life	50	Years
Z	0.30	
R_{ULS}	1	
R_{SLS}	0.25	
N(T,D)	1	

$C_d(T_1)$ Horizontal design action coefficient

				ULS			SLS
	μ	1	1.25	2	3.5	6	1
	Sp	1	0.925	0.7	0.7	0.7	0.7
Period	**Ch(T)**						
0.0	1.12	0.34	0.31	0.24	0.24	0.24	0.06
0.1	3.00	0.90	0.80	0.55	0.46	0.37	0.16
0.2	3.00	0.90	0.78	0.49	0.37	0.26	0.16
0.3	3.00	0.90	0.75	0.44	0.30	0.20	0.16
0.4	3.00	0.90	0.73	0.40	0.26	0.16	0.158
0.5	3.00	0.90	0.71	0.37	0.23	0.14	0.16
0.6	2.84	0.85	0.65	0.32	0.19	0.11	0.15
0.7	2.53	0.76	0.56	0.27	0.15	0.09	0.13
0.8	2.29	0.69	0.51	0.24	0.14	0.08	0.12
0.9	2.09	0.63	0.46	0.22	0.13	0.07	0.11
1.0	1.93	0.58	0.43	0.20	**0.12**	0.07	**0.10**
1.5	1.43	0.43	0.32	0.15	**0.09**	0.05	0.07
2.0	1.07	0.32	0.24	0.11	0.06	0.04	0.06
2.5	0.86	0.26	0.19	0.09	0.05	0.03	0.04
3.0	0.71	0.21	0.16	0.07	0.04	0.02	0.04
3.5	0.52	0.16	0.12	0.06	0.03	0.02	0.03
4.0	0.40	0.12	0.09	0.04	0.02	0.01	0.02
4.5	0.32	0.10	0.07	0.03	0.02	0.01	0.02

Structural Characteristics

Seismic Weight

$$W_i = G_i + \Sigma \psi_E Q_i$$

where 0.6 for storage
0.3 otherwise

Calculations

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PARTS AND COMPONENTS LOADS $F_{ph} = C_p(T_p)\, C_{ph}\, R_p\, W_p \leq 3.6\, W_p$

Assumed Period	T_P	0.4 s	*Assumed horizontal parts period*
	Ch(T=0)	1.12 s	
	Z	0.30	
	R	1.00	
	N(T,D)	1.00	
Site Hazard Coefficient	Cd(T=0)	0.336	
Spectral Shape Coefficient	$C_i(T_P)$	2.0	*For parts with T <= 0.75s*
Part Risk Factor	Rp	1.0	*ULS case*
	Hn	12 m	

Horizontal Design Coefficients

μ_p	1	1.25	1.5	2	3
C_{ph}	1	0.85	0.75	0.55	0.45
H(m)	Fph/Wp				
12	2.02	1.71	1.51	1.11	0.91
11	1.90	1.62	1.43	1.05	0.86
10	1.79	1.52	1.34	0.99	0.81
9	1.68	1.43	1.26	0.92	0.76
8	1.57	1.33	1.18	0.86	0.71
7	1.46	1.24	1.09	0.80	0.66
6	1.34	1.14	1.01	0.74	0.60
5	1.23	1.05	0.92	0.68	0.55
4	1.12	0.95	0.84	0.62	0.50
3	1.01	0.86	0.76	0.55	0.45
2	0.90	0.76	0.67	0.49	0.40
1	0.78	0.67	0.59	0.43	0.35
0	0.67	0.57	0.50	0.37	0.30

Tv	0.15	s	*Assumed vertical parts period*
α	1.5		
	Ch(T)	C(T)	
T=0	1.12	0.34	
T=0.1	3	0.90	
T=3	0.71	0.21	

Vertical Design Coefficients

μ_p	$\mu_p = 1$	$\mu_p = 1.25$	$\mu_p = 2$	$\mu_p = 3$
C_{pv}	1	0.85	0.55	0.45
Vertical part period, Tpv (s)				
0	0.50	0.43	0.28	0.23
0.05	1.35	1.15	0.74	0.61
0.1	1.35	1.15	0.74	0.61
0.23	0.98	0.83	0.54	0.44
0.3	0.80	0.68	0.44	0.36
0.4	0.65	0.55	0.36	0.29
0.5	0.55	0.47	0.30	0.25
0.6	0.48	0.41	0.26	0.21
0.7	0.43	0.36	0.23	0.19
0.8	0.38	0.33	0.21	0.17
0.9	0.35	0.30	0.19	0.16
1	0.33	0.28	0.18	0.15
1.1	0.30	0.26	0.17	0.14
1.2	0.28	0.24	0.16	0.13
1.3	0.27	0.23	0.15	0.12
1.4	0.25	0.21	0.14	0.11
1.5	0.24	0.20	0.13	0.11

Calculations

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Materials

Concrete	f'_c	f_{prob}	E_{prob}
- Tilt-panels	25 MPa	40 MPa	29725 MPa
- Floor Slab	20 MPa	30 MPa	25740 MPa
- Postholes	20 MPa	30 MPa	

Reinforcing Steel	f_y	f_{prob}	ϕ_o	
H Bars	500 MPa	540 MPa	1.25	E = 200 GPa
D Bars	300 MPa	324 MPa	1.25	

CF purlins	f_y	f_{prob}
DHS 200/12		
DHS 250/13		

Structural Steel		
EA	300	345

Calculations

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Wall in-plane

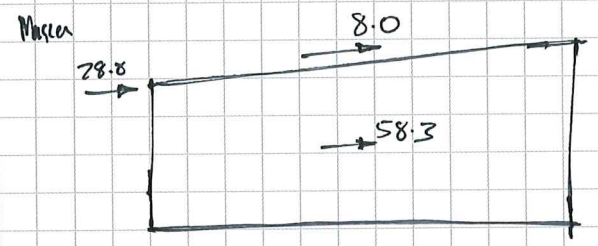
Panels resisting in-plane seismic force

In-plane forces are from the roof, return walls, and panel self weight.
to clause 11.3.5.2.2

$$X_s = \frac{N}{f_c A_g} + f_d \frac{f_y}{f'_c} = 0.0321$$

$$= \frac{2.2\,m}{L_w A_g f'_c} = 0.0126$$

where m =



$$M_{E\,\mu1.25} = 0.73 \times (28.8 \times 3.2 + 8 \times 3.5 + 58.3 \times 1.7) = 160\ kNm$$

$$k_{f_b} = 1$$

$$\frac{h_f t h_n}{t} = \frac{1 \times 3500}{100} = 35 < 12\sqrt{\frac{3500/7500}{0.0126}} = 73 \quad \text{OK.}$$

$$h_n/t = 35 < 75 \quad \text{- OK.}$$

$$\frac{l_u t h_n}{t} = 35 < 65 \quad \text{- OK.}$$

Calculations

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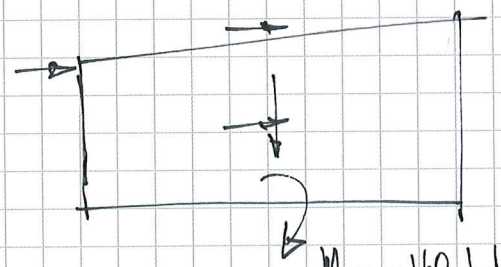
Wall in-plane action



$M_{OT} = 160$ kNm at μ1.25 sp 0.9

$M_{res} = 58 \times \frac{7.5}{2} = 218$ kNm from panel self-weight only

$M_{res} > M_{OT}$ - overturning OK.

Wall in-plane bending,

$M_{cr} = 3558$ kNm

$\phi M_n = 2096$ kNm

/ ref spreadsheet calc - OK by inspection.

Calculations

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Wall Panel 1

Material properties	ϕ,m	1				
	ϕ,v	1				
	f'c	40	MPa			
	f'r	3.79	MPa			
	fy	600	MPa	vert bars		
	fyt	600	MPa	horizontal bars		
	fyh	600	MPa	stirrups / ties		
Wall geometry	Lw	7500	mm			
	tw	100	mm			
	hw	3500	mm			
Wall actions	μ	1.25		wall ductility		
	M*	160	kNm			
	V*	69.2	kN			
	N*	58	kN			
	Ig	3.516E+12	mm4			
	Mcr	3558	kNm	Wall remains uncracked		
Flexural reinforcement	e	50	mm	end distance to first vertical bar		
	s	150	mm	vertical bar spacing		
	nl	1	#	vertical bar layers		
	nf	0	#	number of 'flange' bars		
	nw	51	#	ro,min	ro,prov	
	df	6.3	mm	0.0053	0.0000	NG
	dw	6.3	mm	0.0026	0.0030	OK
	c	**166**	**mm**			
	ϕMn	**2096**	**kNm**	OK		
ransverse reinforcement	s	150	mm	horizontal bar spacing		
	nt	1	mm	horizontal bar layers		
	dbt	6.3	mm	horizontal bar diameter		
	λ	1		cl 11.4.6.3		
	ϕVn	**1393**	**kN**	OK		
Confinement	s	100	mm	Tie spacing exceeds limit!		
	αr	1		cl 11.4.3.2		
	beta	5		cl 11.4.3.2		
	dbh	0	mm			
	Ate,req	**10.3**	**mm2**			
	Ate,prov	**0.0**	**mm2**	NG		
	λ	1		cl 11.4.6.3		
	α	0.175		cl 11.4.5.5		
	Ash,req	**-36.2**	**mm2**			
	Ash,prov	**0.0**	**mm2**	OK		
Curvature limits	Kd	6				
	Δ	4	mm			
	φmax	4.80	rad/mm			
	φactual	3.30	rad/mm	OK		

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Calculations

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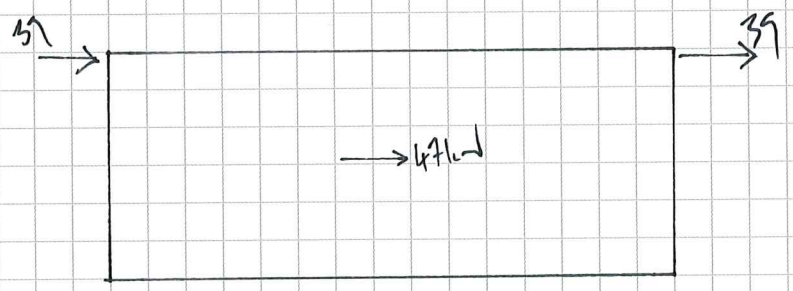
Wall panel 3 In-plane actions

.. Seismic Mass 1

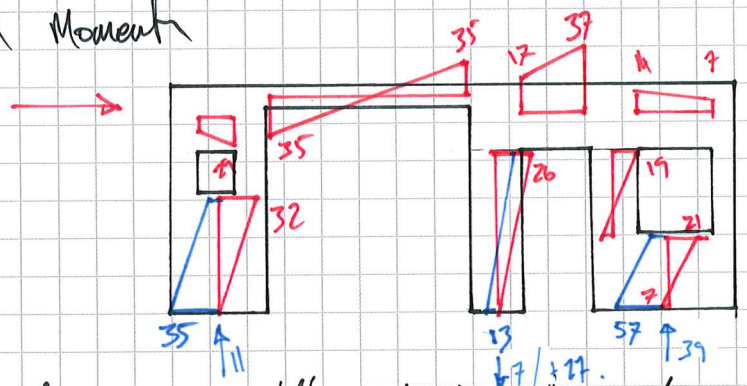


Seismic horizontal coefficient

$C_{\mu 1\,part} = 0.87$

$C_d(T)_{\mu 1.25} = 0.73$

Panel Moments



— Based on low stiffness at base of panel - ie upper bound actions on panel.

— Based on high stiffness at base of panel - ie upper bound actions on panel to slab connection.

Calculations

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Wall Panel 3 door spandrel

Material properties	ϕ,m	1				
	ϕ,v	1				
	f'c	40	MPa			
	f'r	3.79	MPa			
	fy	540	MPa	vert bars		
	fyt	324	MPa	horizontal bars		
	fyh	324	MPa	stirrups / ties		
Wall geometry	Lw	400	mm			
	tw	150	mm			
	hw	2700	mm			
Wall actions	μ	1.25		wall ductility		
	M*	35	kNm			
	V*	25.4	kN			
	N*	12	kN			
	Ig	800000000	mm4			
	Mcr	15	kNm	Wall cracks		
Flexural reinforcement	e	50	mm	end distance to first vertical bar		
	s	150	mm	vertical bar spacing		
	nl	2	#	vertical bar layers		
	nf	0	#	number of 'flange' bars		
	nw	4	#	ro,min	ro,prov	
	df	12	mm	0.0059	0.0000	NG
	dw	12	mm	0.0029	0.0215	OK
	c	**52**	**mm**			
	ϕMn	**61**	**kNm**	OK ✓		
ransverse reinforcement	s	150	mm	horizontal bar spacing		
	nt	2	mm	horizontal bar layers		
	dbt	6	mm	horizontal bar diameter		
	λ	1		cl 11.4.6.3		
	ϕVn	**91**	**kN**	OK ✓		
Confinement	s	150	mm	Tie spacing exceeds limit!		
	αr	1		cl 11.4.3.2		
	beta	5		cl 11.4.3.2		
	dbh	6	mm			
	Ate,req	49.1	mm2			
	Ate,prov	28.3	mm2	NG		
	λ	1		cl 11.4.6.3		
	α	0.175		cl 11.4.5.5		
	Ash,req	**-17.8**	**mm2**			
	Ash,prov	**28.3**	**mm2**	OK		
Curvature limits	Kd	6				
	Δ	4	mm			
	φmax	81.00	rad/mm			
	φactual	**7.41**	**rad/mm**	OK		

$\frac{28.3}{49.1} = 0.58$

→ Walls are not ductile therefore ok.

Engenium Ltd, 395 Madras Street, PO Box 3911, Christchurch, New Zealand
Tel 03 366 0434, Fax 03 379 3 -335544.3

Calculations

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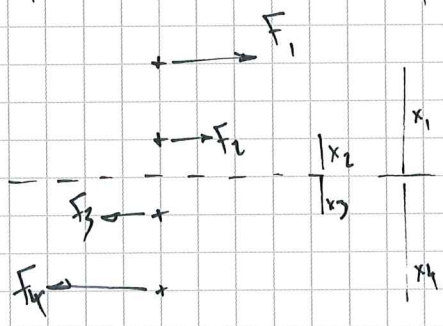
Actions on panel to foundation connection,

$M^* = 57$ kNm over 2m

$= 13$ kNm over 0.7m

$= 35$ kNm over 1.2m

Actions on individual anchors,



$M = \Sigma F x$

$F_i = \frac{x_i}{x_1} \cdot F_1$

Overturning actions on each fixing in panel type 3

Fixing	dx	xt	xi	F
	mm	mm	m	kN
1	0	0	-0.413	-38.2
2	275	275	-0.138	-12.7
3	275	550	0.138	12.7
4	275	825	0.413	38.2
		M	35.00	kNm

Fixing	dx	xt	xi	F
	mm	mm	m	kN
1	0	0	-0.150	-43.3
2	300	300	0.150	43.3
		M	13.00	kNm

Fixing	dx	xt	xi	F
	mm	mm	m	kN
1	0	0	-0.900	-40.7
2	300	300	-0.600	-27.1
3	300	600	-0.300	-13.6
4	300	900	0.000	0.0
5	300	1200	0.300	13.6
6	300	1500	0.600	27.1
7	300	1800	0.900	40.7
		M	57.00	kNm

— Based on fully rigid supports

Ref 5-8 for flexible support model.

