CHRISTCHURCH CITY COUNCIL
BU 1103-001 to 011 EQ2
Maurice Carter Courts
16 Dundee Place, Spreydon

QUANTITATIVE ASSESSMENT REPORT

FINAL

- Rev B
- 17 June 2013

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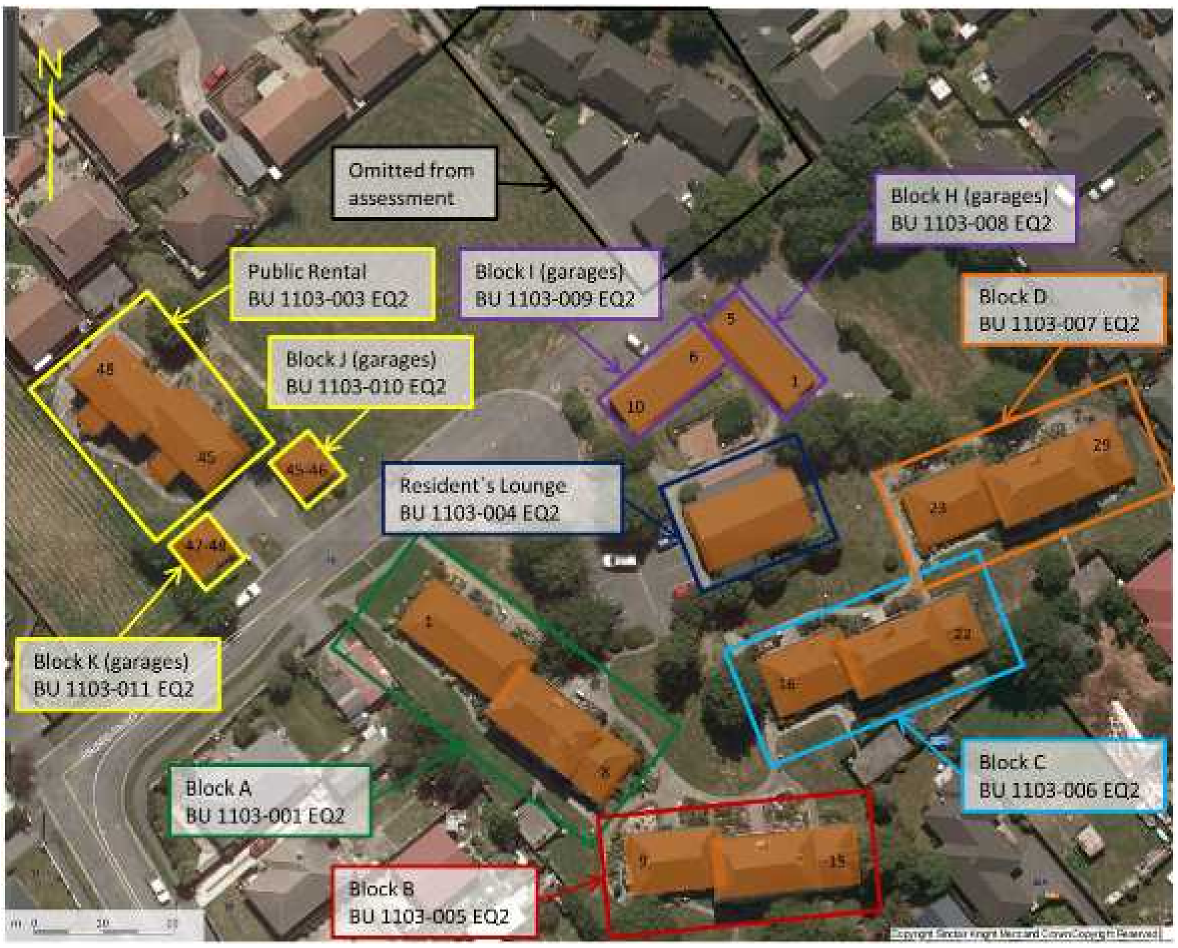
SINCLAIR KNIGHT MERZ

1. Executive Summary

1.1. Background

A Quantitative Assessment was carried out on the buildings located at 16 Dundee place, Spreydon. An aerial photograph illustrating the area is shown below in Figure 1. Detailed descriptions outlining the buildings and construction types are given in Section 5 of this report.

- **Figure 1 Aerial Photograph of Maurice Carter Courts**



This report for the building structures is based on the Engineering Advisory Group's "Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings" (from July 2011) visual inspection on 15/10/2012, limited available existing drawings by Christchurch City Council dated November 1989 and intrusive inspections and drawings by BuildQual NZ during April 2013.

1.2. Key Damage Observed

Hairline cracking was noted to elements in Blocks A and B. Non-structural damage was noted to all blocks. Refer to Section 6 Building Damage for a detailed account of the damage.

1.3. Critical Structural Weaknesses

No critical structural weaknesses have been discovered.

1.4. Indicative Building Strength

As described in the Engineering Advisory Group's "Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings" (from July 2011) we have assessed the capacity of the building using the quantitative method. Our assessment included consideration of geotechnical conditions, existing earthquake damage to the buildings and structural engineering calculations to assess both strength and ductility/resilience.

The assessments were based on the following:

- On-site investigation to assess the extent of existing earthquake damage including limited intrusive investigation.
- Architectural drawings of some of the buildings produced by CCC in 1989. See section 5 and Appendix B for details.
- Intrusive investigation - building measure-ups and details by BuildQual in April 2013. See Appendix C for details
- Qualitative assessment of critical structural weaknesses (CSWs) based on review of available structural drawings and inspection where drawings were not available.
- Levels survey results (by Woods in December 2012) were used for evaluation of the property. The survey covered ground floors of all properties in subject – for details see Appendix D.
- Geotechnical Interpretative Report produced by SKM in December 2012. This report was primarily issued to provide recommendations for proposed new build residential units located in the vicinity of the existing buildings in subject. See Appendix E for details.

Two of the five buildings are Earthquake Prone and three are of moderate risk. The structures with the worst anticipated seismic performance are listed below in approximate order of priority:

STRUCTURE NAME	ESTIMATED %NBS STRENGTH	DETAILING DEFICIENCIES IDENTIFIED
Block H, I	22% NBS	In plane longitudinal shear
Block J, K	22% NBS	In plane longitudinal shear
Public Rental	42% NBS	In plane longitudinal shear
Block A, B, C, D	44% NBS	Ground Floor longitudinal in plane shear
Residents Lounge	47% NBS	In plane transverse shear

1.5. Conclusions and Recommendations

The capacities calculated are generally less than 100%NBS due to changes in design codes, resulting in greater seismic loads than were considered at the time of design. There were no drawings available for three of the five buildings in the table and these were surveyed by BuildQual and therefore the assumptions made during the assessment of the structures may be conservative and need confirmation prior to designing the strengthening solutions.

It is recommended that:

a) There is no damage to the buildings that would cause them to be unsafe to occupy.

b) Options to strengthen the buildings to a target of 67% should be investigated.

c) Barriers around the building are not necessary.

2. Introduction

Sinclair Knight Merz was engaged by Christchurch City Council to carry out a Quantitative Assessment of the seismic performance of the buildings at Maurice Carter Courts located at 16 Dundee place, Spreydon.

The scope of this quantitative analysis includes the following:

- Analysis of the seismic load carrying capacity of the buildings compared with current seismic loading requirements or New Buildings Standard (NBS). It should be noted that this analysis considers the building in its damaged state where appropriate.
- Identify any critical structural weaknesses which may exist in the building and include these in the assessed %NBS of the structure.
- Preparation of a summary report outlining the areas of concern in the building

The recommendations from the Engineering Advisory Group[1] were followed to assess the likely performance of the structures in a seismic event relative to the New Building Standard (NBS). 100% NBS is equivalent to the strength of a building that fully complies with current codes. This includes a recent increase of the Christchurch seismic hazard factor from 0.22 to 0.3[2].

At the time of this report, only architectural drawings by Christchurch City Council dated August 1989 were made available for two buildings, for the other buildings an intrusive investigation and measure up was completed by BuildQual in April 2013. These have been used in our evaluation of the building. The building description below is based on a review of the drawings and our visual inspections.

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[1] EAG 2011, *Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury - Draft*, p 10
[2] http://www.dbh.govt.nz/seismicity-info

3. Compliance

This section contains a summary of the requirements of the various statutes and authorities that control activities in relation to buildings in Christchurch at present.

3.1. Canterbury Earthquake Recovery Authority (CERA)

CERA was established on 28 March 2011 to take control of the recovery of Christchurch using powers established by the Canterbury Earthquake Recovery Act enacted on 18 April 2011. This act gives the Chief Executive Officer of CERA wide powers in relation to building safety, demolition and repair. Two relevant sections are:

Section 38 – Works

This section outlines a process in which the chief executive can give notice that a building is to be demolished and if the owner does not carry out the demolition, the chief executive can commission the demolition and recover the costs from the owner or by placing a charge on the owners' land.

Section 51 – Requiring Structural Survey

This section enables the chief executive to require a building owner, insurer or mortgagee carry out a full structural survey before the building is re-occupied.

We understand that CERA will require a detailed engineering evaluation to be carried out for all buildings (other than those exempt from the Earthquake Prone Building definition in the Building Act). It is anticipated that CERA will adopt the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011. This document sets out a methodology for both qualitative and quantitative assessments.

The qualitative assessment is a desk-top and site inspection assessment. It is based on a thorough visual inspection of the building coupled with a review of available documentation such as drawings and specifications. The quantitative assessment involves analytical calculation of the buildings strength and may require non-destructive or destructive material testing, geotechnical testing and intrusive investigation.

It is anticipated that factors determining the extent of evaluation and strengthening level required will include:

- The importance level and occupancy of the building
- The placard status and amount of damage
- The age and structural type of the building

- Consideration of any critical structural weaknesses
- The extent of any earthquake damage

3.2. Building Act

Several sections of the Building Act are relevant when considering structural requirements:

3.2.1. Section 112 – Alterations

This section requires that an existing building complies with the relevant sections of the Building Code to at least the extent that it did prior to any alteration. This effectively means that a building cannot be weakened as a result of an alteration (including partial demolition).

3.2.2. Section 115 – Change of Use

This section requires that the territorial authority (in this case Christchurch City Council (CCC)) be satisfied that the building with a new use complies with the relevant sections of the Building Code 'as near as is reasonably practicable'. Regarding seismic capacity 'as near as reasonably practicable' has previously been interpreted by CCC as achieving a minimum of 67%NBS however where practical achieving 100%NBS is desirable. The New Zealand Society for Earthquake Engineering (NZSEE) recommend a minimum of 67%NBS.

3.2.3. Section 121 – Dangerous Buildings

The definition of dangerous building in the Act was extended by the Canterbury Earthquake (Building Act) Order 2010, and it now defines a building as dangerous if:

- in the ordinary course of events (excluding the occurrence of an earthquake), the building is likely to cause injury or death or damage to other property; or
- in the event of fire, injury or death to any persons in the building or on other property is likely because of fire hazard or the occupancy of the building; or
- there is a risk that the building could collapse or otherwise cause injury or death as a result of earthquake shaking that is less than a 'moderate earthquake' (refer to Section 122 below); or
- there is a risk that that other property could collapse or otherwise cause injury or death; or
- a territorial authority has not been able to undertake an inspection to determine whether the building is dangerous.

3.2.4. Section 122 – Earthquake Prone Buildings

This section defines a building as earthquake prone if its ultimate capacity would be exceeded in a 'moderate earthquake' and it would be likely to collapse causing injury or death, or damage to other property. A moderate earthquake is defined by the building regulations as one that would generate ground shaking 33% of the shaking used to design an equivalent new building.

3.2.5. Section 124 – Powers of Territorial Authorities

This section gives the territorial authority the power to require strengthening work within specified timeframes or to close and prevent occupancy to any building defined as dangerous or earthquake prone.

3.2.6. Section 131 – Earthquake Prone Building Policy

This section requires the territorial authority to adopt a specific policy for earthquake prone, dangerous and insanitary buildings.

3.3. Christchurch City Council Policy

Christchurch City Council adopted their Earthquake Prone, Dangerous and Insanitary Building Policy in 2006. This policy was amended immediately following the Darfield Earthquake of the 4th September 2010.

The 2010 amendment includes the following:

- A process for identifying, categorising and prioritising Earthquake Prone Buildings, commencing on 1 July 2012;
- A strengthening target level of 67% of a new building for buildings that are Earthquake Prone. Council recognises that it may not be practicable for some repairs to meet that target. The council will work closely with building owners to achieve sensible, safe outcomes;
- A timeframe of 15-30 years for Earthquake Prone Buildings to be strengthened; and,
- Repair works for buildings damaged by earthquakes will be required to comply with the above.

The council has stated their willingness to consider retrofit proposals on a case by case basis, considering the economic impact of such a retrofit.

We anticipate that any building with a capacity of less than 34%NBS (including consideration of critical structural weaknesses) will need to be strengthened to a target of 67%NBS of new building standard as recommended by the Policy.

If strengthening works are undertaken, a building consent will be required. A requirement of the consent will require upgrade of the building to comply 'as near as is reasonably practicable' with:

- The accessibility requirements of the Building Code.
- The fire requirements of the Building Code. This is likely to require a fire report to be submitted with the building consent application.

3.4. Building Code

The building code outlines performance standards for buildings and the Building Act requires that all new buildings comply with this code. Compliance Documents published by The Department of Building and Housing can be used to demonstrate compliance with the Building Code.

After the February Earthquake, on 19 May 2011, Compliance Document B1: Structure was amended to include increased seismic design requirements for Canterbury as follows:

a) Hazard Factor increased from 0.22 to 0.3 (36% increase in the basic seismic design load),
b) Serviceability Return Period Factor increased from 0.25 to 0.33 (80% increase in the serviceability design loads when combined with the Hazard Factor increase),
c) The increase in the above factors has resulted in a reduction in the level of compliance of an existing building relative to a new building despite the capacity of the existing building not changing.

4. Earthquake Resistance Standards

For this assessment, the building's earthquake resistance is compared with the current New Zealand Building Code requirements for a new building constructed on the site. This is expressed as a percentage of new building standard (%NBS).

The likely capacity of this building has been derived in accordance with the New Zealand Society for Earthquake Engineering (NZSEE) guidelines 'Assessment and Improvement of the Structural Performance of Buildings in Earthquakes' (AISPBE), 2006. These guidelines provide an Initial Evaluation Procedure that assesses a buildings capacity based on a comparison of loading codes from when the building was designed and currently. It is a quick high-level procedure that can be used when undertaking a Qualitative analysis of a building. The guidelines also provide guidance on calculating a modified Ultimate Limit State capacity of the building which is much more accurate and can be used when undertaking a Quantitative analysis.

The New Zealand Society for Earthquake Engineering has proposed a way for classifying earthquake risk for existing buildings in terms of %NBS and this is shown in Figure 2 below.

- **Figure 2: NZSEE Risk Classifications Extracted from table 2.2 of the NZSEE 2006 AISPBE Guidelines**

Description	Grade	Risk	%NBS	Existing Building Structural Performance	Improvement of Structural Performance	
					Legal Requirement	NZSEE Recommendation
Low Risk Building	A or B	Low	Above 67	Acceptable (improvement may be desirable)	The Building Act sets no required level of structural improvement (unless change in use) This is for each TA to decide. Improvement is not limited to 34%NBS.	100%NBS desirable. Improvement should achieve at least 67%NBS
Moderate Risk Building	B or C	Moderate	34 to 66	Acceptable legally. Improvement recommended		Not recommended. Acceptable only in exceptional circumstances
High Risk Building	D or E	High	33 or lower	Unacceptable (Improvement	Unacceptable	Unacceptable

Table 1 below provides an indication of the risk of failure for an existing building with a given percentage NBS, relative to the risk of failure for a new building that has been designed to meet current Building Code criteria (the annual probability of exceedance specified by current earthquake design standards for a building of 'normal' importance is 1/500, or 0.2% in the next year, which is equivalent to 10% probability of exceedance in the next 50 years).

Table 1: %NBS compared to relative risk of failure

Percentage of New Building Standard (%NBS)	Relative Risk (Approximate)
>100	<1 time
80-100	1-2 times
67-80	2-5 times
33-67	5-10 times
20-33	10-25 times
<20	>25 times

5. Building Details

The complex under consideration consists of a number of residential buildings, garages and resident`s lounge as shown on aerial view in Figure 1. Buildings to the north corner of the compound were omitted from the assessment. For the purpose of this report; Table 2 shows the notations adopted (in line with CCC notations):

- **Table 2 - Building notations**

CCC notation	Local notation	Purpose	Note	Available Drawings
BU 1103-001 EQ2	Block A	block of flats	Similar to B, C & D**	Original electrical drawing of Block B, C or D (CCC 1989) & new survey drawings (BuildQual 2013)
BU 1103-003 EQ2	Public Rental (PR)	block of flats		Original architectural /structural drawings (CCC 1989)
BU 1103-004 EQ2	Resident`s Lounge (RL)	lounge		Original architectural drawing (CCC 1989) & new survey drawings (BuildQual 2013)
BU 1103-005 EQ2	Block B	block of flats	Similar*	Original electrical drawing of Block B, C or D (CCC 1989) & new as-built drawings (BuildQual 2013)
BU 1103-006 EQ2	Block C	block of flats		
BU 1103-007 EQ2	Block D	block of flats		
BU 1103-008 EQ2	Block H	garage	Similar*	new survey drawings (BuildQual 2013)
BU 1103-009 EQ2	Block I	garage		
BU 1103-010 EQ2	Block J	garage	Similar*	Original architectural /structural drawings (CCC 1989)
BU 1103-011 EQ2	Block K	garage		

*   Buildings are of similar layout and construction.

** Similar construction, but slightly different layout and size.

Building description and our evaluation is based on the visual inspection of external surfaces, original architectural drawings (by CCC in 1989 – Appendix C) and newly completed as-built drawings (by BuildQual in 2013 – Appendix D).

5.1. Design Criteria and Assumptions

The following design criteria and assumptions made in undertaking the assessment of all the buildings include:

- The building was built according to the drawings and according to good practice at the time. We have reviewed the building and from our visual inspection the structure appears to be built in accordance with the drawings.
- The soil on site is class D as described in AS/NZS1170.5:2004, Clause 3.1.3, Soft Soil. This is a conservative assumption based on the desktop study. The ultimate bearing capacity on site is in order of 200kPa.
- Standard design assumptions for residential type buildings as described in AS/NZS 1170.0 :2002:
- 50 year design life.
- Structure Importance Level 2. This level of importance is described as 'normal' with medium or considerable consequence for loss of human life, or considerable economic, social or environmental consequence of failure.
- Site hazard factor, Z = 0.3, NZBC, Clause B1 Structure, Amendment 11 effective from 1 August 2011.
- The following material properties were estimated and used in the analyses:
- **Table 3: Material Properties**

Material	Nominal Strength
Structural Steel	f_y = 250MPa
Concrete	f_c' = 30MPa
Timber – No 1 Framing	f_b' = 10MPa
Masonry	f_m' = 12MPa

The detailed engineering analysis is a post construction evaluation therefore it has the following limitations:

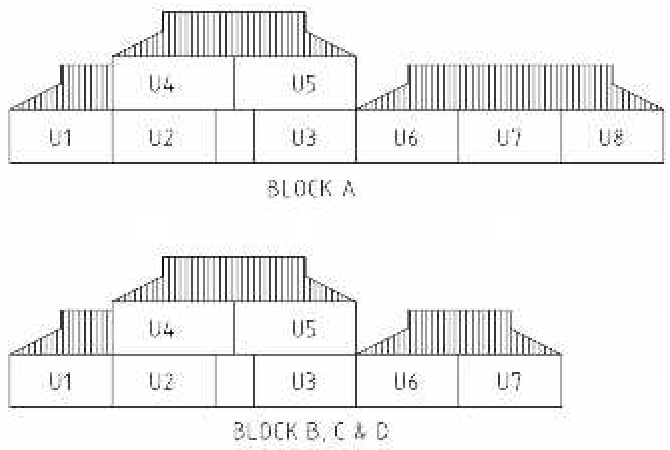
- It is not likely to pick up on any concealed construction errors (if they exist).
- Other possible issues that could affect the performance of the building such as corrosion and modifications to the structure will not be identified unless they are visible and have been specifically mentioned in this report.

The detailed engineering evaluation deals only with the structural aspects of the structure. Other aspects such as building services are not covered.

5.2. Block A, B, C & D

Due to similarities of the blocks A, B, C & D, the below description is given for Block A containing 8 units. Blocks B, C & D are of similar construction, but having one of the single storey wings shorter by one unit – see Figure 3.

- **Figure 3: Schematic front elevations on Blocks A, B, C & D**



The building is a block of flats with the central part being two-storey high and with single storey wings extending to each side from the central part (PHOTOS 1-10).

The central core of the building is constructed of concrete walls, columns, stairs and precast floor slab (PHOTOS 5-9), supplemented by timber framework clad with brick veneer or weatherboard to the perimeter of the upper floor. The wings are constructed of timber framework clad with brick veneer or weatherboard and separated from central part by concrete wall. The internal face of the walls is generally lined with gib board.

The hip roof is constructed of series of timber trusses spanning in transverse direction, supporting timber purlins ply sarking and corrugated metal sheeting. The plasterboard ceiling is attached to the underside of the roof trusses. There is a roof skylight located above the central stair (PHOTO 10).

The building is founded on strip footings and a ground bearing slab.

Refer to PHOTOS 11-20 for general images of Blocks B, C & D.

5.2.1.Gravity load resisting system

Weight of roof is transferred to the perimeter walls (typically timber framework) through transversely spanning timber trusses. The upper floor loads are transferred to the transverse walls and concrete columns through precast concrete slab. The ground floor is directly supported by supporting ground.

Weight of walls and applied loads are transferred into concrete strip footing and resisted by sub-soil.

5.2.2. Seismic load resisting system

Lateral loads at roof level are distributed to the supporting walls by action of roof diaphragm 13 mm gib board attached to the underside of the roof trusses.

Lateral loads at upper floor level are distributed to the supporting walls by diaphragm action of precast concrete floor slab.

Horizontal forces are transferred to foundation level by means of combination of concrete walls and timber stud walls with plasterboard linings, acting as shear walls.

Horizontal forces at foundation level are resisted by friction and ground pressures between the surrounding soil and foundations.

5.2.3. Analysis Assumptions

Longitudinal direction	Transverse direction
▪ Period, T < 0.4s	▪ Period, T < 0.4s
▪ Ductility, μ = 2.0	▪ Ductility, μ = 1.25

- No panel joints were found in the concrete walls - it has been assumed that the walls were cast in-situ.
- The concrete walls were found through intrusive investigation to be singly reinforced with:
  - 12 mm bars at 300 mm centres vertically
  - 12 mm bars at 300 mm centres horizontally
- It is assumed that the plywood roof diaphragm is sufficiently connected to the perimeter supports (i.e. concrete walls, sarking in other planes). Although the architectural sections show the sarking is in close contact with perimeter elements, detail of the connection is not available.
- It is assumed that all the concrete walls are connected to the diaphragm and therefore contribute to the transverse and longitudinal capacity of the building. This will need to be confirmed during the detailed design of strengthening works.

5.3. Public Rental

This building is a two-storey high block of flats constructed of timber frame walls clad with brick veneer or weatherboards externally and with plasterboard or customwood internally (PHOTOS 37-42). Each unit occupies two stories and the units are separated by reinforced concrete masonry wall 190 mm thick.

The hip roof is constructed of series of timber trusses spanning in transverse direction, supporting timber purlins, ply sarking and corrugated metal sheeting. The plasterboard ceiling is attached to the underside of the roof trusses.

The floor is constructed of timber floor joists, supporting customwood floor and plasterboard ceiling. The span of floor joists is variable throughout the plan of the building (typically spanning longitudinally south-west half of the plan and transversely to the north-east half of the plan).

The building is founded on strip footings and ground bearing slab.

5.3.1. Gravity load resisting system

Weight of roof is transferred to the perimeter walls (typically timber framework) through transversely spanning timber trusses. The upper floor loads are transferred to the supporting walls and timber beams through timber floor construction.

Weight of walls and applied loads are transferred into concrete strip footing and resisted by sub-soil.

5.3.2. Seismic load resisting system

Lateral loads at roof level are distributed to the supporting walls by action of roof diaphragm (12 mm ply sarking) and ceiling panels attached to the underside of the roof trusses (10 mm plasterboard).

Lateral loads at upper floor level (are distributed to the supporting walls by diaphragm action of customwood flooring (20 mm) and plasterboard ceiling (10 mm) attached directly to the floor joists.

Horizontal forces are transferred to foundation level by means of combination of concrete block walls and timber stud walls with plasterboard linings (10 mm), ply bracing (12 mm), weatherboard boards (6 mm) or angle braces - acting as shear walls.

Horizontal forces at foundation level are resisted by friction and ground pressures between the surrounding soil and foundations.

5.3.3. Design Assumptions

- Period T < 0.4 seconds
- Ductility, $\mu = 2$
- It is assumed that the timber roof diaphragm is sufficiently connected to the perimeter supports (i.e. concrete walls, sarking in other planes). Although the architectural

sections show the sarking is in close contact with perimeter elements, detail of the connection is not available.

- The size and specification of the foundations has been estimated and scaled off architectural drawings as follows:
  - Width of typical strip footing = 600 mm;
  - Depth of typical strip footing = 300 mm;
  - Reinforcement: No reinforcement (however some may exist)

5.4. Resident`s Lounge

The building is a single storey community hall of rectangular plan, constructed primarily of timber frame walls clad with brick veneer or weatherboard externally and with plasterboard internally (PHOTOS 43-50). The timber framework to gable walls is additionally supplemented by 2No 100x100 SHS posts spanning vertically from base to rafter (one to each side of the large opening).

The duo pitch roof is constructed of series of timber trusses (with raised bottom chord) spanning in transverse direction, supporting timber purlins, ply sarking and corrugated metal sheeting. The plasterboard ceiling is attached to the underside of the roof trusses. The roof is partially extended to the north-west forming a veranda (with eaves supported by series of timber columns) and to the south-east forming a small entrance shelter.

The building is founded on strip footings and a ground bearing slab.

5.4.1.Gravity load resisting system

Weight of roof is transferred to the perimeter walls and beams (typically timber framework) through transversely spanning timber trusses.

Weight of walls and applied loads are transferred into concrete strip footing and resisted by sub-soil.

5.4.2.Seismic load resisting system

Lateral loads at roof level are distributed to the supporting walls by action of roof diaphragm 13 mm gib board attached to the underside of the roof trusses.

Horizontal forces are primarily transferred to foundation level by means of timber stud walls with either 10 mm gib board to gable walls or angle braces to longitudinal walls.

Horizontal forces at foundation level are resisted by friction and ground pressures between the surrounding soil and foundations.

5.4.3. Design Assumptions

- Period T < 0.4 seconds
- Ductility, $\mu = 2$

5.5. Block H & I

The buildings are identical, single storey garages with 5 garages per building divided by plywood partitions, constructed with timber frame walls. The external walls are clad with brick veneer on the sides and weatherboard on the front and rear and exposed internally (PHOTOS 51-57). The mono pitch roof comprises a series of timber joists spanning in transverse direction. The building is founded on strip footing and a ground bearing slab.

5.5.1. Gravity load resisting system

Weight of roof is transferred to the perimeter walls and beams (typically timber framework) through transversely spanning roof joists. The ground floor is directly supported by supporting ground.

Weight of walls and applied loads are transferred into concrete strip footing and resisted by sub-soil.

5.5.2. Seismic load resisting system

Lateral loads at roof level are distributed to the supporting walls by action of roof plane bracing.

Horizontal forces are primarily transferred to foundation level by means of timber stud walls with either angle braces or gib board lining.

Horizontal forces at foundation level are resisted by friction and ground pressures between the surrounding soil and foundations.

5.5.3. Design Assumptions

- Period T < 0.4 seconds
- Ductility, $\mu = 2$

5.6. Block J & K

The buildings are identical, single story garages (Two garages per building divided by plasterboard partition), constructed of timber frame walls clad with brick veneer (sides and rear) or weatherboard (front) externally and exposed internally (PHOTOS 64-71). The hip roof is made of series of timber roof trusses. The building is founded on strip footing and a ground bearing slab.

5.6.1. Block J & K – Gravity load resisting system

Weight of roof is transferred to the perimeter walls and beams (typically timber framework) through transversely spanning timber trusses. The ground floor is directly supported by supporting ground.

Weight of walls and applied loads are transferred into concrete strip footing and resisted by sub-soil.

5.6.2. Block J & K – Seismic load resisting system

Lateral loads at roof level are distributed to the supporting walls by action of roof plane bracing (angle brace 22x22x1.2 and hip rafters).

Horizontal forces are primarily transferred to foundation level by means of timber stud walls with either angle braces (22x22x1.2 to sides and rear) or weatherboards (front).

Horizontal forces at foundation level are resisted by friction and ground pressures between the surrounding soil and foundations.

5.6.3. Design Assumptions

- Period $T < 0.4$ seconds
- Ductility, $\mu = 2$

6. Building Damage

The list of damage items observed during the time of inspection and upon evaluation of levels survey results (by Woods on 14/12/2012 – Appendix D) is as follows:

6.1. Block A

Structural damage	
A-1	Hairline cracking in the transition from concrete slab to supporting concrete wall was observed (PHOTOS 21). The precast concrete slab is simply supported by steel angle protruding from the supporting wall – the cracking indicates some rotation at this interface.
A-2	Cracking (up to 0.4 mm) through the concrete slab was observed on the balcony (PHOTOS 22-23).
Non-structural damage	
A-3	Superficial cracking to plasterboard lining throughout the building.
A-4	Cracking to ceiling panel to the underside of the roof overhang (PHOTO 24)
A-5	Timber lining has detached from the end of concrete walls (PHOTOS 6 & 25)
A-6	Localised variation in levels was observed in **Unit 5**: ▪ 18 mm over the distance of 3.2 m - gradient 1:177 (lino – kitchen) Localised variations in levels were observed in **Unit 8**: ▪ 26 mm over the distance of 2.7 m - gradient 1:103 (lino – kitchen) ▪ 15 mm over the distance of 3.2 m - gradient 1:213 (carpet – living room)

6.2. Block B

Structural damage	
B-1	Hairline cracking through the concrete columns was observed to the front elevation of the central part of the building (PHOTO 28).
Non-structural damage	
B-2	Non-structural hairline cracking through the concrete slab was observed on the balcony (PHOTOS 26-27).
B-3	Superficial cracking to plasterboard lining throughout.
B-4	Cracking to ceiling panel to the underside of the roof near skylight (PHOTO 29)
B-5	Timber lining has detached from the end of concrete walls (PHOTO 30)

B-6	Localised variation in levels was observed in **Unit 9**: ▪ 16 mm over the distance of 2.7 m - gradient 1:170 (lino – bathroom) Localised variation in levels was observed in **Unit 15**: ▪ 18 mm over the distance of 2.7 m - gradient 1:150 (lino – bathroom)

6.3. Block C

Structural damage	
-	Not observed.
Non-structural damage	
C-1	Non-structural hairline cracking through the concrete slab was observed on the balcony (PHOTOS 31-32).
C-2	Superficial cracking to plasterboard lining throughout.
C-3	Brick veneer has slightly separated from the concrete wall to the rear of the building – upper floor, near central staircase (PHOTO 33)
C-4	Localised variation in levels was observed in **Unit 19**: ▪ 22 mm over the distance of 2.7 m - gradient 1:122 (lino – bathroom) Localised variation in levels was observed in **Unit 21**: ▪ 16 mm over the distance of 2.7 m - gradient 1:170 (lino – bathroom) Localised variation in levels was observed in **Unit 22**: ▪ 31 mm over the distance of 2.7 m - gradient 1:87 (lino – bathroom)

6.4. Block D

Structural damage	
-	Not observed.
Non-structural damage	
D-1	Superficial cracking to plasterboard lining throughout.
D-2	Non-structural hairline cracking through the concrete slab was observed on the balcony (PHOTOS 34-36).

D-3	Localised variation in levels was observed in **Unit 23**: ▪ 30 mm over the distance of 2.7 m - gradient 1:90 (lino – bathroom) Localised variation in levels was observed in **Unit 27**: ▪ 35 mm over the distance of 2.7 m - gradient 1:77 (lino – bathroom) Localised variation in levels was observed in **Unit 29**: ▪ 29 mm over the distance of 2.5 m - gradient 1:86 (lino – bathroom) Localised variation in levels was observed in **Unit 24**: ▪ 21 mm over the distance of 3.6 m - gradient 1:171 (carpet – living room)

6.5. Public Rental

Structural damage	
-	Not observed.
Non-structural damage	
PR-1	Superficial cracking to plasterboard lining throughout.
PR-2	Localised variation in levels was observed in **Unit 45**: ▪ 20 mm over the distance of 4.0 m - gradient 1:200 (variable finish – corridor/living room) Localised variation in levels was observed in **Unit 46**: ▪ 28 mm over the distance of 4.0 m - gradient 1:143 (variable finish – kitchen/living room) Localised variation in levels was observed in **Unit 48**: ▪ 20 mm over the distance of 4.0 m - gradient 1:200 (variable finish – kitchen/living room)

6.6. Resident`s Lounge

Structural damage	
-	Not observed.
Non-structural damage	
RL-1	Glazing to the south-east gable wall has been broken and boarded up (PHOTO 50)

6.7. Block H

Structural damage	
-	Not observed.

Non-structural damage	
H-1	Gap has developed between weatherboards and brick veneer to the rear corner of Garage 1 (PHOTO 58).

6.8. Block I

Structural damage	
-	Not observed.
Non-structural damage	
I-1	Gap has developed between weatherboards and brick veneer to the front corner of Garage 6 (PHOTO 59).
I-2	Number of weatherboards to the front and rear elevation cracked (PHOTOS 60-62).
I-3	Some garage door frames ruptured in corners (PHOTO 63)

6.9. Block J

Structural damage	
-	Not observed.
Non-structural damage	
J-1	Weatherboard to the bottom of the front elevation is crushed and coming off (PHOTO 71)

6.10. Block K

No damage observed.

7. Results and Discussion

7.1. Critical Structural Weaknesses

These buildings have no critical structural weaknesses.

7.2. Analysis Results

The equivalent static force method was used to analyse the demands or loads applied to these buildings. These were then compared to the capacities of the structural elements to assess the seismic capacity of the buildings. The results of the analysis are reported in the following table as %NBS.

Table 4: DEE Results

Building	Seismic Resisting Element	Action	Seismic Rating %NBS
Block A,B,C,D	Longitudinal	First Floor - In Plane Shear	>100%
		Ground Floor - In Plane Shear	44%
	Transverse	First Floor - In Plane Shear	>100%
		Ground Floor - In Plane Shear	>100%
	Floor Diaphragm	Capacity	>100%
	Concrete Wall	Flexural Capacity	>100%
	Timber Wall Studs	Flexural Capacity	>100%
Residents Lounge	Transverse	In Plane Shear	47%
	Longitudinal	In Plane Shear	62%
Block H, I	Transverse	Brace Capacity	22%
	Longitudinal	In Plane Shear	>100%
	Roof Bracing	Axial Capacity	>100%

Building	Seismic Resisting Element	Action	Seismic Rating %NBS
Block J, K	Transverse	Brace Capacity	22%
	Longitudinal	In Plane Shear	>100%
	Timber Wall Studs	Flexural Capacity	>100%
Public Rental	Longitudinal	In Plane Shear	42%
	Transverse	In Plane Shear	>100%
	Timber Wall Studs	Flexural Capacity	51%
	Masonry Wall	Flexural Capacity	83%
	Floor Diaphragm	Capacity	>100%

7.3. Discussion

The buildings at Maurice Courts were built in the late 1980's, therefore it is assumed they were designed prior to NZS 3604:1990, *Timber framed buildings*. The building mass was assessed by normal structural engineering methods with seismic live load in accordance with AS/NZS1170.0:2002 *Structural Design Actions: General Principles* and AS/NZS 1170.1:2002 *Structural Design Actions: Permanent, Imposed and Other Actions*. These were converted to seismic lateral load for each orthogonal direction using the Equivalent Static Procedure defined in NZS1170.5:2004 *Structural Design Actions: Earthquake Actions - New Zealand*.

Blocks H, I, J and K have large openings in the front wall that limits the area available for bracing to be placed in the longitudinal direction to the back wall only.

The residents lounge is a large open area with very few internal walls and large windows and openings in the exterior walls, again this limits the available wall lengths to provide sufficient bracing capacity.

The public rental and blocks A, B, C and D, rely on the concrete party walls in the transverse direction and their connection to the diaphragms which provide sufficient capacity. In the longitudinal direction they rely on out of plane capacity of the concrete walls and on the number and lengths of available timber walls to provide bracing capacity to the building.

8. Conclusions and Recommendations

SKM carried out a quantitative assessment on the buildings at Maurice Carter Courts located at 16 Dundee Place, Spreydon. This assessment concluded that Block H, I, J and K buildings are classified as Earthquake Prone.

The Public Rental, Block A, B, C, D and Residents Lounge are ‘Moderate Risk’ having a capacity between 33% and 67% NBS.

▪ **Table 5: Quantitative assessment summary**

Description	Grade	Risk	%NBS
Block H,I	D	High	22%
Block J,K	D	High	22%
Public Rental	C	Moderate	42%
Block A,B,C,D	C	Moderate	44%
Residents Lounge	C	Moderate	47%

It is recommended that:

a) There is no damage to the buildings that would cause them to be unsafe to occupy.

b) Options to strengthen the buildings to a target of 67% should be investigated.

c) Barriers around the building are not necessary.

9. Limitation Statement

This report has been prepared on behalf of, and for the exclusive use of, SKM's client, and is subject to, and issued in accordance with, the provisions of the contract between SKM and the Client. It is not possible to make a proper assessment of this report without a clear understanding of the terms of engagement under which it has been prepared, including the scope of the instructions and directions given to, and the assumptions made by, SKM. The report may not address issues which would need to be considered for another party if that party's particular circumstances, requirements and experience were known and, further, may make assumptions about matters of which a third party is not aware. No responsibility or liability to any third party is accepted for any loss or damage whatsoever arising out of the use of or reliance on this report by any third party.

Without limiting any of the above, in the event of any liability, SKM's liability, whether under the law of contract, tort, statute, equity or otherwise, is limited in as set out in the terms of the engagement with the Client.

It is not within SKM's scope or responsibility to identify the presence of asbestos, nor the responsibility of SKM to identify possible sources of asbestos. Therefore for any property pre-dating 1989, the presence of asbestos materials should be considered when costing remedial measures or possible demolition.

Should there be any further significant earthquake event, of a magnitude 5 or greater, it will be necessary to conduct a follow-up investigation, as the observations, conclusions and recommendations of this report may no longer apply Earthquake of a lower magnitude may also cause damage, and SKM should be advised immediately if further damage is visible or suspected.

10. Site Inspection Report Photos

PHOTO 1: Block A – Exterior view of the property - front

PHOTO 2: Block A – Exterior view of the property - front

PHOTO 3: Block A – Exterior view of the property - rear

PHOTO 4: Block A – Exterior view of the property - rear

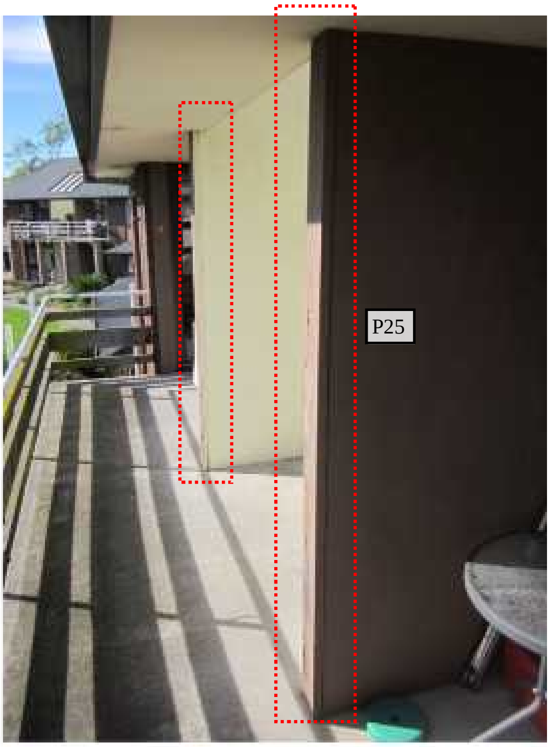
	
PHOTO 5: Block A – View at the concrete stair in the central part of the block - front	PHOTO 6: Block A – View at the balcony at upper floor - front
	
PHOTO 7: Block A – View at the concrete stair in the central part of the block – from half landing	PHOTO 8: Block A – View at the concrete stair in the central part of the block – from rear balcony

	
PHOTO 9: Block A – View at the concrete stair in the central part of the block – from rear balcony	PHOTO 10: Block A – View at the skylight above the central stair

PHOTO 11: Block B – Exterior view of the property - front

PHOTO 12: Block B – Exterior view of the property - front

PHOTO 13: Block B – Exterior view of the property - rear

PHOTO 14: Block B – Exterior view of the property – side wing

PHOTO 15:Detail of Photo 14 – view at concrete foundations (drainage repair works – not EQ related damage)

PHOTO 16: Block C – Exterior view of the property - front

PHOTO 17: Block C – Exterior view of the property – side wing

PHOTO 18: Block C – Exterior view of the property – rear

PHOTO 19: Block D – Exterior view of the property – front

PHOTO 20: Block D – Exterior view of the property – rear

PHOTO 21: Block A - Hinge has developed at the angle support to slab to wall transition.

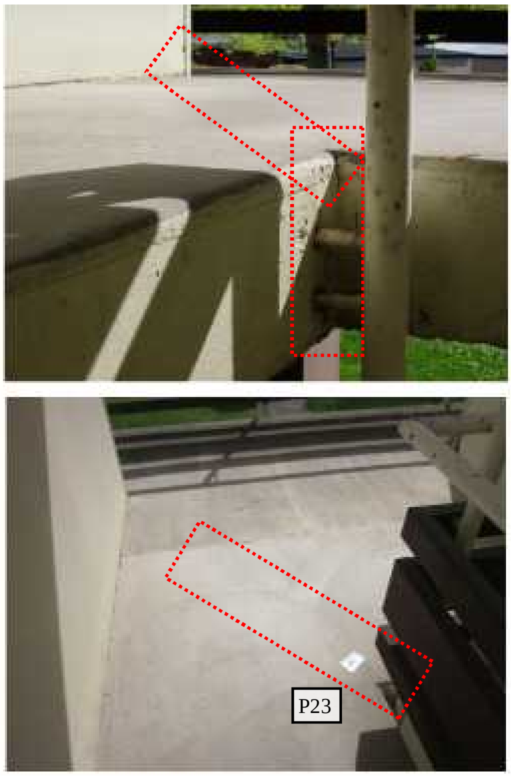


PHOTO 22: Block A – Crack in concrete slab - upper floor landing

PHOTO 23: Detail of Photo 22

PHOTO 24: Block A - Cracking to lining to underside of the roof overhang

PHOTO 25: Block A – Detail of Photo 6. Timber planks separated from the end of the concrete wall.

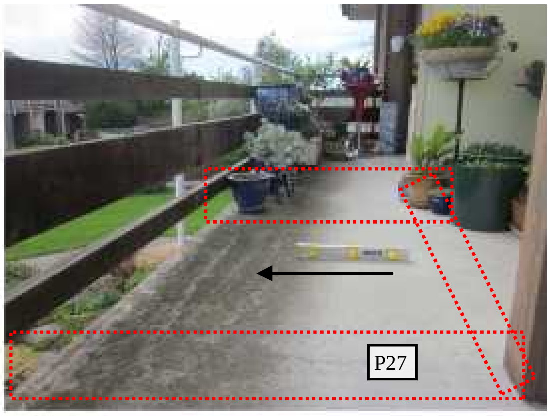


PHOTO 26: Block B - Cracks in concrete slab - upper floor balcony. Balcony sloping down to the left (likely on purpose).

PHOTO 27: Block B – Detail of Photo 26.

PHOTO 28: Block B – Hairline cracking in concrete column to the front elevation.

	
PHOTO 29: Block B – Cracking in ceiling panels to the underside of the skylight	PHOTO 30: Block B –Timber planks separated from the end of the concrete wall.
	
PHOTO 31: Block C - Cracks in concrete slab - upper floor balcony.	PHOTO 32: Block C – Detail of Photo 31.

PHOTO 33: Block C - Brick veneer separated from the concrete wall - rear of the building upstairs

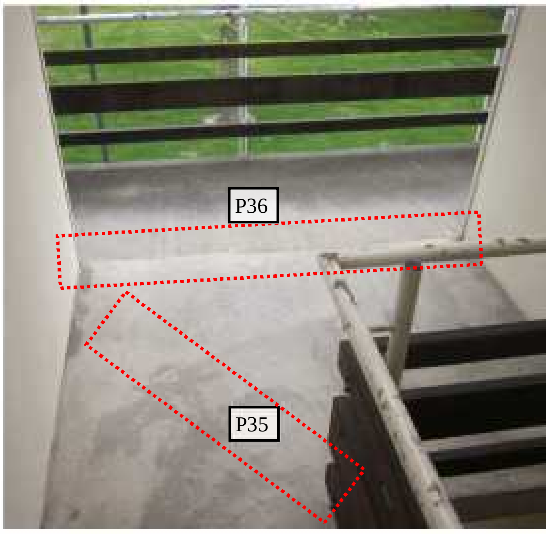


PHOTO 34: Block D - Cracks in concrete slab - upper floor balcony.

PHOTO 35: Block D – Detail of Photo 34.

PHOTO 36: Block D – Detail of Photo 34.

PHOTO 37: Public Rental (to the right) – Exterior north-east view of the property.

Blocks J & K (garages) to the left

PHOTO 38: Public Rental – Exterior south-west view of the property.

PHOTO 39: Public Rental – Exterior north view of the property.

PHOTO 40: Public Rental – Exterior east view of the property.

PHOTO 41: Public Rental – Interior view of the property at ground floor.

PHOTO 42: Public Rental – Interior view of the property at ground floor.

PHOTO 43: Resident`s Lounge – Exterior north view of the property.

PHOTO 44: Resident`s Lounge – Exterior west view of the property.

PHOTO 45: Resident`s Lounge – Exterior south view of the property (gable).

PHOTO 46: Resident`s Lounge – Exterior south view of the property (entrance).

PHOTO 47: Resident`s Lounge – Interior view of the property.

PHOTO 48: Resident`s Lounge – Interior view of the property.

PHOTO 49: Resident`s Lounge – View into roof space.

PHOTO 50: Resident`s Lounge – Broken glazing (south-west gable).

PHOTO 51: Blocks H & I (garages) – Exterior north view of the property.

PHOTO 52: Blocks H & I (garages) – Exterior west view of the property.

PHOTO 53: Blocks H & I (garages) – Exterior south view of the property.

PHOTO 54: Block H – typical view at roof.

PHOTO 55: Block I – View into garage No 7.

PHOTO 56: Block I – View into garage No 7.

PHOTO 57: Block I – View into garage No 7.

PHOTO 58: Block H - Gap has developed between weatherboards and brick veneer to the rear corner of Garage 1

PHOTO 59: Block I - Gap has developed between weatherboards and brick veneer to the front corner of Garage 6

PHOTO 60: Block I – Weatherboard cracked on the front elevation.

PHOTO 61: Block I – Weatherboard ruptured on the rear elevation.

PHOTO 62: Block I – Weatherboard ruptured on the rear elevation.

PHOTO 63: Block I – Garage door frame ruptured in corners.

PHOTO 64: Blocks J & K (garages) – Exterior south-east view of the property.

PHOTO 65: Block J - Exterior south view of the property.

PHOTO 66: Block K - Exterior south view of the property.

	
PHOTO 67: Block J – Interior view of the property.	PHOTO 68: Block J – Interior view of the property (roof trusses and bracing).
	
PHOTO 69: Block J – Interior view of the property (wall bracing – angle brace).	PHOTO 70: Block J – Interior view of the property (wall bracing – angle brace).

PHOTO 71: Block J - Weatherboard to the bottom of the front elevation is crushed and is spalling off

Appendix A   CERA Standardised Report Forms

Location

Building Name:	Maurice Carter Court - Block A,B,C,D	Reviewer:	J Carter
Building Address:	Unit: No: 16 Street: Dundee Place, Maurice Carter Court	CPEng No:	1017618
Legal Description:		Company:	Sinclair Knight Merz
		Company project number:	ZB01276.218
		Company phone number:	03 940 4919
GPS south: (Degrees / Min / Sec)		Date of submission:	17/06/2013
GPS east:		Inspection Date:	15/10/2012
		Revision:	B
Building Unique Identifier (CCC):	PRO 1103-001; PRO 1103-005; PRO 1103-006; PRO 1103-007	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	mixed	Soil Profile (if available):	
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):		If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	

Building

No. of storeys above ground:	2	single storey = 1	Ground floor elevation (Absolute) (m):	
Ground floor split?	no		Ground floor elevation above ground (m):	
Storeys below ground	0			
Foundation type:	other (describe)		if Foundation type is other, describe:	Slab on grade with perimeter footings and deep foundation pads under concrete panels
Building height (m):	6.00		height from ground to level of uppermost seismic mass (for IEP only) (m):	
Floor footprint area (approx):	280			
Age of Building (years):			Date of design:	1976-1992
Strengthening present?	no		If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):	multi-unit residential		Brief strengthening description:	
Use (upper floors):	multi-unit residential			
Use notes (if required):				
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber framed	rafter type, purlin type and cladding	Timber truses @ 1200crs, pulins @ 900crs
Floors:	concrete flat slab	slab thickness (mm)	
Beams:	none	overall depth x width (mm x mm)	
Columns:			
Walls:	load bearing concrete	#N/A	

Lateral load resisting structure

Lateral system along:	lightweight timber framed walls	**Note: Define along and across in detailed report!**	note typical wall length (m)	2
Ductility assumed, μ:	2.00			
Period along:	0.40	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):	5		estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	concrete shear wall		enter wall data in "IEP period calcs" worksheet for period calculation	120mm thick
Ductility assumed, μ:	1.25			
Period across:	0.40	##### enter height above at H31	estimate or calculation?	estimated
Total deflection (ULS) (mm):	5		estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

north (mm):		leave blank if not relevant
east (mm):		
south (mm):		
west (mm):		

Non-structural elements

Stairs:	precast, full flight	describe supports	
Wall cladding:	brick or tile	describe (note cavity if exists)	70mm clay brick on 40mm cavity on ground floor, weatherboard on first floor
Roof Cladding:	Metal	describe	Lightweight roofing iron
Glazing:	timber frames		
Ceilings:	plaster, fixed		
Services(list):			

Available documentation

Architectural	none	original designer name/date	
Structural	none	original designer name/date	
Mechanical	none	original designer name/date	
Electrical	none	original designer name/date	
Geotech report	none	original designer name/date	

Damage

Site: (refer DEE Table 4-2)

Site performance:		Describe damage:	
Settlement:	none observed	notes (if applicable):	
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	none apparent	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	none apparent	notes (if applicable):	

Building:

	Current Placard Status:	green		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	
	Describe (summary):	refer to report for full outline		
Across	Damage ratio:	0%		
	Describe (summary):	refer to report for full outline		
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	yes	Describe:	

$$Damage\_Ratio = \frac{(\%NBS\,(before) - \%NBS\,(after))}{\%NBS\,(before)}$$

Recommendations

	Level of repair/strengthening required:	minor structural	Describe:	Relining of walls, longitudinally
	Building Consent required:	yes	Describe:	
	Interim occupancy recommendations:	full occupancy	Describe:	
Along	Assessed %NBS before e'quakes:	44% ##### %NBS from IEP below	If IEP not used, please detail assessment methodology:	SKM calculations
	Assessed %NBS after e'quakes:	44%		
Across	Assessed %NBS before e'quakes:	100% ##### %NBS from IEP below		
	Assessed %NBS after e'quakes:	100%		

Detailed Engineering Evaluation Summary Data V1.14

Location

Building Name:	Maurice Carter Court - Block H,I	Reviewer:	J Carter
	Unit / No: 16 / Street: Dundee Place	CPEng No:	1017618
Building Address:	Maurice Carter Court	Company:	Sinclair Knight Merz
Legal Description:		Company project number:	ZB01276.218
		Company phone number:	03 940 4919
	Degrees / Min / Sec		
GPS south:		Date of submission:	17/06/2013
GPS east:		Inspection Date:	15/10/2012
		Revision:	B
Building Unique Identifier (CCC):	PRO 1103-008; PRO 1103-009	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	mixed	Soil Profile (if available):	
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):		If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	

Building

No. of storeys above ground:	1	single storey = 1 / Ground floor elevation (Absolute) (m):	
Ground floor split?	no	Ground floor elevation above ground (m):	
Storeys below ground	0		
Foundation type:	strip footings	if Foundation type is other, describe:	Slab on grade with perimeter footings
Building height (m):	2.50	height from ground to level of uppermost seismic mass (for IEP only) (m):	
Floor footprint area (approx):	100		
Age of Building (years):		Date of design:	1976-1992
Strengthening present?	no	If so, when (year)?	
		And what load level (%g)?	
Use (ground floor):	parking	Brief strengthening description:	
Use (upper floors):			
Use notes (if required):			
Importance level (to NZS1170.5):	IL2		

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber framed	rafter type, purlin type and cladding	150 x 50 Rafters @ 800crs
Floors:	concrete flat slab	slab thickness (mm)	
Beams:	none	overall depth x width (mm x mm)	
Columns:			
Walls:			

Lateral load resisting structure

Lateral system along:	lightweight timber framed walls	**Note: Define along and across in detailed report!**	note typical wall length (m)	
Ductility assumed, μ:	2.00			
Period along:	0.40	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):	5		estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	lightweight timber framed walls		note typical wall length (m)	
Ductility assumed, μ:	2.00			
Period across:	0.40	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):	5		estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

north (mm):		leave blank if not relevant
east (mm):		
south (mm):		
west (mm):		

Non-structural elements

Stairs:			
Wall cladding:	brick or tile	describe (note cavity if exists)	70mm clay brick on 40mm cavity
Roof Cladding:	Metal	describe	Lightweight roofing iron
Glazing:	timber frames		
Ceilings:			No ceiling
Services(list):			

Available documentation

Architectural	none	original designer name/date	
Structural	none	original designer name/date	
Mechanical	none	original designer name/date	
Electrical	none	original designer name/date	
Geotech report	none	original designer name/date	

Damage

Site: (refer DEE Table 4-2)

Site performance:		Describe damage:	
Settlement:	none observed	notes (if applicable):	
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	none apparent	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	none apparent	notes (if applicable):	

Building:

	Current Placard Status:	green		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	
	Describe (summary):	refer to report for full outline		
Across	Damage ratio:	0%		
	Describe (summary):	refer to report for full outline		
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	no	Describe:	

$$Damage\_Ratio = \frac{(\%NBS\,(before) - \%NBS\,(after))}{\%NBS\,(before)}$$

Recommendations

	Level of repair/strengthening required:	minor structural	Describe:	bracing of walls, longitudinally and transversely
	Building Consent required:	yes	Describe:	
	Interim occupancy recommendations:	full occupancy	Describe:	
Along	Assessed %NBS before e'quakes:	47%	##### %NBS from IEP below / If IEP not used, please detail assessment methodology:	SKM calculations
	Assessed %NBS after e'quakes:	47%		
Across	Assessed %NBS before e'quakes:	62%	##### %NBS from IEP below	
	Assessed %NBS after e'quakes:	62%		

Detailed Engineering Evaluation Summary Data V1.14

Location

Building Name:	Maurice Carter Court - Block J,K	Reviewer:	J Carter
Building Address:	Unit: / No: 16 / Street: Dundee Place; Maurice Carter Court	CPEng No:	1017618
Legal Description:		Company:	Sinclair Knight Merz
		Company project number:	ZB01276.218
		Company phone number:	03 940 4919
GPS south:	Degrees / Min / Sec	Date of submission:	17/06/2013
GPS east:		Inspection Date:	15/10/2012
		Revision:	B
Building Unique Identifier (CCC):	PRO 1103-010; PRO 1103-011	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	mixed	Soil Profile (if available):	
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):		If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	

Building

No. of storeys above ground:	1	single storey = 1 Ground floor elevation (Absolute) (m):	
Ground floor split?	no	Ground floor elevation above ground (m):	
Storeys below ground	0		
Foundation type:	strip footings	if Foundation type is other, describe:	Slab on grade with perimeter footings
Building height (m):	2.50	height from ground to level of uppermost seismic mass (for IEP only) (m):	
Floor footprint area (approx):	40		
Age of Building (years):		Date of design:	1976-1992
Strengthening present?	no	If so, when (year)?	
		And what load level (%g)?	
Use (ground floor):	parking	Brief strengthening description:	
Use (upper floors):			
Use notes (if required):			
Importance level (to NZS1170.5):	IL2		

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber framed	rafter type, purlin type and cladding	
Floors:	concrete flat slab	slab thickness (mm)	
Beams:	none	overall depth x width (mm x mm)	
Columns:			
Walls:			

Lateral load resisting structure

Note: Define along and across in detailed report!

Lateral system along:	lightweight timber framed walls		note typical wall length (m)	
Ductility assumed, μ:	2.00			
Period along:	0.40	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):	5		estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	lightweight timber framed walls		note typical wall length (m)	
Ductility assumed, μ:	2.00			
Period across:	0.40	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):	5		estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

north (mm):		leave blank if not relevant
east (mm):		
south (mm):		
west (mm):		

Non-structural elements

Stairs:			
Wall cladding:	brick or tile	describe (note cavity if exists)	70mm clay brick on 40mm cavity and weatherboard
Roof Cladding:	Metal	describe	Lightweight roofing iron
Glazing:	timber frames		
Ceilings:			No ceiling
Services(list):			

Available documentation

Architectural	partial	original designer name/date	Christchurch City Council
Structural	partial	original designer name/date	Christchurch City Council
Mechanical	none	original designer name/date	
Electrical	none	original designer name/date	
Geotech report	none	original designer name/date	

Damage

Site: (refer DEE Table 4-2)

Site performance:		Describe damage:	
Settlement:	none observed	notes (if applicable):	
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	none apparent	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	none apparent	notes (if applicable):	

Building:

	Current Placard Status:	green		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	
	Describe (summary):	refer to report for full outline		
Across	Damage ratio:	0%		
	Describe (summary):	refer to report for full outline		
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	no	Describe:	

$$Damage\_Ratio = \frac{(\%NBS\,(before) - \%NBS\,(after))}{\%NBS\,(before)}$$

Recommendations

	Level of repair/strengthening required:	minor structural	Describe:	bracing of walls transversely
	Building Consent required:	yes	Describe:	
	Interim occupancy recommendations:	full occupancy	Describe:	
Along	Assessed %NBS before e'quakes:	22% ##### %NBS from IEP below	If IEP not used, please detail assessment methodology:	SKM calculations
	Assessed %NBS after e'quakes:	22%		
Across	Assessed %NBS before e'quakes:	100% ##### %NBS from IEP below		
	Assessed %NBS after e'quakes:	100%		

Detailed Engineering Evaluation Summary Data V1.14

Location

Building Name:	Maurice Carter Court - Public Rental (PR)	Reviewer:	J Carter
	Unit / No: 16 / Street	CPEng No:	1017618
Building Address:	Maurice Carter Court / 16 / Dundee Place	Company:	Sinclair Knight Merz
Legal Description:		Company project number:	ZB01276.218
		Company phone number:	03 940 4919
	Degrees / Min / Sec		
GPS south:		Date of submission:	17/06/2013
GPS east:		Inspection Date:	15/10/2012
		Revision:	B
Building Unique Identifier (CCC):	PRO 1103-003	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	mixed	Soil Profile (if available):	
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):		If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	

Building

No. of storeys above ground:	2 single storey = 1	Ground floor elevation (Absolute) (m):	
Ground floor split?	no	Ground floor elevation above ground (m):	
Storeys below ground	0		
Foundation type:	other (describe)	if Foundation type is other, describe:	Slab on grade with perimeter footings and deep foundation pads under concrete panels
Building height (m):	6.00	height from ground to level of uppermost seismic mass (for IEP only) (m):	
Floor footprint area (approx):	160		
Age of Building (years):		Date of design:	1976-1992
Strengthening present?	no	If so, when (year)?	
		And what load level (%g)?	
Use (ground floor):	multi-unit residential	Brief strengthening description:	
Use (upper floors):	multi-unit residential		
Use notes (if required):			
Importance level (to NZS1170.5):	IL2		

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber framed	rafter type, purlin type and cladding	
Floors:	concrete flat slab	slab thickness (mm)	
Beams:	none	overall depth x width (mm x mm)	
Columns:			
Walls:	load bearing concrete	#N/A	

Lateral load resisting structure

Lateral system along:	lightweight timber framed walls	Note: Define along and across in detailed report! note typical wall length (m)	2
Ductility assumed, μ:	2.00		
Period along:	0.40 0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):	5	estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):		estimate or calculation?	
Lateral system across:	fully filled CMU	note total length of wall at ground (m):	7.5m
Ductility assumed, μ:	1.25		
Period across:	0.40 ##### enter height above at H31	estimate or calculation?	estimated
Total deflection (ULS) (mm):	5	estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):		estimate or calculation?	

Separations:

north (mm):		leave blank if not relevant
east (mm):		
south (mm):		
west (mm):		

Non-structural elements

Stairs:	precast, full flight	describe supports	300 x 50 Timber Stringers
Wall cladding:	brick or tile	describe (note cavity if exists)	90mm brickwork and weatherboard
Roof Cladding:	Metal	describe	Lightweight roofing iron
Glazing:	timber frames		
Ceilings:	plaster, fixed		
Services(list):			

Available documentation

Architectural	partial	original designer name/date	Christchurch City Council
Structural	partial	original designer name/date	Christchurch City Council
Mechanical	none	original designer name/date	
Electrical	none	original designer name/date	
Geotech report	none	original designer name/date	

Damage

Site:
(refer DEE Table 4-2)

Site performance:		Describe damage:	
Settlement:	none observed	notes (if applicable):	
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	none apparent	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	none apparent	notes (if applicable):	

Building:

	Current Placard Status:	green		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	
	Describe (summary):	refer to report for full outline		
Across	Damage ratio:	0%		
	Describe (summary):	refer to report for full outline		
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	yes	Describe:	

$$Damage\_Ratio = \frac{(\%NBS\,(before) - \%NBS\,(after))}{\%NBS\,(before)}$$

Recommendations

Level of repair/strengthening required:	minor structural	Describe:	Relining of walls, longitudinally
Building Consent required:	yes	Describe:	
Interim occupancy recommendations:	full occupancy	Describe:	

Along	Assessed %NBS before e'quakes:	42% ##### %NBS from IEP below	If IEP not used, please detail assessment methodology:	SKM calculations
	Assessed %NBS after e'quakes:	42%		
Across	Assessed %NBS before e'quakes:	100% ##### %NBS from IEP below		
	Assessed %NBS after e'quakes:	100%		

Detailed Engineering Evaluation Summary Data V1.14

Location

Building Name:	Maurice Carter Court - Residents Lounge (RL)
Building Address:	Unit: — No: 16 Street: Dundee Place; Maurice Carter Court
Legal Description:	
GPS south: (Degrees / Min / Sec)	
GPS east:	
Building Unique Identifier (CCC):	PRO 1103-004
Reviewer:	J Carter
CPEng No:	1017618
Company:	Sinclair Knight Merz
Company project number:	ZB01276.218
Company phone number:	03 940 4919
Date of submission:	17/06/2013
Inspection Date:	15/10/2012
Revision:	B
Is there a full report with this summary?	yes

Site

Site slope:	flat
Soil type:	mixed
Site Class (to NZS1170.5):	D
Proximity to waterway (m, if <100m):	
Proximity to clifftop (m, if < 100m):	
Proximity to cliff base (m,if <100m):	
Max retaining height (m):	
Soil Profile (if available):	
If Ground improvement on site, describe:	
Approx site elevation (m):	

Building

No. of storeys above ground:	1 (single storey = 1)
Ground floor split?	no
Storeys below ground	0
Foundation type:	other (describe)
Building height (m):	5.00
Floor footprint area (approx):	140
Age of Building (years):	
Ground floor elevation (Absolute) (m):	
Ground floor elevation above ground (m):	
if Foundation type is other, describe:	Slab on grade with perimeter footings
height from ground to level of uppermost seismic mass (for IEP only) (m):	
Date of design:	1976-1992
Strengthening present?	no
If so, when (year)?	
And what load level (%g)?	
Brief strengthening description:	
Use (ground floor):	public
Use (upper floors):	public
Use notes (if required):	
Importance level (to NZS1170.5):	IL2

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber framed	rafter type, purlin type and cladding	Trusses @ 1200crs
Floors:	concrete flat slab	slab thickness (mm)	
Beams:	none	overall depth x width (mm x mm)	
Columns:			
Walls:	load bearing concrete	#N/A	

Lateral load resisting structure

Note: Define along and across in detailed report!

Lateral system along:	lightweight timber framed walls		note typical wall length (m)	
Ductility assumed, μ:	2.00			
Period along:	0.40	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):	5		estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	lightweight timber framed walls		note typical wall length (m)	
Ductility assumed, μ:	2.00			
Period across:	0.40	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):	5		estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

leave blank if not relevant

north (mm):	
east (mm):	
south (mm):	
west (mm):	

Non-structural elements

Stairs:			
Wall cladding:	brick or tile	describe (note cavity if exists)	90mm brickwork and weatherboard
Roof Cladding:	Metal	describe	Lightweight roofing iron
Glazing:	timber frames		
Ceilings:	plaster, fixed		
Services(list):			

Available documentation

Architectural	none	original designer name/date	
Structural	none	original designer name/date	
Mechanical	none	original designer name/date	
Electrical	none	original designer name/date	
Geotech report	none	original designer name/date	

Damage

Site: (refer DEE Table 4-2)

Site performance:		Describe damage:	
Settlement:	none observed	notes (if applicable):	
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	none apparent	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	none apparent	notes (if applicable):	

Building:

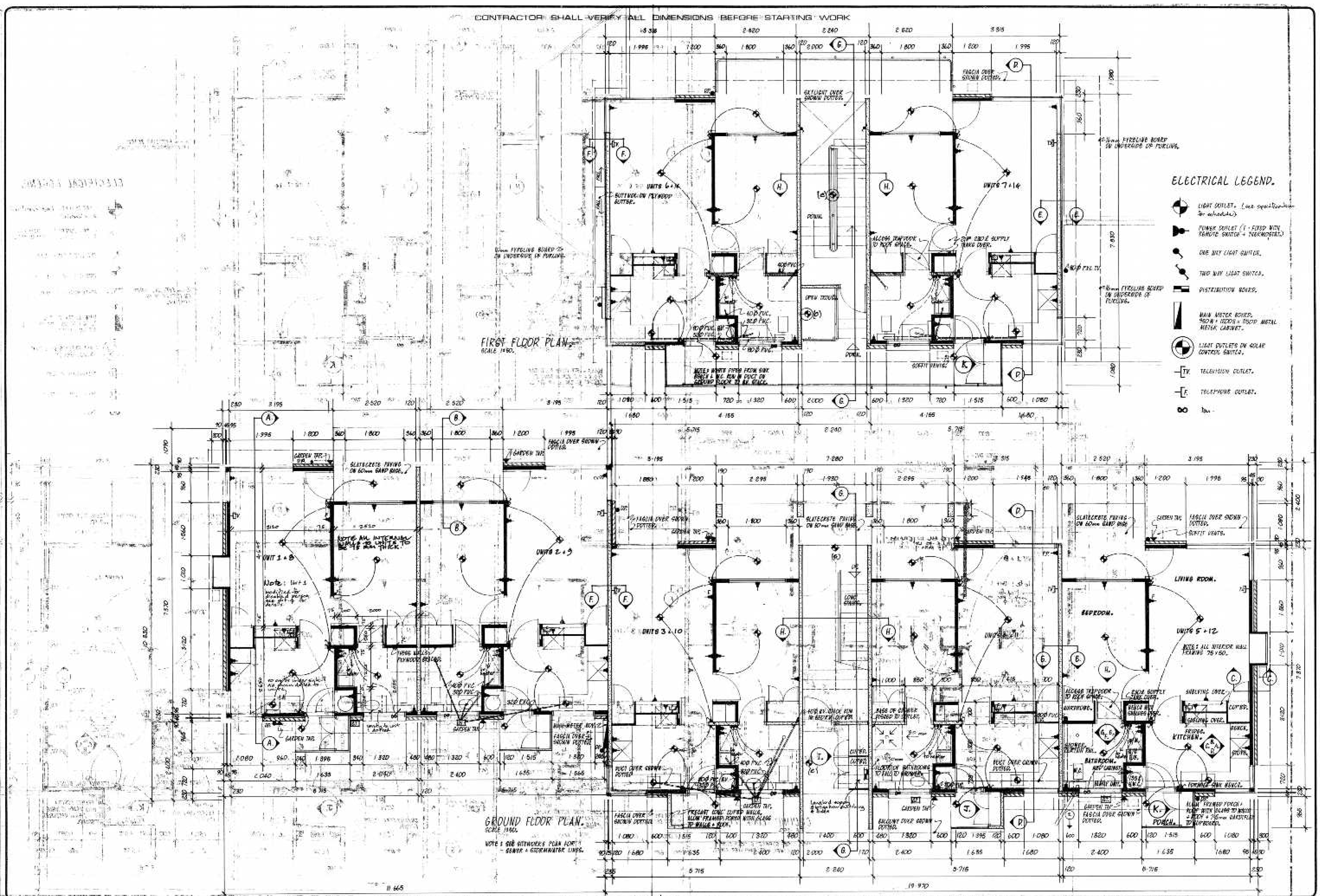
	Current Placard Status:	green		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	
	Describe (summary):	refer to report for full outline		
Across	Damage ratio:	0%		
	Describe (summary):	refer to report for full outline		
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	yes	Describe:	

$$Damage\_Ratio = \frac{(\%NBS\,(before) - \%NBS\,(after))}{\%NBS\,(before)}$$

Recommendations

	Level of repair/strengthening required:	minor structural	Describe:	Relining of walls, longitudinally
	Building Consent required:	yes	Describe:	
	Interim occupancy recommendations:	full occupancy	Describe:	
Along	Assessed %NBS before e'quakes:	47%	##### %NBS from IEP below	If IEP not used, please detail assessment methodology: SKM calculations
	Assessed %NBS after e'quakes:	47%		
Across	Assessed %NBS before e'quakes:	62%	##### %NBS from IEP below	
	Assessed %NBS after e'quakes:	62%		

Appendix B Original drawings (by CCC in 1989)



CHRISTCHURCH CITY COUNCIL

CITY ARCHITECTS DIVISION · CITY WORKS & PLANNING DEPARTMENT

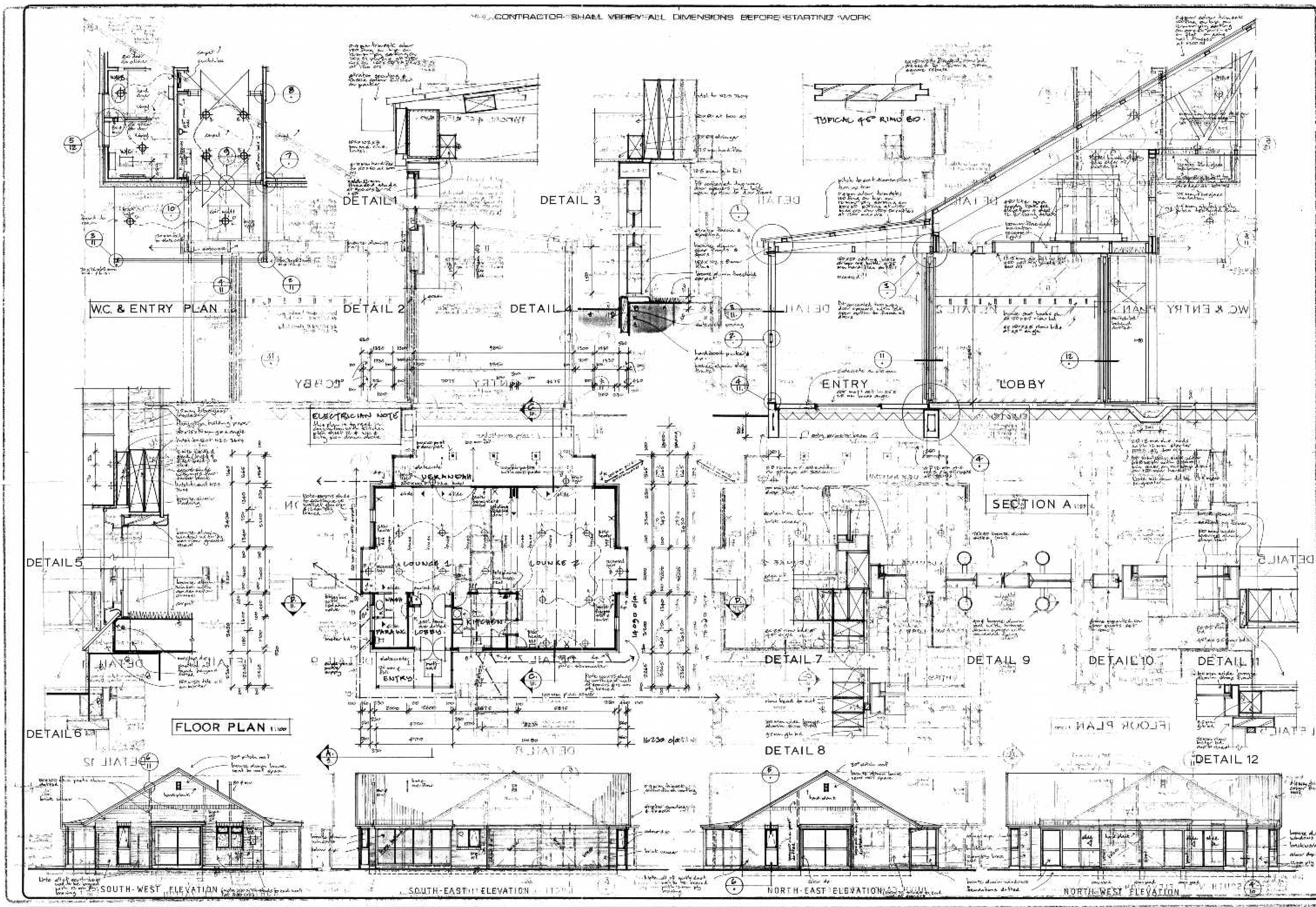
DUNDEE PLACE HOUSING DEVELOPMENT — STAGE TWO · GROUND & FIRST FLOOR PLAN

SCALE 1:50 DATE

DESIGN DRAWN TRACED

JOB NO. FILE NO.