$\frac{9.4}{0.7} = 0.823$

$\frac{12.4}{0.7} = 0.30$

Due to shear distortion / expansion.

Calculations

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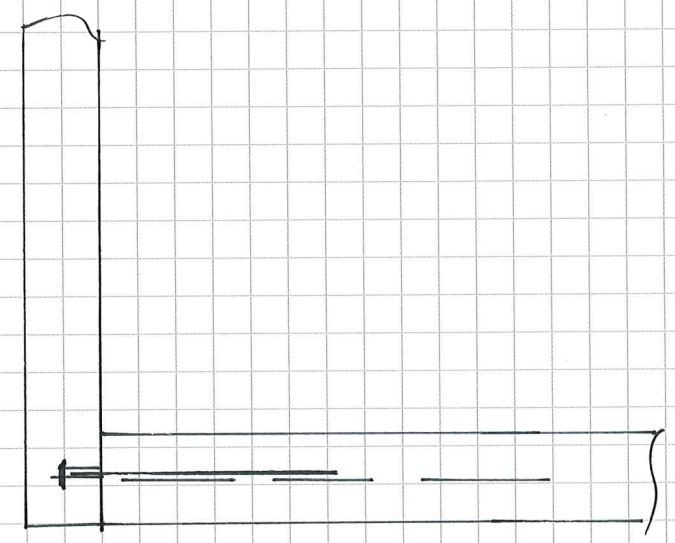
Wall panel to slab connection.



- panel to slab interface can carry up to 12.4 kN in shear friction,
- Slab weight = 2.88 kN/m²
- Slab flexural strength, - 665 mesh c in 120mm slab

$M_{slab} = 5.14$ kNm/m

$$M^* = \frac{W^* L}{2}$$

$$L = \frac{2 \times M}{W} = \frac{2 \times 5.14}{2.88} = 3.57\text{m}$$

$$V^* = \frac{3.57}{2} \times 2.88 = 5.14 \text{ kN/m}$$

Therefore the slab would fail before shear friction failure or anchor failure in the wall.

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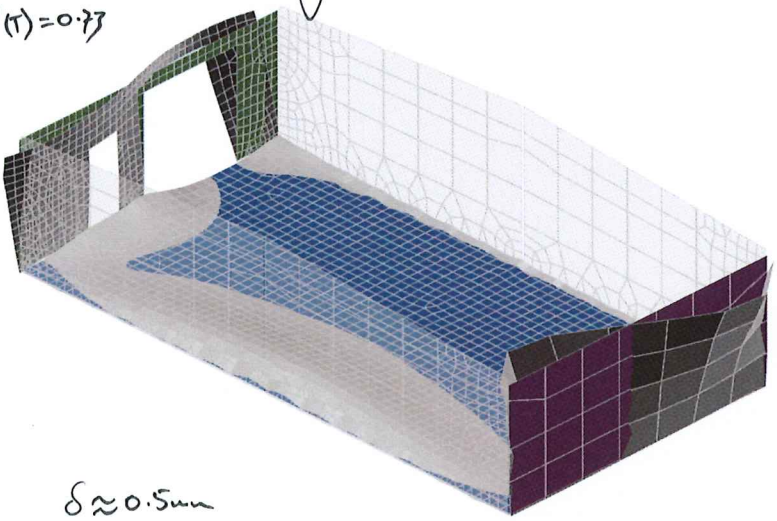
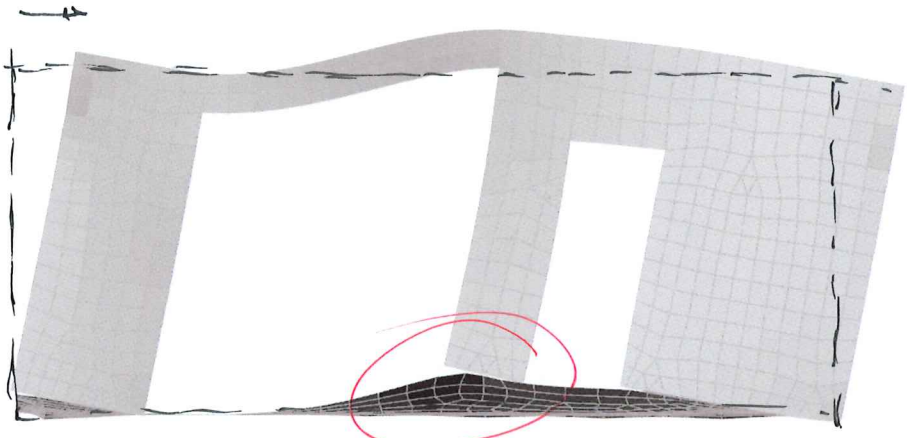
Full unit model to identify how front facade wall & slab interact.

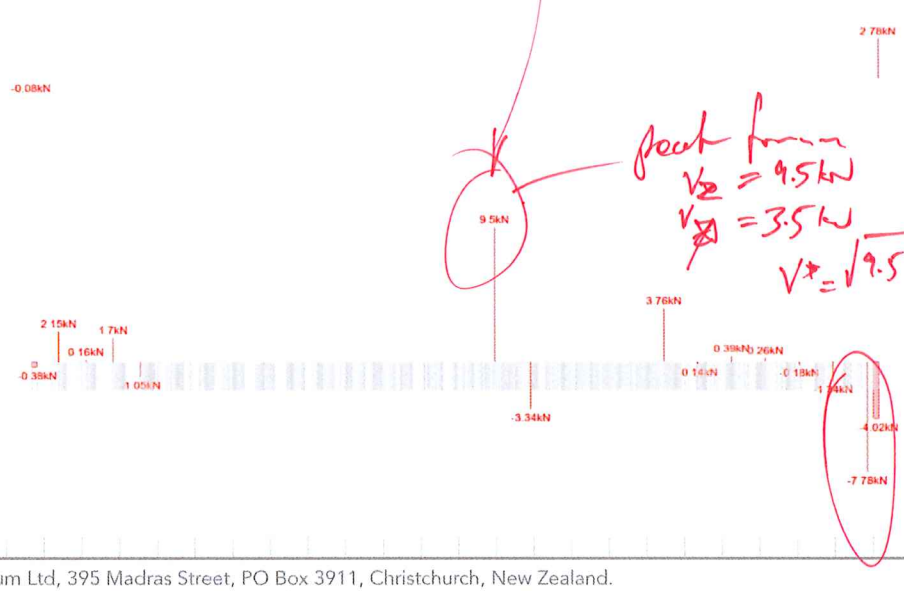
$C_d(T) = 0.73$



It can be seen that the front wall to slab connection forces from $\mu = 1.25$ seismic actions are lower than the shear friction capacity of the connection, therefore the connection does not 'let go' and bare shear to the slab, it still a viable load path.

$\delta \approx 0.5mm$





peak forces
$V_z = 9.5kN$
$V_x = 3.5kN$
$V^* = \sqrt{9.5^2 + 3.5^2} = 10.1kN$
- OK.

$\rightarrow \sqrt{12.1^2 + 4.1^2} = 12.8kN$

$\frac{12.4}{12.8} = 0.97$

Calculations

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Wall out-of-plane

Wall stability to 3101 cl 11.3.5.1.2

$N^* = 0.1 \times 2.4 \times 1.5 = 3.6$ kN/m $\quad << 0.06 f'_c$

$M^* = M^*_A + N^* \Delta_u = 6.24$ kNm

where $\Delta_u = \dfrac{5 w h^2}{0.75 \times 48 E_c I_{cr}} = 65.4$ mm

and $M^* = \dfrac{M^*_a}{1 - \dfrac{5 N^* h_w^2}{0.75 \times 48 E_c I_{cr}}} = 6.24$ kNm

and $I_{cr} = n A_{se} (d - kd)^2 + \dfrac{L_w (kd)^3}{3} = 4.52 \times 10^6$ mm^4

$\boxed{I_{cr}/I_g = 0.0542}$

which does not align with a rational assessment of the wall

and $n = \dfrac{E_s}{E_c} = \dfrac{200000}{50000} = 6.67$

$A_{se} = \dfrac{N^* + A_t f_y}{f_y} = \dfrac{3.6 + 208 \times 600}{600} = 214$ mm^2

where $M_a = 6.01$ kNm/m at the yeldline

Wall cracking moment is 6.3 kNm/m

I_c/I_g is likely to be closer to 0.25 to 0.5, in which case

$M^* = 6.02$ kNm

Therefore out of plane stability is not a significant component of minor axis bending.

Calculations

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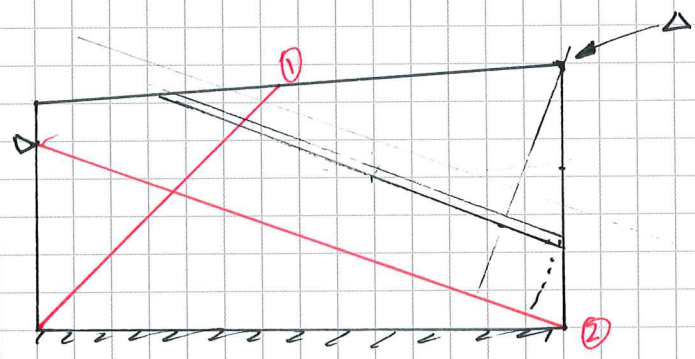
Panels Resisting Out-of-plane Seismic Force

Panel actions - End/IT Walls. "Type 1"



$\alpha = \arctan\left(\frac{2.7}{7}\right) = 21.1°$

$\tan = O/A$

$O = A\tan\alpha$

$A' = \frac{O \cdot A}{2} = \frac{A^2 \tan\alpha}{2}$

$A = \sqrt{\frac{2A'}{\tan\alpha}} = 6.5\,m.$

$O = 6.5 \tan 21 = 2.5\,m$

$2.5 \cos 21 = 2.33\,m$

$3.5 \cos 21 = 3.27\,m$

$3.27 - 2.33 = 0.94\,m$

$\Delta = \Delta_{max} \times \frac{0.94}{3.27} = 0.287$

$2.02 / 2.09 = 0.96$

@ $\mu = 1$

① $w_u = \frac{6}{L_z^2\left(3\frac{L_y}{L_x} - 1\right)} \cdot (m_x + m_y + k_3)$

$m_x = 6.01 + 9.57 = 15.6\,kN/m/m$

$m_y = 6.01\,kN/m/m$

$k_3 = 0$

$w_u = 2.02\,kPa$

$w^r_{\mu=1} = 0.1 \times 24 \times 0.87 = 2.09\,kPa$

② $\Sigma P\Delta = \Sigma M l \theta$

$P = Aw^*$ $\qquad \Delta = 0.287\Delta_{max}.$

$A = 1 \times 7 + \frac{1}{2} \times 2.67 \times 7 = 16.3\,m^2$

$l = \sqrt{2.67^2 + 7^2} = 7.5\,m$

$\theta = \Delta / 3.3$

$16.3 \times w^* \times 0.287 \times \Delta = M \times 7.5 \times \frac{\Delta}{3.3}$

$4.68 w^* = 2.27 M$

$M = \frac{4.68}{2.27} w^* = 2.06\,w^*$

$M^r = 2.09 \times 2.06 = 4.3\,kN/m/m$ $\qquad < M_{prob} = 6.01\,kN/m/m$ - Ok.

Calculations

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Panel type 2 - lower walls.

① $w_u = \dfrac{6}{L_x^2\left(3\frac{L_y}{L_x} - 2\right)} \times \left[2(m_x + m_y) + k_1\right]$

$M_x = 6.01$ kNm/m
$m_y = 6.01$ kNm/m

$k_1 = 0$

$W_u = 2.91$ kPa $> w^*_{\mu=1} = 2.09$ kPa - Ok.

② $\Sigma P\Delta = \Sigma M\theta$

or $M^* = wL^2/8$ in this case,

$M = 2.09 \times 2.4^2/8 = 1.50$ kNm/m $<$ 6.01 kNm/m - Ok.

$2.91/2.09 = 1.39$

Calculations

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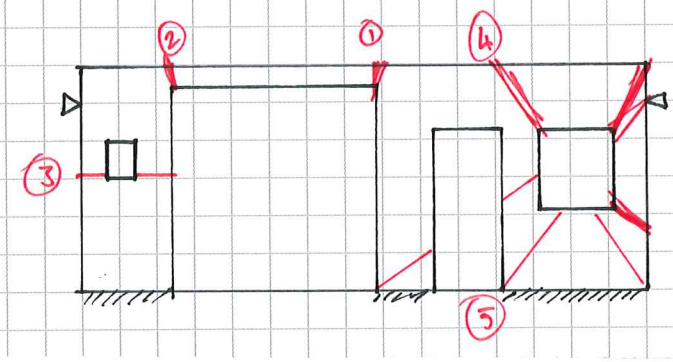
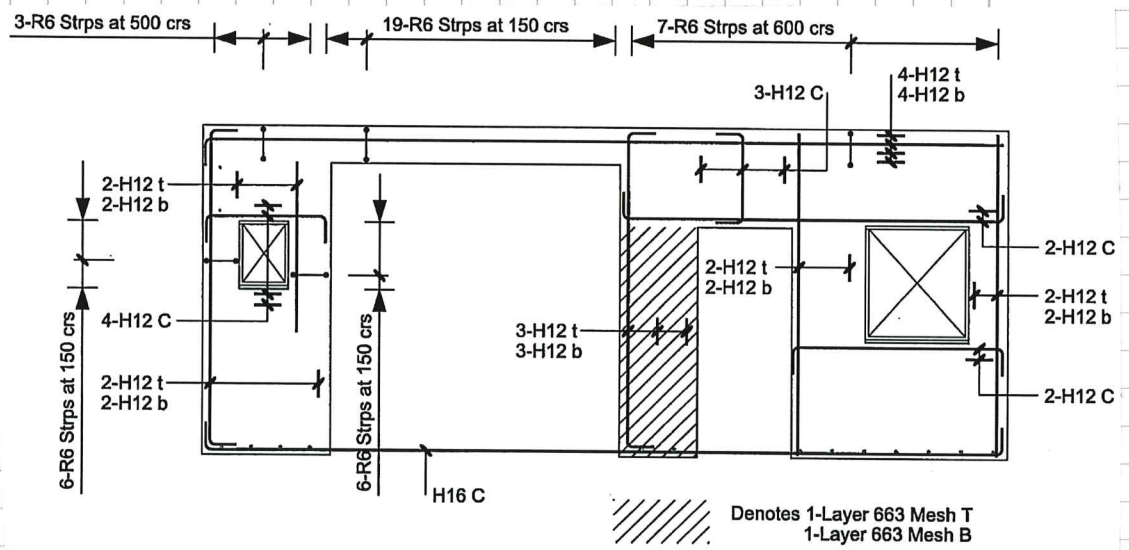
Panel type 3 - Front wall panels.