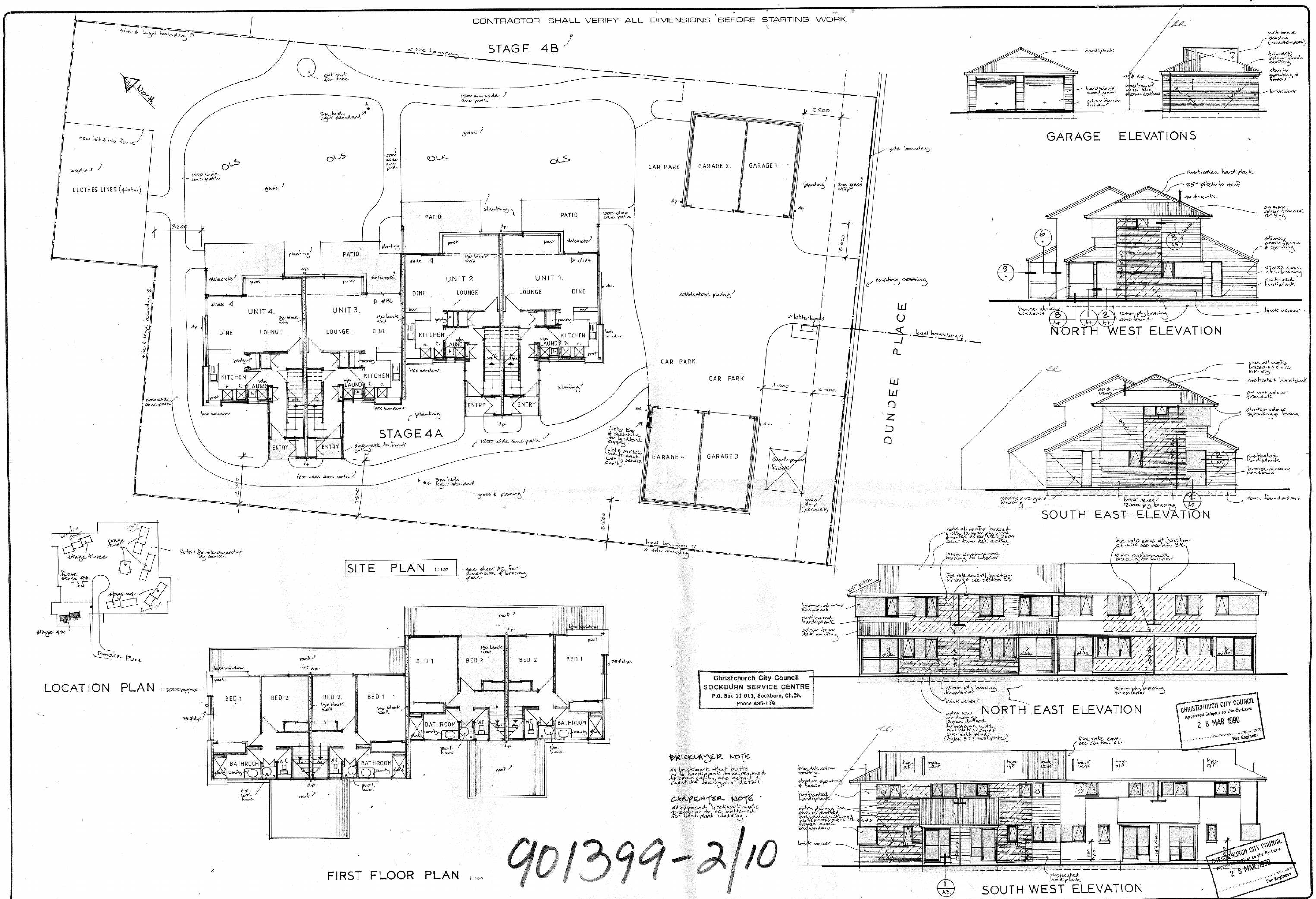
SHEET NUMBER 2



CHRISTCHURCH CITY COUNCIL
CITY ARCHITECTS DIVISION CITY WORKS & PLANNING DEPARTMENT

DUNDEE PLACE HOUSING DEVELOPMENT STAGE 2 RESIDENTS LOUNGE

SCALE DESIGN DRAWN TRACED DATE JOB No. FILE No. SHEET NUMBER OF SHEETS 9

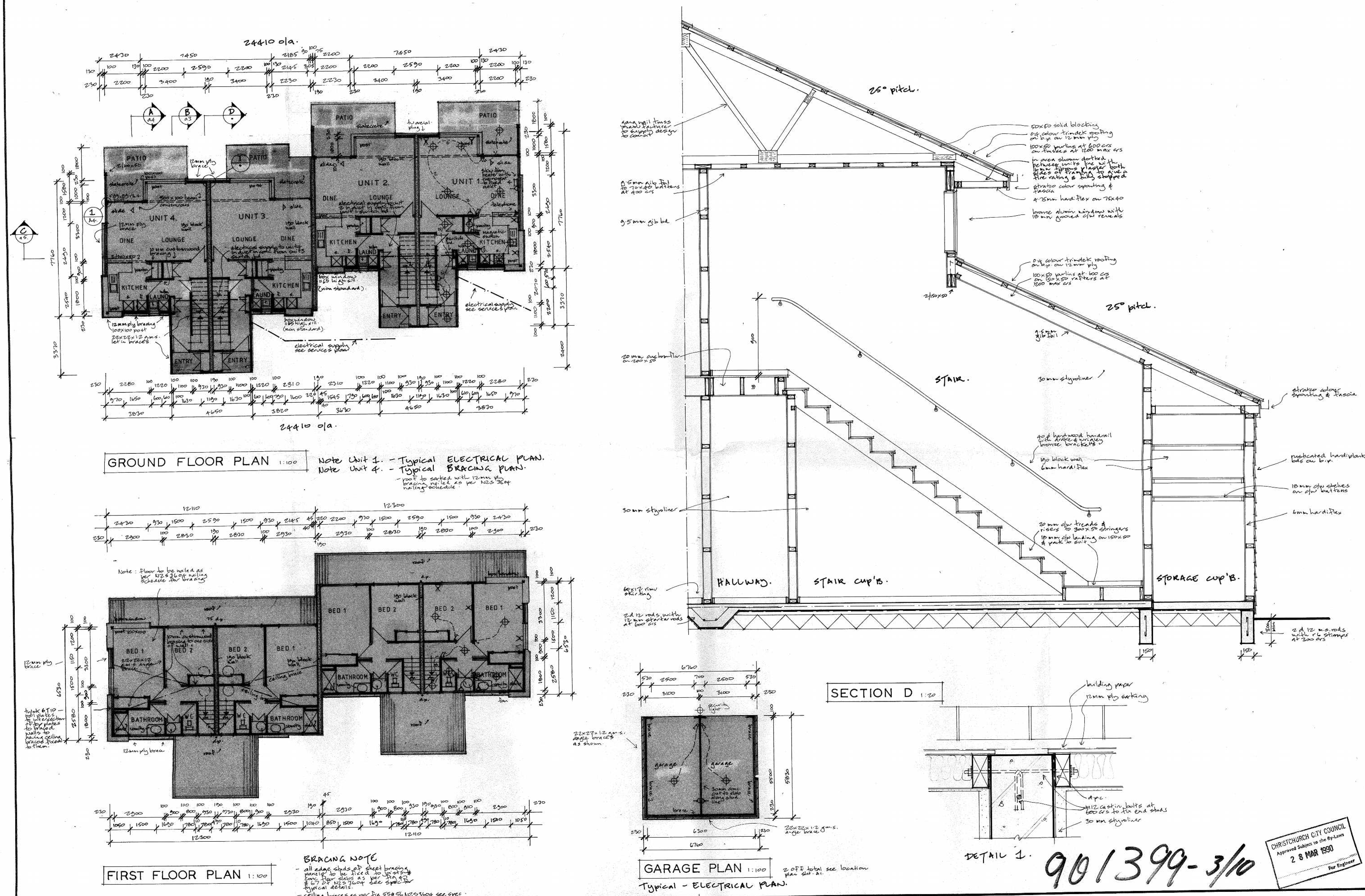


CHRISTCHURCH CITY COUNCIL
CITY ARCHITECTS DIVISION : CITY WORKS & PLANNING DEPARTMENT

DUNDEE PLACE HOUSING DEVELOPMENT · STAGE 4 4 PUBLIC RENTAL UNITS

SCALE 1:100 DESIGN DRAWN TRACED JOB NO SHEET NUMBER A1
DATE May 1989 FILE NO OF SHEETS

CONTRACTOR SHALL VERIFY ALL DIMENSIONS BEFORE STARTING WORK

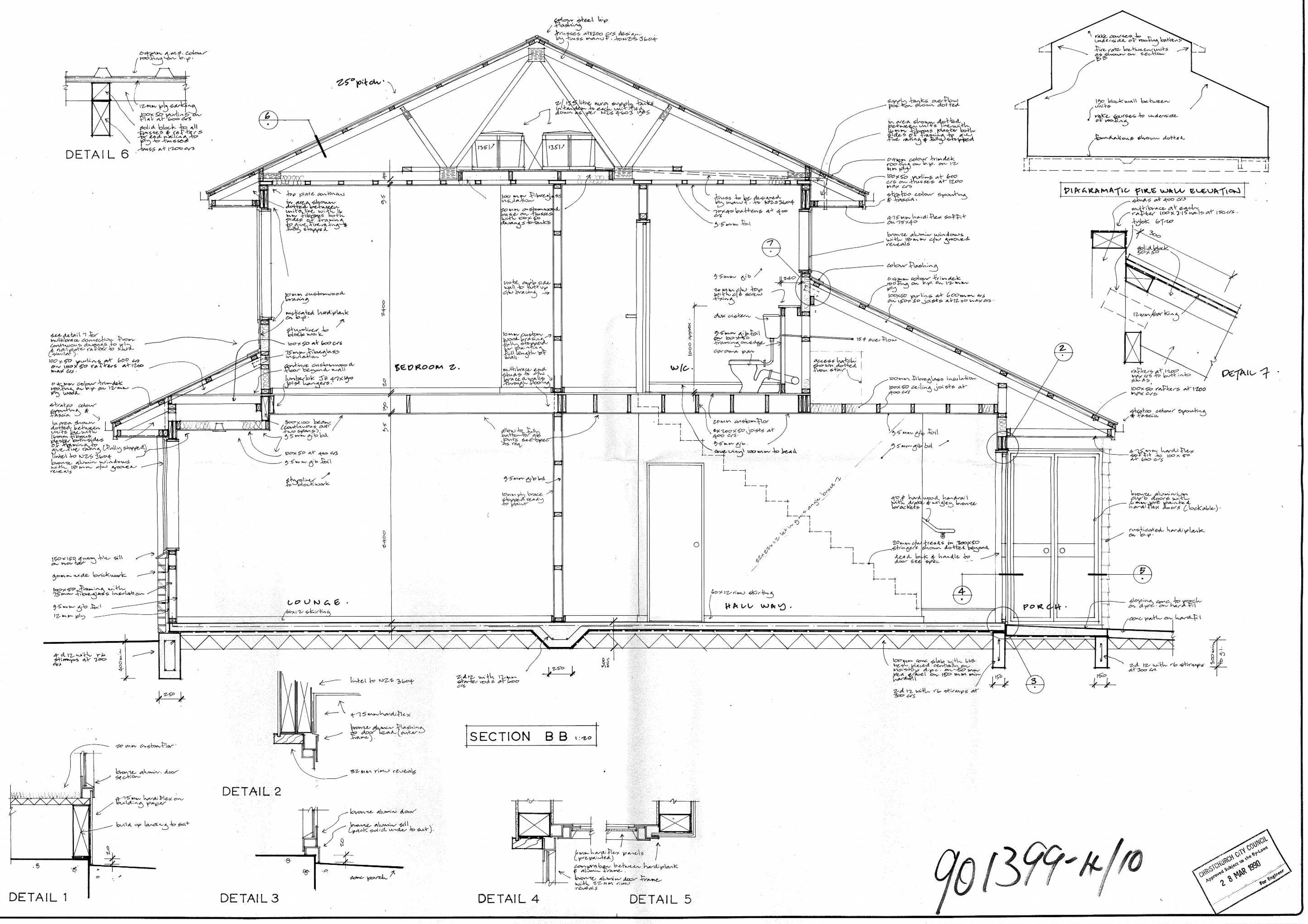


CHRISTCHURCH CITY COUNCIL
CITY ARCHITECTS DIVISION : CITY WORKS & PLANNING DEPARTMENT

DUNDEE PLACE HOUSING DEVELOPMENT STAGE 4 — PLANS & SECTION

SCALE 1:20, 100 DESIGN DRAWN TRACED DATE 15/10/89. JOB NO FILE NO SHEET NUMBER A2 OF 8 SHEETS

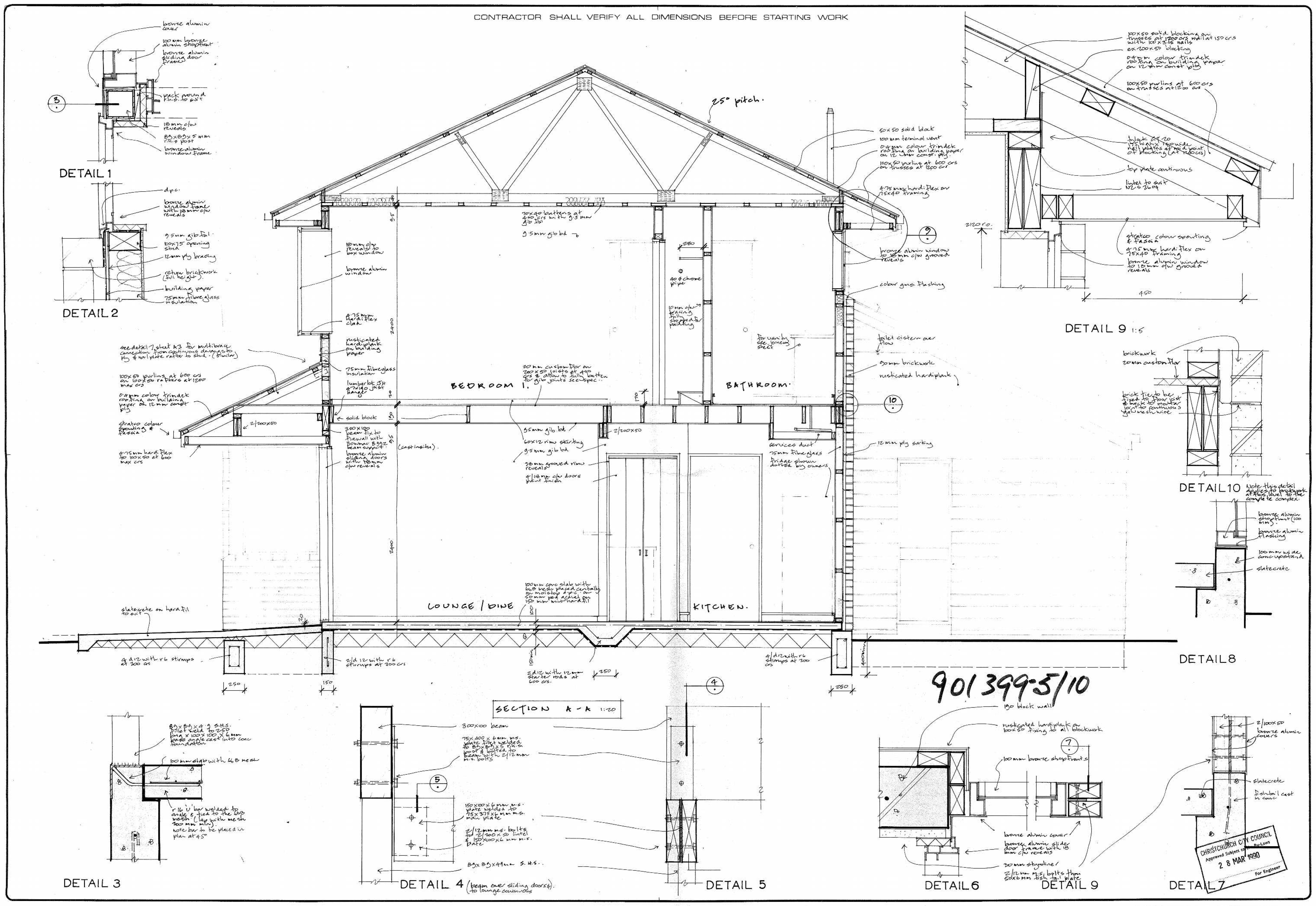
CONTRACTOR SHALL VERIFY ALL DIMENSIONS BEFORE STARTING WORK



CHRISTCHURCH CITY COUNCIL
CITY ARCHITECTS DIVISION · CITY WORKS & PLANNING DEPARTMENT

DUNDEE PLACE HOUSING DEVELOPMENT · STAGE 4 - CROSS SECTION & DETAILS

SCALE 1:20 &	DESIGN	JOB NO	SHEET NUMBER A3
DATE	DRAWN TRACED	FILE NO	OF 8 SHEETS

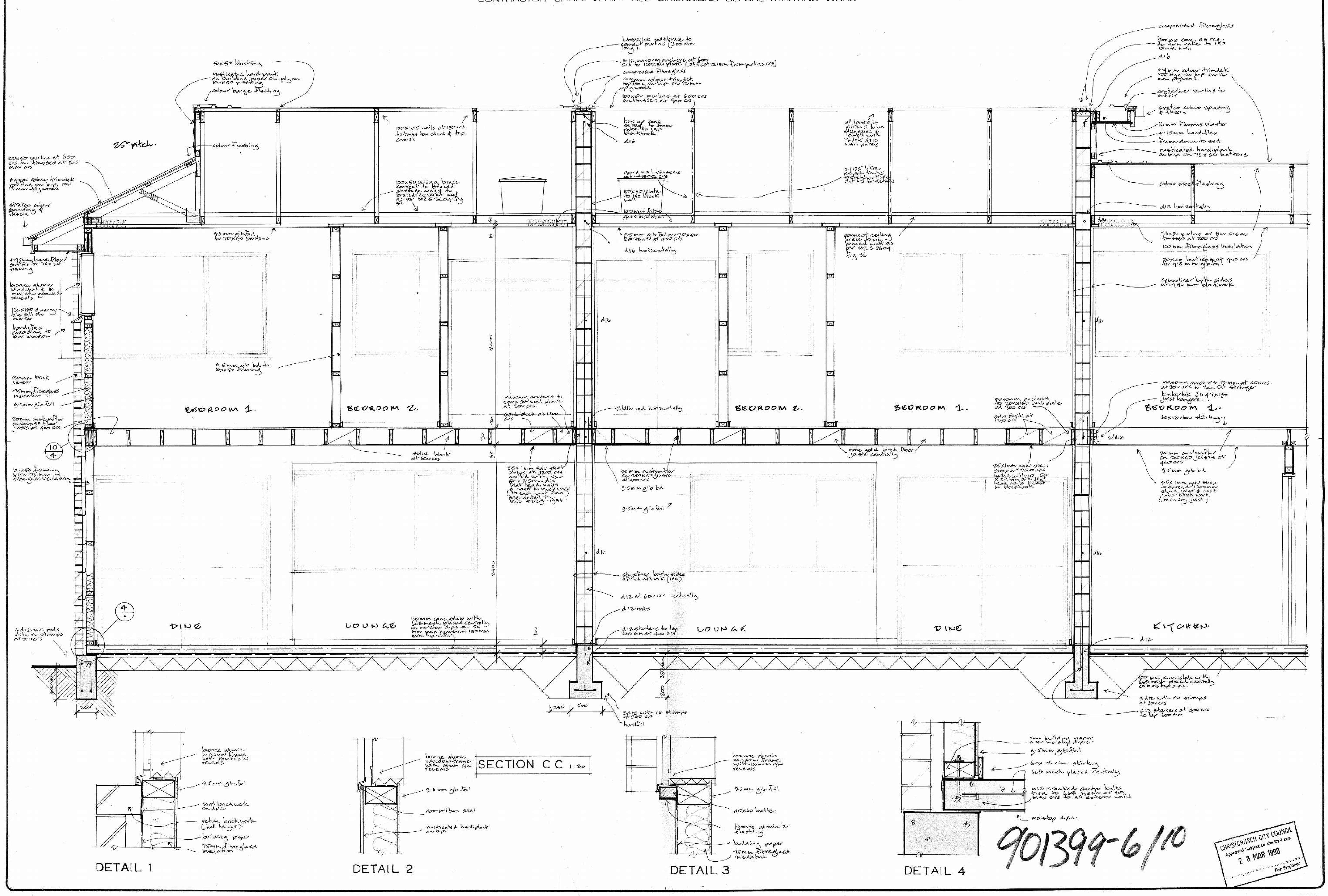


CHRISTCHURCH CITY COUNCIL
CITY ARCHITECTS DIVISION : CITY WORKS & PLANNING DEPARTMENT

DUNDEE PLACE HOUSING DEVELOPMENT · STAGE 4 - CROSS SECTION & DETAILS

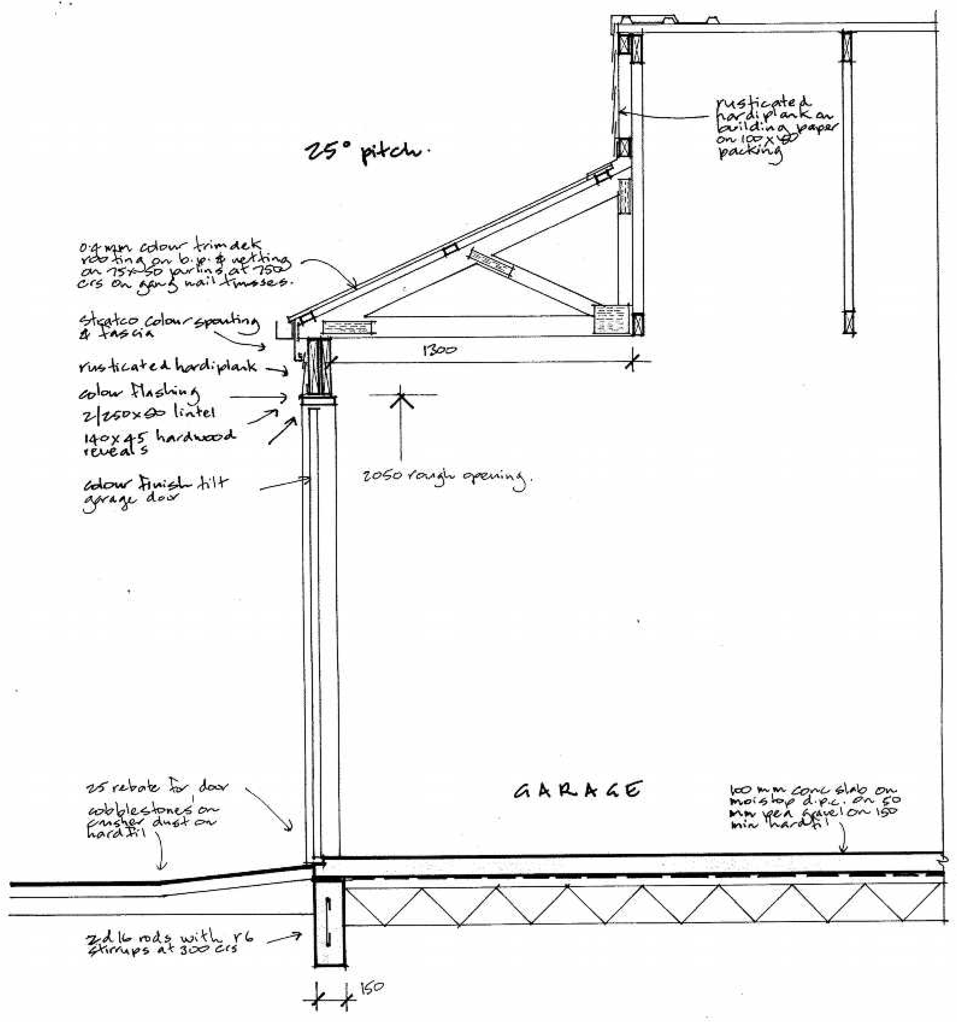
SCALE 1:20, 5 | DESIGN | JOB NO | SHEET NUMBER A4
DATE 9.11.89 | DRAWN | TRACED | FILE NO | OF 8 SHEETS

CONTRACTOR SHALL VERIFY ALL DIMENSIONS BEFORE STARTING WORK

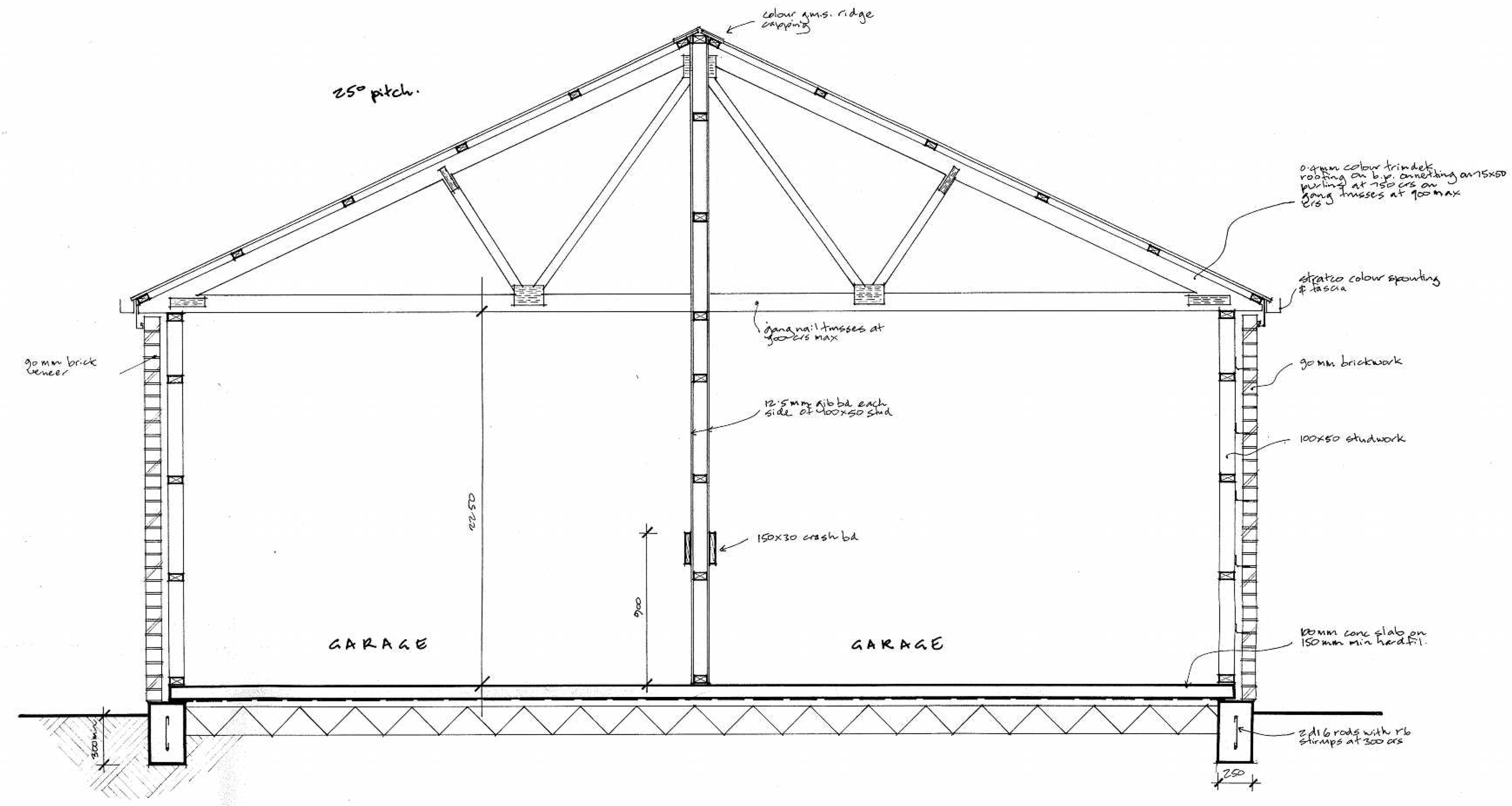


CHRISTCHURCH CITY COUNCIL CITY ARCHITECTS DIVISION : CITY WORKS & PLANNING DEPARTMENT	DUNDEE PLACE HOUSING DEVELOPMENT · STAGE 4 · - CROSS SECTION & DETAILS	SCALE 1:20 5 DATE 9 11 89	DESIGN G.F.H. DRAWN TRACED	JOB NO 502/54 FILE NO	SHEET NUMBER A5 OF 8 SHEETS

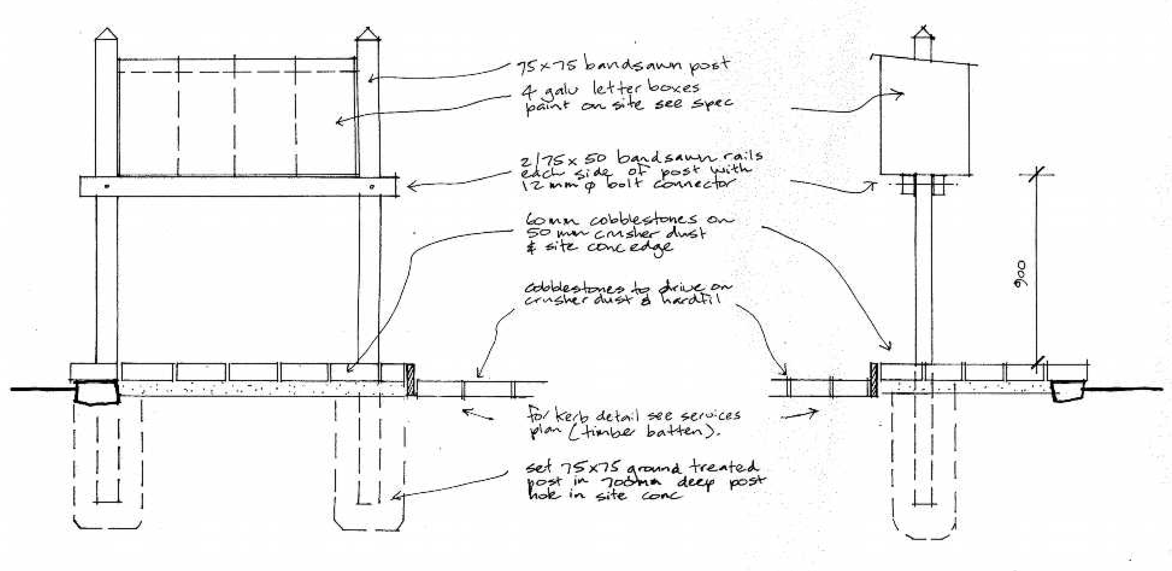
CONTRACTOR SHALL VERIFY ALL DIMENSIONS BEFORE STARTING WORK



TYPICAL GARAGE DOOR SECTION 1:20



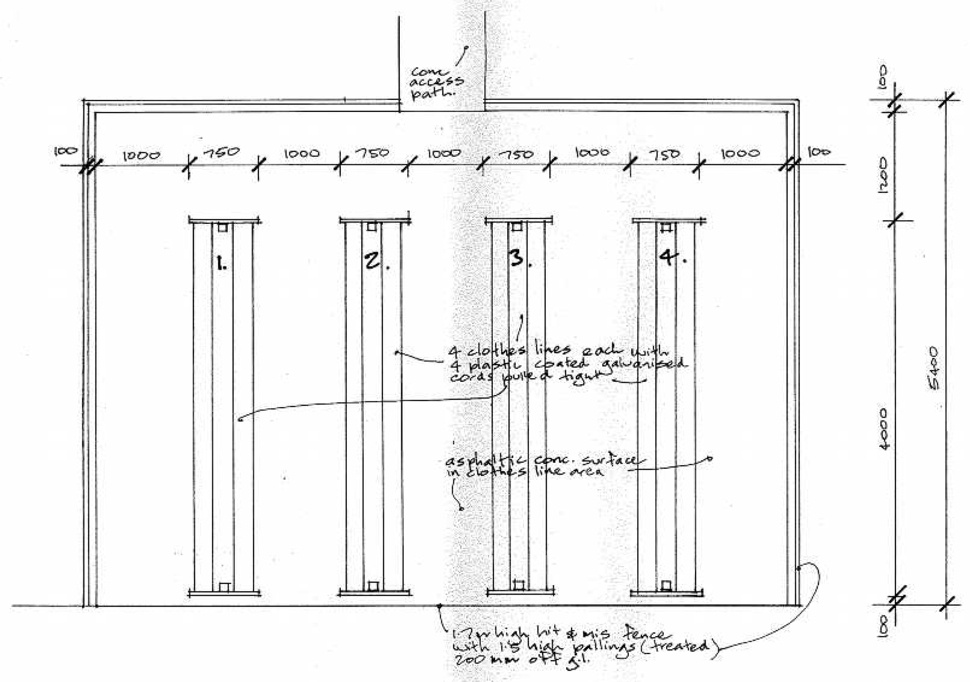
TYPICAL GARAGE CROSS SECTION 1:20



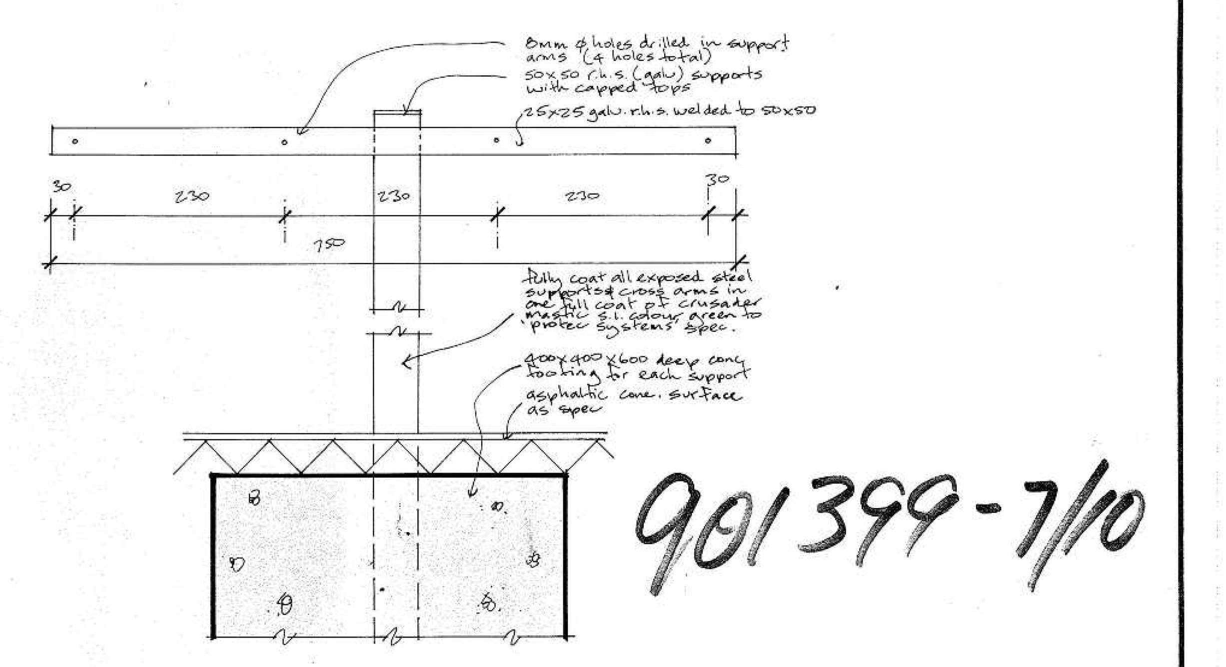
LETTER BOXES

REAR ELEVATION

SIDE ELEVATION



PLAN OF CLOTHES LINE AREA



FOOT & STEEL SUPPORT DETAILS TO CLOTHES LINES

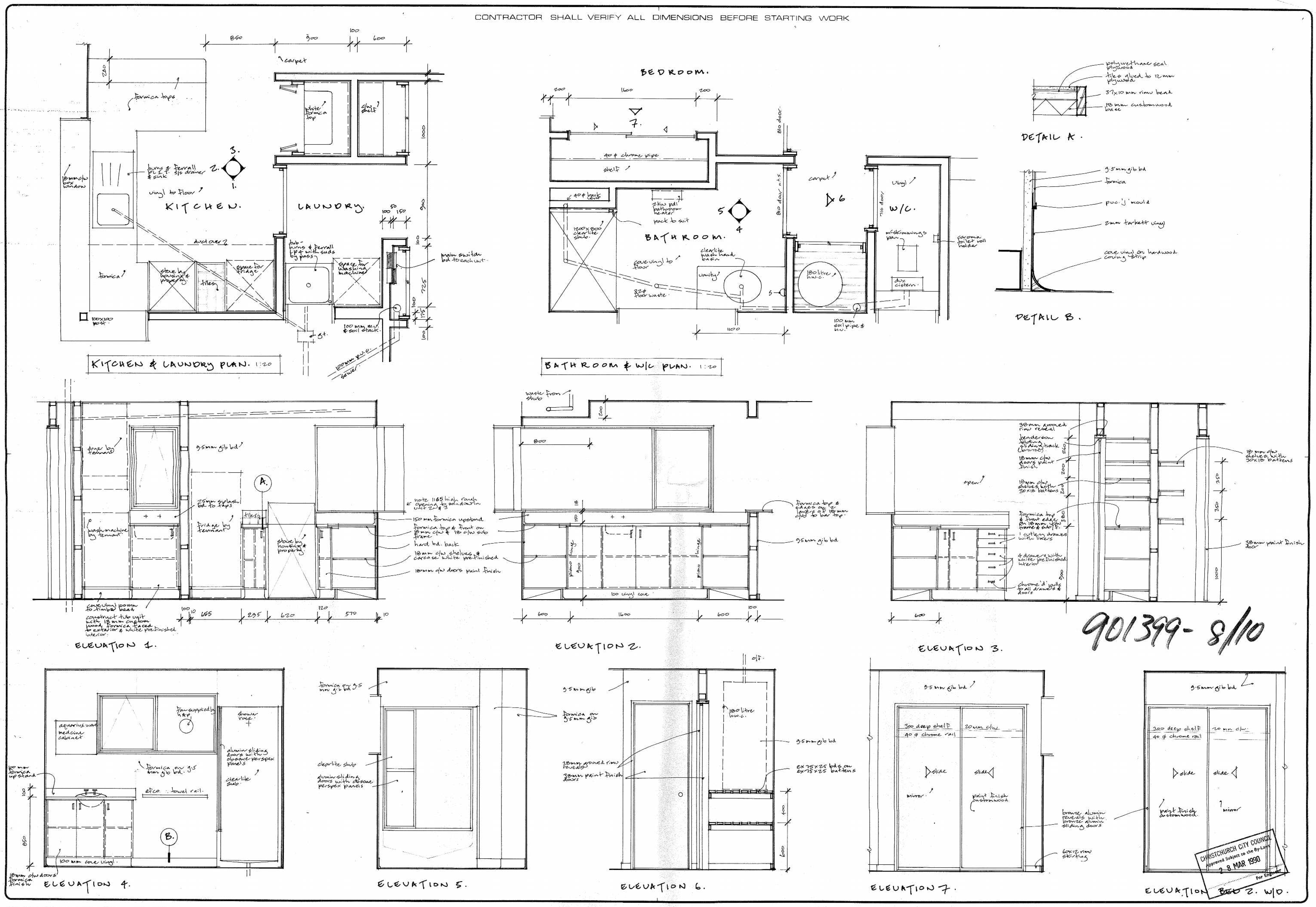
901399-7/10

CHRISTCHURCH CITY COUNCIL Approved Subject to the By-Laws 28 MAR 1990 For Engineer

CHRISTCHURCH CITY COUNCIL
CITY ARCHITECTS DIVISION : CITY WORKS & PLANNING DEPARTMENT

DUNDEE PLACE HOUSING DEVELOPMENT · STAGE 4 - CROSS SECTION & DETAILS

SCALE | DESIGN | JOB NO 502.54 | SHEET NUMBER A6
DATE 27.11.89 | DRAWN | FILE NO | OF 8 SHEETS
TRACED

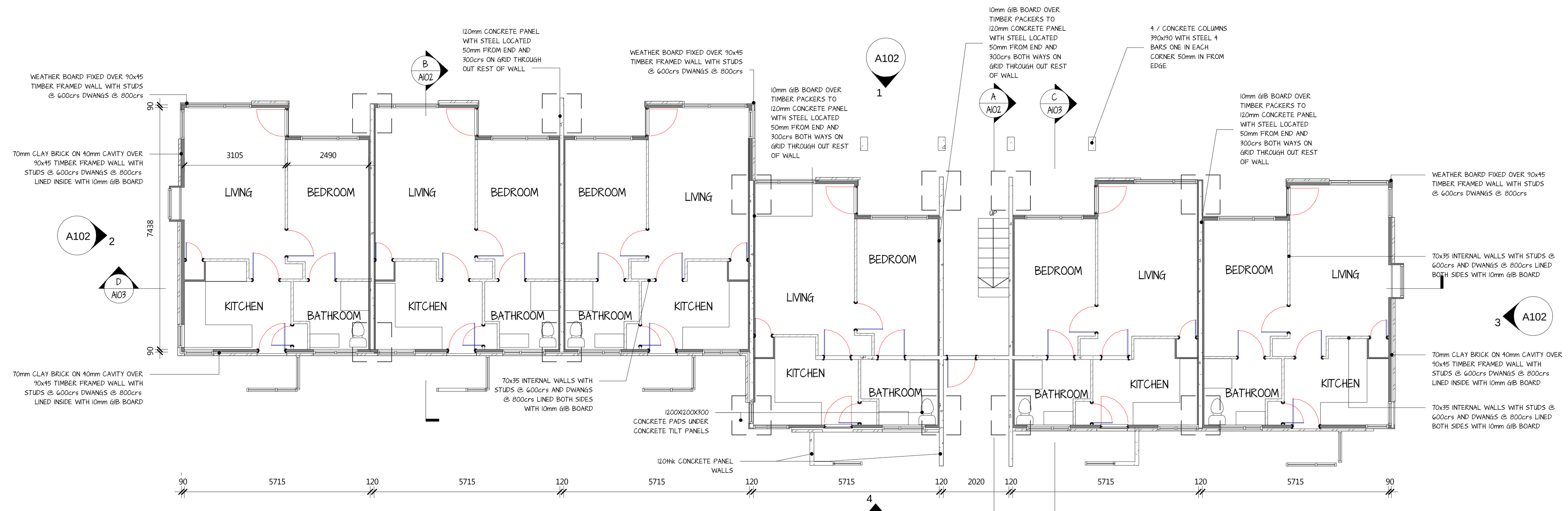


CHRISTCHURCH CITY COUNCIL
CITY ARCHITECTS DIVISION : CITY WORKS & PLANNING DEPARTMENT

DUNDEE PLACE HOUSING DEVELOPMENT • STAGE 4. - JOINERY PLANS & ELEVATIONS

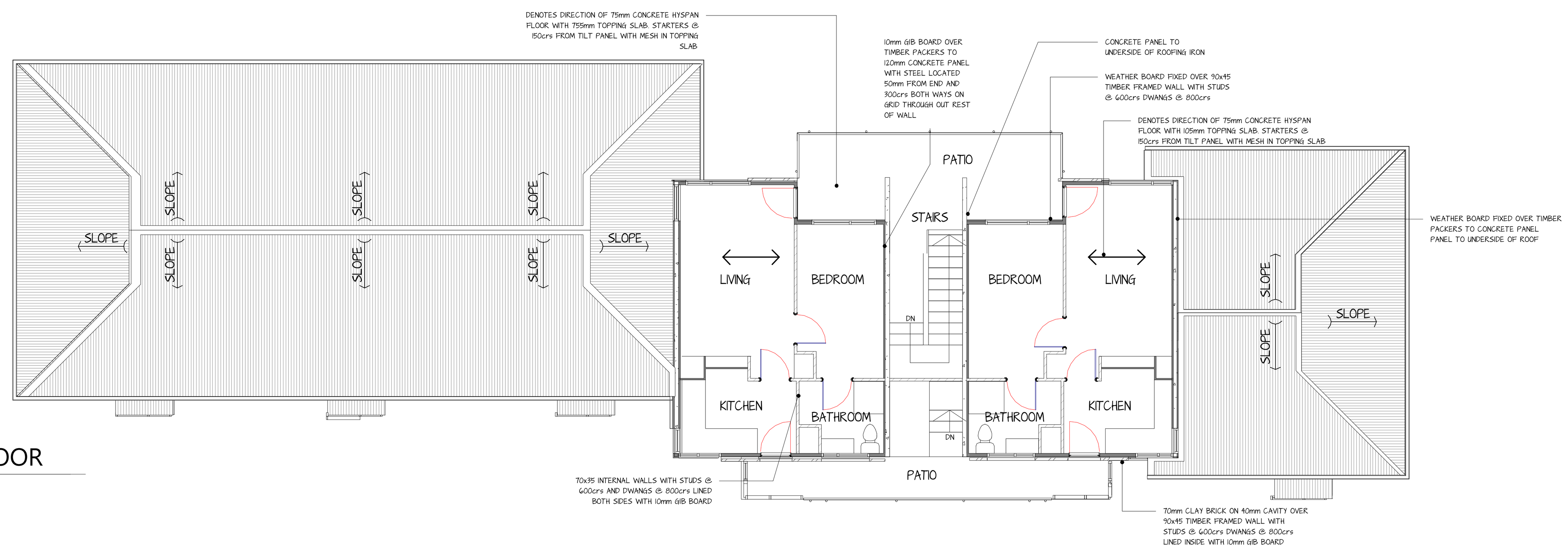
SCALE 1:20	DESIGN / DRAWN / TRACED	JOB NO	SHEET NUMBER A7
DATE 7/11/89.		FILE NO	OF 8 SHEETS

Appendix C New as-built drawings (by BuildQual in 2013)



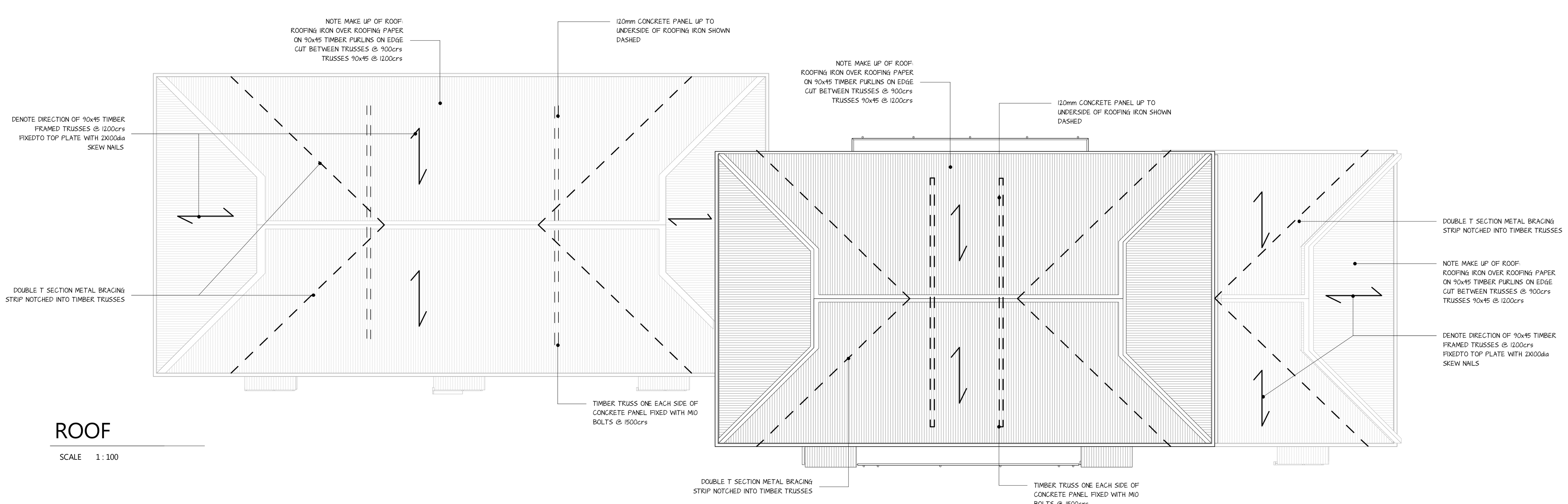
GROUND FLOOR

SCALE 1 : 100



FIRST FLOOR

SCALE 1 : 100



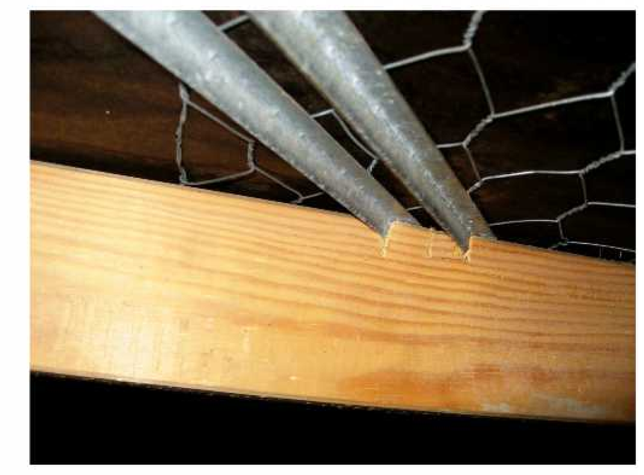
ROOF

SCALE 1 : 100

TRUSS TO CONCRETE PARTY WALL

TRUSS TO TOP PLATE DETAIL

PURLIN TO TRUSS DETAIL



ROOF BRACING ANGLE DETAIL

PURLIN TO TRUSS DETAIL

TRUSS TO TOP PLATE DETAIL

TRUSS TO CONCRETE PARTY WALL

TRUSS APEX DETAIL

DUTCH GABLE TRUSS TO CONCRETE PARTY WALL

DRAFT

BUILDQUALNZ LTD

REV	DATE	DESCRIPTIONS
2	Date 2	ISSUE FOR INFORMATION TO STRUCTURAL ENGINEER
1	08.03.13	ISSUE FOR INFORMATION STRUCTURAL ENGINEER

REVISIONS / ISSUES

PROJECT
MAURICE CARTER COURTS EXISTING PLANS

CLIENT
SMK

TITLE
BLOCK A / PLANS

DO NOT SCALE FROM DRAWING, USE DIMENSIONS SHOWN. ALL DISCREPANCIES MUST BE REPORTED

SCALE
1 : 100 @A1

DRAWN
Author

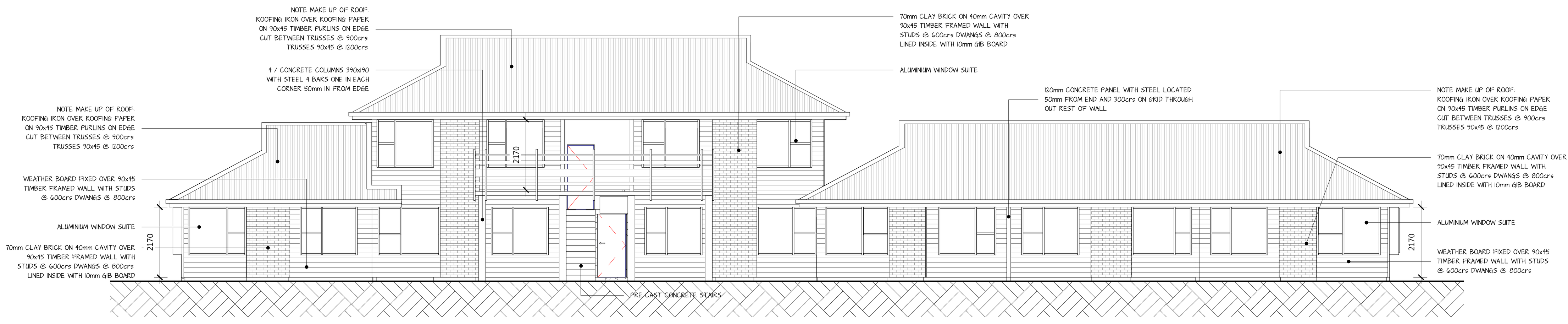
JOB No.
500-001

STAGE
INFORMATION

DRAWING No.
A101

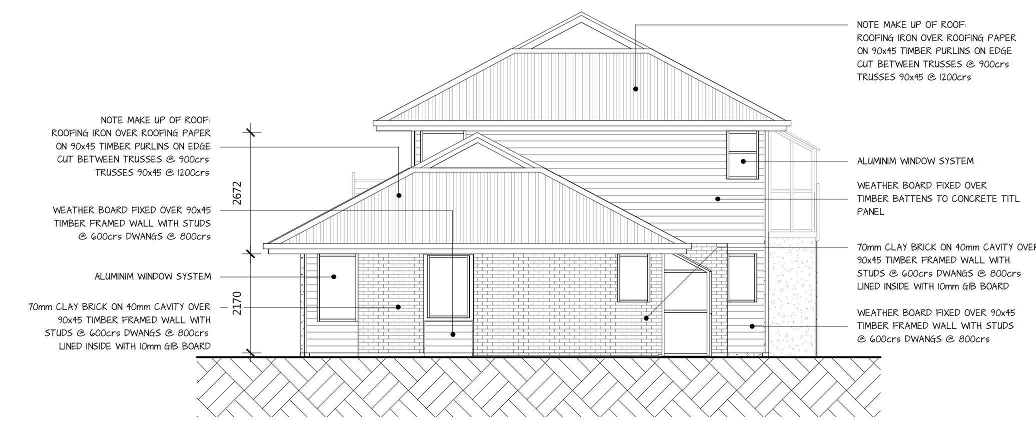
REV
2

DATE PRINTED 11/04/2013 1:22:25 p.m.



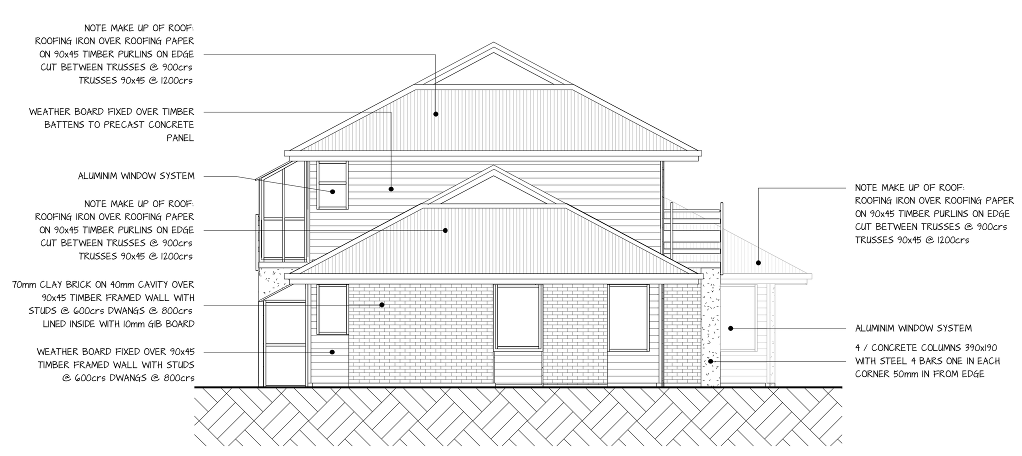
ELEVATION A

SCALE 1 : 100



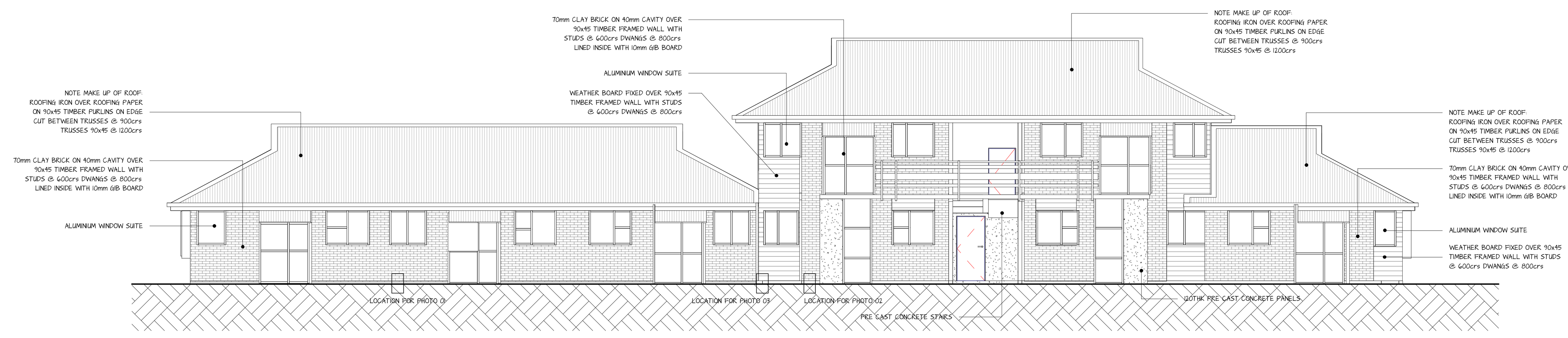
ELEVATION B

SCALE 1 : 100



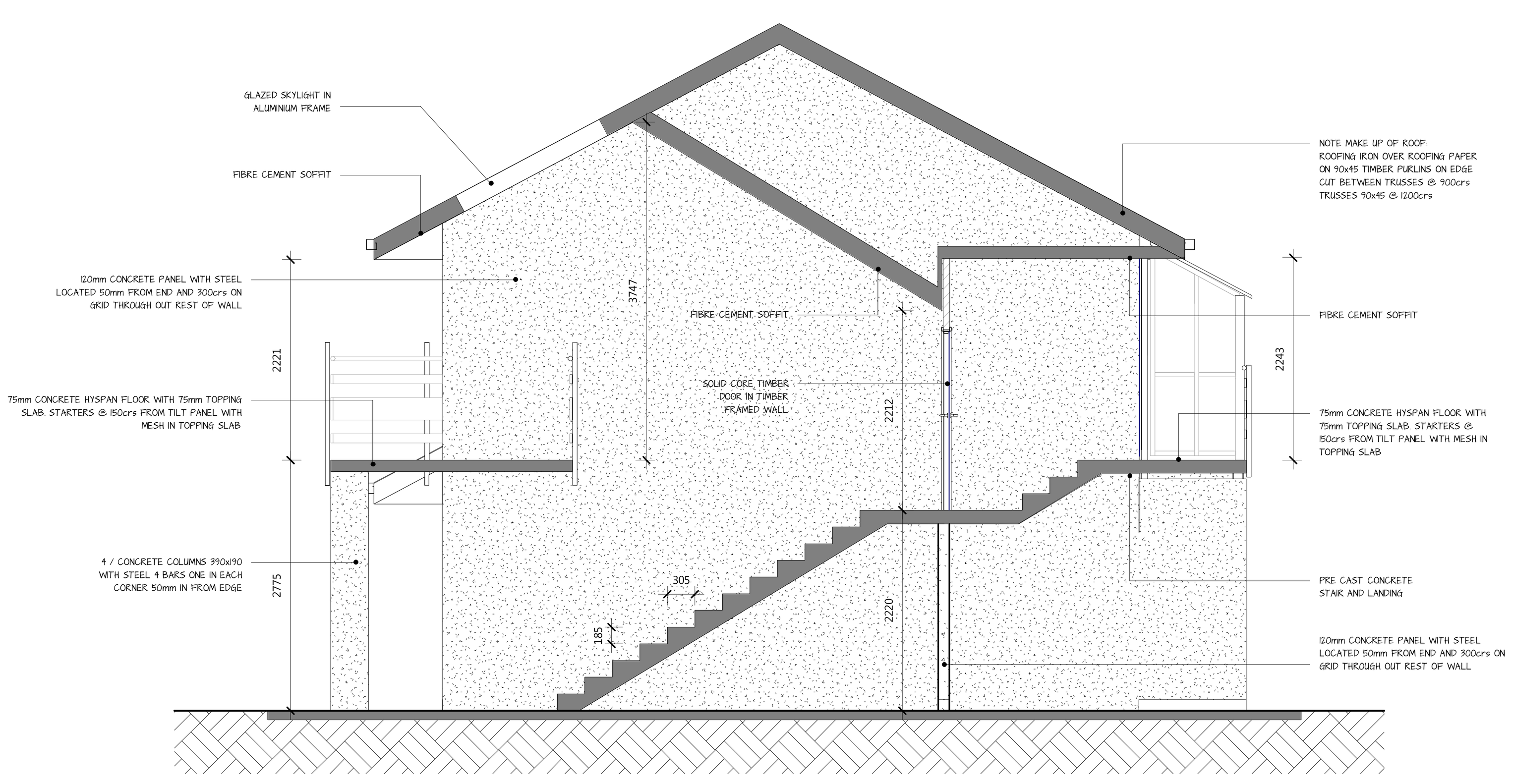
ELEVATION C

SCALE 1 : 100



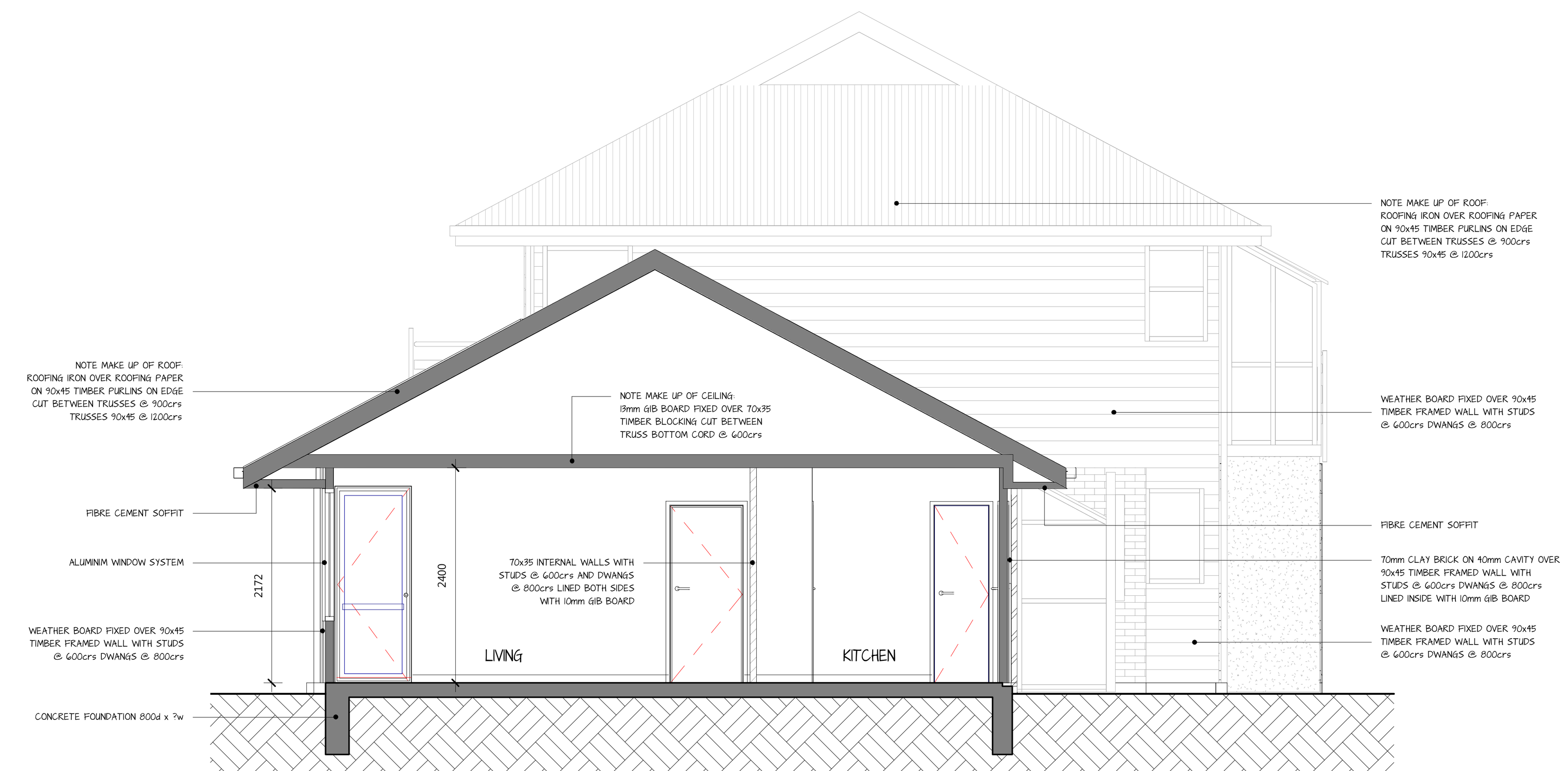
ELEVATION D

SCALE 1 : 100



A SECTION

A101 1 : 50



B SECTION

A101 1 : 50

STEEL IN CONCRETE TITL PANEL

TRUSS CONNECTION AT DUTCH GABLE END

FOUNDATION DETAIL PHOTO 01

FOUNDATION DETAIL PHOTO 03

DRAFT

REV	DATE	DESCRIPTIONS
2	Date 2	ISSUE FOR INFORMATION TO STRUCTURAL ENGINEER
1	08.03.13	ISSUE FOR INFORMATION STRUCTURAL ENGINEER

REVISIONS / ISSUES

BUILDQUALNZ LTD

PROJECT
MAURICE CARTER COURTS EXISTING PLANS

CLIENT
SMK

TITLE
BLOCK A / ELEVATIONS

DO NOT SCALE FROM DRAWING, USE DIMENSIONS SHOWN. ALL DISCREPANCIES MUST BE REPORTED

SCALE
As indicated @A1

DRAWN
SJ

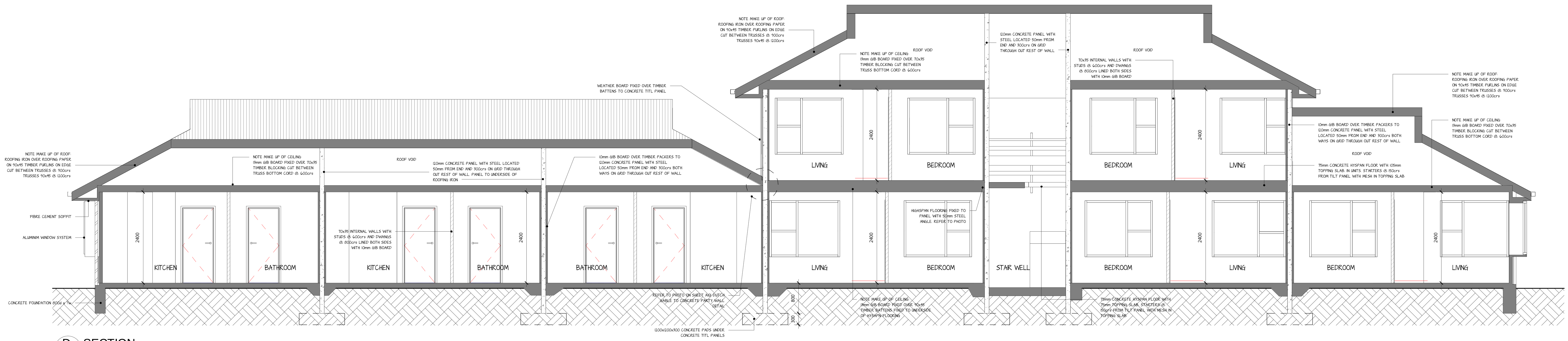
JOB No.
500-001

STAGE
INFORMATION

DRAWING No.
A102

REV
2

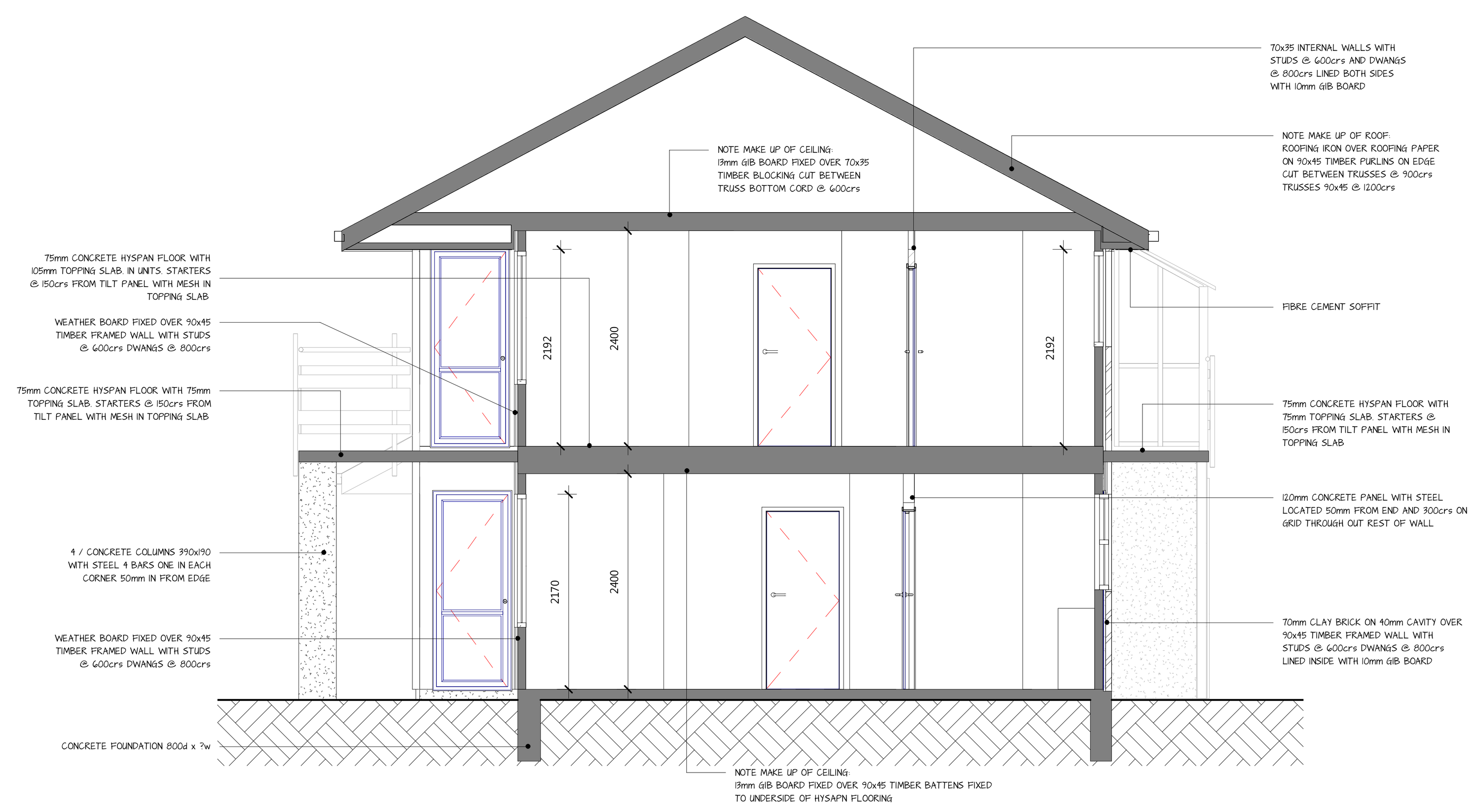
DATE PRINTED 11/04/2013 1:22:26 p.m.