- 150mm thick panel.





PANELS 12 & 64 - REINFORCING ONLY

(1) 4-H12 t/b + R6-150

(2)

$A_s = 452 mm^2$

$f_{prob} = 540$ MPa

$a = \frac{452 \times 540}{0.85 \times 40 \times 400} = 17.9$ mm

$jd = 150 - 40 - 6 - 7 - 17.9/2 = 88$ mm

$M_{prob} = 452 \times 540 \times 88 = 21.5$ kNm

$M^*_{\mu 1 100\%} = 18.8$ kNm/m - ok.

or 13 kNm/m with rigid supports.

or 22.5 kNm/m + 18.3 kNm/m / 2 = 20.4 kNm/m w/A adjust.

Engenium, 395 Madras Street, PO Box 3911, Christchurch, New Zealand.
Tel 03 366 0434, Fax 03 379 3981, Email eng@engenium.co.nz, Internet www.engenium.co.nz

Calculations

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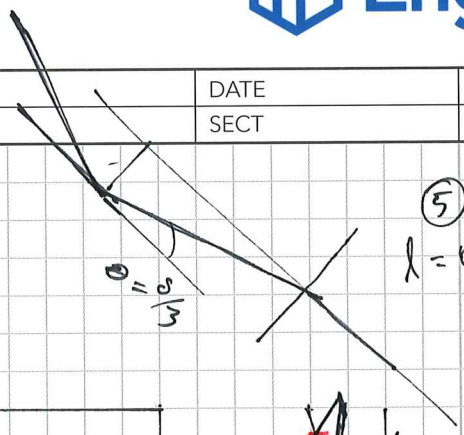
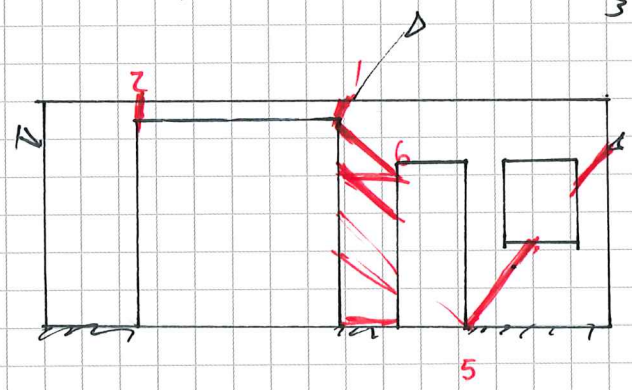
Panel type 3

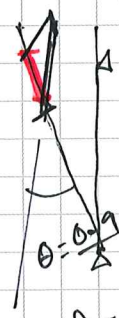
Yield lines ①, ②, & ⑤



⑤

$l = 0.6 + 1.4 = 2.0m$

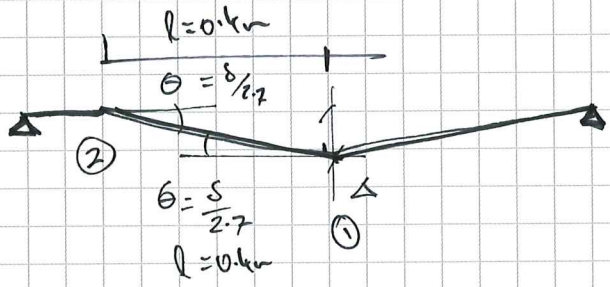




$\theta = \frac{0.9 \times \delta}{2.4}$

$l = 0.9m$

⑥



$\Sigma P\Delta = \Sigma m l \theta$

$P = Aw^*$ ~~$w^* = 0.15 \times 24 \times 0.87 =$~~ w = solve for.

$A_{1-2} = 0.4 \times 2.7 = 1.08\ m^2$

$A_{1-5} = 0.9 \times 3.6 + 0.7 \times 2.1 = 4.71\ m^2$

$A_6 = 0.7 \times 2.1 = 1.47\ m^2$

$w^* \Sigma A\Delta = \Sigma m l \theta$

$m = \frac{wL^2}{8}$

$\frac{0.56 \times 8^2}{8} = 4.48$ kNm/m

$w = \frac{8m}{L^2}$

$= \frac{8 \times 10}{8^2}$

$\frac{5.32}{8} =$

$w = 10/8 = 1.25$

Calculations

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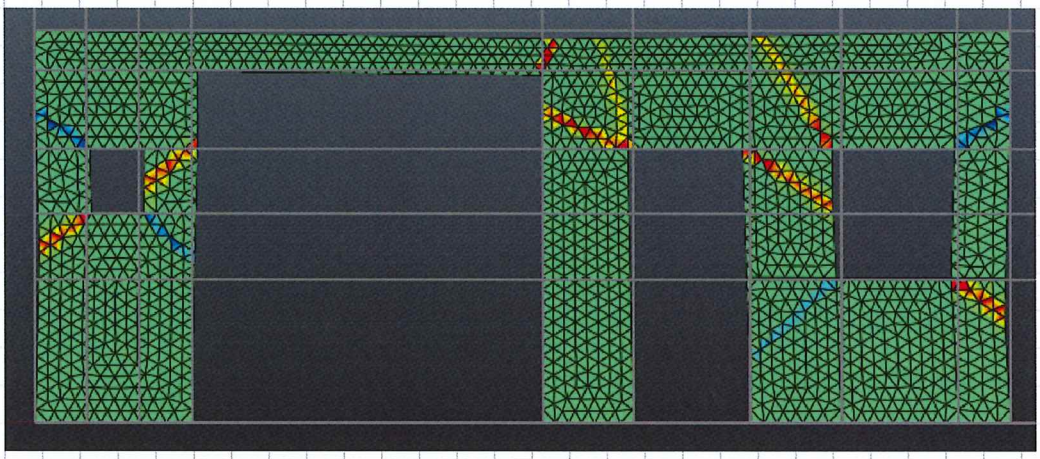
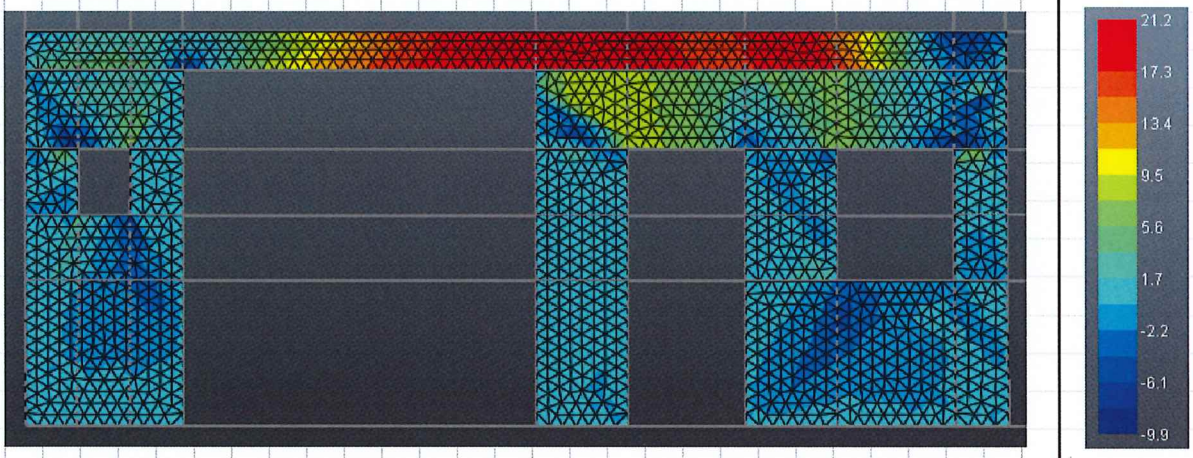
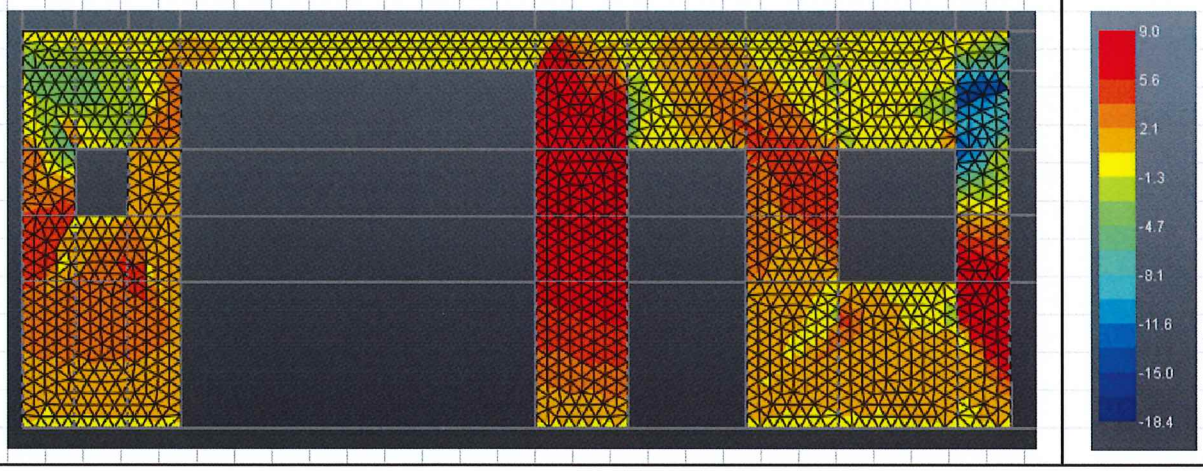
Panel yieldline theory calculation by Optum MP software
Applied load = 3.13 kPa
Capacity/Demand = 1.61



X-direction moments



Y-direction moments



Calculations

Engenium

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Panel 3

Verification

- Hogging yieldline
- Sagging yieldline.

Flexural strength of singly reinforced slabs not subject to significant axial load, designed to NZS3101 Clause 9.

Flexural strength of singly reinforced slabs not subject to significant axial load, designed to NZS3101 Clause 9.	Reinforcing		665	663	SE62	SE72	SE82	SE92	HD10					HD12					HD16				
	db	(mm)	5.3	6.3	6	7	8	9	10	10	10	10	10	12	12	10	12	12	16	16	10	16	16
	s	(mm)	150	150	200	200	200	200	400	300	250	200	150	400	300	250	200	150	400	300	250	200	150
	As	(mm2/m)	147	208	141	192	251	318	196	262	314	393	524	283	377	314	565	754	503	670	314	1005	1340
	fy	(MPa)	600	600	500	500	500	500	540	540	540	540	540	540	540	540	540	540	540	540	540	540	540

ϕ 1

Table 1: Centrally placed reinforcement

f'c (MPa)	f'r (MPa)	t (mm)	d (mm)	Mcr (kNm)	ϕMn (kNm)																				
20	2.68	100	50	4.5	4.18	5.78	3.39	4.54	5.82	7.21	4.97	6.48	7.64	9.28	11.8	6.95	8.96	7.64	12.5	15.5	11.4	14.2	7.64	18.5	20.8
20	2.68	125	62.5	7.0	5.29	7.34	4.27	5.74	7.39	9.2	6.3	8.25	9.76	11.9	15.3	8.86	11.5	9.76	16.3	20.6	14.8	18.8	9.76	25.3	29.8
20	2.68	150	75	10.1	6.39	8.89	5.15	6.94	8.96	11.2	7.62	10	11.9	14.6	18.9	10.8	14	11.9	20.2	25.7	18.2	23.3	11.9	32	38.9
20	2.68	175	87.5	13.7	7.49	10.5	6.04	8.15	10.5	13.2	8.95	11.8	14	17.2	22.4	12.7	16.6	14	24	30.8	21.6	27.8	14	38.8	47.9
20	2.68	200	100	17.9	8.6	12	6.92	9.35	12.1	15.2	10.3	13.5	16.1	19.9	25.9	14.6	19.1	16.1	27.8	35.8	25	32.3	16.1	45.6	57
25	3.00	100	50	5.0	4.23	5.87	3.42	4.59	5.91	7.36	5.04	6.6	7.81	9.54	12.3	7.09	9.2	7.81	13.1	16.5	11.8	15	7.81	20.2	23.9
25	3.00	125	62.5	7.8	5.33	7.43	4.3	5.8	7.48	9.35	6.36	8.37	9.93	12.2	15.8	8.99	11.7	9.93	16.9	21.5	15.2	19.5	9.93	27	32.9
25	3.00	150	75	11.3	6.44	8.99	5.18	7	9.05	11.3	7.69	10.1	12	14.8	19.3	10.9	14.3	12	20.7	26.6	18.6	24.1	12	33.8	42
25	3.00	175	87.5	15.3	7.54	10.5	6.07	8.2	10.6	13.3	9.01	11.9	14.2	17.5	22.9	12.8	16.8	14.2	24.5	31.7	22	28.6	14.2	40.6	51
25	3.00	200	100	20.0	8.64	12.1	6.95	9.4	12.2	15.3	10.3	13.7	16.3	20.1	26.4	14.7	19.4	16.3	28.3	36.8	25.4	33.1	16.3	47.4	60.1
30	3.29	110	55	6.6	4.7	6.55	3.79	5.11	6.6	8.25	5.61	7.38	8.77	10.8	14	7.94	10.4	8.77	15	19.1	13.5	17.3	8.77	24.1	29.5
30	3.29	125	62.5	8.6	5.36	7.49	4.32	5.83	7.54	9.44	6.41	8.44	10	12.4	16.1	9.09	11.9	10	17.3	22.2	15.5	20.1	10	28.2	35
30	3.29	150	75	12.3	6.47	9.05	5.2	7.03	9.12	11.4	7.73	10.2	12.2	15	19.6	11	14.5	12.2	21.1	27.3	18.9	24.6	12.2	34.9	44
30	3.29	175	87.5	16.8	7.57	10.6	6.09	8.24	10.7	13.4	9.06	12	14.3	17.7	23.2	12.9	17	14.3	24.9	32.4	22.3	29.1	14.3	41.7	53.1
30	3.29	200	100	21.9	8.67	12.2	6.97	9.44	12.3	15.4	10.4	13.7	16.4	20.3	26.7	14.8	19.5	16.4	28.7	37.5	25.7	33.6	16.4	48.5	62.1
35	3.55	110	55	7.2	4.72	6.6	3.8	5.14	6.65	8.32	5.64	7.44	8.85	10.9	14.2	8.01	10.5	8.85	15.2	19.6	13.7	17.7	8.85	24.9	31
35	3.55	120	60	8.5	5.16	7.22	4.16	5.62	7.27	9.12	6.17	8.15	9.7	12	15.6	8.77	11.5	9.7	16.8	21.6	15	19.5	9.7	27.6	34.6
35	**3.55**	**150**	**75**	**13.3**	**6.49**	**9.09**	**5.22**	**7.06**	**9.16**	**11.5**	**7.76**	**10.3**	**12.2**	**15.1**	**19.9**	**11.1**	**14.6**	**12.2**	**21.3**	**27.8**	**19.1**	**24.9**	**12.2**	**35.8**	**45.5**
35	3.55	175	87.5	18.1	7.59	10.6	6.1	8.26	10.7	13.5	9.09	12	14.4	17.8	23.4	13	17.1	14.4	25.2	32.8	22.5	29.5	14.4	42.5	54.5
35	3.55	200	100	23.7	8.69	12.2	6.98	9.47	12.3	15.5	10.4	13.8	16.5	20.4	26.9	14.9	19.7	16.5	29	37.9	25.9	34	16.5	49.3	63.6
40	3.79	100	50	6.3	4.3	6.01	3.46	4.67	6.05	7.58	5.14	6.77	8.06	9.94	13	7.29	9.57	8.06	13.9	17.9	12.5	16.2	8.06	22.8	28.5
40	3.79	120	60	9.1	5.18	7.25	4.17	5.64	7.31	9.17	6.2	8.19	9.76	12.1	15.8	8.82	11.6	9.76	17	22	15.2	19.8	9.76	28.2	35.7
40	3.79	150	75	14.2	6.5	9.12	5.23	7.08	9.19	11.6	7.79	10.3	12.3	15.2	20	11.1	14.7	12.3	21.5	28.1	19.3	25.2	12.3	36.4	46.6
40	3.79	175	87.5	19.4	7.61	10.7	6.11	8.28	10.8	13.5	9.11	12.1	14.4	17.9	23.6	13	17.2	14.4	25.3	33.2	22.7	29.7	14.4	43.2	55.6
40	3.79	200	100	25.3	8.71	12.2	7	9.49	12.3	15.5	10.4	13.8	16.5	20.5	27.1	14.9	19.7	16.5	29.2	38.3	26.1	34.3	16.5	50	64.7