D SECTION
A101 1 : 50

HYSPAN FLOORING TO PARTY WALL JUNCTION DETAIL

GROUND FLOOR CEILING TO UNDERSIDE HYSPAN FLOORING



C SECTION
A101 1 : 50

SINGLE STORY CEILING TO INTERNAL WALL JUNCTION DETAIL

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2	Date 2	ISSUE FOR INFORMATION TO STRUCTURAL ENGINEER
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MAURICE CARTER COURTS EXISTING PLANS

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TITLE
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A103

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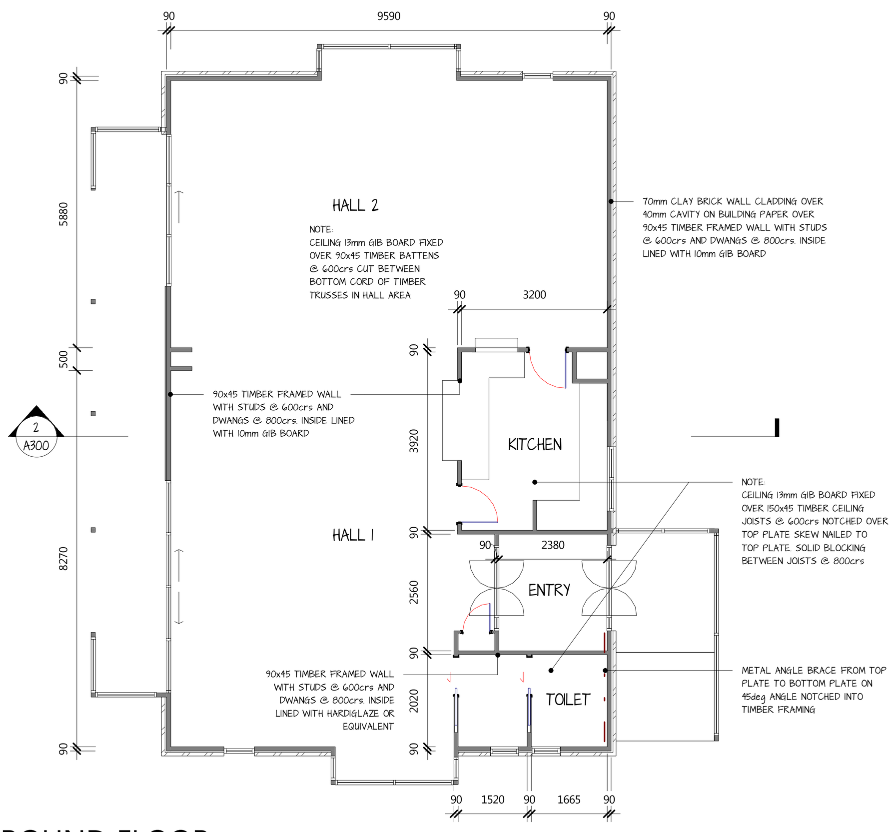

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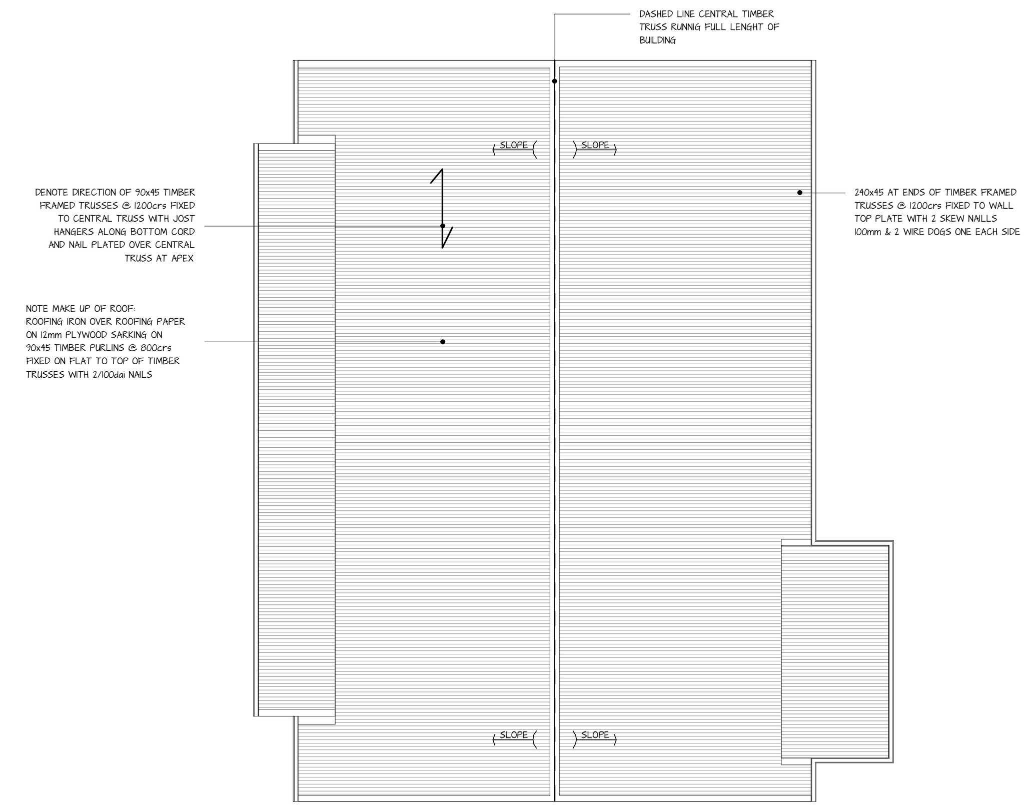

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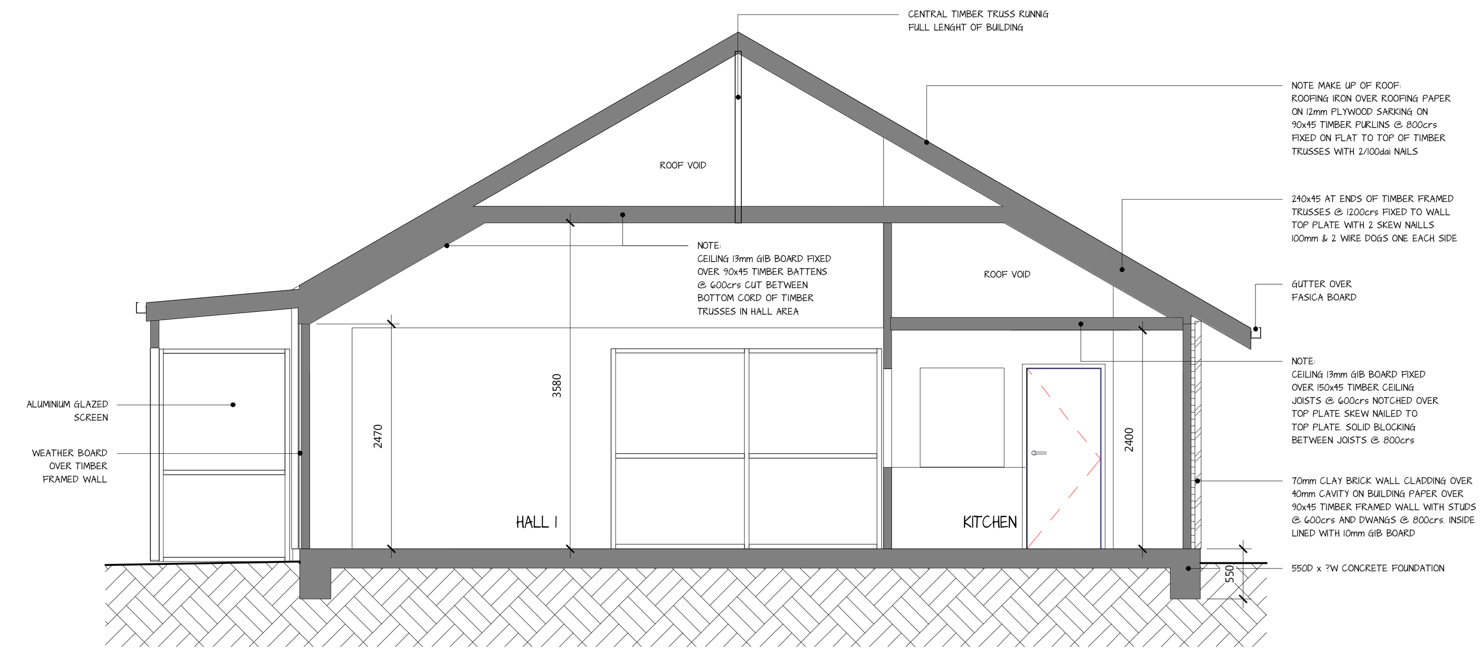

A SECTION

A300

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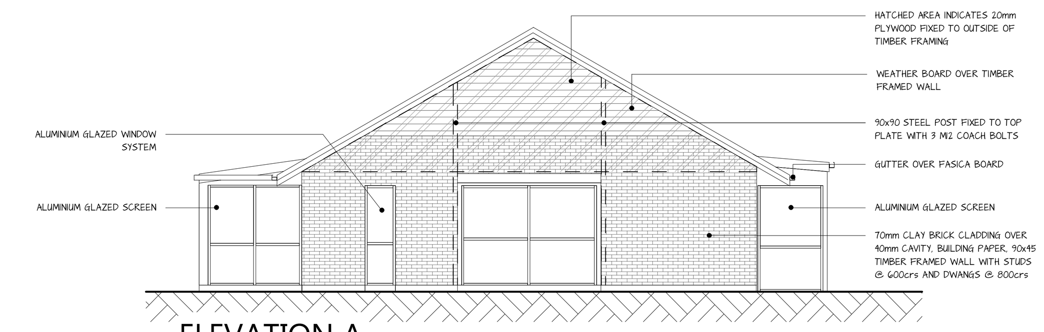

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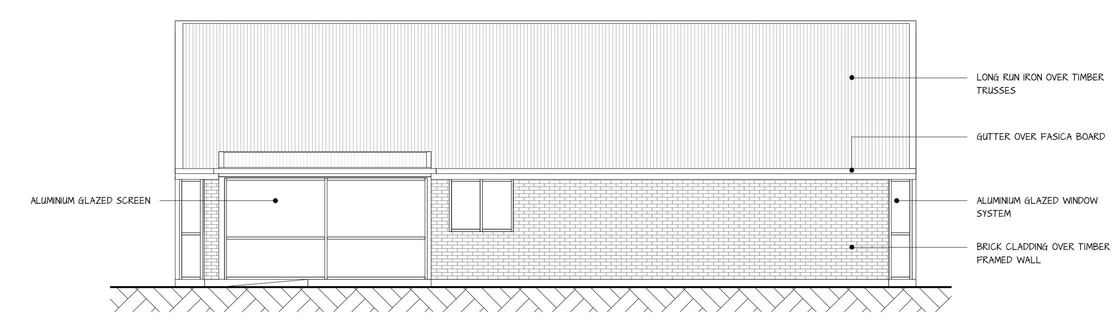

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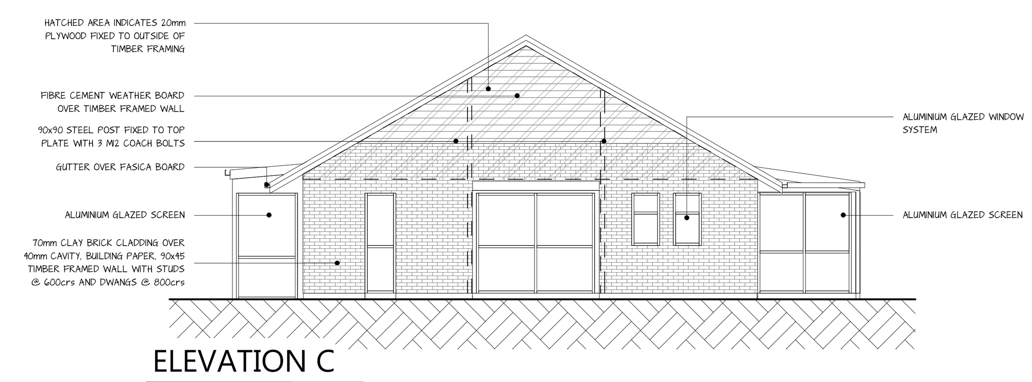

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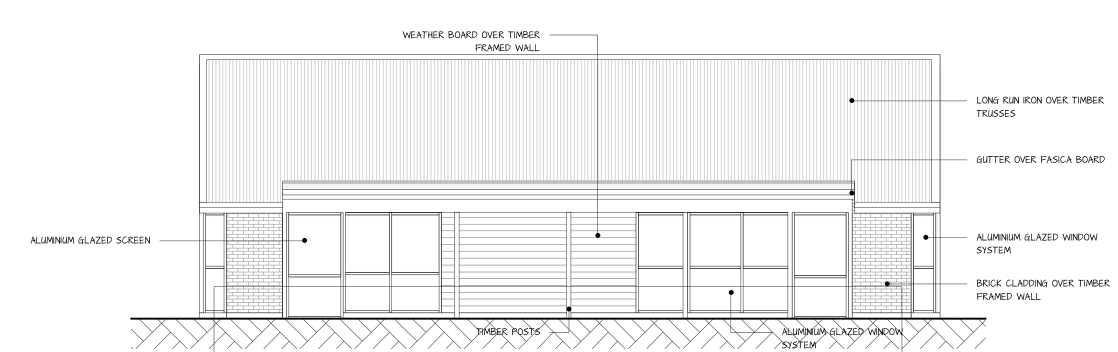

ELEVATION D

SCALE 1 : 100

DRAFT

TRUSS TO TOP PLATE DETAIL

APEX DETAIL

TRUSS LAYOUT

TRUSS TO TOP PLATE DETAIL OVER KITCHEN BATHROOM AREA

CENTRAL TRUSS TO GABLE DETAIL

CENTRAL TRUSS BOTTOM CORD DETAIL

TRUSS TO TOP PLATE FIXING DETAIL

TRUSS TO TOP PLATE FIXING DETAIL

TRUSS TO TOP PLATE FIXING DETAIL

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BOTTOM CORD TRUSS DETAILS


TRUSS TO INTERNAL WALL BETWEEN HALL AND KITCHEN


GABLE END STEEL POST DETAIL


GABLE END STEEL POST DETAIL


GABLE END STEEL POST DETAIL


GABLE END / APEX JUNCTION


GABLE END TO TOP PLATE JUNCTION DETAIL


APEX DETAIL TRUSS MAKE UP


TIMBER FRAMED WALL BETWEEN HALL AND KITCHEN

DRAFT

REV	DATE	DESCRIPTIONS
1	11.04.13	ISSUE FOR INFORMATION TO STRUCTURAL ENGINEER

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PROJECT
MAURICE CARTER COURTS EXISTING PLANS

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A301

REV
1

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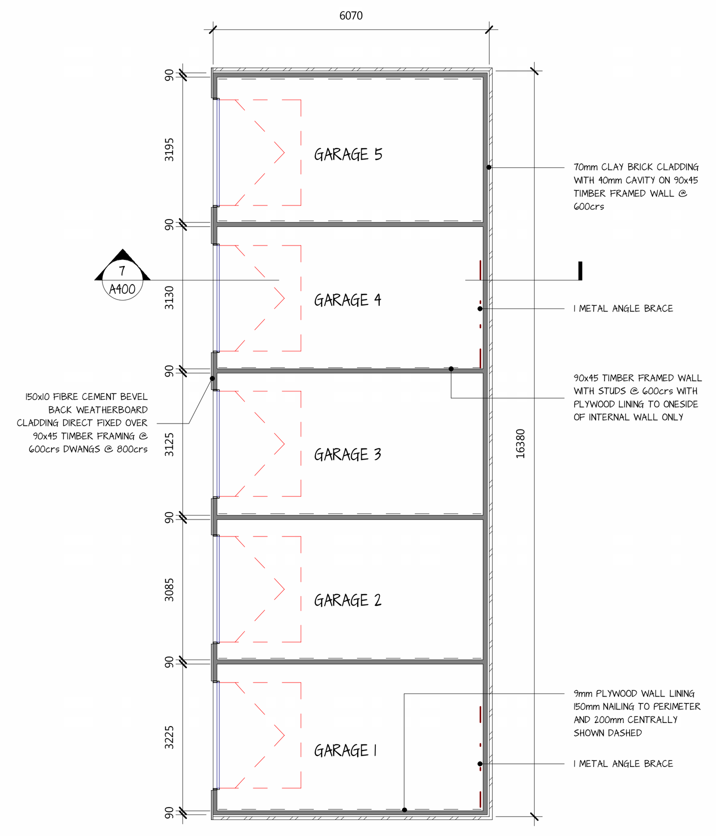
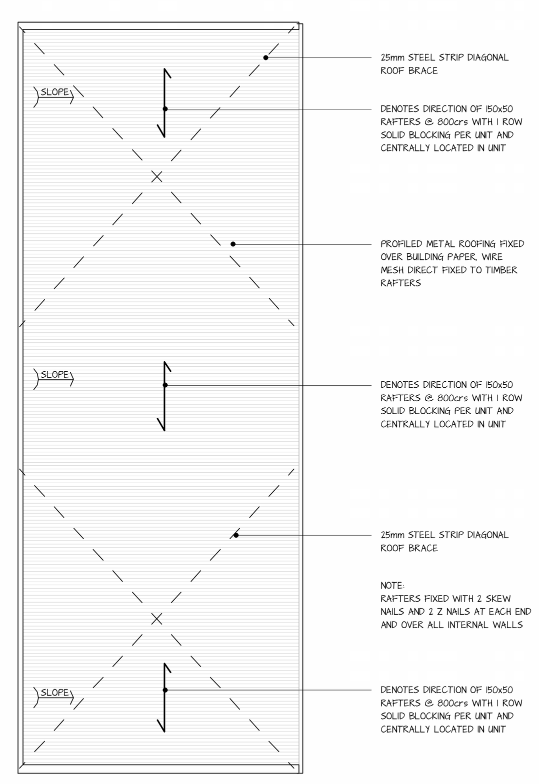


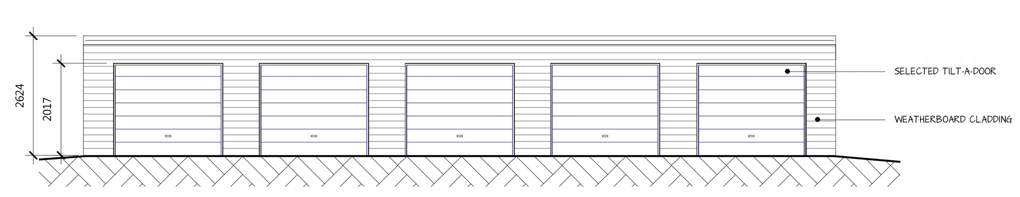
GROUND FLOOR

SCALE 1 : 100



ROOF

SCALE 1 : 100



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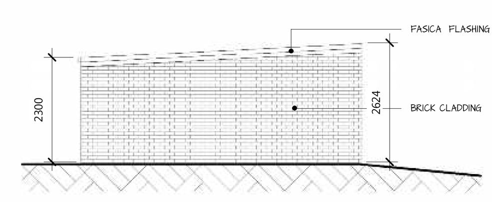

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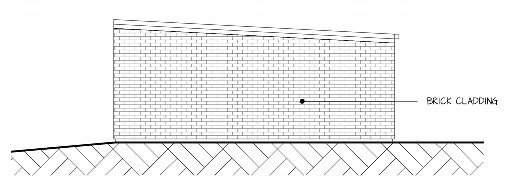
SCALE 1 : 100



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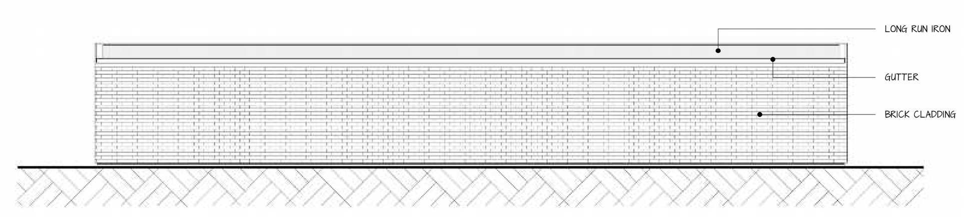

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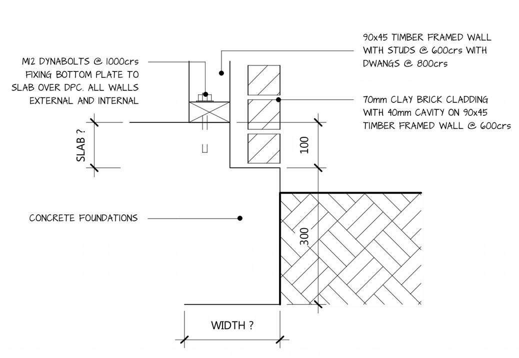
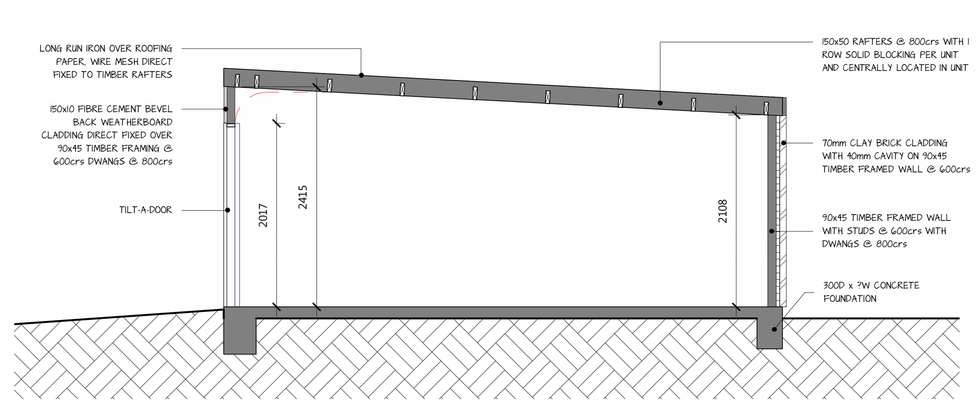
SCALE 1 : 100



FOUNDATION DETAIL

SCALE 1 : 10



X SECTION

SCALE 1 : 50

PHOTOS

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2	10.04.13	ISSUE FOR STRUCTURAL ENGINEER COMMENTS
1	06.03.13	ISSUE FOR INFORMATION STRUCTURAL ENGINEER

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PROJECT MAURICE CARTER COURTS EXISTING PLANS

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REV 2

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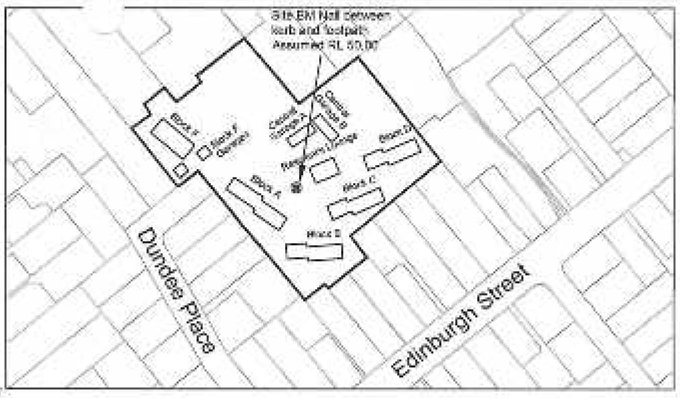
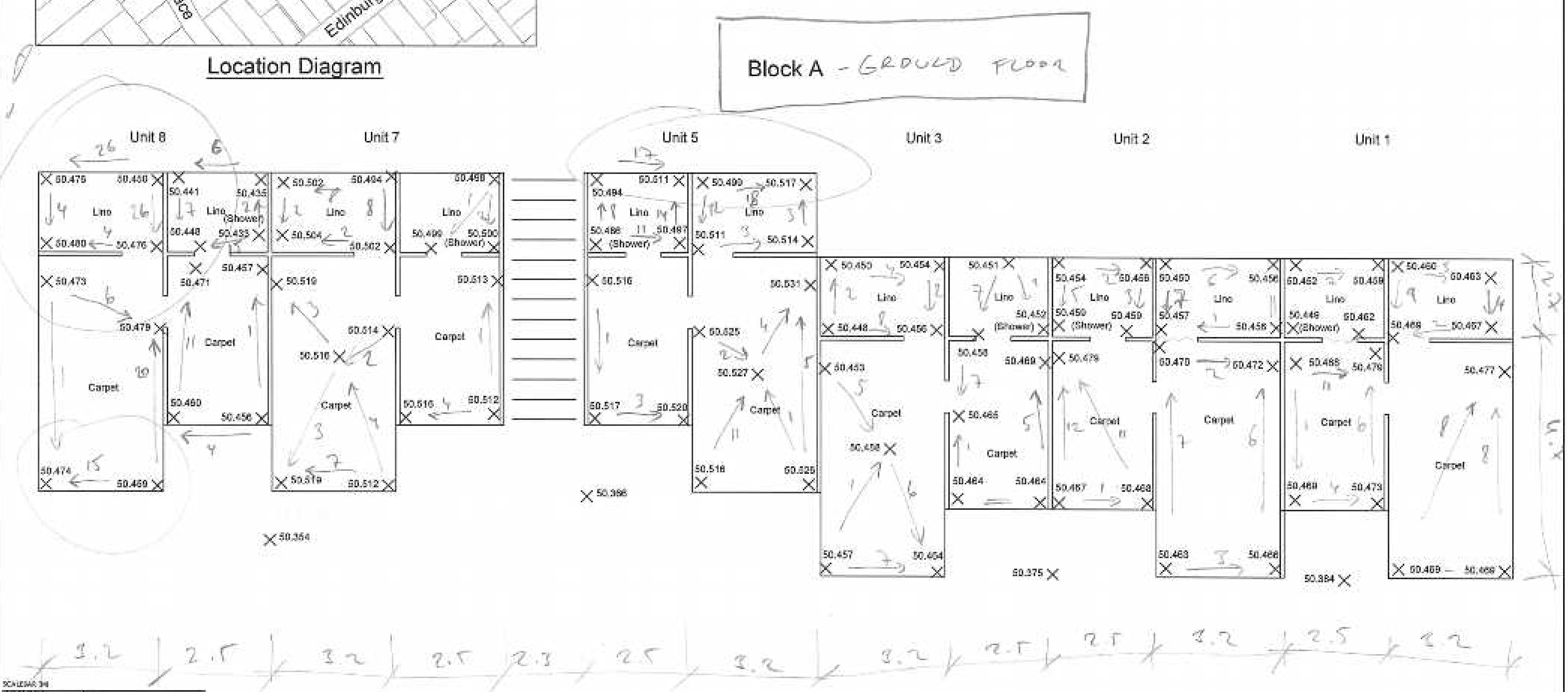
Appendix D Levels Survey Results (by Woods on 14 December 2012)



Location Diagram

NOTES

1. Levels are in terms of an assumed datum. Site BM is a nail in between the footpath and kerb as indicated on Location Diagram. RL 50.000
2. All level measurements have been taken on the finished surface of carpet or linoleum as noted.
3. Levels and wall measurements taken where accessible.
4. The levels and measurements were taken on 3rd-7th December 2012.
5. Equipment was a Leica Automatic Level NA724 #5538825



REVISION DETAILS	NAME	DATE	CLIENT
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Maurice Carter Courts

BUILDING FLOOR LEVELS - BLOCK A

CHRISTCHURCH CITY COUNCIL

WOODS Engineers. Surveyors. Planners.

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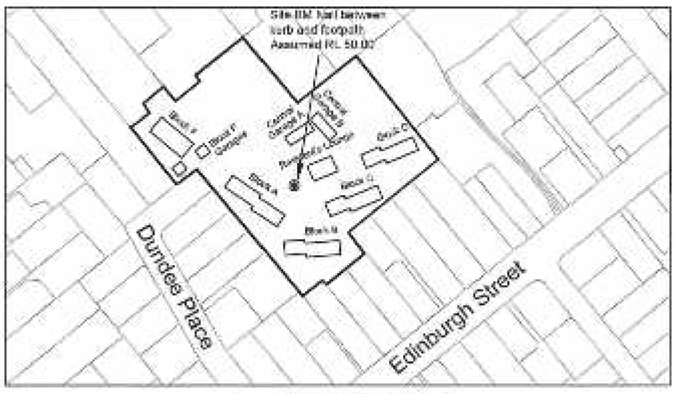
JOB NUMBER:	40154	SCALE:	1:100@ A3
ISSUED:	14/12/12		
DWG. NO:	40154-GE-001	REV:	1



Location Diagram

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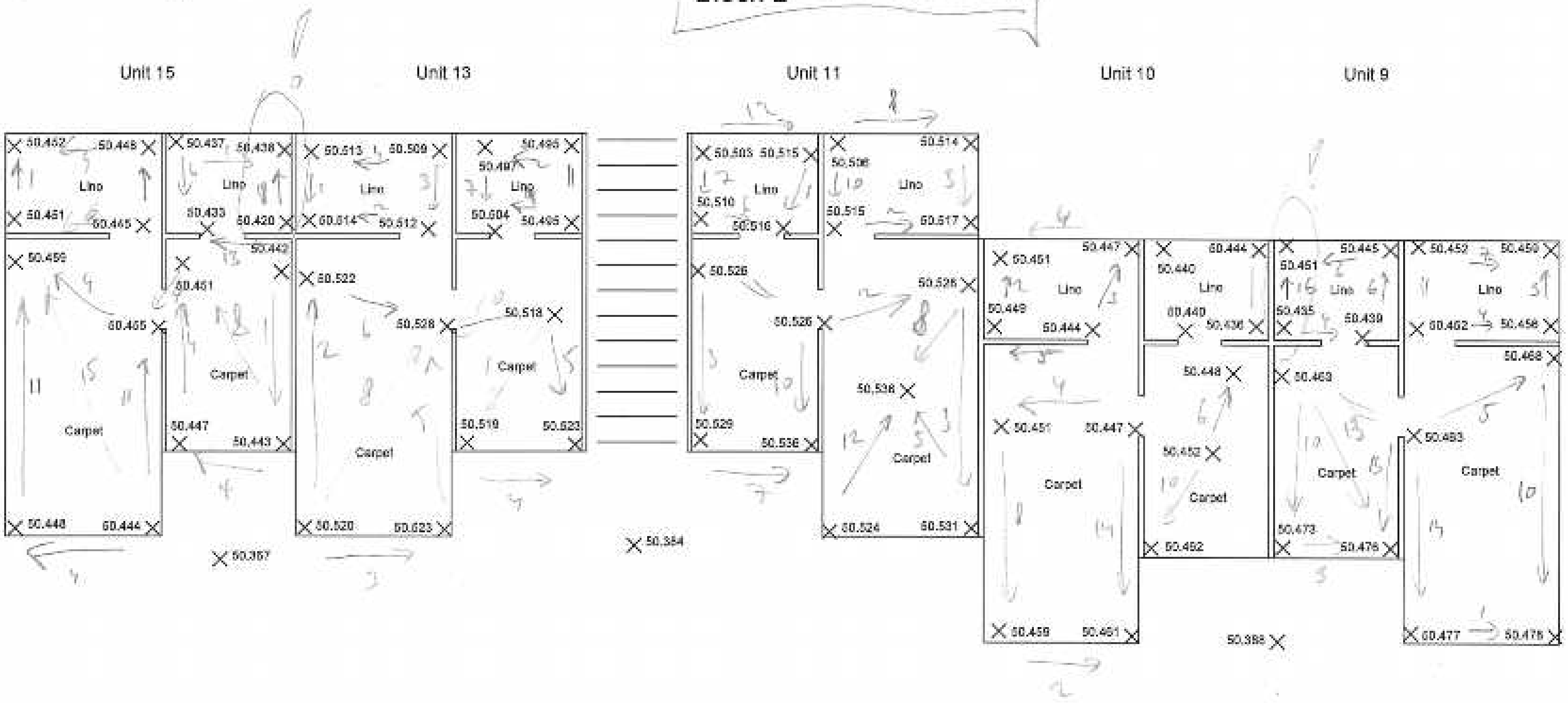
Block B - GROUND FL



REVISION DETAILS	NAME	DATE
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Maurice Carter Courts

BUILDING FLOOR LEVELS - BLOCK B

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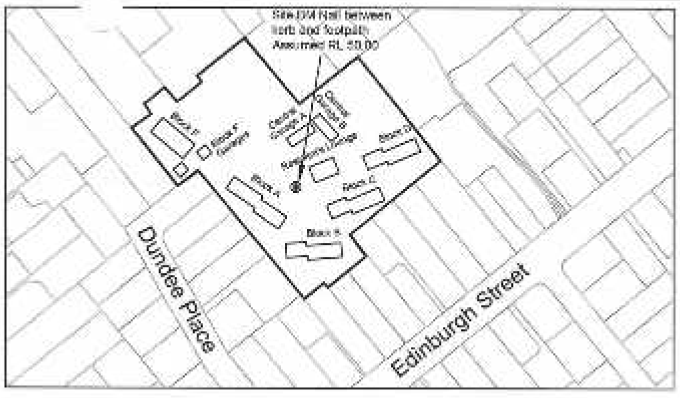
ISSUED: 14/12/12	
DWG. NO. 40154-GE-00	REV. 1



Location Diagram

NOTES

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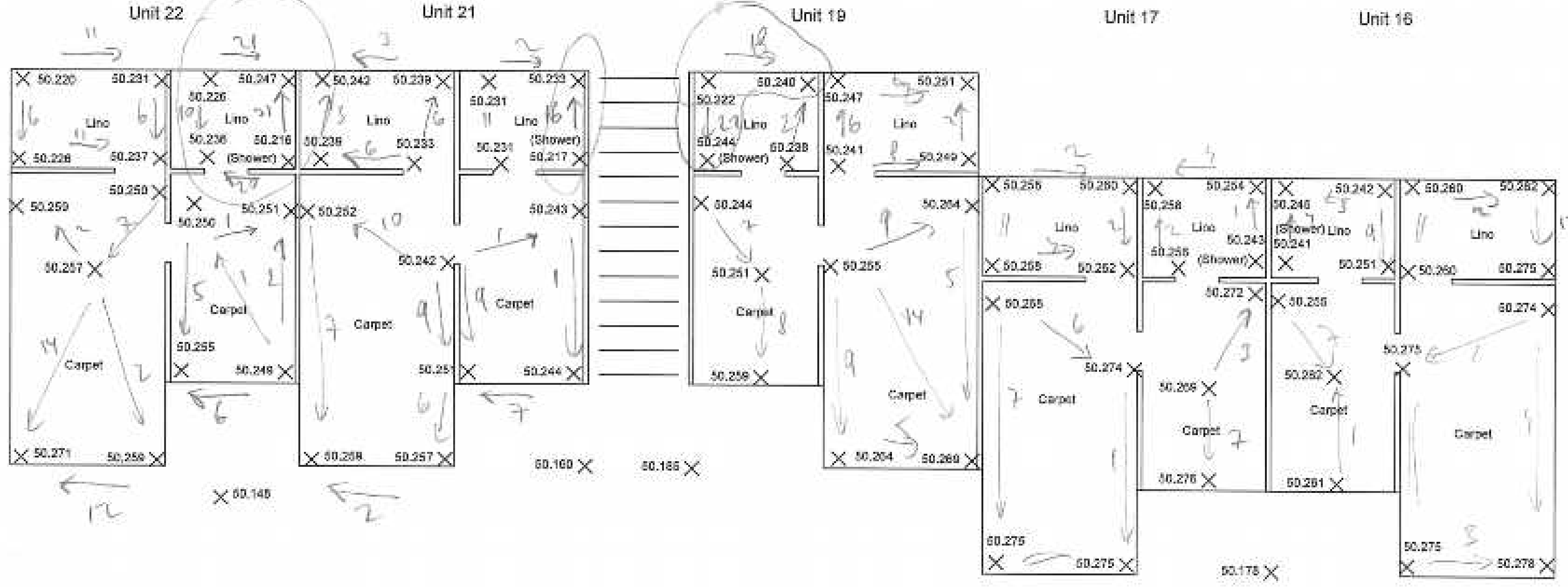
Block C - GROUND FLOOR



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Maurice Carter Courts

BUILDING FLOOR LEVELS-BLOCK C

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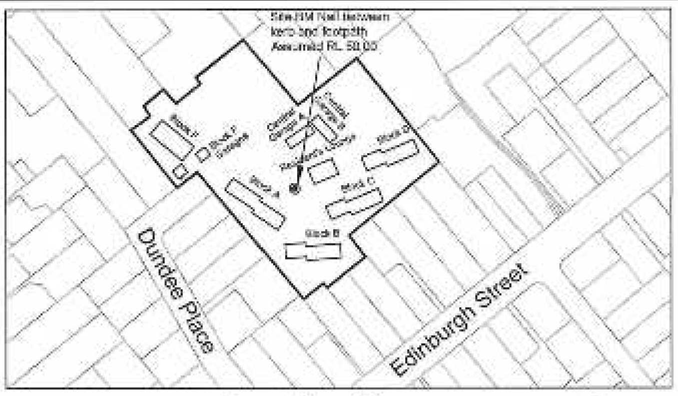
DWG. NO:	40154-GE-003	REV:	1