Table 2: Reinforcement at 40mm cover

f'c (MPa)	f'r (MPa)	t (mm)	d (mm)	Mcr (kNm)	ϕMn (kNm)																				
20	2.68	100	50	4.5	4.18	5.78	3.39	4.54	5.82	7.21	4.97	6.48	7.64	9.28	11.8	6.95	8.96	7.64	12.5	15.5	11.4	14.2	7.64	18.5	20.8
20	2.68	125	75	7.0	6.39	8.89	5.15	6.94	8.96	11.2	7.62	10	11.9	14.6	18.9	10.8	14	11.9	20.2	25.7	18.2	23.3	11.9	32	38.9
20	2.68	150	100	10.1	8.6	12	6.92	9.35	12.1	15.2	10.3	13.5	16.1	19.9	25.9	14.6	19.1	16.1	27.8	35.8	25	32.3	16.1	45.6	57
20	2.68	175	125	13.7	10.8	15.1	8.69	11.8	15.2	19.1	12.9	17.1	20.4	25.2	33	18.4	24.2	20.4	35.4	46	31.8	41.4	20.4	59.2	75.1
20	2.68	200	150	17.9	13	18.2	10.5	14.2	18.4	23.1	15.6	20.6	24.6	30.5	40.1	22.2	29.3	24.6	43.1	56.2	38.5	50.4	24.6	72.8	93.2
25	3.00	100	50	5.0	4.23	5.87	3.42	4.59	5.91	7.36	5.04	6.6	7.81	9.54	12.3	7.09	9.2	7.81	13.1	16.5	11.8	15	7.81	20.2	23.9
25	3.00	125	75	7.8	6.44	8.99	5.18	7	9.05	11.3	7.69	10.1	12	14.8	19.3	10.9	14.3	12	20.7	26.6	18.6	24.1	12	33.8	42
25	3.00	150	100	11.3	8.64	12.1	6.95	9.4	12.2	15.3	10.3	13.7	16.3	20.1	26.4	14.7	19.4	16.3	28.3	36.8	25.4	33.1	16.3	47.4	60.1
25	3.00	175	125	15.3	10.8	15.2	8.72	11.8	15.3	19.3	13	17.2	20.5	25.4	33.5	18.5	24.5	20.5	36	47	32.2	42.2	20.5	60.9	78.2
25	3.00	400	350	80.0	30.7	43.3	24.6	33.5	43.6	55.1	36.8	49	58.7	73.2	97.1	52.9	70.3	58.7	104.7	138.6	93.3	123.6	58.7	183.1	241
30	3.29	110	60	6.6	5.14	7.18	4.14	5.59	7.23	9.05	6.14	8.09	9.61	11.8	15.4	8.7	11.4	9.61	16.5	21.2	14.8	19.1	9.61	26.8	33.2
30	3.29	120	70	7.9	6.02	8.42	4.85	6.55	8.49	10.6	7.2	9.5	11.3	14	18.2	10.2	13.4	11.3	19.5	25.3	17.6	22.8	11.3	32.2	40.4
30	3.29	150	100	12.3	8.67	12.2	6.97	9.44	12.3	15.4	10.4	13.7	16.4	20.3	26.7	14.8	19.5	16.4	28.7	37.5	25.7	33.6	16.4	48.5	62.1
30	3.29	175	125	16.8	10.9	15.3	8.74	11.8	15.4	19.4	13	17.3	20.6	25.6	33.8	18.6	24.6	20.6	36.3	47.6	32.5	42.7	20.6	62.1	80.2
30	3.29	350	300	67.1	26.3	37.1	21.1	28.7	37.4	47.2	31.6	42	50.3	62.7	83.3	45.3	60.3	50.3	89.8	118.9	80	106	50.3	157.1	206.9
35	3.55	110	60	7.2	5.16	7.22	4.16	5.62	7.27	9.12	6.17	8.15	9.7	12	15.6	8.77	11.5	9.7	16.8	21.6	15	19.5	9.7	27.6	34.6
35	3.55	120	70	8.5	6.05	8.47	4.86	6.58	8.53	10.7	7.23	9.56	11.4	14.1	18.4	10.3	13.6	11.4	19.8	25.7	17.8	23.1	11.4	33	41.9
35	**3.55**	**150**	**100**	**13.3**	**8.69**	**12.2**	**6.98**	**9.47**	**12.3**	**15.5**	**10.4**	**13.8**	**16.5**	**20.4**	**26.9**	**14.9**	**19.7**	**16.5**	**29**	**37.9**	**25.9**	**34**	**16.5**	**49.3**	**63.6**
35	3.55	175	125	18.1	10.9	15.3	8.75	11.9	15.4	19.5	13.1	17.3	20.7	25.8	34	18.7	24.8	20.7	36.6	48.1	32.7	43	20.7	62.9	81.7
35	3.55	200	150	23.7	13.1	18.4	10.5	14.3	18.6	23.4	15.7	20.9	25	31.1	41.1	22.5	29.8	25	44.2	58.3	39.5	52.1	25	76.5	99.8
40	3.79	100	50	6.3	4.3	6.01	3.46	4.67	6.05	7.58	5.14	6.77	8.06	9.94	13	7.29	9.57	8.06	13.9	17.9	12.5	16.2	8.06	22.8	28.5
40	3.79	120	70	9.1	6.06	8.5	4.87	6.6	8.56	10.8	7.26	9.6	11.5	14.2	18.6	10.3	13.6	11.5	20	26.1	17.9	23.4	11.5	33.7	43
40	3.79	150	100	14.2	8.71	12.2	7	9.49	12.3	15.5	10.4	13.8	16.5	20.5	27.1	14.9	19.7	16.5	29.2	38.3	26.1	34.3	16.5	50	64.7
40	3.79	175	125	19.4	10.9	15.4	8.76	11.9	15.5	19.5	13.1	17.4	20.8	25.8	34.2	18.7	24.8	20.8	36.8	48.5	32.8	43.3	20.8	63.5	82.8
40	3.79	200	150	25.3	13.1	18.5	10.5	14.3	18.6	23.5	15.7	20.9	25	31.1	41.2	22.6	29.9	25	44.4	58.6	39.6	52.4	25	77.1	100.9

Flexural strength of singly reinforced slabs not subject to significant axial load, designed to NZS3101 Clause 9.

Flexural strength of singly reinforced slabs not subject to significant axial load, designed to NZS3101 Clause 9.	Reinforcing		665	663	SE62	SE72	SE82	SE92	HD10					HD12					HD16				
	db	(mm)	5.3	6.3	6	7	8	9	10	10	10	10	10	12	12	10	12	12	16	16	10	16	16
	s	(mm)	150	150	200	200	200	200	400	300	250	200	150	400	300	250	200	150	400	300	250	200	150
	As	(mm2/m)	147	208	141	192	251	318	196	262	314	393	524	283	377	314	565	754	503	670	314	1005	1340
	fy	(MPa)	600	600	500	500	500	500	324	324	324	324	324	324	324	324	324	324	324	324	324	324	324

ϕ 1

Table 1: Centrally placed reinforcement

f'c (MPa)	f'r (MPa)	t (mm)	d (mm)	Mcr (kNm)	ϕMn (kNm)																			
20	2.68	100	50	4.5	4.18	5.78	3.39	4.54	5.82	7.21	3.06	4.03	4.78	5.89	7.64	4.33	5.67	4.78	8.17	10.5	7.36	9.47	4.78	13.2	16.2
20	2.68	125	62.5	7.0	5.29	7.34	4.27	5.74	7.39	9.2	3.86	5.09	6.06	7.48	9.76	5.48	7.2	6.06	10.5	13.5	9.4	12.2	6.06	17.2	21.6
20	2.68	150	75	10.1	6.39	8.89	5.15	6.94	8.96	11.2	4.65	6.15	7.33	9.07	11.9	6.62	8.72	7.33	12.8	16.6	11.4	14.9	7.33	21.3	27
20	2.68	175	87.5	13.7	7.49	10.5	6.04	8.15	10.5	13.2	5.45	7.21	8.6	10.7	14	7.77	10.2	8.6	15	19.6	13.5	17.6	8.6	25.4	32.5
20	2.68	200	100	17.9	8.6	12	6.92	9.35	12.1	15.2	6.24	8.27	9.87	12.2	16.1	8.91	11.8	9.87	17.3	22.7	15.5	20.3	9.87	29.5	37.9
25	3.00	100	50	5.0	4.23	5.87	3.42	4.59	5.91	7.36	3.09	4.07	4.85	5.98	7.81	4.38	5.76	4.85	8.37	10.8	7.52	9.75	4.85	13.8	17.3
25	3.00	125	62.5	7.8	5.33	7.43	4.3	5.8	7.48	9.35	3.88	5.13	6.12	7.57	9.93	5.53	7.28	6.12	10.7	13.9	9.55	12.5	6.12	17.9	22.7
25	3.00	150	75	11.3	6.44	8.99	5.18	7	9.05	11.3	4.68	6.19	7.39	9.16	12	6.67	8.81	7.39	13	16.9	11.6	15.2	7.39	21.9	28.1
25	3.00	175	87.5	15.3	7.54	10.5	6.07	8.2	10.6	13.3	5.47	7.25	8.66	10.8	14.2	7.82	10.3	8.66	15.2	20	13.6	17.9	8.66	26	33.6
25	3.00	200	100	20.0	8.64	12.1	6.95	9.4	12.2	15.3	6.27	8.31	9.93	12.3	16.3	8.96	11.9	9.93	17.5	23	15.7	20.6	9.93	30.1	39
30	3.29	110	55	6.6	4.7	6.55	3.79	5.11	6.6	8.25	3.42	4.52	5.4	6.68	8.77	4.87	6.43	5.4	9.42	12.3	8.44	11	5.4	15.8	20.2
30	3.29	125	62.5	8.6	5.36	7.49	4.32	5.83	7.54	9.44	3.9	5.16	6.16	7.63	10	5.56	7.34	6.16	10.8	14.1	9.66	12.6	6.16	18.3	23.4
30	3.29	150	75	12.3	6.47	9.05	5.2	7.03	9.12	11.4	4.69	6.22	7.43	9.23	12.2	6.71	8.87	7.43	13.1	17.2	11.7	15.4	7.43	22.3	28.9
30	3.29	175	87.5	16.8	7.57	10.6	6.09	8.24	10.7	13.4	5.49	7.28	8.7	10.8	14.3	7.85	10.4	8.7	15.4	20.2	13.7	18.1	8.7	26.4	34.3
30	3.29	200	100	21.9	8.67	12.2	6.97	9.44	12.3	15.4	6.28	8.34	9.98	12.4	16.4	9	11.9	9.98	17.7	23.3	15.8	20.8	9.98	30.5	39.7
35	3.55	110	55	7.2	4.72	6.6	3.8	5.14	6.65	8.32	3.43	4.54	5.42	6.73	8.85	4.9	6.47	5.42	9.51	12.4	8.51	11.2	5.42	16.1	20.7
35	3.55	120	60	8.5	5.16	7.22	4.16	5.62	7.27	9.12	3.75	4.97	5.93	7.36	9.7	5.36	7.08	5.93	10.4	13.7	9.33	12.2	5.93	17.8	22.9
35	**3.55**	**150**	**75**	**13.3**	**6.49**	**9.09**	**5.22**	**7.06**	**9.16**	**11.5**	**4.7**	**6.24**	**7.46**	**9.27**	**12.2**	**6.73**	**8.91**	**7.46**	**13.2**	**17.3**	**11.8**	**15.5**	**7.46**	**22.6**	**29.4**
35	3.55	175	87.5	18.1	7.59	10.6	6.1	8.26	10.7	13.5	5.5	7.3	8.73	10.9	14.4	7.87	10.4	8.73	15.5	20.4	13.8	18.2	8.73	26.7	34.8
35	3.55	200	100	23.7	8.69	12.2	6.98	9.47	12.3	15.5	6.29	8.36	10	12.5	16.5	9.02	12	10	17.8	23.4	15.8	20.9	10	30.8	40.3
40	3.79	100	50	6.3	4.3	6.01	3.46	4.67	6.05	7.58	3.12	4.14	4.94	6.12	8.06	4.46	5.89	4.94	8.67	11.3	7.75	10.2	4.94	14.7	18.9
40	3.79	120	60	9.1	5.18	7.25	4.17	5.64	7.31	9.17	3.76	4.98	5.95	7.4	9.76	5.37	7.11	5.95	10.5	13.8	9.38	12.3	5.95	18	23.3
40	3.79	150	75	14.2	6.5	9.12	5.23	7.08	9.19	11.6	4.71	6.26	7.48	9.3	12.3	6.75	8.94	7.48	13.2	17.4	11.8	15.6	7.48	22.9	29.8
40	3.79	175	87.5	19.4	7.61	10.7	6.11	8.28	10.8	13.5	5.51	7.32	8.75	10.9	14.4	7.89	10.5	8.75	15.5	20.5	13.9	18.3	8.75	26.9	35.2
40	3.79	200	100	25.3	8.71	12.2	7	9.49	12.3	15.5	6.3	8.38	10	12.5	16.5	9.04	12	10	17.8	23.6	15.9	21	10	31	40.7