Location Diagram

NOTES

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3. Levels and wall measurements taken where accessible.
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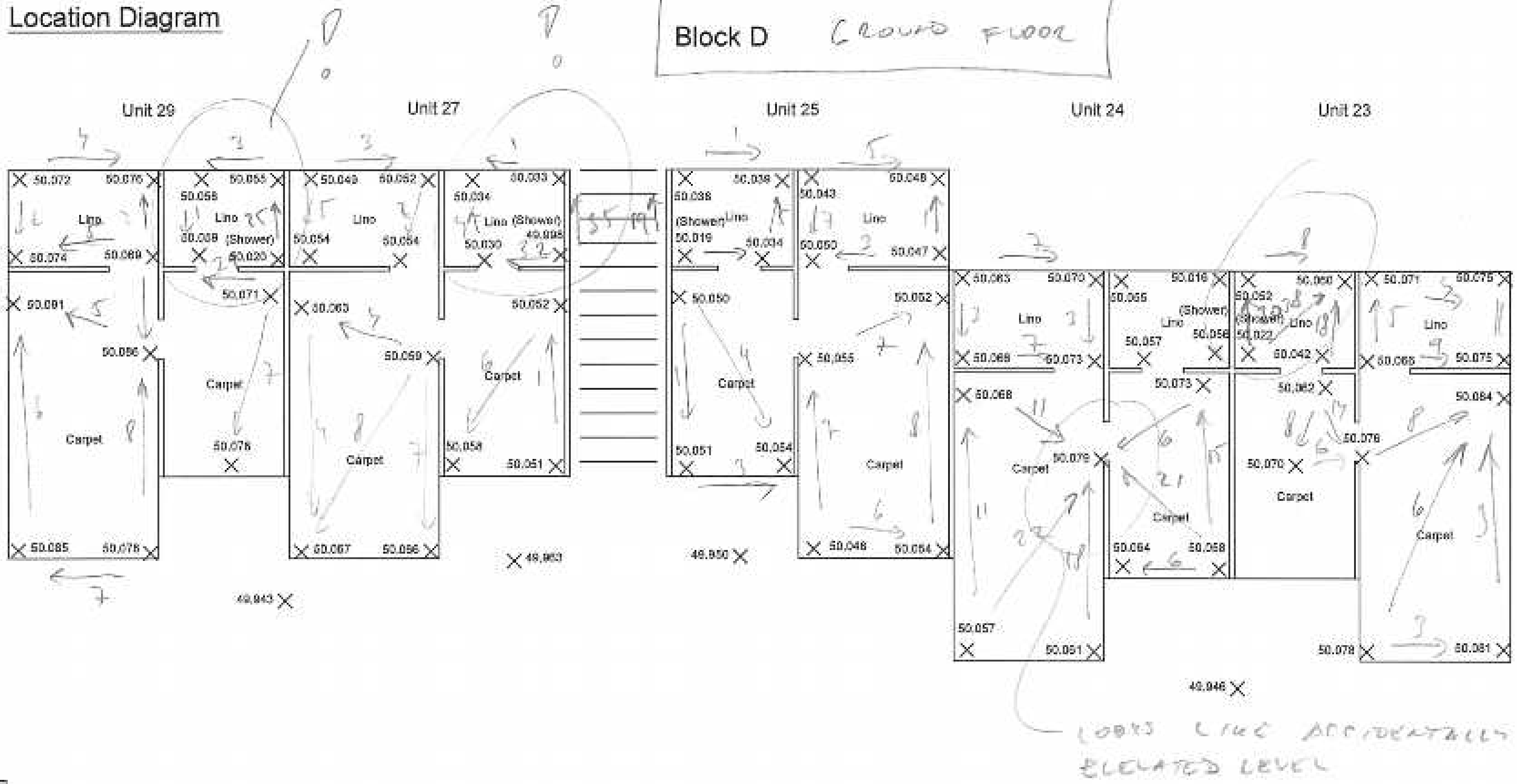
5. Equipment was a Leica Automatic Level NA724 #5538825

Block D GROUND FLOOR



SCALEBAR (M)

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Maurice Carter Courts

BUILDING FLOOR LEVELS -BLOCK D

CHRISTCHURCH CITY COUNCIL

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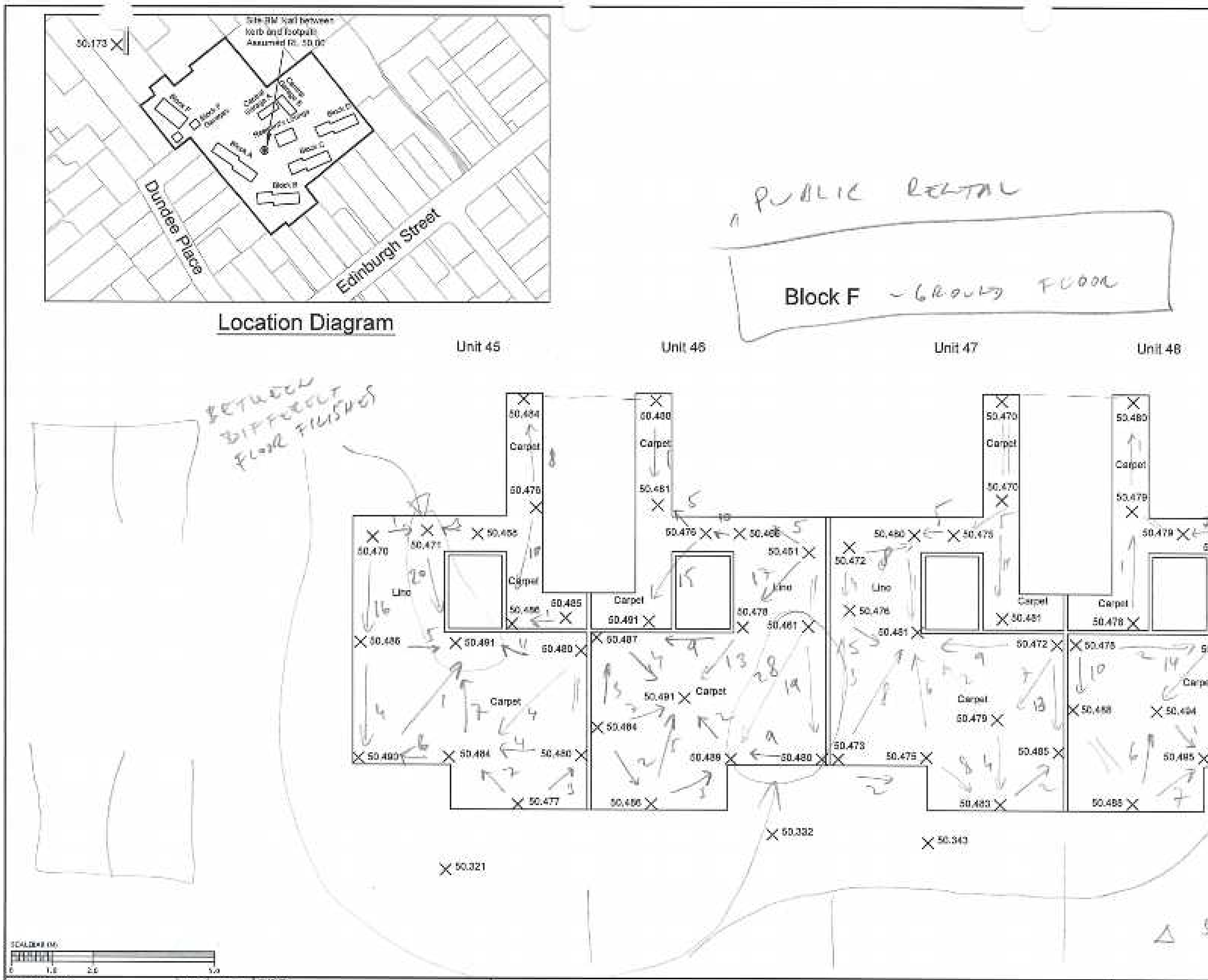
JOB NUMBER:	40154	SCALE:	1:100 @ A3
ISSUED:	14/12/12		
DWG. NO.	40154-GE-0(	REV.	1



NOTES

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Δ 50.501 − 50.473 = 28 mm

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Maurice Carter Courts

BUILDING FLOOR LEVELS -BLOCK F

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Engineers. Surveyors. Planners.

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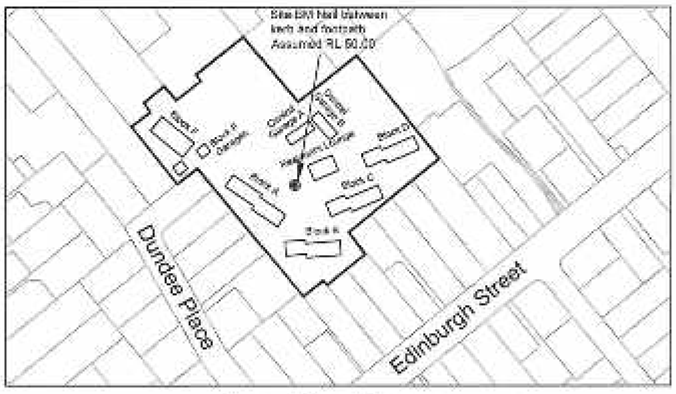
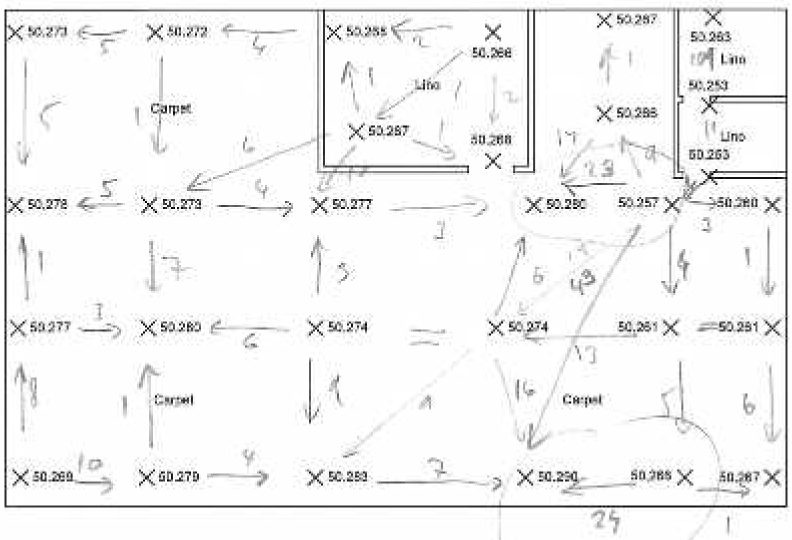


Location Diagram

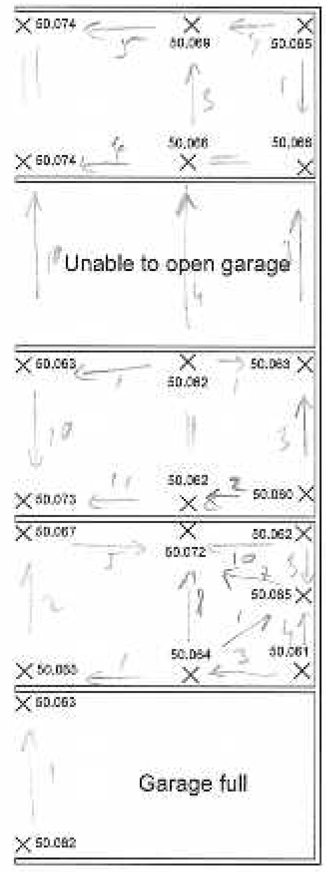
Resident's Lounge



ϕ 50.260 – 50.270 → $\boxed{50.265}$

Block H

Central Garage B



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SCALEBAR (M) 0 1.0 2.0 5.0

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Maurice Carter Courts

BUILDING FLOOR LEVELS-LOUNGE & GARAGES

CHRISTCHURCH CITY COUNCIL

WOODS
Engineers. Surveyors. Planners.

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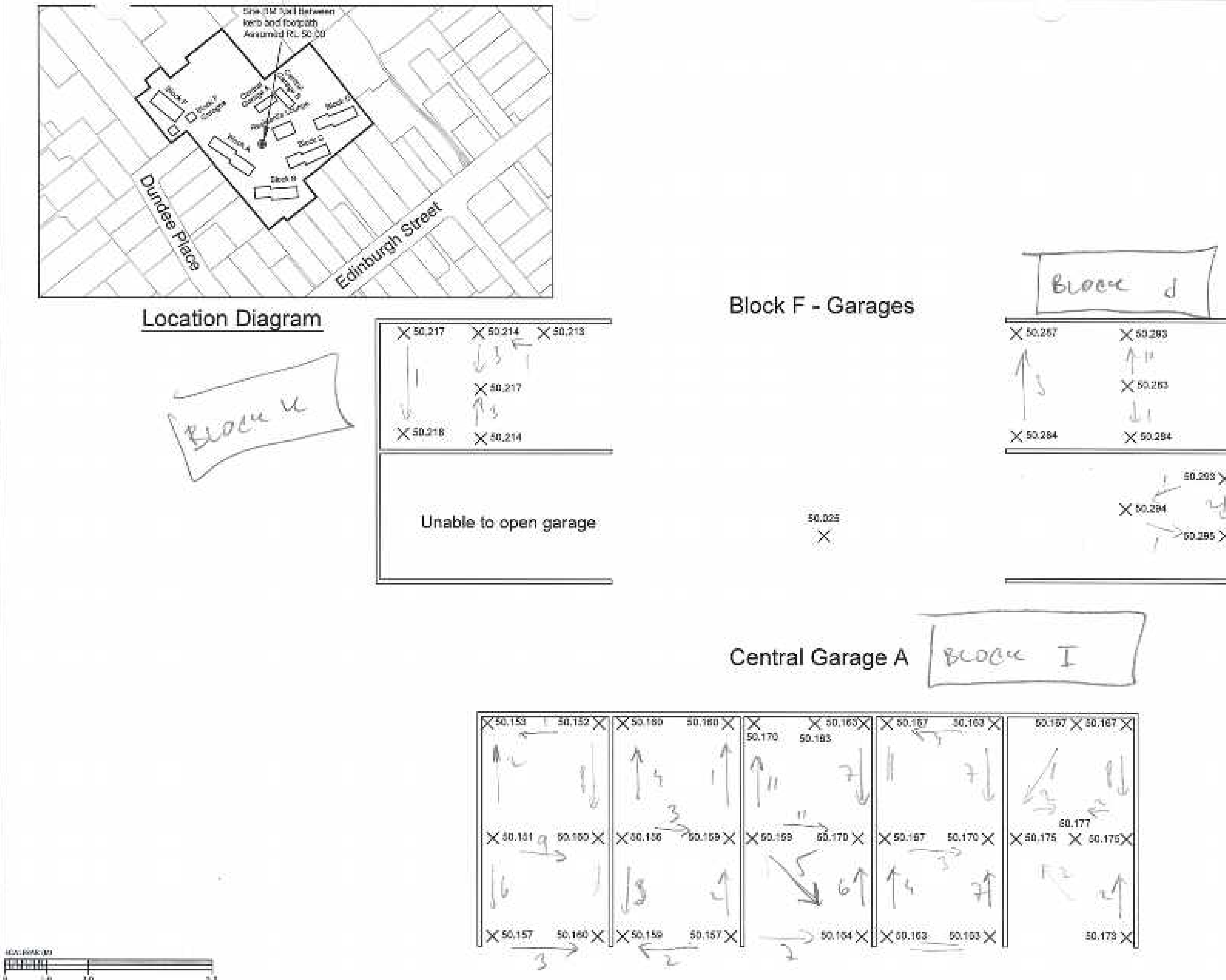
APPROVED:	SURVEYED: CA/JG/DS
JOB NUMBER: 40154	SCALE: 1:100 @ A3
ISSUED: 14/12/12	
DWG. NO: 40154-GE-00	REV: 1



NOTES

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4. The levels and measurements were taken on 3rd-7th December 2012.

5. Equipment was a Leica Automatic Level NA724 #6538825

REVISION DETAILS	NAME	DATE
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Maurice Carter Courts

BUILDING FLOOR LEVELS - GARAGES

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Appendix E Geotechnical Interpretative report (by SKM on 19 December 2012)

Christchurch City Council

BE 1103 EQ2
Maurice Carter Courts
16 Dundee Place, Spreydon

GEOTECHNICAL INTERPRETATIVE REPORT

FINAL

- B
- 19 December 2012

Christchurch City Council
BE 1103 EQ2
Maurice Carter Courts
16 Dundee Place, Spreydon

GEOTECHNICAL INTERPRETATIVE REPORT

FINAL

- B
- 19 December 2012

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Contents

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A	19/12/2012	M Doherty	N Calvert	19/12/2012	Draft for Client Approval
B	21/03/2013	A Martin	A Martin	21/03/2013	Final Issue

Approval

	Signature	Date	Name	Title
Author		21/03/2013	Jon Rabey	Engineering Geologist
Approver		21/03/2013	Alex Martin	Project Manager

Distribution of copies

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Project manager:	Alex Martin
Name of organisation:	Christchurch City Council
Name of project:	Maurice Carter Courts, 16 Dundee Place
Name of document:	Geotechnical Interpretative Report
Document version:	A
Project number:	ZB01276.219

1. Introduction

SKM has been commissioned by Christchurch City Council (CCC) to undertake a geotechnical investigation to provide foundation recommendations for the proposed new build residential units at 16 Dundee Place, Spreydon. It is understood that the findings from this report will be used in a quantitative Detailed Engineering Evaluation (DEE).

The scope of geotechnical works involved:

- Organising a drilling contractor to undertake the geotechnical investigation comprising 2 CPTs.
- Supervising the on-site investigation (CPTs), undertaking hand auger boreholes and Scala penetrometer tests, logging geotechnical data and soil sampling.
- Preliminary assessment of liquefaction potential and settlement at the site.
- Preparation of a geotechnical interpretative report identifying the ground related issues for consideration when building the proposed residential units.
- Recommendation for foundations for the purpose of cost estimating.

2. Site description

The site is located at 16 Dundee Place in Spreydon and comprises a topographically flat, undeveloped area of lawn (approximately 40 m by 50 m) in between residential properties.

- **Table 2.1 – Site Location**

Maurice Carter Courts has been classified as 'urban non-residential' by CERA. However, the site is surrounded by residential housing which has been classified as TC2 so it is suggested that Maurice Carter Courts falls under this category with respect to foundation construction. TC2 refers to the 'Foundation Technical Category 2' which is defined as:

Minor to moderate damage land from liquefaction is possible in future large earthquakes. Lightweight construction or enhanced foundations are likely to be required such as enhanced concrete raft foundations.

3. Existing geotechnical information

3.1. Investigation by third parties

Available map data shows that no boreholes or Cone Penetration Tests (CPTs) have been undertaken previously on the site or if they have, they are not publically available. No boreholes were found in close proximity to the site from a search of all available information. However, Project Orbit shows CPT logs (approximately 250 m away) which indicate silts and sands to at least 16 m below ground level (mbgl).

The liquefaction mapping exercise undertaken by Cubrinovski and Taylor following the 22 February 2011 earthquake found no evidence of liquefaction within or adjacent to the site. EQC interpretation of liquefaction from mapping shows no liquefaction after 22 February 2011 or 23 December 2011, but some minor liquefaction occurred in the nearby area following the 13 June earthquake. Discussions with local residents confirmed that no damage to the properties had occurred and that no liquefaction was observed in the immediate area of the site following any of the major earthquakes in the recent Canterbury earthquake sequence.

3.2. Regional geology

The 1:250,000 geological map of the Christchurch urban area (Brown and Weeber, 1992) indicates that the site is predominantly underlain by alluvial sand and silt deposits of the Springston Formation.

4. Geotechnical investigation

4.1. General

The geotechnical investigation included 2 CPT tests to a target depth of 20 mbgl as detailed in Table 4.1. Prior to commencing the CPTs, hand auger boreholes were excavated at each CPT position to check for the presence of underground services. The boreholes were terminated at 1.5 mbgl and then backfilled with arisings. In addition, 6 Scala penetrometer tests were undertaken to a maximum depth of 3.3 mbgl (see Table 4.3) and 4 further hand auger boreholes were put down to 3 mbgl (see Table 4.2). Please refer to the exploratory hole location plan showing all the test locations (Appendix A).

4.2. Methodology

4.2.1. Cone penetration tests

The CPTs were conducted using a truck mounted CPT rig in accordance with ASTM standard D-5778-07.

Table 4.1 summarises the CPT locations and probe depths. The CPT results are presented in Appendix B.

- **Table 4.1 – CPTs Summary**

CPT	Final depth, mbgl	Coordinates		Termination Remarks
		Eastings	Northings	
CPTu01	19.94	1567691	5177820	Target depth
CPTu02	20.00	1567664	5177793	Target depth

Note: Coordinates to NZTM, derived from aerial photography; CPTu = piezocone

4.2.2. Hand augers

The 4 hand auger boreholes referred to in Section 4.1 above are detailed in Table 4.2 below.

- **Table 4.2 – Hand augers summary**

Hand augerhole	Final depth, mbgl	Coordinates	
		Eastings	Northings
H1	3.2	1567704	5177801
H2	3.2	1567692	5177789
H3	3.0	1567661	5177796
H4	3.2	1567676	5177807

Note: Coordinates to NZTM, derived from aerial photography.

4.2.3. Scala penetrometer tests

The 6 Scala penetrometer tests referred to in Section 4.1 above are detailed in Table 4.3 below.

Table 4.3 – Scala penetrometer summary

Scala penetrometer test	Final depth, mbgl	Coordinates	
		Eastings	Northings
S1	3.3	1567691	5177820
S2	3.3	1567704	5177801
S3	3.3	1567692	5177789
S4	3.3	1567677	5177780
S5	3.3	1567661	5177796
S6	3.3	1567676	5177807

4.3. Groundwater observations

The table below provides a summary of the groundwater levels observed during the investigation.

Table 4.4 – Groundwater levels summary

Test ref.	Date	Groundwater Level (mbgl)
CPTu01	10/12/12	1.0
CPTu01	10/12/12	1.0
H1	11/12/12	1.3
H2	11/12/12	1.4
H3	12/12/12	1.2
H4	12/12/12	1.3

5. Geotechnical interpretation

5.1. Geological model

Based on the above data and the review of published geological information, the following ground model for the site can be inferred.

- **Table 5.1 – Geological ground model**

Depth range (mbgl)	Description	Formation
0.0 – 0.5	SILT / Clayey SILT with subordinate peat bands	Springston
0.5 – 13.0	Silty SAND / Sandy SILT/ Clayey SILT with subordinate peat bands	Springston
13.0 – 20.0	SAND / Silty SAND / SILT	Springston
20 >	Sandy GRAVEL	Riccarton Gravels

Note: Ground model based on CPT logs only

The CPT logs indicate the subsurface to comprise of silts and sands to 20 mbgl. The subsurface material becomes sandy at approximately 13 mbgl.

5.2. Geotechnical parameters

This section provides the geotechnical parameters adopted for use in foundation design. The parameters are based on in-situ test results with empirical correlations.

- **Table 5.2 – Summary of geotechnical parameters**

Unit	Depth (mbgl)	Cohesion (kPa)	Peak undrained shear strength (kPa) (1)	Effective friction Angle (Degrees) (2)	Relative Density (%) (3)
SILT / Clayey SILT	0.0 – 0.5	0	50	35	45
Silty SAND / Sandy SILT / Clayey SILT	0.5 – 13.0	5	80	30	30
SAND / Silty SAND / SILT	13.0 – 20.0	0	-	38	45
Sandy GRAVEL	20 >	0	-	38	65

1) Parameters estimated from CPT correlations – Lunne et al (1997), Scala penetrometer and shear vanes.
2) Parameters estimated from CPT results, shear vanes, published data (Meyerhof G.G. 1956) and experience (1956).
3) Parameters estimated from published data (NZGS guidelines, 2005) and CPT results.

These values are based on site conditions at the time of investigation and may change if the subgrade is disturbed prior to foundation construction, in which case further geotechnical assessment may be required.

It is suggested that the ground parameters listed above together with the seismic subsoil class and liquefaction assessment can be used to assess the existing residential units at 16 Dundee Place for the purposes of writing a quantitative DEE.

5.3. Seismicity

Canterbury is located in a wide zone of active earth deformation associated with collision between the Australian and Pacific plates. The nearest active fault to the site is the Greendale Fault, approximately 22 km west of central Christchurch based on the Institute of Geological and Nuclear Society (GNS) active fault database.

The design seismic actions have been evaluated in accordance with NZS1170.5:2004 considering upgraded Z factors as per recommendations by the Structural Engineering Society (SESOC) following the Canterbury Earthquakes (2010-2011).

The site has been evaluated as Class D due to the consistency and depth of the alluvial formations underlying this site. An Importance Level of 2 has been selected based on the current site use. SKM is not aware of any planned changes to the use of the site.

6. Geotechnical considerations

6.1. Liquefaction

The liquefaction potential of the site has been evaluated based on CPT results using the Modified Robertson Method published in the 1997 Proceedings of NCEER Workshop on Evaluation of Liquefaction Resistance of Soils (TL Youd, 2001).

Estimations of liquefaction-induced ground settlement have been determined using Ishihara & Yoshimine (1992) method. This is strictly an estimate due to limitations involved with the calculation, and the predicted settlements are generally regarded as conservative.

The following tables (Table 6.1 to 6.2) summarise the liquefaction potential of the site and its estimated ground settlement. A groundwater level of 1 mbgl has been used in the liquefaction analysis.

- **Table 6.1 – Evaluation of liquefaction potential from CPT results for a ULS design event (0.35g/M7.5)**

CPT	Sections that have potentially liquefiable layers (mbgl)	Potentially liquefiable thickness (m)	Estimated Ground Settlement (mm)
CPT01	1.5 – 15.2 16.2 – 19.2	16.7	670
CPT02	1.5 – 10.9 11.1 – 15.0 15.2 – 15.9 16.5 – 19.3	16.8	670

- **Table 6.2 – Evaluation of liquefaction potential from CPT results SLS design event 0.13g / M7.5**

CPT	Sections that have potentially liquefiable layers (mbgl)	Potentially liquefiable thickness (m)	Estimated Ground Settlement (mm)
CPT01	1.5 – 7.8 8.2 – 12.8 12.9 – 13.2 13.4 – 14.5 14.8 – 15.2 16.2 – 16.4 16.7 – 19.2	15.4	620
CPT02	1.5 – 10.9 11.1 – 14.8 15.5 – 15.8 16.5 – 17.2 18.0 – 18.2 18.4 – 19.1	15.0	600

Based on our recent investigation the site is unlikely to be susceptible to liquefaction in future earthquakes despite the high estimated ground settlements in the tables above. The estimates above are based upon the 1997 Proceedings of NCEER Workshop on Evaluation of Liquefaction Resistance of Soils (TL Youd, 2001). This procedure does not take into account the percentage of fines which has resulted in the high estimates of estimated ground settlement in the tables above. According to Project Orbit, aerial photography and discussions with local residents, there has been no evidence of liquefaction at the surface following the major earthquakes in the recent Canterbury earthquake sequence. No ejected material, sand boils or uneven ground was identified during the site visit.

Graphical outputs of liquefaction assessments from CPT results are provided in Appendix C for ULS and SLS design events. The results suggest that most of the material in the subsurface is cohesive in nature up to 13 mbgl and therefore does not have the potential to liquefy. It is suggested that the more silty layers (particularly at the ground surface) have confined any liquefiable material at depth preventing any material coming to the ground surface. The sand below 13 mbgl, although liquefiable, has not manifested at the surface due to the cohesive strata above preventing the upward movement of liquefied material.

6.2. Lateral spread

The site is not located near any free faces and is therefore considered to be at a negligible risk of lateral spread.

6.3. Bearing capacity

An assessment of the bearing capacity of the shallow soils can be carried out based on the findings of the Scala penetrometer results and in particular the plots of blow counts with depth. The majority

6.4. Foundations

6.4.1. General

Notwithstanding the findings of the liquefaction assessment and bearing in mind the nature of the proposed development, it is assumed that the recommendations contained within the Department of Building and Housing (DBH) guidance dated November 2011 can be adopted assuming single storey buildings with lightweight cladding and roofing.

The development comprises the construction of eight units (1-8) with associated garages, parking areas, footpaths and soft landscaping. The recommendations provided below relate to the units and any integral garages. In the case of detached garages, consideration could be given to a conventional strip footing and ground bearing slab assuming an ultimate rupture bearing capacity of 200 kPa as indicated by the Scala penetrometer test results.

As previously mentioned, the site is located within an area classified as TC2. The Scala penetrometer test results indicate an ultimate rupture bearing capacity of 200 kPa (i.e. blows counts of 2 or 3 for 100mm penetration). Based on this assessment of the ultimate rupture bearing capacity and referring to the above design guidance, it is recommended that the units are provided with foundations consisting of a TC2 compliant stiffened raft slab as outlined below.

It should be noted that all the below options require detailed consideration to be given to the service lines as they enter and travel within the slab. With careful design, provision could also be included in the design of the raft slabs for re-levelling following a major seismic event, if required.

6.4.2. Raft Options

A detailed description of the TC2 complaint raft slab options is provided in Section 5.3 of the DBH guidance. An overview is provided below.

6.4.2.1. Composite raft and gravel platform

This option involves removing the upper 800mm of soil from below the proposed raft followed by the reinstatement of the excavation to the underside of the raft with well graded and compacted granular fill with a basal geo-grid layer and possibly a further geo-grid layer at the mid-depth of the gravel platform and at least 100mm below the lowest point of the raft. The overlying raft should comprise a NZS3604 reinforced and tied slab foundation with edge beams and local thickenings beneath internal load bearing walls.

6.4.2.2. Thick slab raft

This option involves the construction of a 300mm thick reinforced slab raft with a minimum of two layers of mesh reinforcement (top and bottom). The guidance stipulates that for two storey, heavyweight structures, the thickness of the slab should be increased to 400mm.

6.4.2.3. Generic beam grid and slab formation

This option involves the construction of a 100mm thick reinforced slab supported on a 250mm thick layer of compacted gravel or polystyrene pods tied into external and internal, 600mm deep by 300mm wide, reinforced concrete beams with a maximum span between the beams of 3.5m.

6.4.2.4. Waffle slab raft

This option involves the construction of a 85mm thick slab raft supported on 300mm deep polystyrene pods and tied into 385mm deep by 300mm wide external, reinforced concrete beams and internal, 100mm wide reinforced concrete ribs at spacings not exceeding 1.2m.

6.4.3. Other foundation options

In addition to the above shallow solutions, consideration could be given to piles or ground improvement. However, both options are likely to prove more expensive than the raft slab solutions outlined above. It should be noted that detailed design of the slab rafts will be required by a qualified structural engineer using the information contained in this report.

7. Conclusions and recommendations

7.1. Conclusions

- The site is underlain by silts and sands of the Springston Formation overlying Riccarton Gravels. The subsurface strata are generally cohesive (silts/silty clays) in nature up to 13 mbgl. Sands are encountered between 13 and 20 mbgl.
- The groundwater level has been estimated to be between 1.0 and 1.4 mbgl. A conservative groundwater level of 1.0 mbgl has been used in the liquefaction assessment.
- The site has been evaluated as Class D due to the consistency and depth of the alluvial formations underlying this site.
- The liquefaction assessment indicates the potential for 670 mm of liquefaction induced total free field settlement at the site. However, this does not take into account the percentage of fines. As the subsurface mostly comprises materials with a high percentage of fines between the ground surface and 13 mbgl this material is expected to have a low susceptibility to liquefaction.
- Maurice Carter Courts are not located near any free surfaces and are therefore considered to be at negligible risk of lateral spread.
- It is suggested that the ground parameters listed in this report together with the seismic subsoil class and liquefaction assessment can be used to assess the existing residential units at 16 Dundee Place for the purposes of writing a quantitative DEE.

7.2. Recommendations

- Based on this assessment of the ultimate rupture bearing capacity and referring to the TC2 design guidance, it is recommended that the units and integral garages are provided with foundations consisting of a TC2 compliant stiffened raft slab as outlined in section 6.4.2.For the detached garages, a conventional strip footing and ground bearing floor slab should suffice.
- In addition to a shallow foundation solution, consideration could be given to piles or ground improvement. However, both options are likely to prove more expensive than the raft slab solutions outlined above.
- If significant modifications or relevelling of the existing units is required additional ground investigation is likely to be required.

8. Limitations

This report is project specific. It was prepared to address geotechnical issues relating to Maurice Carter Courts, 16 Dundee Place in accordance with the scope of works as defined in the contract between SKM and our Client. This report has been prepared on behalf of, and for the exclusive use of, our Client, and is subject to, and issued in accordance with, the provisions of the contract between SKM and our Client. The findings presented in this report should not be applied to another site or another development within the same site without consulting SKM.

Geotechnical conditions can change and will vary across any site and between investigation locations. The findings of this geotechnical report reflect the geotechnical conditions at the identified locations and at the time of the investigation. If this report is being referenced after some period of time has elapsed since it was drafted then it is recommended that SKM be consulted regarding the current validity of this report.

Not all of the ground conditions that exist at the site may have been identified in this report. All reports and conclusions that deal with sub-surface conditions are based on interpretation and judgement and as a result have uncertainty attached to them. You should be aware that this report contains interpretations and conclusions which are uncertain due to the nature of the investigations. Sampling techniques, by definition, cannot determine the conditions between the sample points and so this report cannot be taken to be a full representation of the sub-surface conditions. This report only provides an indication of the likely sub surface conditions. No study or investigation can eliminate every risk and conclusively identify all the ground conditions within a site.

This report is based on assumptions that the site conditions as revealed through sampling are indicative of conditions throughout the site. The findings are the result of standard assessment techniques used in accordance with normal practices and standards, and they represent a reasonable interpretation of the current conditions on the site.

This report should be read in full and no excerpts are to be taken as representative of the findings. It must not be copied in parts, have parts removed, redrawn or otherwise altered without the written consent of SKM.

9. References

Brown LJ and Weeber JH. 1992: Geology of the Christchurch Urban Area. Scale 1:25,000. Institute of Geological and Nuclear Sciences geological map1. 1 sheet + 104 p. Institute of Geological and Nuclear Sciences Ltd, Lower Hutt, New Zealand.

Forsyth, P.J., Barrell, D.J.A., Jongens, R. (compilers) 2008. Geology of the Christchurch area. Institute of Geological and Nuclear Sciences 1:250,000 geological map 16. 1 sheet + 67 p. Lower Hutt, New Zealand. GNS Science.

Ishihara, K. And Yoshimine, M. 1992. Evaluation of settlements in sand deposits following liquefaction during earthquakes. Soils and Foundations. Vol.32(1): 173-188.

Lunne, T., Robertson, P.K., & Powell, J.J.M. 1997. Cone Penetration Testing In Geotechnical Practice. London: Blackie Academic & Professional

Meyerhof, G. G. 1956. Penetration tests and bearing capacity of cohesionless soils. JSMFD, ASCE, vol. 82, SM1, Jan pp.1-19.

New Zealand Geotechnical Society, 2005. *Guidelines for the Field Classification and Description of Soil and Rock for Engineering Purposes*.

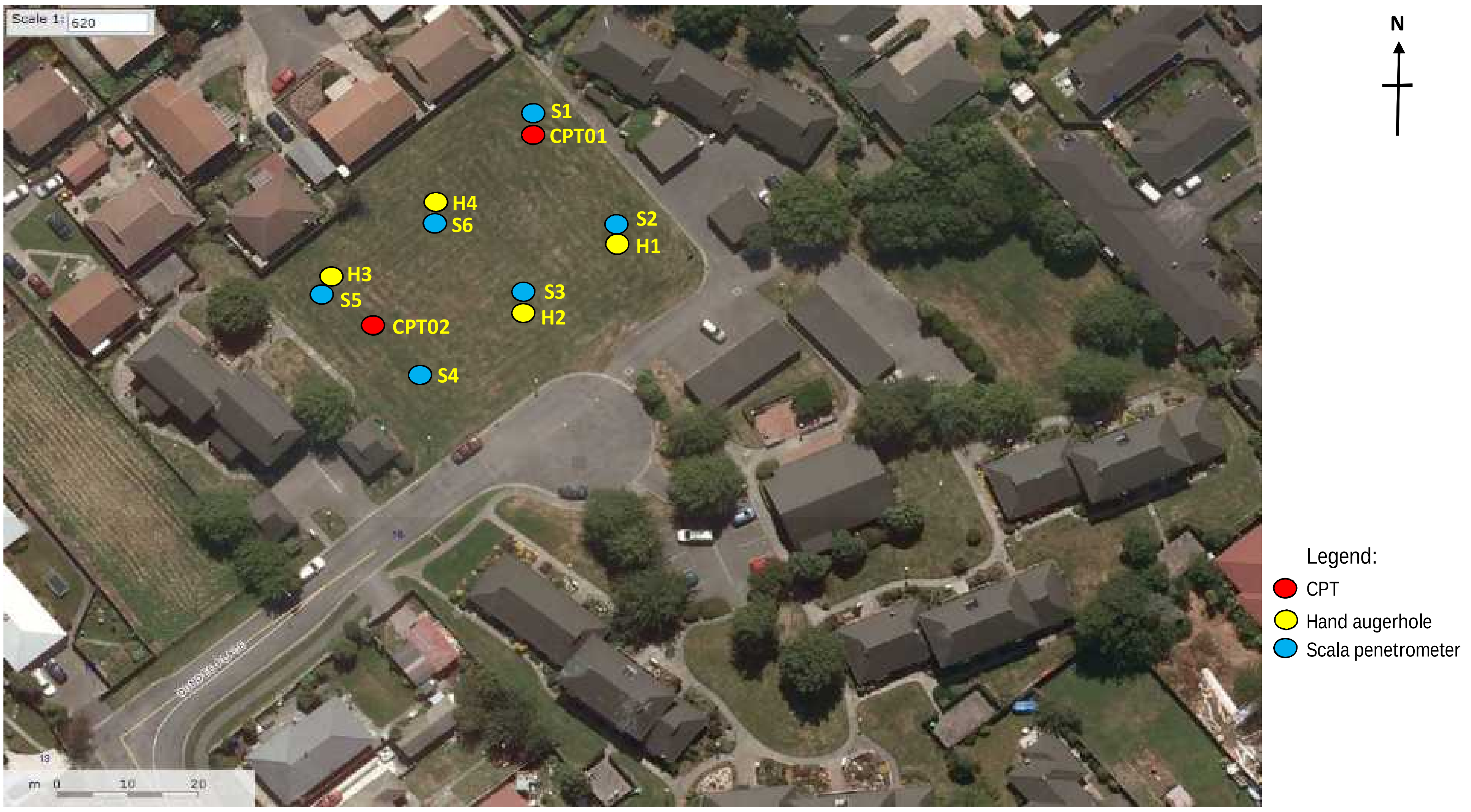
Peck et al. 1967. Foundation Engineering, 2nd Edition, John Wiley, New York, p. 310.