Table 2: Reinforcement at 40mm cover

f'c (MPa)	f'r (MPa)	t (mm)	d (mm)	Mcr (kNm)	ϕMn (kNm)																			
20	2.68	100	50	4.5	4.18	5.78	3.39	4.54	5.82	7.21	3.06	4.03	4.78	5.89	7.64	4.33	5.67	4.78	8.17	10.5	7.36	9.47	4.78	13.2	16.2
20	2.68	125	75	7.0	6.39	8.89	5.15	6.94	8.96	11.2	4.65	6.15	7.33	9.07	11.9	6.62	8.72	7.33	12.8	16.6	11.4	14.9	7.33	21.3	27
20	2.68	150	100	10.1	8.6	12	6.92	9.35	12.1	15.2	6.24	8.27	9.87	12.2	16.1	8.91	11.8	9.87	17.3	22.7	15.5	20.3	9.87	29.5	37.9
20	2.68	175	125	13.7	10.8	15.1	8.69	11.8	15.2	19.1	7.83	10.4	12.4	15.4	20.4	11.2	14.8	12.4	21.9	28.8	19.6	25.8	12.4	37.6	48.7
20	2.68	200	150	17.9	13	18.2	10.5	14.2	18.4	23.1	9.42	12.5	15	18.6	24.6	13.5	17.9	15	26.5	34.9	23.6	31.2	15	45.7	59.6
25	3.00	100	50	5.0	4.23	5.87	3.42	4.59	5.91	7.36	3.09	4.07	4.85	5.98	7.81	4.38	5.76	4.85	8.37	10.8	7.52	9.75	4.85	13.8	17.3
25	3.00	125	75	7.8	6.44	8.99	5.18	7	9.05	11.3	4.68	6.19	7.39	9.16	12	6.67	8.81	7.39	13	16.9	11.6	15.2	7.39	21.9	28.1
25	3.00	150	100	11.3	8.64	12.1	6.95	9.4	12.2	15.3	6.27	8.31	9.93	12.3	16.3	8.96	11.9	9.93	17.5	23	15.7	20.6	9.93	30.1	39
25	3.00	175	125	15.3	10.8	15.2	8.72	11.8	15.3	19.3	7.86	10.4	12.5	15.5	20.5	11.3	14.9	12.5	22.1	29.1	19.7	26	12.5	38.2	49.8
25	3.00	400	350	80.0	30.7	43.3	24.6	33.5	43.6	55.1	22.2	29.5	35.4	44.2	58.7	31.9	42.4	35.4	63.3	84.1	56.4	74.9	35.4	111.5	147.6
30	3.29	110	60	6.6	5.14	7.18	4.14	5.59	7.23	9.05	3.74	4.95	5.9	7.32	9.61	5.33	7.04	5.9	10.3	13.5	9.25	12.1	5.9	17.5	22.4
30	3.29	120	70	7.9	6.02	8.42	4.85	6.55	8.49	10.6	4.37	5.8	6.92	8.59	11.3	6.25	8.26	6.92	12.2	15.9	10.9	14.3	6.92	20.7	26.7
30	3.29	150	100	12.3	8.67	12.2	6.97	9.44	12.3	15.4	6.28	8.34	9.98	12.4	16.4	9	11.9	9.98	17.7	23.3	15.8	20.8	9.98	30.5	39.7
30	3.29	175	125	16.8	10.9	15.3	8.74	11.8	15.4	19.4	7.87	10.5	12.5	15.6	20.6	11.3	15	12.5	22.2	29.4	19.8	26.2	12.5	38.6	50.6
30	3.29	350	300	67.1	26.3	37.1	21.1	28.7	37.4	47.2	19	25.3	30.3	37.9	50.3	27.3	36.4	30.3	54.3	72.1	48.3	64.2	30.3	95.6	126.6
35	3.55	110	60	7.2	5.16	7.22	4.16	5.62	7.27	9.12	3.75	4.97	5.93	7.36	9.7	5.36	7.08	5.93	10.4	13.7	9.33	12.2	5.93	17.8	22.9
35	3.55	120	70	8.5	6.05	8.47	4.86	6.58	8.53	10.7	4.39	5.82	6.95	8.63	11.4	6.27	8.3	6.95	12.3	16.1	11	14.4	6.95	21	27.2
35	**3.55**	**150**	**100**	**13.3**	**8.69**	**12.2**	**6.98**	**9.47**	**12.3**	**15.5**	**6.29**	**8.36**	**10**	**12.5**	**16.5**	**9.02**	**12**	**10**	**17.8**	**23.4**	**15.8**	**20.9**	**10**	**30.8**	**40.3**
35	3.55	175	125	18.1	10.9	15.3	8.75	11.9	15.4	19.5	7.88	10.5	12.5	15.6	20.7	11.3	15	12.5	22.3	29.5	19.9	26.4	12.5	38.9	51.1
35	3.55	200	150	23.7	13.1	18.4	10.5	14.3	18.6	23.4	9.47	12.6	15.1	18.8	25	13.6	18.1	15.1	26.9	35.6	24	31.8	15.1	47.1	62
40	3.79	100	50	6.3	4.3	6.01	3.46	4.67	6.05	7.58	3.12	4.14	4.94	6.12	8.06	4.46	5.89	4.94	8.67	11.3	7.75	10.2	4.94	14.7	18.9
40	3.79	120	70	9.1	6.06	8.5	4.87	6.6	8.56	10.8	4.39	5.83	6.97	8.67	11.5	6.29	8.33	6.97	12.3	16.2	11	14.5	6.97	21.2	27.6
40	3.79	150	100	14.2	8.71	12.2	7	9.49	12.3	15.5	6.3	8.38	10	12.5	16.5	9.04	12	10	17.8	23.6	15.9	21	10	31	40.7
40	3.79	175	125	19.4	10.9	15.4	8.76	11.9	15.5	19.5	7.89	10.5	12.6	15.7	20.8	11.3	15	12.6	22.4	29.7	20	26.4	12.6	39.2	51.5
40	3.79	200	150	25.3	13.1	18.5	10.5	14.3	18.6	23.5	9.48	12.6	15.1	18.8	25	13.6	18.1	15.1	27	35.8	24	31.9	15.1	47.3	62.4

Calculations

Engenium

PROJECT	DATE	FILE
ENGINEER	SECT	PAGE

Foundation

Building wall panels are supported on short posthole footings noted as bearing on 'solid bearing'.

Walls are tied into the floor slabs with starter bars which will form the lateral load resisting element for base shear. (by friction).

Ref previous calcs for wall to foundation/slab connection.

Total building base shear

Roof = 14×7.5×0.142 = 14.9 kN
Walls = 46.8 + 56.1 + 58.3×2 = 220 kN
Slab = 14×7.5×0.12×24 = 302

235 kN (Building)

537 kN for a single unit incl slab

$V^*_{\mu 1.25} = 0.73 \times 235 = 172$ kN. $V^*_{slab} = 0.35 \times 302 = 106$ kN.

$R = W \tan\delta'$

Where $\phi' = 30°$ Assumed

$\delta' = \frac{2}{3}\phi = 20°$

$\tan\delta' = 0.364$

$R = 537 \times 0.364 = 195$ kN $> V_{\mu 1.25}$ building only.

$< 172 + 106 = 278$ kN including slab acceleration

$195 / 278 = 0.70$

But sliding of the building 'mass' is not a life safety issue and therefore does not contribute to the %NBS score.

Taking $\phi = 1$ for assessment purposes.

Foundation Bearing

Footings are $\phi 600 \times 1000$ unreinforced concrete on 'solid bearing'

Assuming $\phi = 30$ & $\gamma' = 18$,

$$q = \cancel{c'N_c}^{\,0} + q'N_q + \tfrac{1}{2}B\gamma N_\gamma$$

$N_q = 18.4$

$N_\gamma = 20.1$

$q' = 18$ kPa

$B = 0.6$

$\gamma = 18$

$$q = 0 + 18 \times 18.4 + 0.5 \times 0.6 \times 18 \times 20.1$$

$= 440$ kPa bearing strength.

$$Q = Aq = 0.283 \times 440$$

$= 124$ kN

Which is greater than $\underline{P^* = 105 \text{ kN}}$ from the precast walls therefore OK.

→ $Q^* = \frac{105}{0.283} = 371$ kPa at base of foundation, assuming 100% of bearing force/pressure goes to base of foundation.

→ Bearing ind slab, say 1m wide effective strip along wall,

$Q^* = 8.88$ kN/m $+ 2.88$ kN/m $= 11.76$ kN/m edges

$= 14.64$ kN/m centre.

Which is equal to a bearing pressure of ~ 15kPa max.

Calculations

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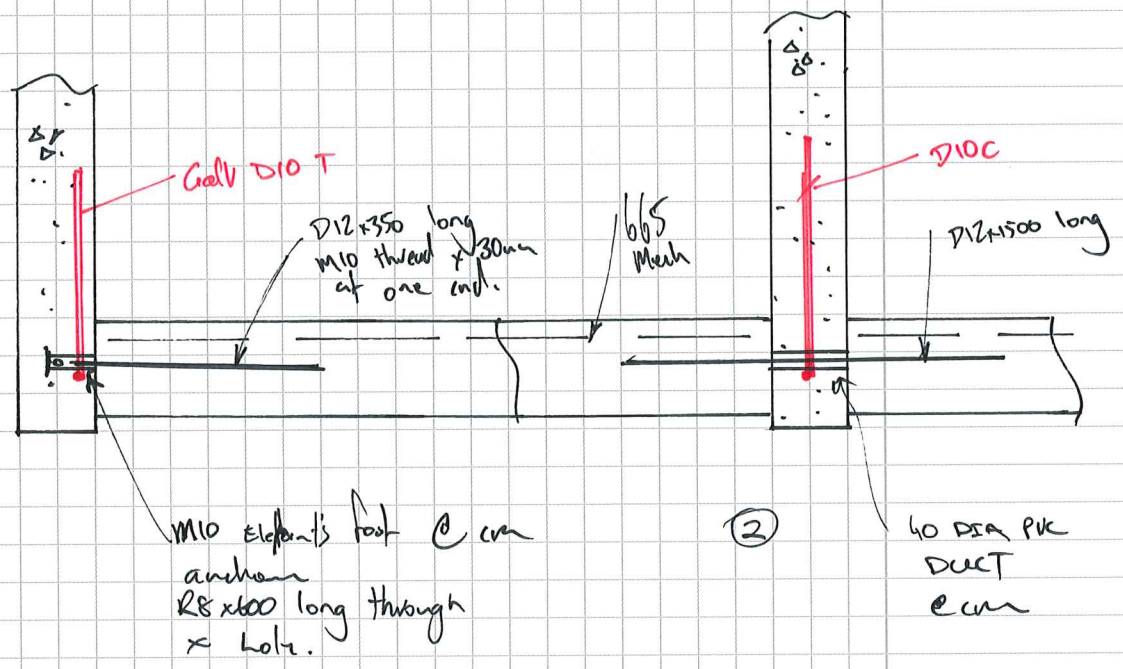
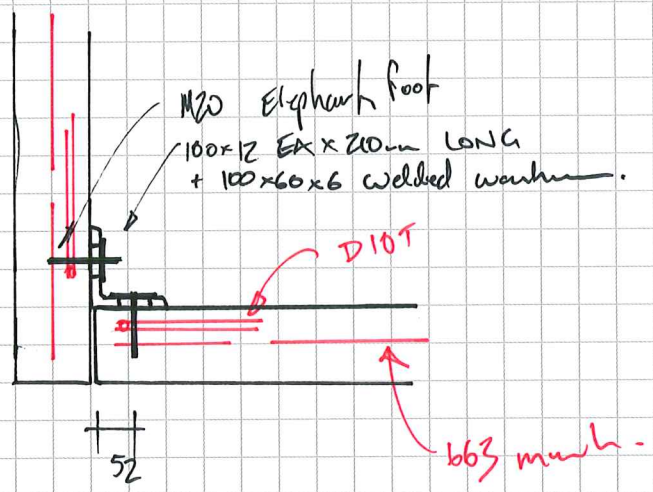
Wall Construction

Base connection



Top Connection



Calculations

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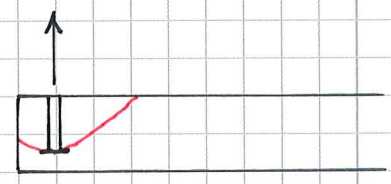
Top connection

M20 Elephant's foot anchor - Assume 70mm long

Capacity - in tension.



$N_b = 10\sqrt{f'_c} \times 70^{1.5}$

$= 37.0$ kN

At 50mm edge distance,

$\psi_1 = 1$

$\psi_2 = 0.7 + 0.3 \times \frac{50}{1.5 \times 70} = 0.84$

$\psi_3 = 1.25$

$A_{no} = 9 \times 70^2 = 44100 mm^2$

$A_n = 32550 mm^2$

$N_{cb} = 37 \times 0.84 \times 1.25 \times \frac{32550}{44100} = 28.7$ kN

$\phi = 1$ for assessment.

$0.75 \times N_{cb} = 21.5$ kN

At 105mm edge distance.

$\psi_2 = 0.99$

$A_n = 43470 mm^2$

$N_{cb} = 37 \times 0.99 \times 1.25 \times \frac{43470}{44100} = 45.1$ kN

$0.75 N_{cb} = 33.9$ kN - Approx capacity of > 1.5 kN.

Calculations

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Capacity - In shear

At 50mm edge distance.

$$V_6 = k_2 \left(\frac{\lambda}{d}\right)^{0.2} \sqrt{d f_c}\ e_1^{1.5}$$

$$= 0.6 \times \left(\frac{70}{26}\right)^{0.2} \sqrt{26 \times 40} \times 50^{1.5} = 8.3 \text{ kN}$$

$\gamma_5 = 1$

$\gamma_6 = 1$

$\gamma_7 = 1.2$

$A_{vo} = 4.5 \times 50^2 = 11250 \text{ mm}^2$

$A_v = 11250 \text{ mm}^2$

$V_{cb} = 10.0$ kN

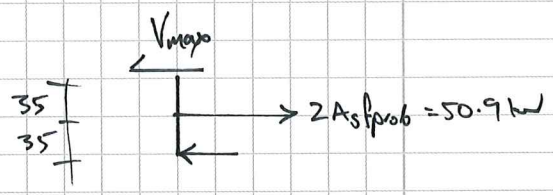
$0.75 \phi V_{cb} = 7.5$ kN.

Hairpin

D10 $\quad A = 78.5 \text{ mm}^2/\text{leg}$

$f_{prob} = 324$ MPa

$2 \times A f_{prob} = 50.9$ kN



Capacity considering prying,

$V_{max} = 50.9/2 = 25.4$ kN

Base connection

outwards - end panels

M10 EF @ 70mm

$0.75\phi N_{cb} = 26.0$ kN

$0.75\phi V_{cb} = 11.8$ kN

$V_{map} = 25.4$ kN) from hairpin.

Shear friction to slab,

$\mu_f = 0.6$

$V_n = (A_s f_y + N^*) \cdot \mu_f$

~~$= 26.0 \times 0.6 = 15.6$ kN~~

Capacity of threaded section of starter bar,

M10 thread,

$d_{minor} = 9.0$ mm

$A_s = 63.6$ mm^2

$f_{yrob} = 324$ MPa

$A_{sprob} = 20.6$ kN — which governs

Shear friction

$V_n = 0.6 \times 20.6 = 12.4$ kN — governs capacity of base connection.

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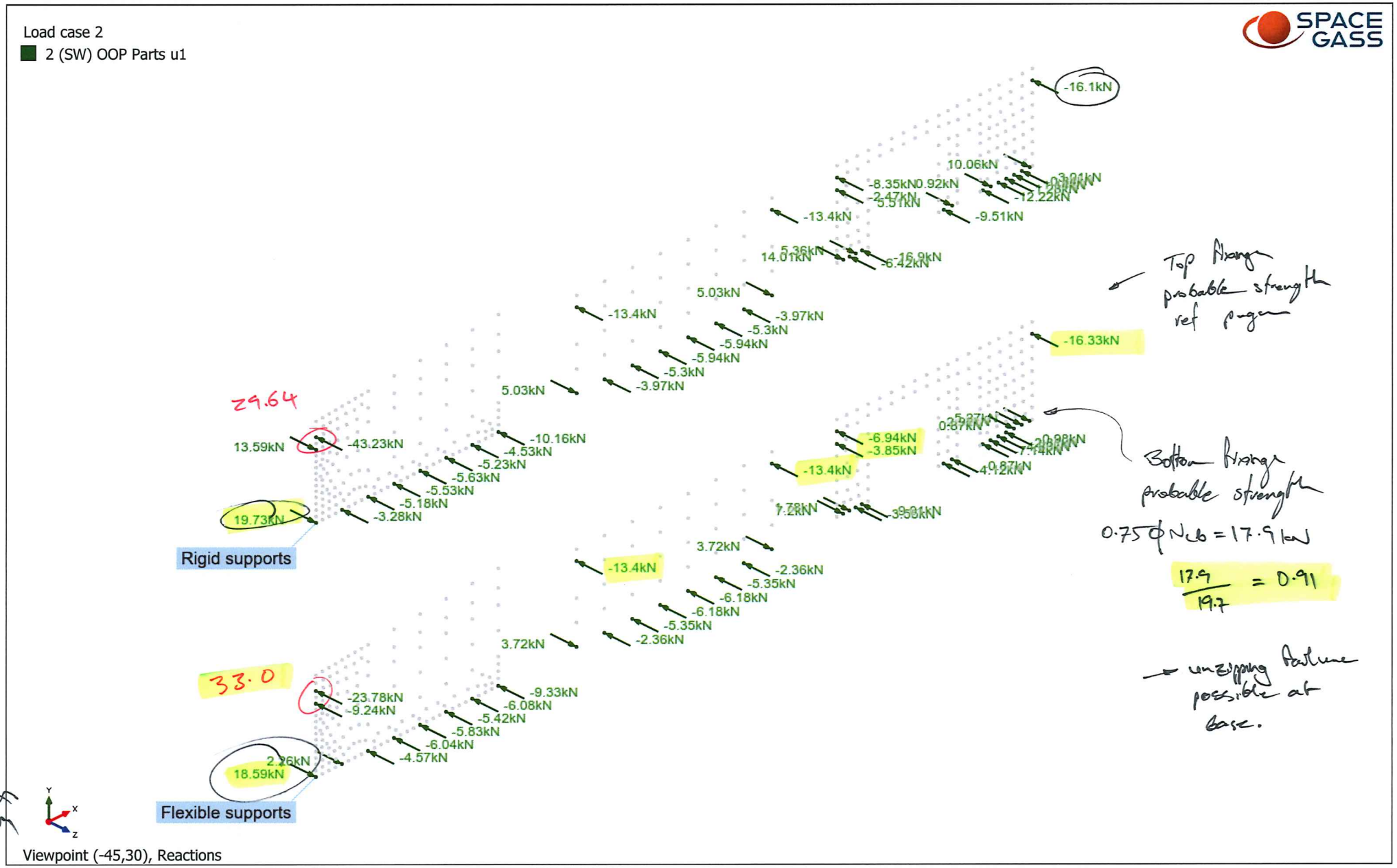
Filter: Restraint Nodes



Concrete cone failure capacity in tension & shear.

Single anchors

Case			A	B	C	D'	E'	F'
Anchor type			M10 EF	M10 EF	M20 EF	M16 EF	M20 EF	M20 EF
concrete	t	mm	100	100	100	100	100	100
	fc	MPa	40	40	40	40	40	40
Anchor	hef	mm	45	45	70	70	70	70
	do	mm	16	16	26	22	26	26
edges	c1	mm	70	70	102	92	52	160
	c2	mm	1000	150	1000	97	400	400
	e	mm	0	0	0	0	0	0
Tension	Ano	mm2	18225	18225	44100	44100	44100	44100
	An	mm2	18225	18225	43470	41370	32970	44100
	ψ1		1.00	1.00	1.00	1.00	1.00	1.00
	ψ2		1.00	1.00	0.99	0.96	0.85	1.00
	ψ3		1.25	1.25	1.25	1.25	1.25	1.25
	Nb	kN	19.1	19.1	37.0	37.0	37.0	37.0
	Ncb	kN	23.9	23.9	45.2	41.8	29.4	46.3
	ϕ		1	1	1	1	1	1
	0.75ϕNcb	**kN**	**17.9**	**17.9**	**33.9**	**31.4**	**22.0**	**34.7**
Shear	Avo	mm2	22050	22050	46818	38088	12168	115200
	Av	mm2	21000	21000	30600	23500	12168	48000
	ψ5		1	1	1	1	1	1
	ψ6		1	1	1	0.91087	1	1
	ψ7		1.2	1.2	1.2	1.2	1.2	1.2
	Vb	kN	10.9	10.9	24.3	19.8	8.8	47.7
	Vcb	kN	12.5	12.5	19.1	13.4	10.6	23.9
	ϕ		1	1	1	1	1	1
	0.75ϕVcb	**kN**	**9.4**	**9.4**	**14.3**	**10.0**	**8.0**	**17.9**

Double anchors

			D	E	F
			M16 EF	M20 EF	M20 EF
	t	mm	100	100	100
	fc	MPa	40	40	40
	hef	mm	70	70	70
	do	mm	22	26	26
	x	mm	120	400	400
	c1	mm	92	52	160
	c2	mm	97	400	400
	e	mm	0	0	0
	Ano	mm2	44100	44100	44100
	An	mm2	65010	65940	88200
	ψ1		1.00	1.00	1.00
	ψ2		0.96	0.85	1.00
	ψ3		1.25	1.25	1.25
	Nb	kN	37.0	37.0	37.0
	Ncb	kN	65.7	58.7	92.6
	ϕ		1	1	1
	0.75ϕNcb	**kN**	**49.3**	**44.1**	**69.5**
	Avo	mm2	38088	12168	115200
	Av	mm2	35500	24336	88000
	ψ5		1	1	1
	ψ6		0.91087	1	1
	ψ7		1.2	1.2	1.2
	Vb	kN	19.8	8.8	47.7
	Vcb	kN	20.2	21.2	43.8
	ϕ		1	1	1
	0.75ϕVcb	**kN**	**15.1**	**15.9**	**32.8**

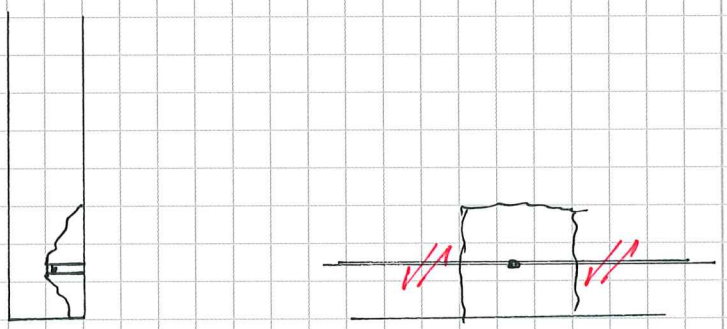
Base restraint. – in tension.

Type 'A' anchor in summary calc,

$0.75\phi N_{ub} = 17.9$ kN

$V^*_{p1} = 19.7$ kN for fully rigid support.

Residual capacity is based on dowel action of the R8 x 600mm bar through the x-hole.



R8 bar

$\phi V_{fn} = 10.1$ kN for $\phi = 1$

$\phi M_{fn} = 0.028$ kNm

Combined action on 2 No. shear planes limits the total force that can be resisted by the pair of shear planes as

$V^*_{max} = 11.2$ kN

which is less than the concrete failure force therefore there is not sufficient as a redundant load-path.

Therefore an unzipping failure would occur at around

$17.9/23.4 = 0.76$

Calculations

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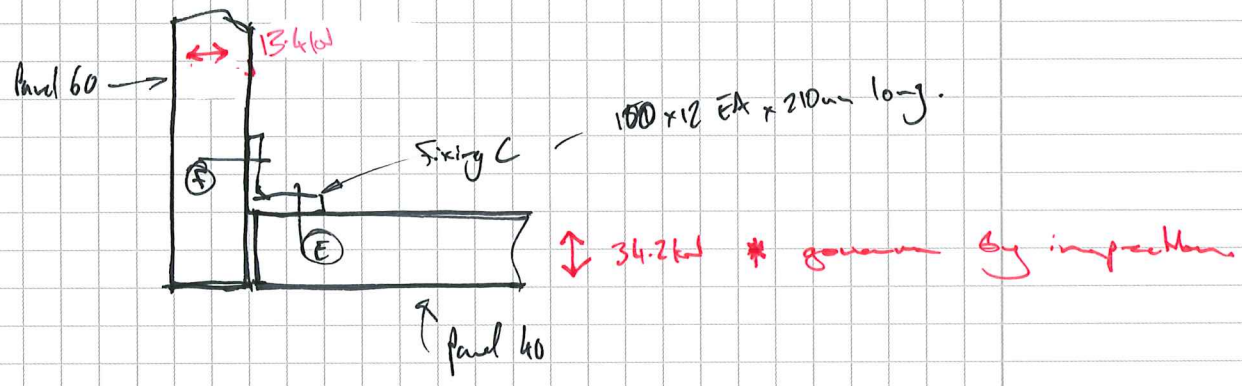
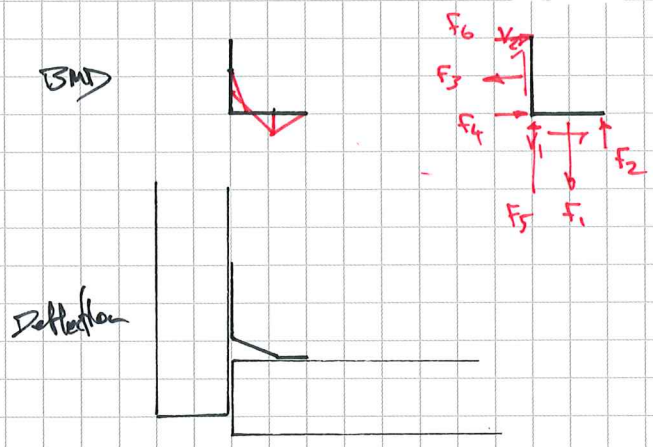
Panel to Panel connection:

Edge case - Grid A2





Reactions are indeterminate

Taking moment about F2

$F_1 \times 0.04 - V_2 \times 0.1 + F_3 \times 0.06 = 0$

From analytic model, with a single fixing modelled.

$F_1 = 35.4$ kN
$F_2 = 4.9$ kN
~~F_5 =~~ * Appears to be caused by plate rotation of the precast panel.

$F_3 = 32.0$ kN
$F_4 = 29.2$ kN
$F_6 = 2.8$ kN *

$M^* = 1.64$ kNm

$V_1 = 0$ kN
$V_2 = 30.6$ kN

Angle Bending strength,

M_{nf} fnob Z

$= 345 \times \frac{210 \times 12^2}{4}$

$= 2.61$ kNm

Which limits prying force

$F_{py} = 2.61 / 0.04 = 65.3$ kN maximum

Continued.

Calculations

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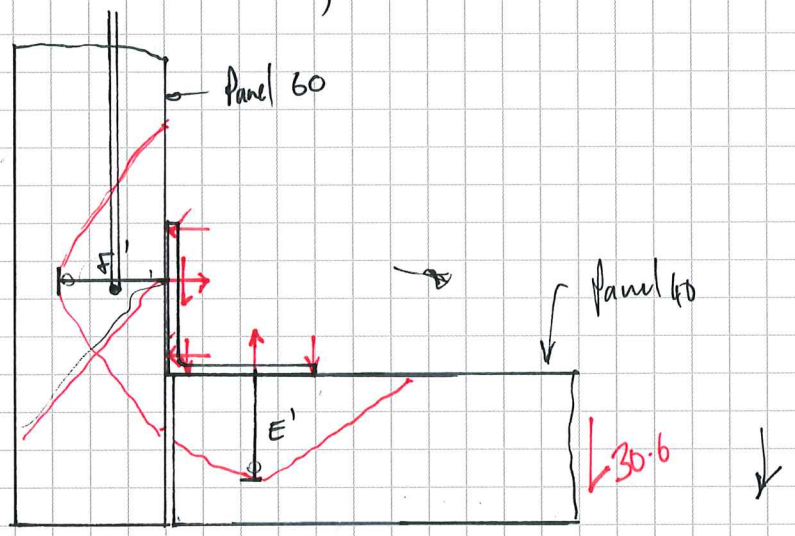
Panel to panel connection - continued

- With two fixings modelled at 400mm apart, the top fixing appears to carry all of the force. This has not been unreasonable as each fixing is expected to resist some proportion of the force.
- The proportion of force shared is indeterminate, and one fixing could be effectively resisting 100% of the force due to bolt hole alignment.
- As there is no flexibility or ductility in the fixings, there is no way of redistributing forces between fixings.
- Therefore consider a single fixing resisting 100% of the panel to panel seismic force.