Robertson. 2010. Guide to Cone Penetration Testing, 4th Edition.

Terzaghi et al. 1996, Soil Mechanics in Engineering Practice, 3rd Edition, Jon Wiley & Sons Inc., New York.

Youd, TL et al. 2001. Liquefaction Resistance of Soils: summary report from the 1996 NCEER and 1998 NCEER/NSE workshops on evaluation of liquefaction resistance of soils. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, vol. 127, no. 10, Oct pp.817 – 833.

Appendix A – Site Plan



Note: scale and layout approximate only.

Client:	Christchurch City Council	Job No:	ZB01276.219
Project:	Maurice Carter Courts, 16 Dundee Place	Date:	19-Dec-12
		Scale:	N/A
Title:	Geotechnical Investigation Site Plan	By:	Jon Rabey
	SKM	Figure:	Appendix A

Appendix B – CPT logs

CPT ANALYSIS NOTES

Soil Type

Interpretation using chart of Robertson & Campanella (1983). This is a simple but well proven interpretation using cone tip resistance (q_C) and friction ratio (f_R) only. No normalisation for overburden stress is applied. Cone tip resistance measured with the piezocone is corrected with measured pore pressure (u_C).

- sand (and gravel)
- silt-sand
- silt
- clay-silt
- clay
- peat

Liquefaction Screening

The purpose of the screening is to highlight susceptible soils, that is sand and silt-sand in a relatively loose condition. This is not a full liquefaction risk assessment which requires knowledge of the particular earthquake risk at a site and additional analysis. The screening is based on the chart of Shibata and Teparaksa (1988).

- high susceptibility
- medium susceptibility
- low susceptibility

High susceptibility is here defined as requiring a shear stress ratio of 0.2 to cause liquefaction with D_{50} for sands assumed to be 0.25 mm and for silty sands to be 0.05 mm.

Medium susceptibility is here defined as requiring a shear stress ratio of 0.4 to cause liquefaction with D_{50} for sands assumed to be 0.25 mm and for silty sands to be 0.05 mm.

Low susceptibility is all other cases.

Relative Density (D_R)

Based on the method of Baldi et. al. (1986) from data on normally consolidated sand.

Undrained Shear Strength (S_U)

Derived from the bearing capacity equation using $S_U = (q_C - \sigma_{VO})/15$.

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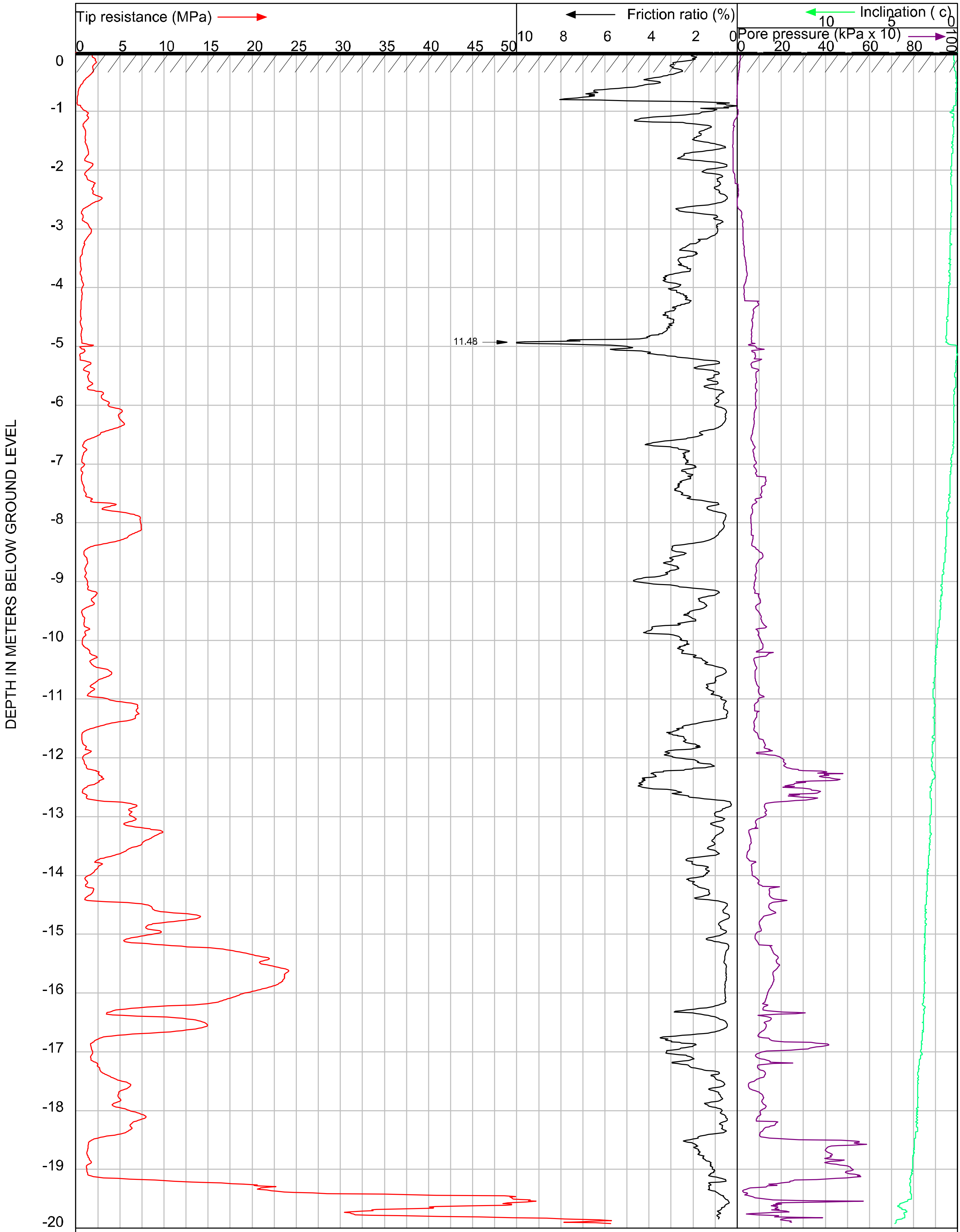
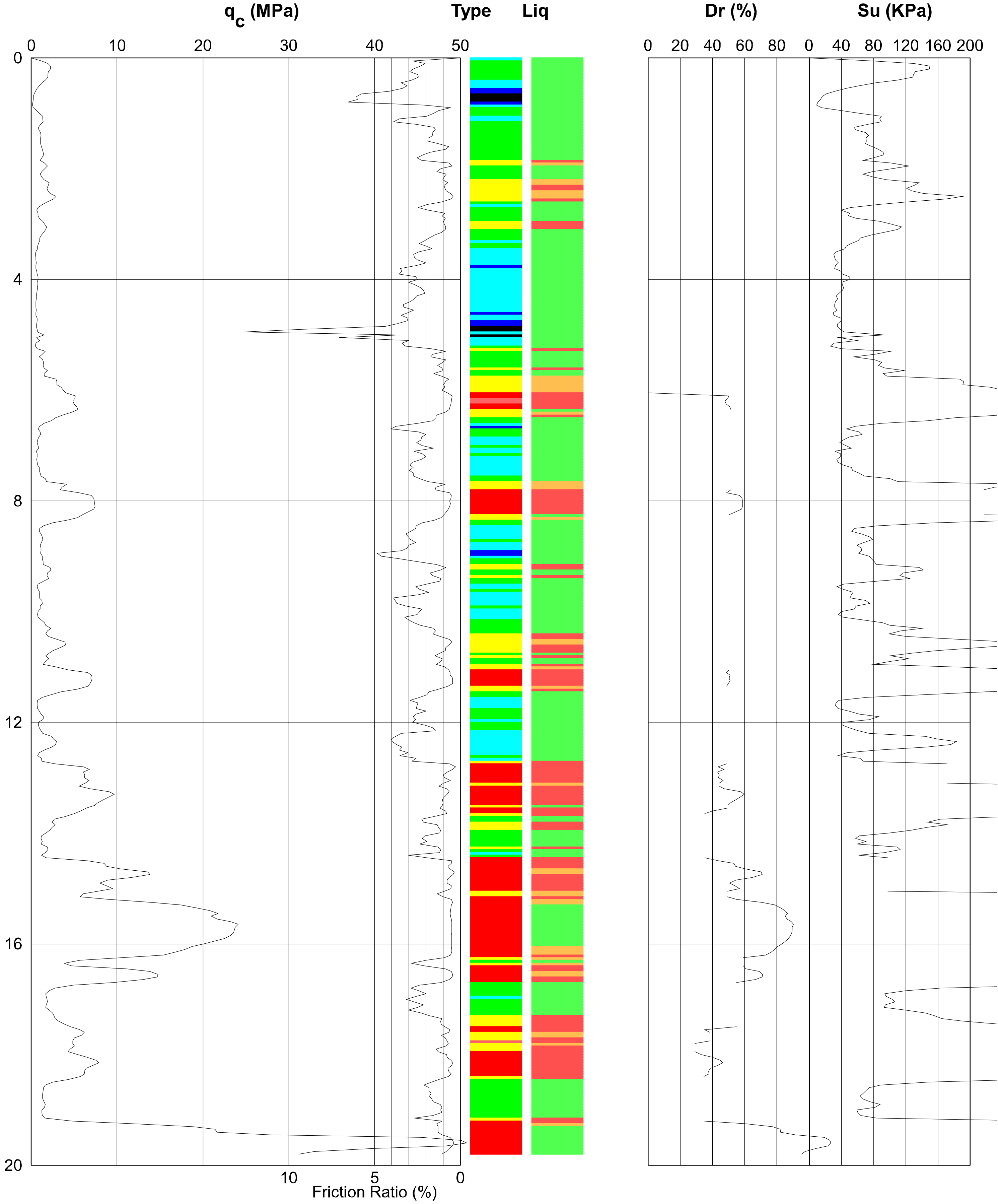


CLIENT : Sinclair Knight Merz
LOCATION : 16 Dundee Place, Christchurch
DATE : 10-12-2012
OPERATOR : S.Cardona
REMARK 1 : CPTu001
REMARK 2 : Effective Refusal

JOB # : 11448
TEST # : 1
CONE TYPE/SERIAL #: I-CFXYP20-10/ 120523T

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

PIEZOCONE PENETROMETER TEST (CPTU) INTERPRETIVE REPORT



Job No:	11448	Date:	10-12-2012
CPT No:	CPTu001	Operator:	S.Cardona
Project:	Sinclair Knight Merz	Remark:	Effective Refusal
Location:	16 Dundee Place, Christchurch		

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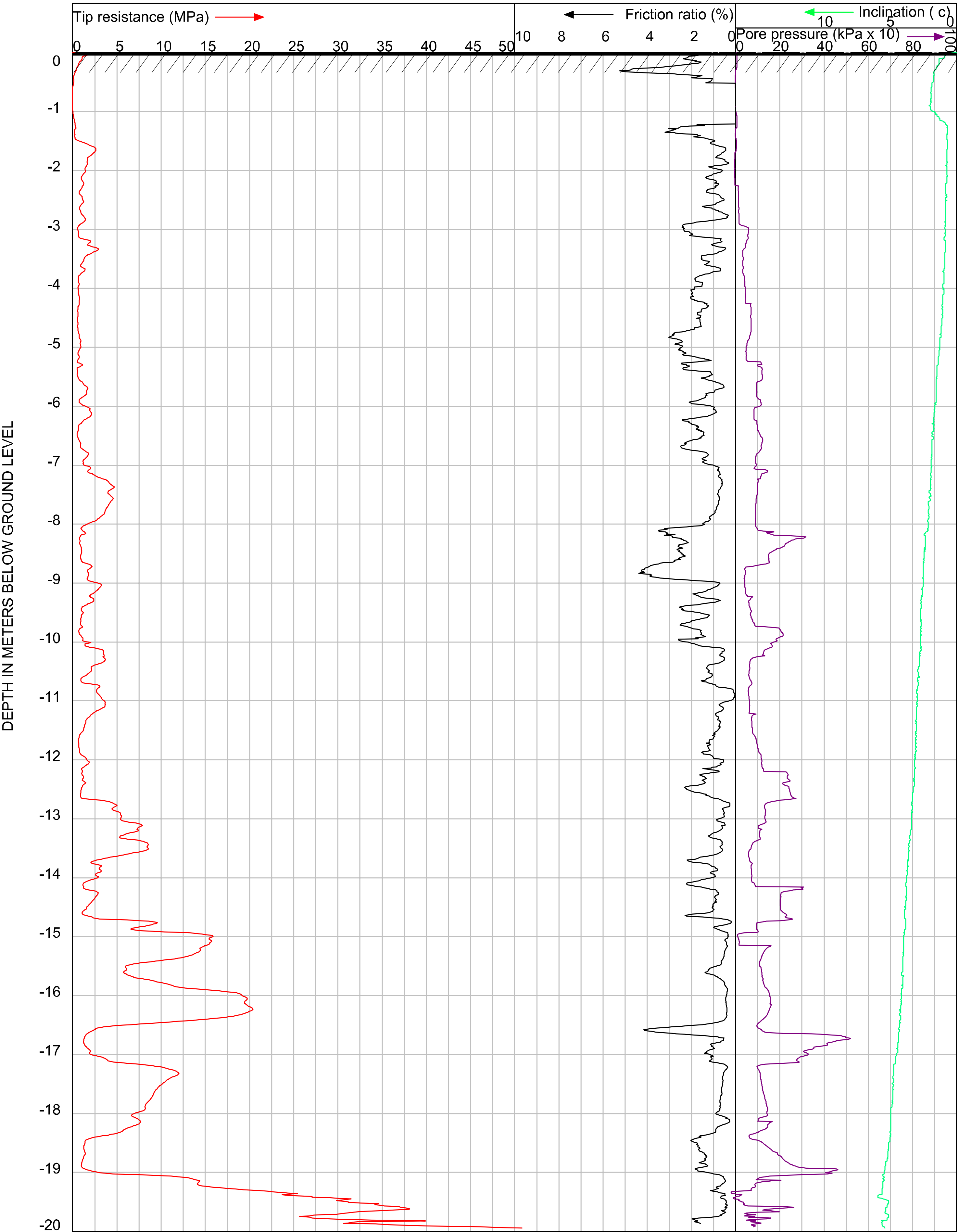

CLIENT : Sinclair Knight Merz
LOCATION : 16 Dundee Place, Christchurch
DATE : 10-12-2012
OPERATOR : S.Cardona
REMARK 1 : CPTu002
REMARK 2 : Target Depth

JOB # : 11448
TEST # : 2
CONE TYPE/SERIAL #: I-CFXYP20-10/ 120523T

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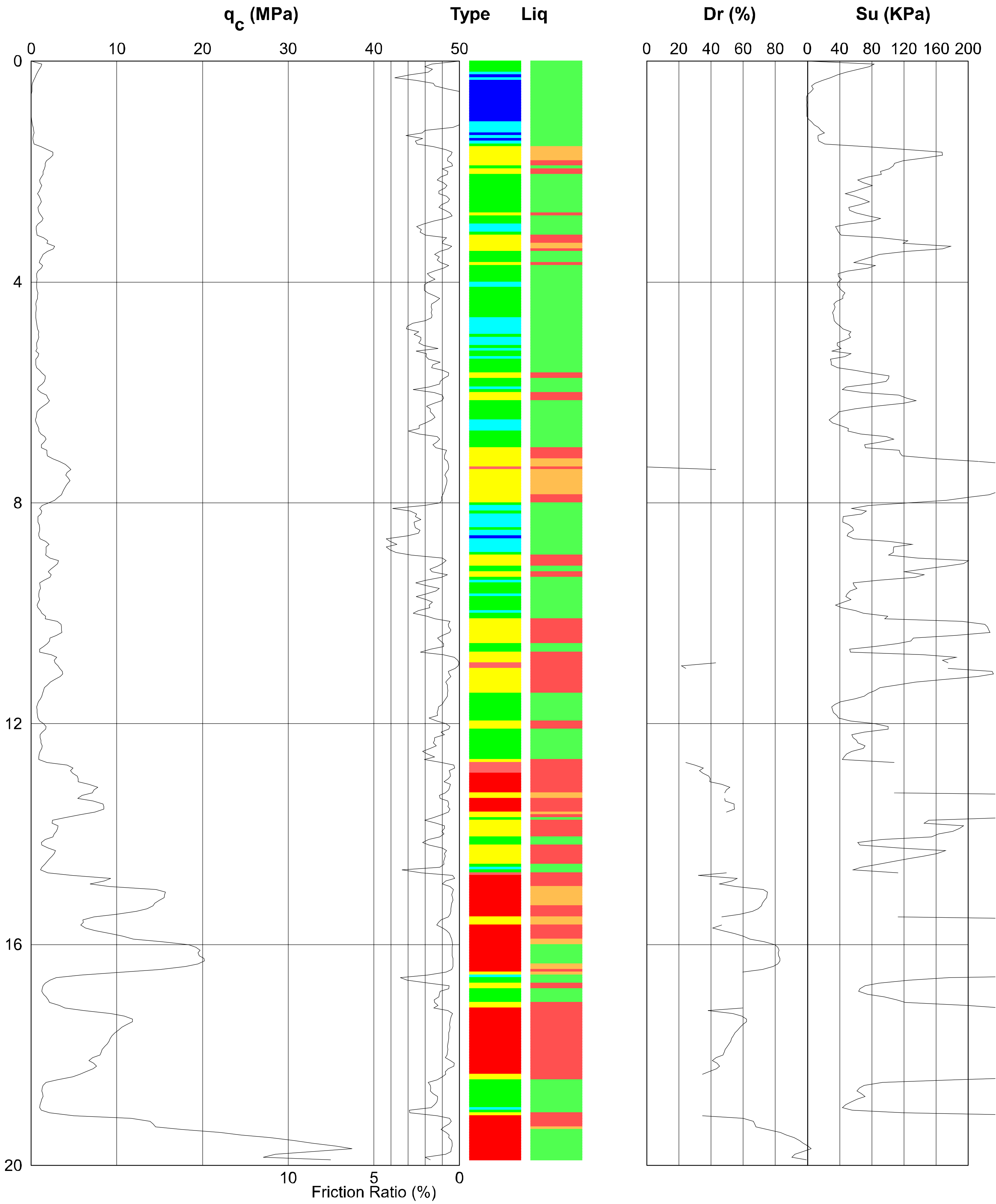
PIECOZONE PENETROMETER TEST (CPTU) INTERPRETIVE REPORT



Job No:	11448	Date:	10-12-2012
CPT No:	CPTu002	Operator:	S.Cardona
Project:	Sinclair Knight Merz	Remark:	Target Depth
Location:	16 Dundee Place, Christchurch		

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CPT CALIBRATION AND TECHNICAL NOTES

These notes describe the technical specifications and associated calibration references pertaining to the following cone types:

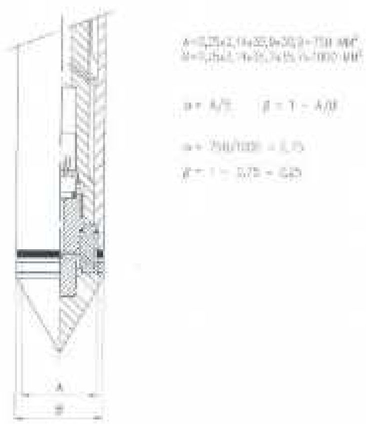
- ELCI-10CFXY measuring cone resistance, sleeve friction and inclination (standard cone);
- ELCI-CFXYP20-10 measuring cone resistance, sleeve friction, inclination and pore pressure (piezo cone).

Dimensions

Dimensional specifications for both cone types are detailed below. All tolerances are routinely checked prior to testing and measurements taken are manually recorded on CPT field sheets. All field sheets are kept on file and available on request.

A.P. van den Berg Machinefabriek b.v. tel. :0513-631355 fax. :0513-631212	DEVIATION of Straightness + MINIMAL Dimensions tip, (friction)jacket, thread adapter	Standards: EN ISO 22476-1 NEN 5140 APB standard
Type of cone:	10 cm^2	
Diameter of tip: (acc. to EN ISO 22476-1)	$35{,}3 \le d1 \le 36{,}0$	
Diameter friction jacket:	$d_1 \le d_2 < d_1 + 0{,}35$	
Tip: (production dimension)	$d_1 = 35{,}7\ ^{+0{,}2}_{0}$	
Jacket (C-cone):	$d_2 = 35{,}7\ ^{+0{,}2}_{0}$	
Friction jacket (CF-cone):	$d_2 = 35{,}9\ ^{+0{,}1}_{0}$	
Tip for used cone:	$d_1 = 35{,}5\ ^{+0{,}1}_{0}$	
Minimal diameter jacket: (C-cone)	$d_2 = 35{,}2$ (APB std.)	
Minimal diameter of friction jacket: (CF-cone)	$d_2 = 35{,}3$	
Use "used cone"-tip when friction jacket diameter:	$d_2 \le 35{,}65$	
Minimal diameter of thread adaptor:	$d = 35{,}3$	
Height dimension tip edge:	$7 \le h_e \le 10$	
Maximal deviation of straightness:	1 mm on a length of 1000 mm (max. oscillation 1,0 mm.)	

Cone surface ratio



CPT CALIBRATION AND TECHNICAL NOTES (cont.)

Calibration

Each cone has a unique identification number that is electronically recorded and reported for each CPT test. The identification number enables the operator to compare 'zero-load offsets' to manufacturer calibrated zero-load offsets.

The recommended maximum zero-load offset for each sensor is determined as ± 10% of the maximum measuring range although the more conservative trigger point adopted by McMillan Drilling Services is ± 10% of the nominal range.

In addition to maximum zero-load offsets, McMillan Drilling Services also limits the difference in zero load offset before and after the test as ± 1% of the maximum measuring range. See table below:

	Tip (MPa)	Friction (MPa)	Pore Pressure (MPa)
Maximum Measuring Range:	150	1.50	3.00
Nominal Measuring Range:	100	1.00	2.00
Max. 'zero-load offset':	10	0.10	0.20
Max 'before and after test':	1.5	0.015	0.03

Note: The zero offsets are electronically recorded and reported for each test in the same units as that of each sensor.

TEST CERTIFICATE
Icone (all versions)

Supplier:	A.P. v.d. Berg Machinefabriek, Heerenveen The Netherlands
Production-order:	57152
Client:	McMillan
Cone-type:	ELCS-CFXYP 20-10
Cone-number:	120523

To test / To check item	Required value	Checked value
Isolation-resistance	>0.5 G-Ohm	. Gohm
Straightness	S=<0,2 mm	1 mm
Zero-Value Tip	Good	-4,546 MPa
Zero-Value Local Friction	Good	-0,1696 MPa
Zero-Value Pore Pressure	Good	-233 kPa
Zero-Value Inclination X Zero-Value Inclination Y	-2°< X <+2° -2° < Y <+2°	-0,1 ° 0,1 °
Measurements Tip resistance OK?	Yes	0-50 MPa
Influence of Tip on Local Friction and Pore Pressure? Tip: **Max Load**; Mantle free? 10cm2:**150 kN.** // 15 cm2:**225 kN.**	No influence	
Measurements Local Friction OK?	Yes	0-0,75 MPa
Local Friction: **Max Load**	O.K.	
Measurements Pore Pressure OK?	Yes	0-2000 kPa
Measurements Inclination OK?	Yes	± 1,2°
Cone recognition on disconnecting and connecting Icone again?	Yes	
Software version **1.8** installed? Check at opening screen. **Uitzondering: GEO LYNBY gebruikt v. 1.7 ! NOTEER versienr.**	Version:	1.8
Check alarm-settings Icone. Alarm values are set. (Kill Shutdown)	O.K.	

Remarks:

Calibrated by:	J.E. Tenhage	Date:	14.05.12	Sign.:	[signature]
Final check:	C.J. Ouweman	Date:	15.05.12	Sign.:	[signature]

Appendix C – Hand auger logs

Preliminary Log of Investigation

Project: **Maurice Carter Courts**

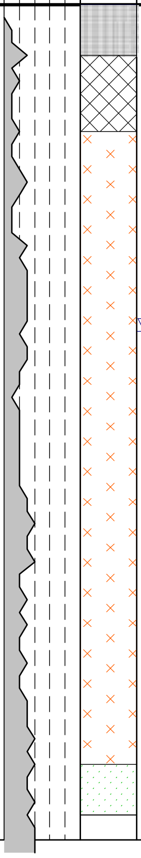
Location: **16 Dundee Place** Project No: **ZB01276.219**

Hole ID: **H1**

Client: **Christchurch City Council**

Date: **11/12/2012**

R.L. (m)	Depth (m)	Drilling Method (Shift Details / Casing Diameter (mm))	In-Situ Testing	Sampling	DCP (Blows per Drive) 2 4 6 8	Geology Legend	GroundWater	Description of Strata	Geological Unit	Backfill / Installation
								SILT with trace sand, brown. Soft, dry, low plasticity **(Topsoil)**	R	
								SILT with gravel. Soft, dry, low plasticity. Gravel is fine to medium, subangular **(Fill)**	Q1nc	
			I_{vp}65/I_{vr}26					*0.20m: With subangular, fine to medium gravel (Fill)*		
								0.30m: Minor sand. Sand becomes coarse.		
	1.0		I_{vp}207/I_{vr}207					*0.40m: Absence of gravel*		
								SILT, grey mottled orange. Soft, dry, low plasticity **(Alluvium)**	Q1a	
							1	*0.60m: Becomes moist, moderate plasticity.*		
			I_{vp}39/I_{vr}25					*0.90m: Becomes high plasticity*		
								1.00m: Becomes low plasticity, sandy.		
								1.10m: Becomes firm, moderate plasticity.		
	2.0		I_{vp}23/I_{vr}19					*1.30m: Becomes high plasticity.*		
								1.80m: Becomes very soft, wet.		
			I_{vp}58/I_{vr}48							
	3.0							SAND with silt, grey. Loose, wet **(Alluvium)**	Q1a	



H1 terminated at 3.30m. Target Depth

Started: 11/12/2012

Finished: 11/12/2012

Driller: N/A

Plant:

Logged: JR

Checked: LAB

Depth Related Remarks

From *Remarks*

Groundwater Observations

No.	*Struck (m)*	*Date*	*Observations*	*Standing (m)*
1.	1.3m	11/12/2012		

Remarks

NZTM coordinates derived from aerial photography.

Co-ordinates:

5177801.00mN

1567704.00mE

Inclination: -90°

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

Preliminary Log of Investigation

Project: **Maurice Carter Courts**

Location: **16 Dundee Place** Project No: **ZB01276.219**

Hole ID: **H2**

Client: **Christchurch City Council**

Date: **11/12/2012**

R.L. (m)	Depth (m)	Drilling Method (Shift Details / Casing Diameter (mm))	In-Situ Testing	Sampling	DCP (Blows per Drive)	Geology Legend	GroundWater	Description of Strata	Geological Unit	Backfill / Installation
								SILT with trace sand, brown. Soft, dry, low plasticity. **(Topsoil)**	R	
								0.20m: Sand becomes fine to medium.		
			I_{vp}134/I_{vr}61					SILT with minor sand, grey mottled orange. Firm, dry, low plasticity. **(Alluvium)**		
								0.60m: Becomes soft, moderate plasticity. Absence of sand.		
								0.70m: Becomes moist. Trace of sand		
	1.0		I_{vp}135/I_{vr}55					*0.90m: Becomes firm*		
							1	*1.15m: Becomes high plasticity. Absence of sand.*		
			I_{vp}104/I_{vr}30					*1.40m: Becomes wet.*		
									Q1a	
	2.0		I_{vp}80/I_{vr}35					*1.80m: Becomes very soft. Trace of sand.*		
								2.40m: Becomes sandy, grey.		
								2.70m: Wood fragments.		
	3.0		I_{vp}90/I_{vr}84							
								SAND with silt. Loose, wet. **(Alluvium)**	Q1a	

H2 terminated at 3.30m. Target Depth

Started: 11/12/2012 Finished: 11/12/2012 Driller: N/A Plant: Logged: JR Checked: LAB	Depth Related Remarks *From* *Remarks*	Groundwater Observations *No. Struck (m) Date Observations Standing (m)* 1. 1.4m 11/12/2012	Co-ordinates: 5177789.00mN 1567692.00mE Inclination: -90°
	Remarks NZTM coordinates derived from aerial photography.		

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

Preliminary Log of Investigation

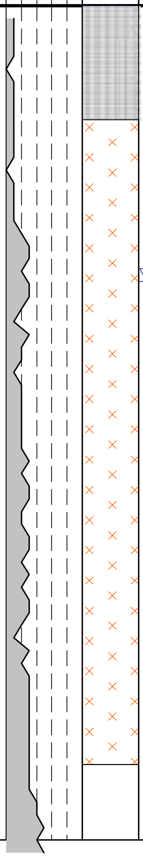
SKM

Project: Maurice Carter Courts

Location: 16 Dundee Place | **Project No:** ZB01276.219 | **Hole ID:** H3

Client: Christchurch City Council | **Date:** 12/12/2012

R.L. (m)	Depth (m)	Drilling Method (Shift Details / Casing Diameter (mm))	In-Situ Testing	Sampling	DCP (Blows per Drive) 2 4 6 8	Geology Legend	GroundWater	Description of Strata	Geological Unit	Backfill / Installation
	0.0–0.45							SILT with trace sand, brown. Soft, dry, low plasticity. **(Topsoil)**	R	
			I_{vp}54/I_{vr}38					SILT with sand, grey mottled orange. Soft, moist, low plasticity. **(Alluvium)**	Q1a	
								0.50m: Becomes moderate plasticity. Trace of sand.		
	1.0		I_{vp}64/I_{vr}26				1 ▽			
								1.10m: Becomes wet.		
			I_{vp}57/I_{vr}39							
								1.50m: Becomes sandy.		
								1.60m: Becomes high plasticity.		
	2.0							*2.00m: Becomes sandy, low plasticity.*		
								2.30m: Becomes high plasticity, absence of sand.		
			I_{vp}26/I_{vr}26							
	3.0		I_{vp}62/I_{vr}46					*2.90m: Becomes sandy, low plasticity.*		
								3.00m: Becomes high plasticity. Absence of sand		
								H3 terminated at 3.30m. Target Depth		



Started:	12/12/2012
Finished:	12/12/2012
Driller:	N/A
Plant:	
Logged:	JR
Checked:	LAB

Depth Related Remarks
From *Remarks*

Groundwater Observations

No.	*Struck (m)*	*Date*	*Observations*	*Standing (m)*
1.	1.1m	12/12/2012		

Co-ordinates:
5177796.00mN
1567661.00mE

Inclination: -90°

Remarks
NZTM coordinates derived from aerial photography.

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

Preliminary Log of Investigation

SKM

Project: **Maurice Carter Courts**

Location: **16 Dundee Place** | Project No: **ZB01276.219** | Hole ID: **H4**

Client: **Christchurch City Council** | Date: **12/12/2012**

R.L. (m)	Depth (m)	Drilling Method (Shift Details / Casing Diameter (mm))	In-Situ Testing	Sampling	DCP (Blows per Drive) 2 4 6 8	Geology Legend	GroundWater	Description of Strata	Geological Unit	Backfill / Installation
								SILT with trace sand, brown. Soft, dry, low plasticity. Sand is fine. **(Topsoil)**	R	
			I_{vp}101/I_{vr}49					SILT with gravel. Soft, dry, low plasticity. Gravel is fine to medium, subangular **(Fill)** *0.20m: With subangular gravel.* *0.30m: Becomes dark brown.* *0.40m: Absence of gravel*	Q1nc	
	1.0		I_{vp}58/I_{vr}33 I_{vp}196/I_{vr}98 I_{vp}88/I_{vr}45 I_{vp}47/I_{vr}34				1	SILT with trace sand, grey mottled orange. Soft, moist, high plasticity. Sand is fine. **(Alluvium)** *1.30m: Becomes wet.* *1.50m: Becomes firm.* *1.60m: Becomes stiff.* *2.00m: Becomes firm.* *2.40m: Becomes soft.*	Q1a	
	2.0							Silty SAND, grey. Loose, wet. **(Alluvium)**	Q1a	
								SILT, grey mottled orange, soft, wet, high plasticity **(Alluvium)**	Q1a	
	3.0		I_{vp}52/I_{vr}47					Becomes sandy **(Alluvium)**	Q1a	

H4 terminated at 3.30m. Target Depth

Started: 12/12/2012
Finished: 12/12/2012
Driller: N/A
Plant:
Logged: JR
Checked: LAB

Depth Related Remarks
From *Remarks*

Groundwater Observations

No.	*Struck (m)*	*Date*	*Observations*	*Standing (m)*
1.	1.3m	12/12/2012		

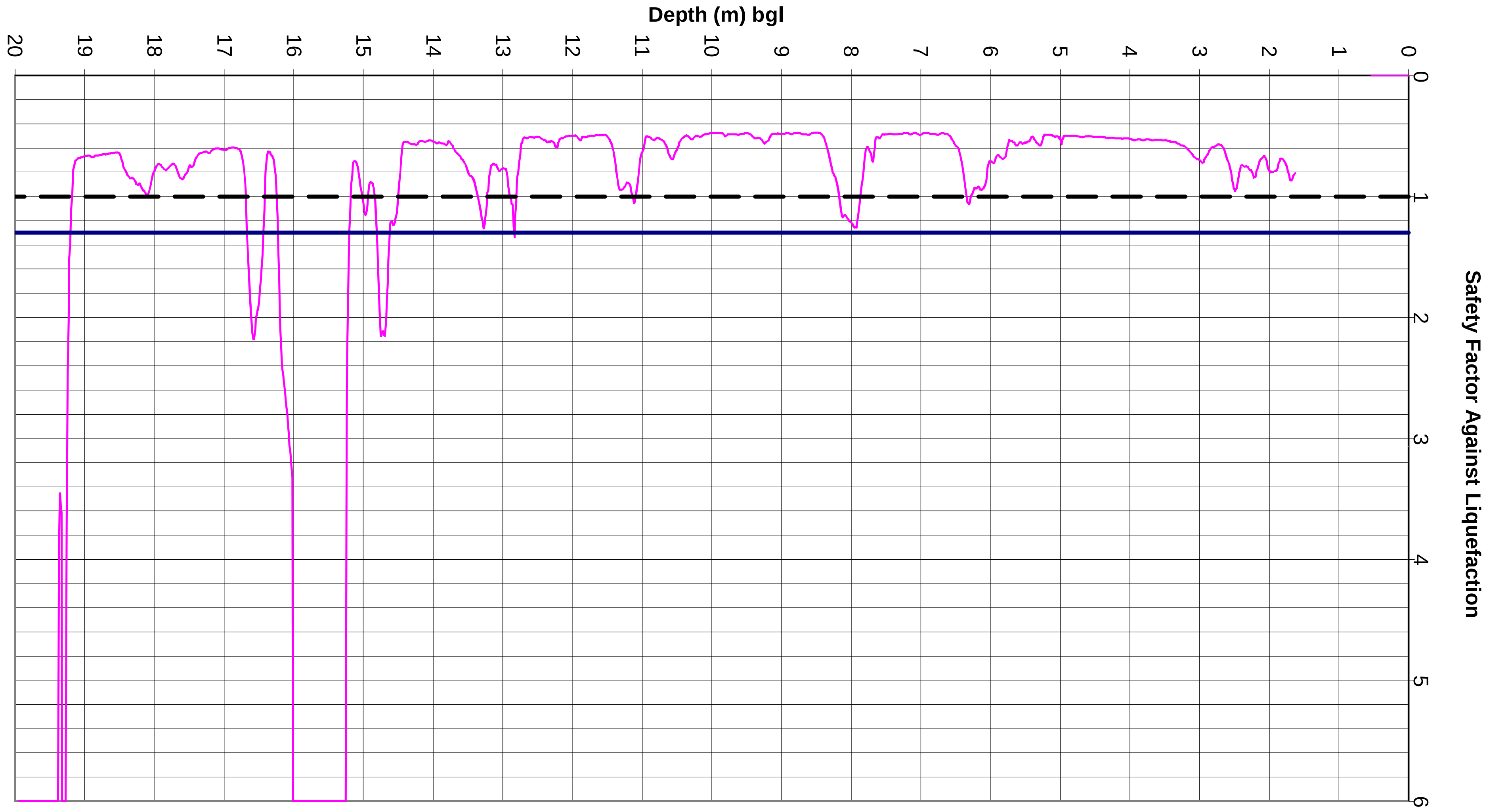
Remarks
NZTM coordinates derived from aerial photography.