Therefore Case Grid A2,



E' $N^* = 35.4$ kN
$0.75 N_{cb} = 22.0$ kN

Concrete failure
$22/35.4 = 0.62$

F' $N^* = 32.0$ kN
$V^* = 30.6$ kN

$0.75 N_{cb} = 34.7$ kN ✓
$0.75 V_{cb} = 17.9$ kN 0.58

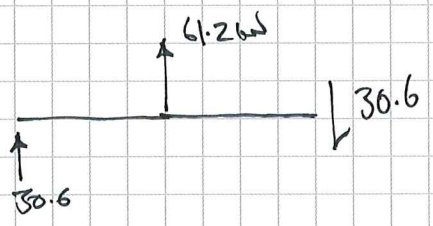
$$\frac{32.0}{34.7} + \frac{30.6}{17.9} = 2.63 > 1.2$$

Concrete failure
$\frac{1.2}{2.63} = 0.46$

However cone failure in shear has supplementary reinforcement,

D10 hairpin @ 35mm

$V^* = 30.6$ kN



Single Hairpin capacity

$50.9/61.2 = 0.83$

$T_{hairpin} = 61.2$ kN $>$ $N_{tprob} = 50.9$ kN

Performance at higher levels of displacement,

The probable sequence of events is

1. Seismic forces out of plane initially act on one of the two brackets due to uneven gaps in the bolt holes.
2. Combined shear/tension failure occurs in the F' insert (or E' insert due to tension cone failure) due to concrete cone failure, resulting in a small amount of displacement. The connection has a small residual strength due to the hairpin bar and x-hole bar.
3. The majority of the seismic force redistributes to the adjacent flying 400mm away, which will also fail as the ultimate force is the same (or similar) and the capacity of the connection is the same.
4. After concrete failure has occurred in both flanges, the seismic forces are resisted by the reinforcement anchored into the panel.

Calculations

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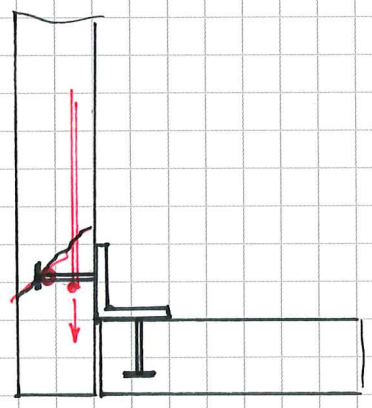
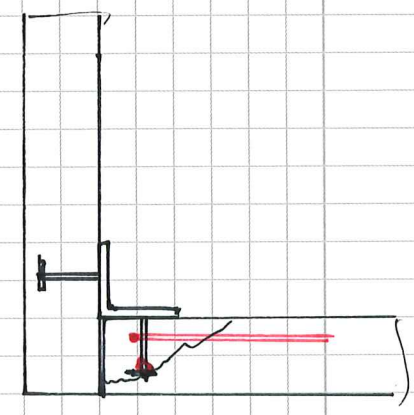
Residual Capacity

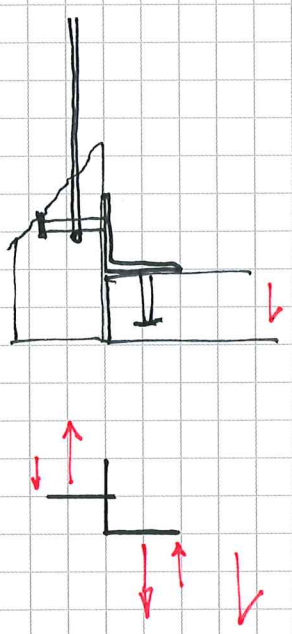


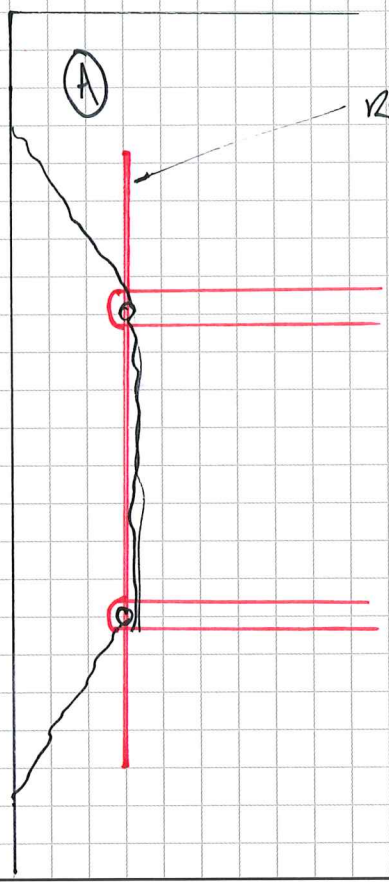
(A) Failure in panel 60



(B) Failure in panel 40.

(A) Residual force is resisted by 2 No. D10 hairpin plus R10x800mm bar through the x-hole.





R10 dowel $V_{prob} = 0.6 \times \frac{\pi \times 10^2}{4} \times 324 = 15.3\,kN$

$M_{prob} = \ldots = 0.0540\,kNm$

D10 hairpin $N_{prob} = 50.9\,kN$

D10 hairpin $N_{prob} = 50.9\,kN$

→ hairpin are a stiffer loadpath.

$\frac{2 \times 50.9}{61.2} = 1.66$

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Calculations

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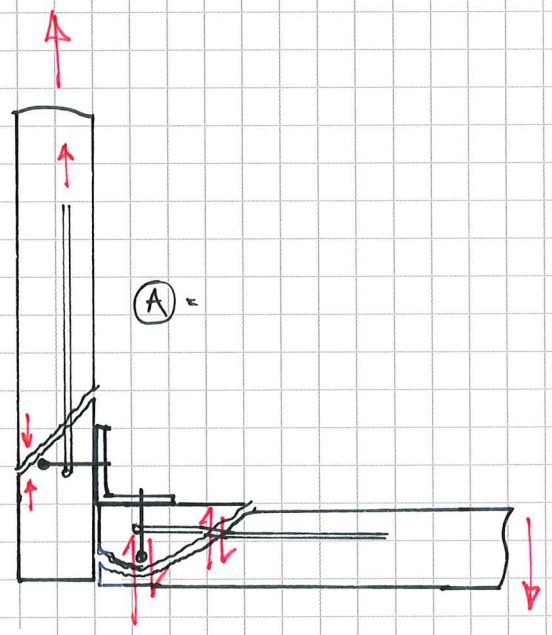
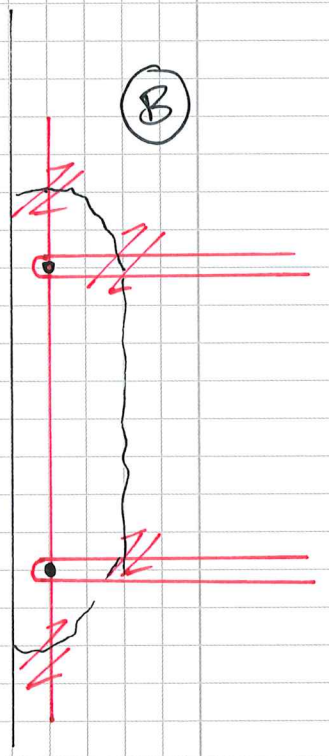
Forces redistribute and if any significant prying occurs then a failure in panel 40 would also occur.





Not a reliable mechanism

By inspection, the next failure mechanism is dowel action of the two D10 hairpins and R10 whole bars.

$V_{fn} = 15.8$ kN

$M_{fn} = 0.054$ kNm

And 6 No. shear planes to resist forces. Based on bolt bearing equations by Scanty to calculate moments, the combined actions probable strength of the 6 bars is 52.3 kN.

$\frac{52.3}{30.6} = 1.71$

Which exceeds the connection force of $\mu = 1$ panel.

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	fy MPa	fu MPa
4.6	240	400
8.8	660	830
500E	500	675

V*	52.3
n	6
ϕ	1

Actions on each bolt/pin

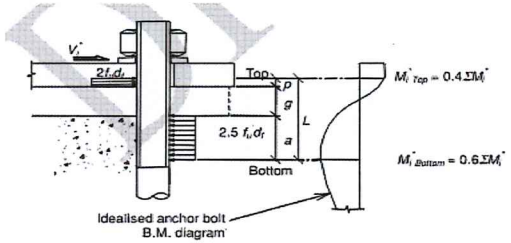
g	0	mm
N*	0	kN
V*	8.717242	kN
M*	0	kNm

based on simple cantilever

fu	500
fc	40

material that the bolt terminates into,
35 for anchor bolts into concrete,
340 for bolts into steel ferrules

Bolt	Fastener di df mm	pitch p mm	minor dia ds mm	A mm2	Ze mm3	fy MPa	fu MPa	Bolt equations ϕNtf kN	ϕVfn kN	ϕMfm kNm	check	Pin equations ϕVfp kN	ϕMfm kNm	check	Bolt BM calculation, ref Scarry paper for theory p mm	a mm	L mm	Mi* kNm	check
M12 4.6	12	1.75	10.4	84	185	240	400	33.7	19	0.04	0.46	10	0.04	0.87	0.73	4.84	5.57	0.03	1.12
M16 4.6	16	2	14.1	157	470	240	400	62.7	36	0.11	0.24	19	0.09	0.47	0.54	3.63	4.18	0.02	0.44
M20 4.6	20	2.5	17.7	245	917	240	400	97.9	56	0.22	0.16	29	0.18	0.30	0.44	2.91	3.34	0.02	0.24
M24 4.6	24	3	21.2	353	1585	240	400	141.0	80	0.38	0.11	42	0.30	0.21	0.36	2.42	2.78	0.01	0.15
M30 4.6	30	3.5	26.7	561	3178	240	400	224.2	129	0.76	0.07	67	0.61	0.13	0.29	1.94	2.23	0.01	0.08
M36 4.6	36	4	32.2	817	5589	240	400	326.7	188	1.34	0.05	97	1.07	0.09	0.24	1.61	1.86	0.01	0.05
M12 8.8	12	1.75	10.4	84	185	660	830	69.9	39.2	0.12	**0.22**	27.6	0.10	**0.32**	0.73	4.84	5.57	0.03	**0.46**
M16 8.8	16	2	14.1	157	470	660	830	130.0	74.2	0.31	**0.12**	51.3	0.25	**0.17**	0.54	3.63	4.18	0.02	**0.19**
M20 8.8	**20**	**2.5**	**17.7**	245	**917**	**660**	**830**	**203.2**	**115.9**	**0.61**	**0.08**	**80.1**	**0.48**	**0.11**	0.44	2.91	3.34	0.02	**0.10**
M24 8.8	24	3	21.2	353	1585	660	830	292.6	167	1.05	0.05	115	0.84	0.08	0.36	2.42	2.78	0.01	0.07
M30 8.8	30	3.5	26.7	561	3178	660	830	465.3	267	2.10	0.03	184	1.68	0.05	0.29	1.94	2.23	0.01	0.04
M36 8.8	36	4	32.2	817	5589	660	830	677.9	391	3.69	0.02	267	2.95	0.03	0.24	1.61	1.86	0.01	0.02
RB12	12	0	12	113	288	500	675	76.3	47	0.14	0.18	28	0.12	0.31	0.73	4.84	5.57	0.03	0.39
RB16	16	0	16	201	683	500	675	135.7	84	0.34	0.10	50	0.27	0.17	0.54	3.63	4.18	0.02	0.17
RB20	20	0	20	314	1333	324	437.4	137.4	85	0.43	0.10	50	0.35	0.17	0.44	2.91	3.34	0.02	0.14
RB25	25	0	25	491	2604	500	675	331.3	205	1.30	0.04	122	1.04	0.07	0.35	2.32	2.67	0.01	0.05
RB32	32	0	32	804	5461	500	675	542.9	337	2.73	0.03	199	2.18	0.04	0.27	1.82	2.09	0.01	0.03
R10	10	0	10	79	167	324	324	25.4	15.78	0.0540	0.55	13	0.04	0.69	0.87	5.81	6.68	0.03	1.20
R12	12	0	12	113	288	300	405	45.8	28	0.09	0.31	17	0.07	0.52	0.73	4.84	5.57	0.03	0.64
R16	16	0	16	201	683	300	405	81.4	50	0.20	0.17	30	0.16	0.29	0.54	3.63	4.18	0.02	0.28
R20	20	0	20	314	1333	300	405	127.2	79	0.40	0.11	47	0.32	0.19	0.44	2.91	3.34	0.02	0.15
R25	25	0	25	491	2604	300	405	198.8	123	0.78	0.07	73	0.63	0.12	0.35	2.32	2.67	0.01	0.09



Total moment demand $\Sigma M_i^* = V_i^*(L - 0.5P) - 2.5f_c^*d_f\frac{a^2}{2}$

8-13

Calculations

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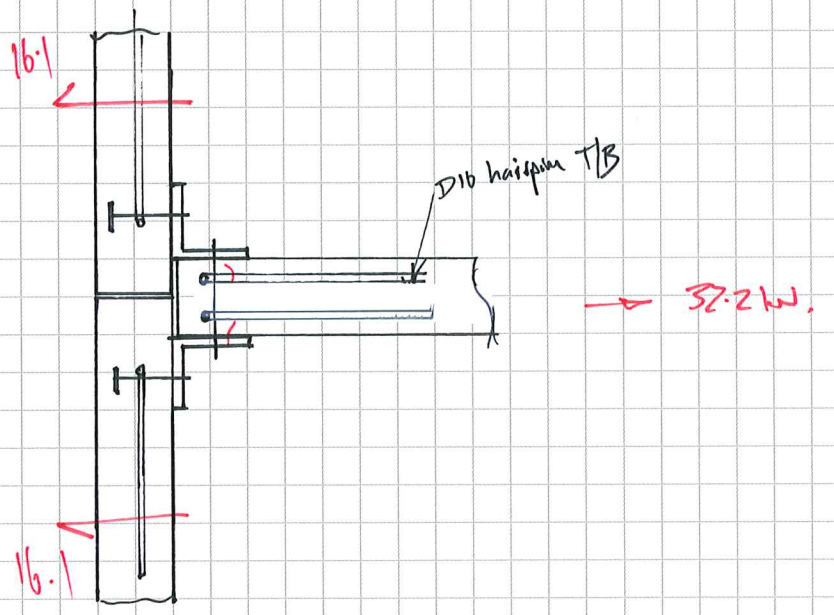
Panel to panel connection, typical interfenency wall,



By inspection the 25 mm dia PVC duct at 52mm edge distance does not have sufficient concrete cone strength and fails at around 20 kN. Both sides must fail together as a single side failure would redistribute.

Forces are resisted by the hairpin bars,

$N_t = 50.9$ kN per hairpin

Capacity of the M20 EF anchors in front/rear panels, (102mm edge distance).

$0.75N_b = 33.9$ kN each - Ok by inspection.

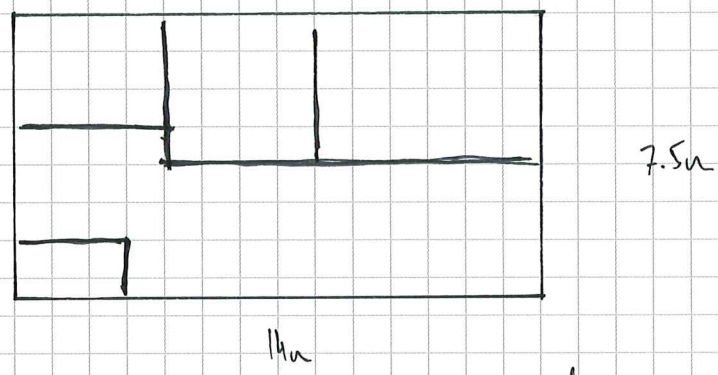
- Other direction always loads into an angle/away from edge.

$\frac{50.9}{37.2} = > 1$

$\frac{37.7}{16.1} = > 1$

Timber Framed Partition walls

- Several units have lightweight timber framed partitions built in which contribute to the seismic mass outway on the buildings.
- Timber framed partitions are typically 90mm timber with GIB or plywood linings on one or both sides.
- A typical arrangement would be,



- Walls span simply between floor & roof, therefore additional seismic mass to the roof of the units are

$$W^* = 0.4 \times 3.7/2 = 0.74 \text{ kN/m}.$$

- The walls load onto purlins & into the face of wall panels, adding to a roof mass of,

$$W^* = 0.142 \times 7.5 = 1.07 \text{ kN/m}.$$

- Total mass acting on top edge of panels is,

$$W^* = 1.81 \text{ kN/m}$$

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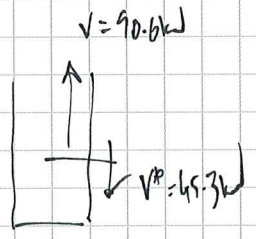
Additional forces on panels due to framing/ceilings are Ok, but additional forces on panel connections are significant.

Corner @ Top, $V^* = 45.3$ kN

Force on hairpin, $V^* = 90.6$ kN



$\frac{2 \times 50.9}{90.6} = 1.12$ - Ok.

Corner @ base $V^* = 31.6$ kN

$R = 17.9$ kN

$17.9 / 31.6 = 0.566$

Which results in an unzipping failure. - Based on elastic spaceframe modelling.

Based on FEM yield line modelling, $R^*_{base} = 19$ kN, $R^*_{top} = 26$ kN

Therefore $\frac{17.9}{19} = 0.94$

and $\frac{2 \times 50.9}{26} = 3.9$

- 0.94.

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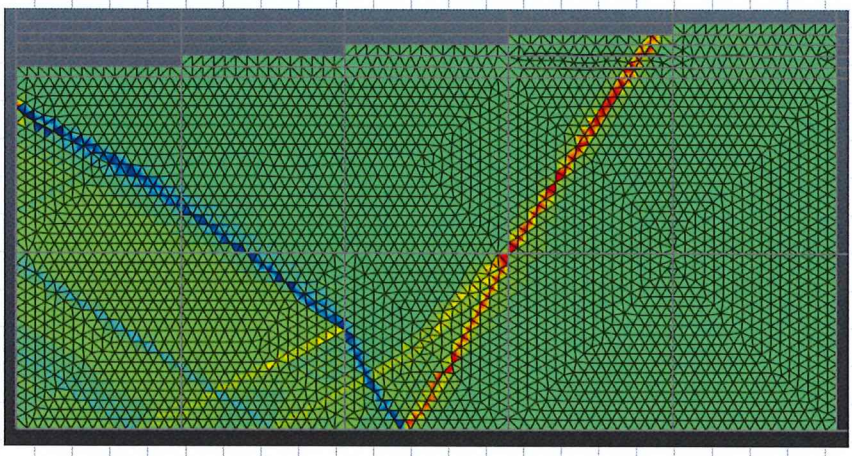
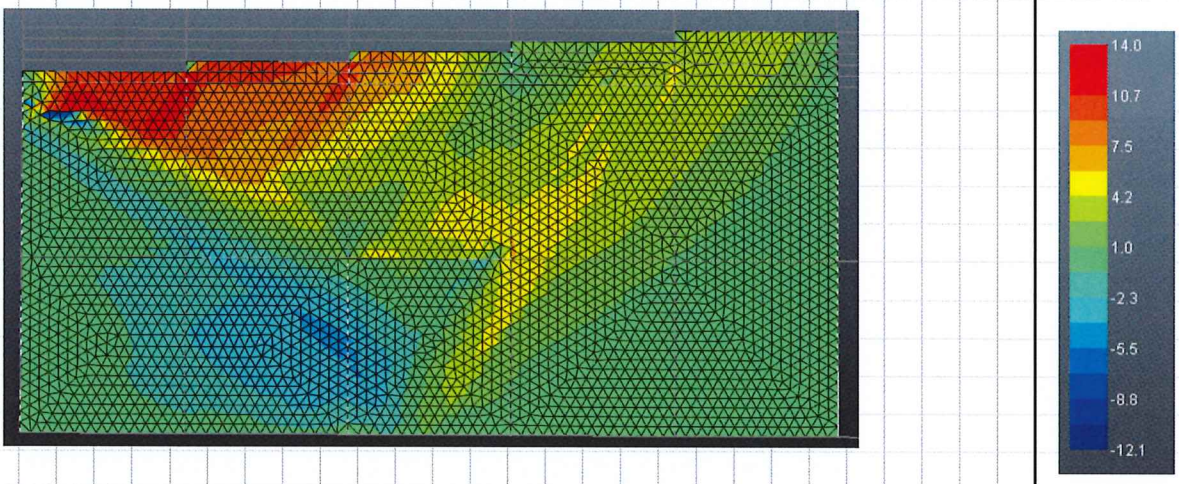
Panel yieldline theory calculation by Optum MP software
Applied load = 2.09 kPa face load + 1.10 kN/m top edge load (u=1 parts)
Capacity/Demand = 0.67



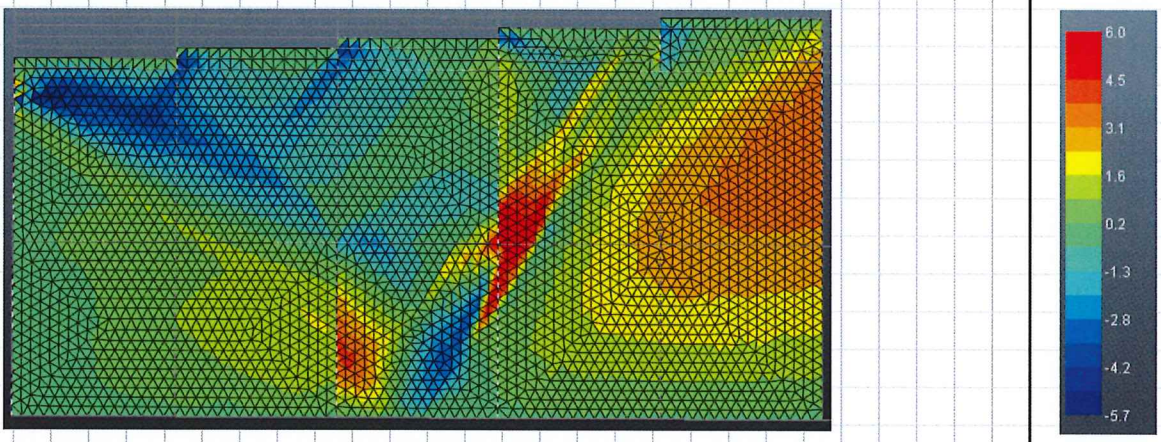
→ Panel capacity at a nominal ductility would be $\mu = 1.75$

$0.67 \times \frac{0.87}{0.74} = 0.79$

X-direction moments



Y-direction moments



Engenium Ltd, 395 Madras Street, PO Box 3911, Christchurch, New Zealand.
Tel 03 366 0434, Fax 03 379 3981, Email eng@engenium.co.nz, www.engenium.co.nz

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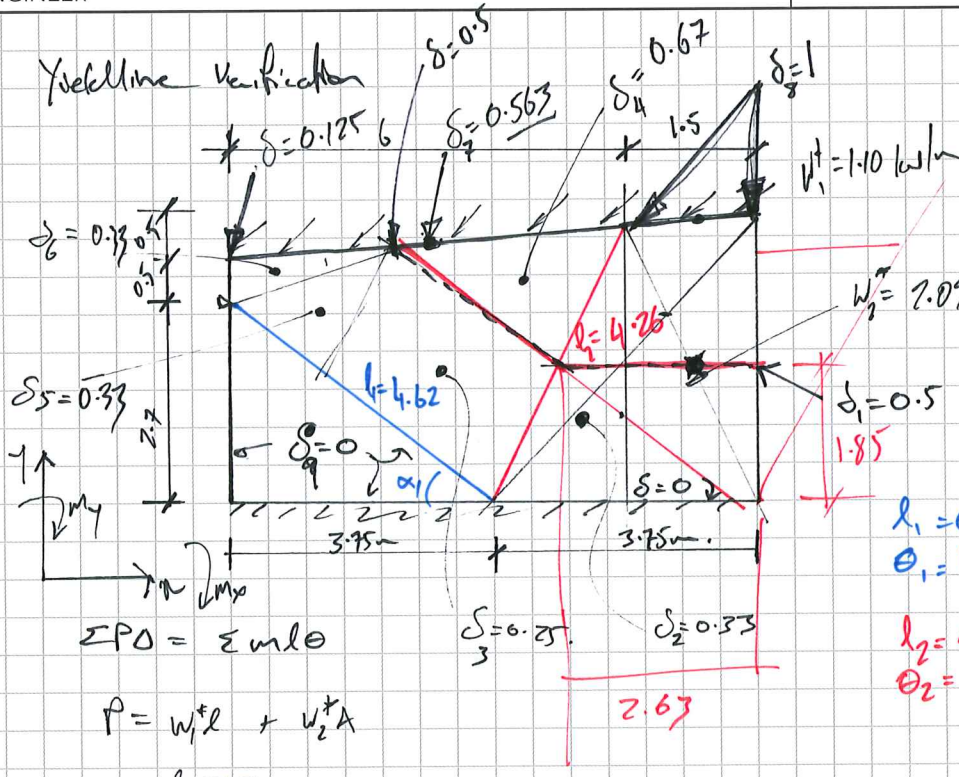
Yieldline Verification



$w_1^* = 1.10$ kN/m to top edge.

$w_2^* = 2.09$ kN/m² face load.

$l_1 = 4.62$ m

$\theta_1 = \delta / 4.26$

$l_2 = 4.26$ m

$\theta_2 = \frac{3.75 + 1.5}{2} = 2.63$

$\sqrt{2.63^2 + 1.85^2} = 3.22$

$\theta_2 = \frac{0.5\delta}{3.22}$

$M_2 = 6.01$ kNm/m.

$\alpha_1 = \tan\left(\frac{2.7}{3.75}\right) = 35.8°$

$M_x = 6.01$ kNm/m

$M_y = 6.01 + 9.57 = 15.6$ kNm/m.

Take $M = 6.01$ for simplicity.

$\Sigma P\Delta = \Sigma m l \theta$

$P = w_1^* l + w_2^* A$

$l = 7.5$

$A = 25.9$

$P = 8.25 + 54.1 = 62.4$ kN total

$P_1 = 2.09 \times 1.5 \times 3.7 = 11.6$ kN		$\delta_1 = 0.5\delta$
$P_2 = 2.09 \times 2.25 \times 3.6/2 = 8.46$ kN		$\delta_2 = 0.33\delta$
$P_3 = 2.09 \times 4.26/2 \times 4.62 \times 2/3 = 13.7$ kN		$\delta_3 = 0.25\delta$
$P_4 = 2.09 \times 4.26/2 \times 4.62 \times 2/3 \times 1/2 = 6.85$ kN		$\delta_4 = 0.67\delta$
$P_5 = 2.09 \times 4.26/2 \times 4.62 \times 1/3 \times 1/2 = 3.43$ kN		$\delta_5 = 0.33\delta$
$P_6 = 2.09 \times 0.5 \times 4.62 \times 1/3 \times 1/2 = 0.80$ kN		$\delta_6 = 0.33\delta$
$P_7 = 1.10 \times 6 = 6.6$ kN		$\delta_7 = 0.563\delta$
$P_8 = 1.10 \times 1.5 = 1.65$ kN		$\delta_8 = 1\delta$
$P_9 = 3.75 \times 2.7/2 \times 2.09 = 10.6$ kN		$\delta_9 = 0\delta$
Total	63.7 kN	

$\Sigma P\delta = 23.4\delta \times f$

f - factor representing Demand/capacity.

$\Sigma P\Delta = \Sigma m l \theta$

$= 6.01 \times 4.62 \times \frac{\delta}{4.26} + 6.01 \times 4.26 \times \frac{0.5\delta}{3.22}$

~~15.6~~ 12.74

$= 6.51\delta + 3.98\delta$

~~14.4~~ 13.8

$23.4\delta f = 10.5\delta$

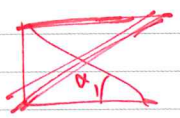
~~23.4~~ 17.8

$\frac{17.8}{23.4} = 0.76$ based on an estimate of moment capacity.

$f = \frac{10.5}{23.4} = 0.449$ based on isotropic reinforcement.

1.17

optimum MP answer appears reasonable.



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Roof

The roof are lightweight metal cladding on steel purlins spanning between concrete side walls.

There is no roof bracing in the structure so roof mass is supported by the panels supporting purlins on a tributary width basis. This includes out-of-plane forces on 'side panels'.

Roof forces transmit through EA brackets into the concrete walls, through M12 EF anchors or fixed both sides through PVC duct.

Forces from a single purlin are

$$V_{max} = 0.142 \times 1.5 \times 7.5/2 = 0.799 \text{ kN}$$

- Bolts/fixings OK by inspection.