Co-ordinates:
5177807.00mN
1567676.00mE

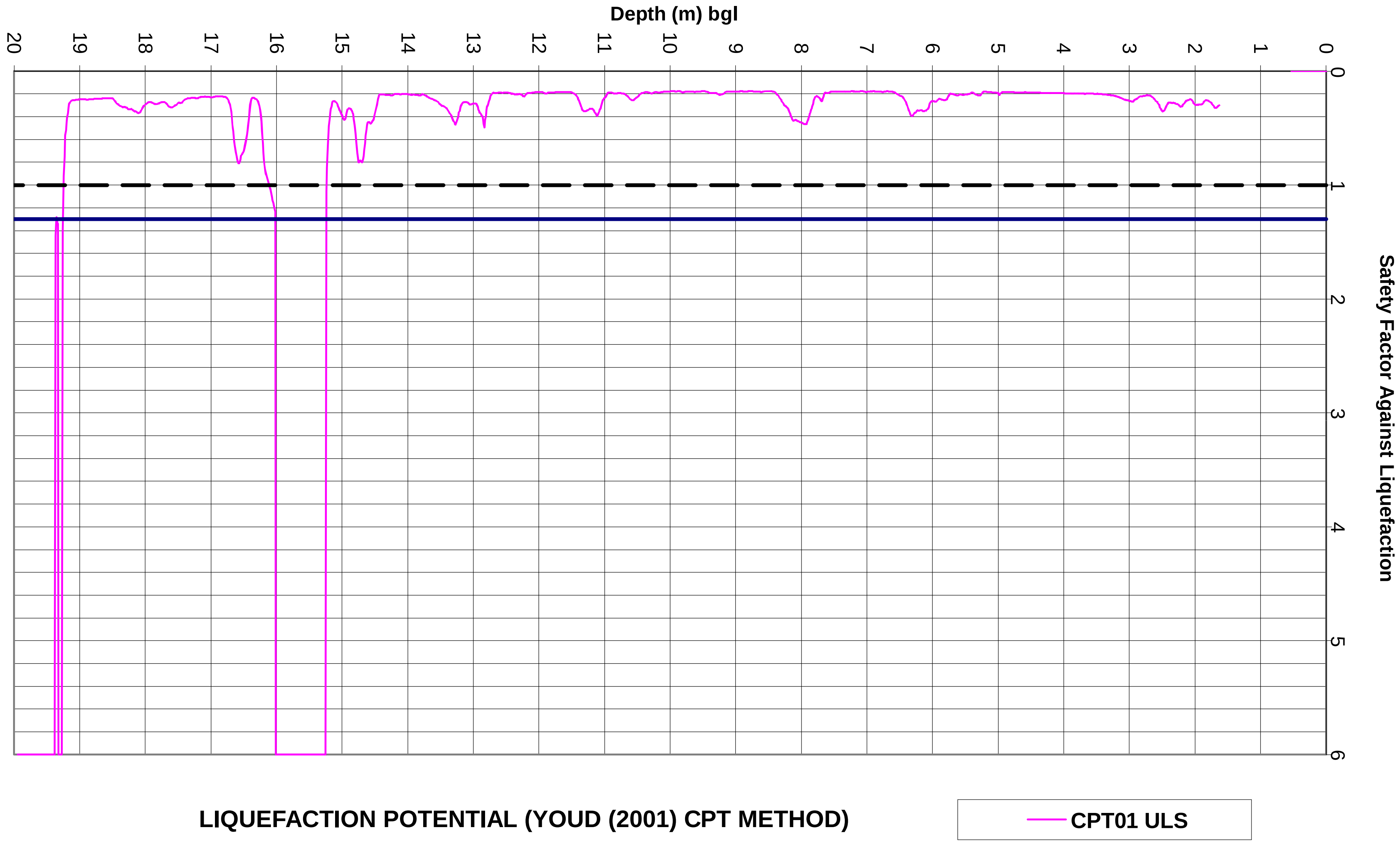
Inclination: -90°

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

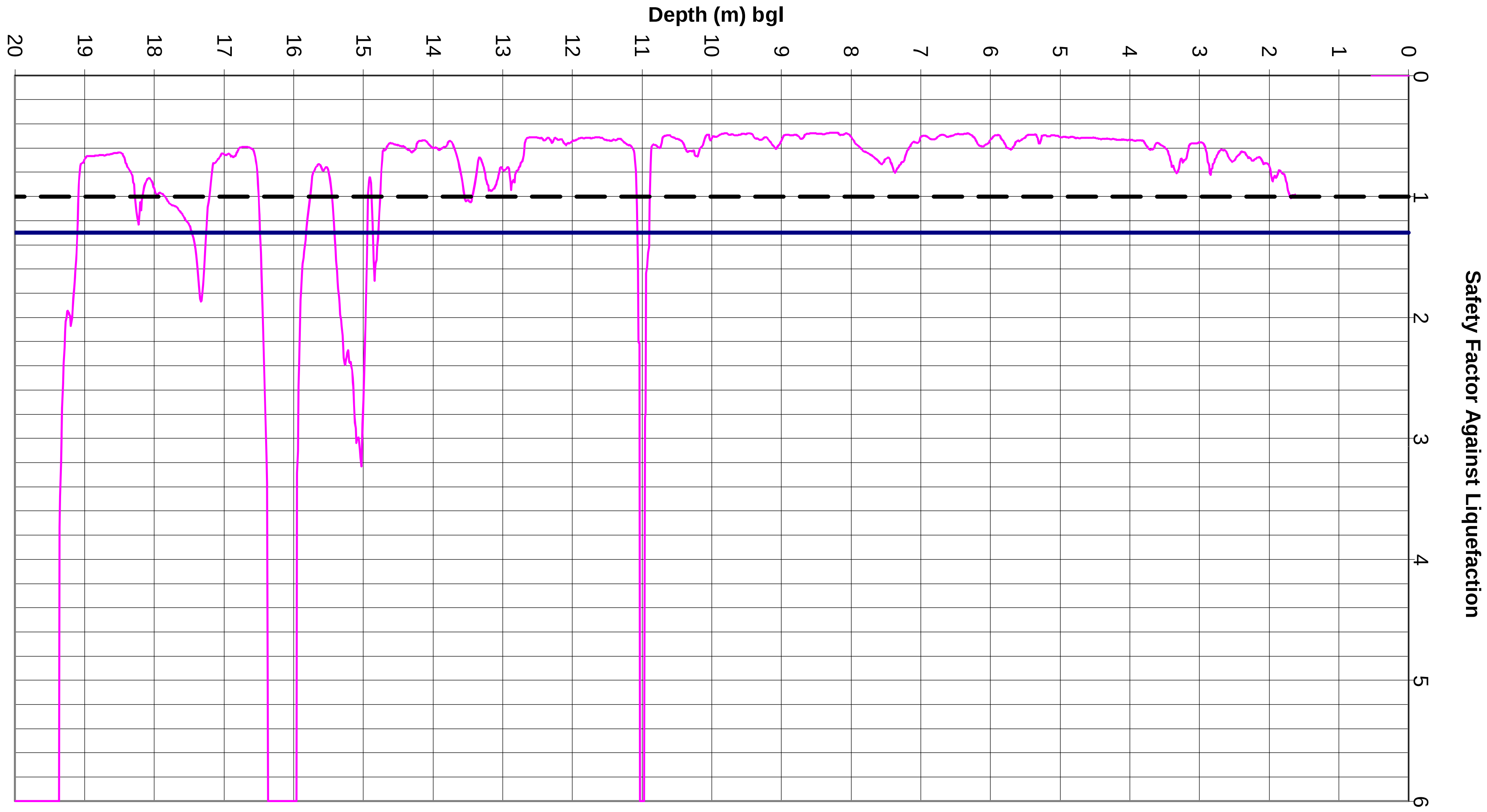
Appendix D – Liquefaction Analysis



LIQUEFACTION POTENTIAL (YOUD (2001) CPT METHOD)

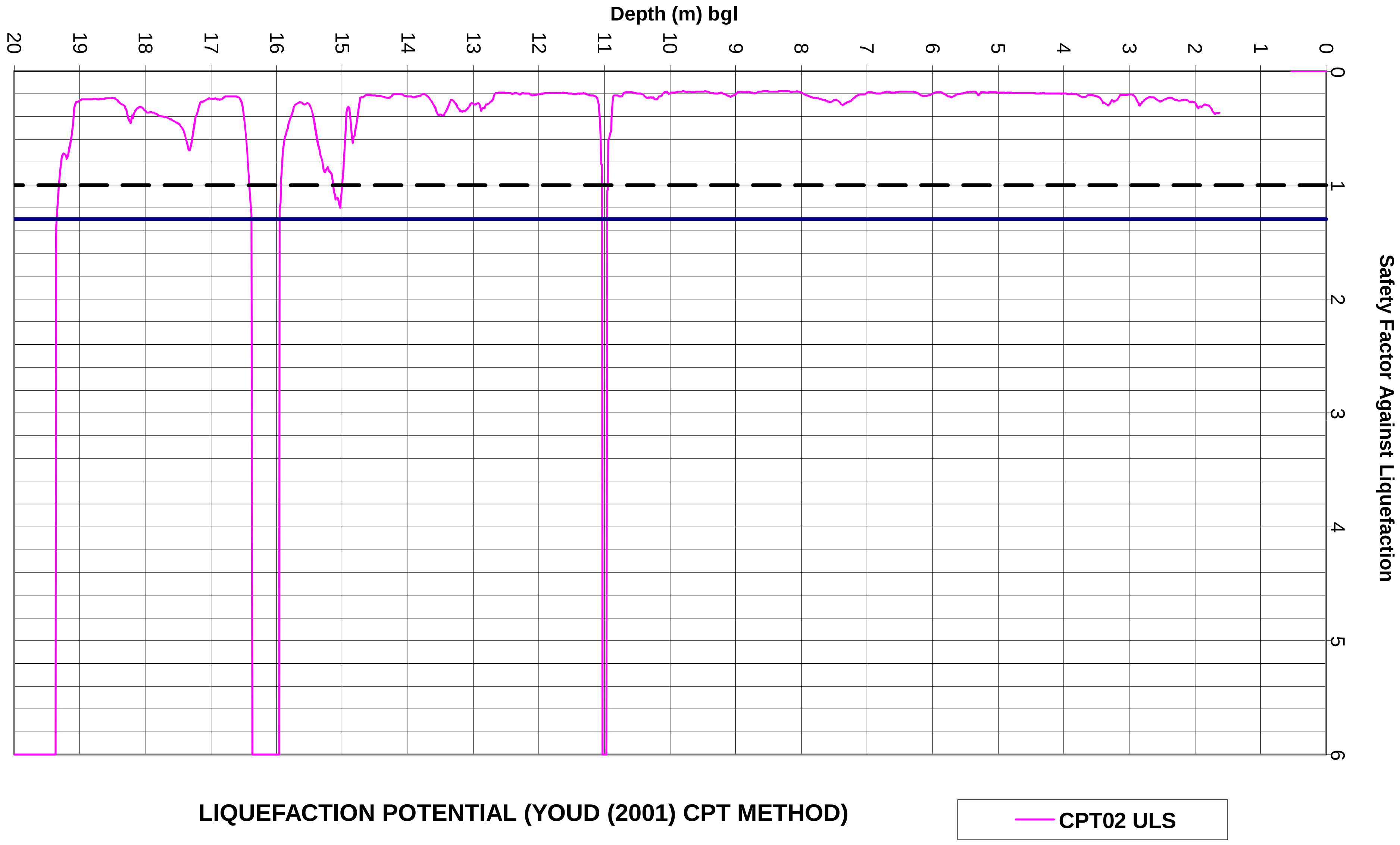


LIQUEFACTION POTENTIAL (YOUD (2001) CPT METHOD)



LIQUEFACTION POTENTIAL (YOUD (2001) CPT METHOD)

CPT02 SLS



LIQUEFACTION POTENTIAL (YOUD (2001) CPT METHOD)

Appendix F   Structural Calculations

SKM

	Job No. 2301276.215
	Calc. Series
Client CCC	Page 1
Job Name Maurice Courts	By KS
Calcs Title Block A, B, C, D	Date 19/4/13

Block A Summary:

- μ = 2 for timber framed & μ = 1.25 for concrete walls
- Cd(T) = 0.37 & 0.71 (IL 2, T ≤ 0.5sec, Soil Class D)
- No live loads on Roof, 2kPa at 1st Floor.
- Assume party wall connection okay because exact connection between party wall & ceiling diaphragm is unknown, will need to confirm.
- Gib bracing capacity from "Gib Ezy Brace System" catalogue, June 2011.
- Design guides used - AS/NZ 1170, NZS 3101, NZSEE
- NZS 3604, NZS 3603

Results:

				Timber	Timber + Concrete
Longitudinal	%NBS	GF	=	44% NBS	62%
		1st	≥	100% NBS	12%
Transverse	%NBS	GF	≥	53% NBS	≥ 100%
		1st	≥	51% NBS	≥ 100%

Diaphragm	%NBS	≥	100%
Timber Studs	%NBS	≥	100%

Blocks A:

• Roof Build up:

- Roofing iron - 0.15 kPa
- Building paper, netting, insulation - 0.03 kPa
- 90 x 45 purlins @ 900 crs - 0.02 kPa
- Timber trusses @ 1200 crs - 0.10 kPa

Σ 0.30 kPa

Area 1 - 17.5 x 7.5 = 131 m² ⇒ 40 kN

2 - 5.8 x 7.5 = 44 m² ⇒ 13 kN

3 - 14.0 x 7.5 = 105 m² ⇒ 32 kN

• Ceiling:

- 13mm Gib-board - 0.07 kPa
- 75 x 35 Blocking @ 600 crs - 0.02 kPa
- Insulation & Services - 0.05 kPa

Σ 0.15 kPa

Area 1 - 131 m² ⇒ 20 kN

2 - 44 m² ⇒ 7 kN

3 - level 1 - 5.8 x 7.5 = 44 m² x 2 ⇒ 13 kN

Roof - 87 m² ⇒ 13 kN

Σ 138 kN

SKM

Job No. 6130/276-218
Calc. Series
Client
Page 3
Job Name
By KS
Calcs Title
Date 19/4/13

- 1st Floor:
  - 75mm Unispan - 1.88kPa
  - 105mm topping - 2.52kPa Σ 4.4kPa

Area - level 1 - 87m² ⇒ 383 kN

- Stair:
  - 200 thick concrete - 4.8kPa

Area - 6.5m long × 2.02m = 12.5m² ⇒ 60 kN

- Patio
  - 75mm Unispan - 1.88kPa
  - 75mm topping - 1.8 kPa

Σ 3.68kPa

Area - (2.5 + 2.5 + 2.25) × 2.2m = 16m² ⇒ 59 kN

- Concrete Walls
  - 120mm thick - 2.88kPa

Yellow wall Area - 7.5m × 3.6m × 2 walls ⇒ 155 kN
Green wall Area - 7.5m × 5.1m × 2 walls ⇒ 220 kN
Pink wall Area - 7.5m × 5.1m × 2 walls ⇒ 220 kN
+ ½ × 3.75m × 2m × 4 walls ⇒ 43 kN
Pink wall Area - 5.7m × 2.2m × 1 wall ⇒ 36 kN
4.5

Σ 1176 kN

SKM

Job No. ZB01276.218

Calc. Series

Client

Page 4

Job Name

By KS

Calcs Title

Date 19/4/13

- Internal timber walls

  - 70 x 35 Studs @ 600 crs - 0.02 kPa
  - 70 x 35 dwangs @ 800 crs - 0.015 kPa
  - 10mm Gib board both sides - 0.14 kPa

  Σ 0.175 kPa

Length per unit (8 units)

- 2.7m + 1.4m + 1.2m + 1.0m + 1.0m + 1.0m = 8.3m

Height = 2.4m

GF weight = 6 units × 2.4 × 8.3m × 0.175 kPa ⇒ 21 kN

1st Floor weight = 2 units × 2.4 × 8.3m × 0.175 kPa ⇒ 7 kN

- Cladding

  - Weatherboard - 0.25 kPa

Length per unit

- 2m + 2.5m = 4.5m

Height = 2.2m

GF weight = 6 units × 4.5m × 2.2m × 0.25 kPa → 15 kN

1st floor = 2 units × " " " → 5 kN

- Cladding

  - Brick Veneer - 70mm clay - 1.35 kPa

length per unit

- 1.0 + 5.7 = 6.7m

Height = 2.2m

Σ 48 kN

SKM

Job No. ZB01276.218

Calc. Series

Client

Page 5

Job Name

By KS

Calcs Title

Date 19/4/13

End units

7.5m long.

GF weight = 6units × 2.2 × 6.7 × 1.35 ⇒ 165kN

+ 2unit × 2.2 × 7.5 × 1.35

1st floor = 2units × 2.2 × 6.7 × 1.35 ⇒ 40kN

- Weatherboard end units

7.5m long

1st floor = 2units × 2.2 × 7.5 × 0.25kPa ⇒ 8.5kN

Σ 214kN

SKM

Job No. 2301176.218

Calc. Series

Client

Page 6

Job Name

By KS

Calcs Title

Date 19/4/13

- External Framed Walls

- 90 x 45 studs @ 600crs	-	0.03kPa
- 90 x 45 dwangs @ 800crs	-	0.02kPa
- Insulation & services	-	0.05kPa
- 10mm Gibbard 1side	-	0.07kPa
		0.17kPa

Length per unit (8 units)

- $(7.5 \times 2) + (5.7 \times 2) = 26m$

Height = 2.4m

1st Floor - 2units × 0.17 × 26 × 2.4 ⇒ 21kN

GF - 6units × 0.17 × 26 × 2.4 ⇒ 64kN

Live load

1st floor - 2kPa

Area - 3.7 × 7.5 × 2units ⇒ 171kN

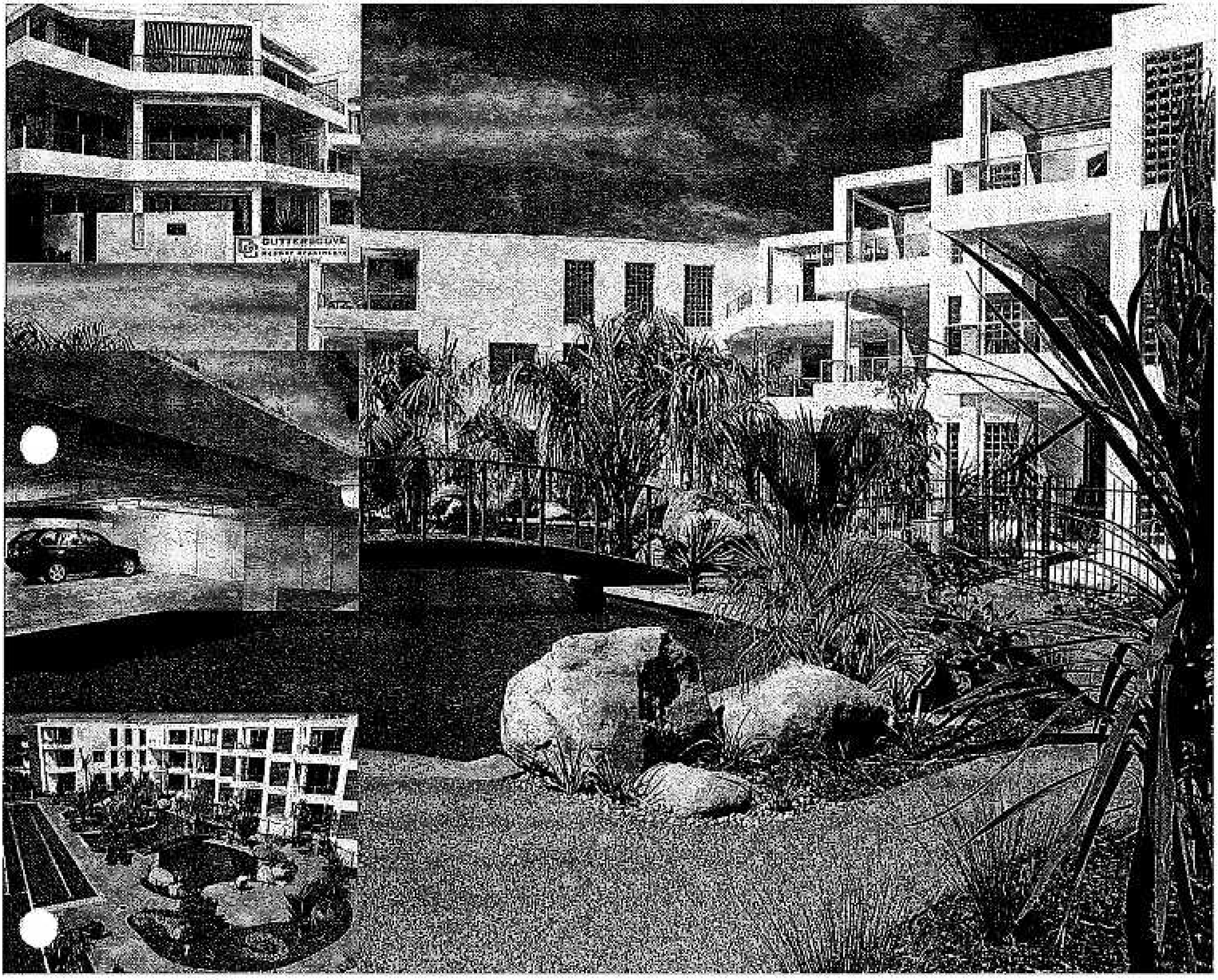
× 0.3 = 51kN

$ 136kN

total weight = 136 + 138 + 1176 + 48 + 214kN

TW = 1712kN

ZB01276.218
page 7



UNISPAN SECTION PROPERTIES

Section properties are based on a 1200mm wide section of floor. The modular ratio for topping concrete is assumed to be 0.71 for 20MPa concrete for calculating the composite section properties A, Y_b, I, Z_b and Z_t.

	Area x10^3 mm^2	Y_b mm	I x10^9 mm^4	Z_b x10^6 mm^3	Z_t x10^6 mm^3	1200mm wide mass kg/m	weight kPa
Bare unit	90	37.5	0.042	1.12	1.12	230	1.88
With 65mm topping	145	64	0.230	3.61	3.03	430	3.51
With 75mm topping	154	68	0.283	4.15	3.44	461	3.77
With 90mm topping	167	75	0.377	5.02	[illegible]	507	4.14
With 110mm topping	184	84	0.532	6.31	5.25	568	4.64
With 130mm topping	201	94	0.723	7.72	6.54	629	5.15

Cutters Cove, Mount Maunganui. Hollowcore and Unispan floors were used along with precast Shell Beams. A lap pool, lagoon pool, spa, gym and sauna are some of the many luxurious features at Cutters Cove.

ZB01276.218
page 8

GIB® Standard plasterboard is an economical lining material available in 10mm and 13mm thicknesses.

13mm GIB® Standard plasterboard is recommended for use on ceilings as it is stronger and more rigid than 10mm GIB Board® Standard plasterboard.

OCTOBER 2010 -
NEW GIB® Standard - learn more about the new re-engineered GIB® Standard plasterboard

Safety Data Sheet
GIB Plasterboard
24/02/12

Environmental Choice Certification
01/05/12

Literature Archive

masterspec

Download Masterspec for GIB® Plasterboard

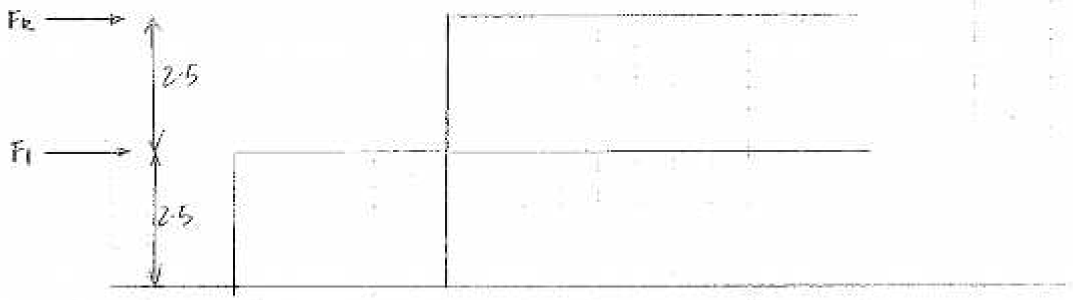
GIB® Standard sheet sizes

	Board Thickness (mm)	Sheet Width (mm)	2400	2700	3000	3300	3600	4200	4800	6000	Max kg/m2
TE/TE	10	1200									7.0
TE/TE	13	1200									8.7
TE/SE	10	1200									7.0
GIB Wideline TE/SE	10	1350									7.0
GIB Wideline TE/SE	13	1350									8.7

SKM

Job No.	9130-1276.218
Calc. Series	
Client	
Page	9
Job Name	
By	KS
Calcs Title	
Date	14/4/13

Total weight



- F_1 = half wall height level 1 + half wall height GF + Roof GF + 1st Floor

$$F_1 = \underbrace{40 + 13}_{\text{Roof}} + \underbrace{20 + 7 + 13}_{\text{ceiling}} + \underbrace{383 + 60 + 59}_{\text{1st Floor, Stair, Patio}}$$

$$+ \underbrace{155/2 + 220/2 + 220/2 + 36}_{\text{concrete walls}}$$

$$+ 21/2 + 7/2 + 15/2 + 5/2 + 165/2 + 40/2 + 8.5/2$$

$$+ 21/2 + 64/2 + 0.3 \times 171$$

F_1 = 1153 kN

- F_R = Roof at level 1 + half level 1 walls

$$F_R = \underbrace{32 + 13}_{\text{Roof + ceiling}} + \underbrace{220/4 + 220/4 + 43}_{\text{concrete walls}}$$

$$+ 7/2 + 3/2 + 40/2 + 8.5/2 + 21/2$$

F_R = 239 kN

$F_G = 21/2 + 15/2 + 165/2 + 64/2 + 135/2 + 220/2$

$F_G = 320\,kN$

Total weight = 1153 + 239 + 320 = 1712 kN

Equivalent Static Method

$$F = (0.08 V_b) + 0.92 V_b \frac{F w_i h_i}{\Sigma F w_i h_i}$$

Level	hi	Wi	Whi	Force (L)	Force (T)
GF	0	320	0		
1	2.5	1153	2883	412 kN	790 kN
Roof	5.0	239	1195	221 kN	425 kN
		1712	4078	633 kN	1215 kN

$V_b = 0.37 \times 1712\,kN$ (For longitudinal

$V_b = 633\,kN$ where $\mu = 2$)

$V_b = 0.71 \times 1712\,kN$ (For transverse

$V_b = 1215\,kN$ where $\mu = 1.25$)

SKM

Job No. 3301276.218

Calc. Series

Client

Page 11

Job Name

By KS

Calcs Title

Date 19/4/13

Demand:

1st Floor	Transverse	-	425 kN	or	8500 BU
	Longitudinal	-	221 kN	or	4420 BU
Ground Floor	Transverse	-	790 kN	or	15800 BU
	Longitudinal	-	412 kN	or	8240 BU

Capacity

1st Floor : T - Concrete walls / Wall lining (Check connection)

L - Wall lining

G Floor T - Concrete walls / Wall lining (Check connection)

L - Wall lining

SKM

Job No. 2B01276.218

Calc. Series

Client

Page 12

Job Name

By KS

Calcs Title

Date 19/4/13

Lined wall Capacity:

Internal walls -

- 10 mm Gib on both sides - 0.4m ⇒ 65 BU/m

  min length - 1.2m ⇒ 80 BU/m

External walls

- 10 mm Gib on one side - 0.4m ⇒ 55 BU/m

  min length - 1.2m ⇒ 60 BU/m

Values from Gib Braceline manual

SKM

Job No. 2B01276.218

Calc. Series

Client

Page 13

Job Name

By KJ

Calcs Title

Date 19/4/13

Concrete Wall Capacity.

In Plane:

120mm - Steel at 300 centres

Assume singly reinforced with $\phi 12$'s

Shear Capacity

$\phi V = \phi (V_c + V_s)$

$V_c = 0.17\sqrt{f_c'}\, A_{cv}$

$A_{cv} = 0.8\, l_w \times b_w$

$A_{cv} = 0.8 \times 7500 \times 120 = 720{,}000 \text{ mm}^2$

$V_c = 0.17\sqrt{25} \times 720{,}000$

$V_c = 612 \text{ kN}$

SKM

Client	Job No. 7801276.218
Job Name	Calc. Series
Calcs Title	Page 14
	By KS
	Date 19/4/13

Out of Plane

Moment Capacity

$A = \frac{\pi}{4} \times 12^2 / 0.3 = 376 mm^2$ /m length

$f_y = 300 MPa$

$a = 376 \times 300 / 0.85 \times 1000 \times 25 = 5.3 mm$

$d = 120/2 = 60 mm$

$\phi M = 1.0 \times (300 \times 376) \times (60 - \frac{5.3}{2})$

$\phi M = 6.5$ kN.m /per metre width of wall.

Because the wall is lined, out of plane %NBS will not be an issue.

SKM

Client	Job No. 7B01276.218
Job Name	Calc. Series
Calcs Title	Page 15
	By KJ
	Date 19/4/13

Longitudinal Demand vs Capacity.

Ground Floor Demand : 412 kN or 8240 BU
First Floor Demand : 221 kN or 4420 BU
(from page 10)

Capacity of 1 unit

- Longitudinal Walls - See page 18 for marked up dwg

	A	-	0.8m	×	55 BU/m	⇒ 44 BU
	B	-	1.4m	×	60 BU/m	⇒ 84 BU
	C	-	1.0m	×	65 BU/m	⇒ 65 BU
(x2)	D	-	0.6m	×	65 BU/m	⇒ 78 BU
	E	-	1.2m	×	80 BU/m	⇒ 96 BU
	F	-	1.2m	×	60 BU/m	⇒ 72 BU
(x2)	G	-	0.4m	×	55 BU/m	⇒ 44 BU

Σ 483 BU per unit

6 units × 483 BU = 2898 BU

GROUND FLOOR

% NBS = 2898 / (8240 + 4420) = 22% NBS

FIRST FLOOR

% NBS = (483×2) / 4420 = 22% NBS

Look into using out of plane capacity of concrete walls on next page

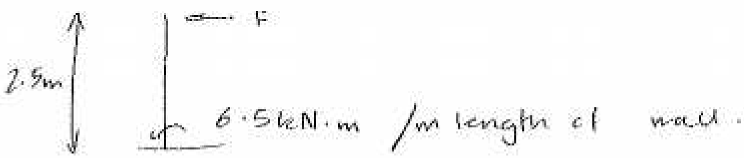
SKM

Client	
Job Name	
Calcs Title	
Job No.	ZB01276.218
Calc. Series	
Page	15a
By	K.S
Date	7/5/13

Out of plane capacity of walls:

Capacity out of plane: at Ground floor level.

$\phi M = 6.5 kN.m/m$ (from page 14)



$F = \frac{6.5}{2.5} = 2.6 kN$ /per metre width of wall.

6 walls, × 7.5m long × 2.6kN/m = 117kN

% NBS at Ground floor:

$2898 + (117 \times 20) / (8240 + 4420) = 41\%$

% NBS at first floor:

$2898 + (4 \times 7.5 \times 2.6 \times 20) / (4420) \geq 100\%$

SKM

Job No.	EB01276.214
Calc. Series	
Client	
Page	16
Job Name	
By	KS
Calcs Title	
Date	19/4/13

Transverse Demand vs Capacity

Ground Floor Demand 412kN or 8240BU
First Floor Demand 221kN or 4420BU
(from Page 10)

Capacity of 1 unit (Internal)

- Transverse walls - See page 18 for marked up dwg

	1	-	2.7m	×	80 BU/m	⇒	216 BU
(5)	2	-	4.1m	×	60 BU/m	⇒	492 BU
(7)	3	-	2.0m	×	60 BU/m	⇒	240 BU
	4	-	1.2m	×	80 BU/m	⇒	96 BU
(x2)	6	-	0.6m	×	65 BU/m	⇒	72 BU

Σ 1122 BU

Capacity of 1 unit (External)

Transverse walls

	1	-	2.7m	×	80 BU/m	⇒	216 BU
	2	-	2.4m	×	60 BU/m	⇒	144 BU
	3	-	3.1m	×	60 BU/m	⇒	186 BU
	4	-	1.2m	×	80 BU/m	⇒	96 BU
	5	-	4.1m	×	60 BU/m	⇒	246 BU
(x2)	6	-	0.6m	×	65 Bu/m	⇒	72 BU
	7	-	2.0m	×	60 BU/m	⇒	120 BU

Σ 1086 BU

Capacity GF = 4 × 1121 + 2 × 1086 = 6656 BU

1st F = 2 × 1121 = 2242 BU

GROUND FLOOR

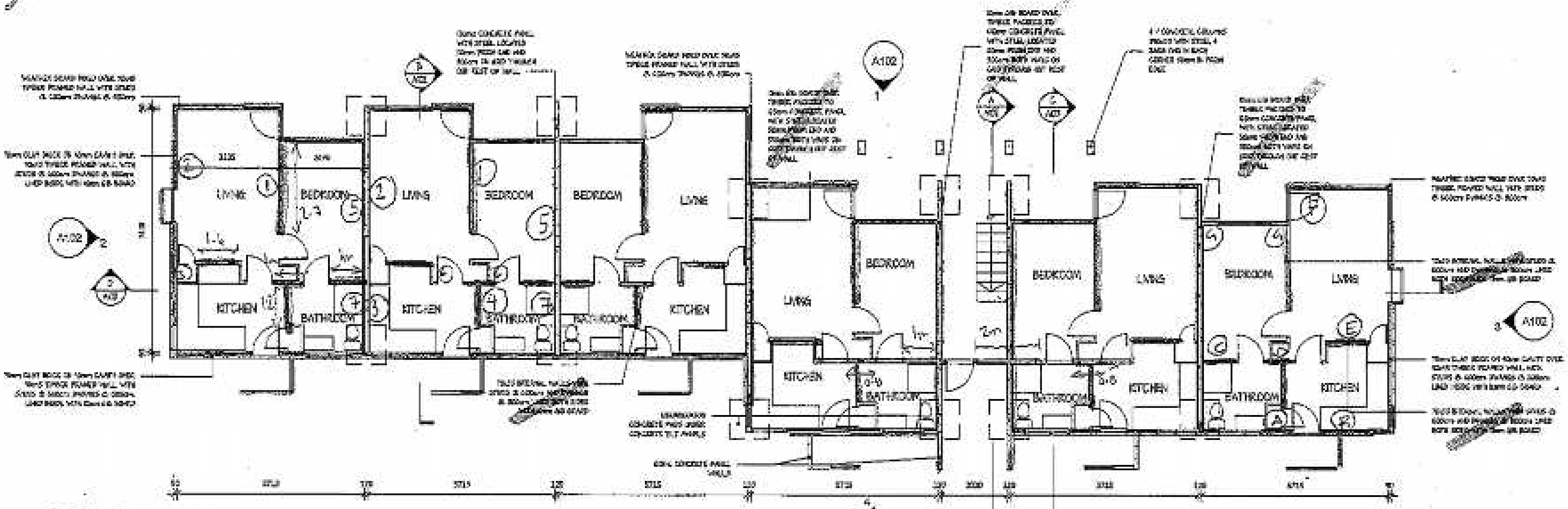
% NBS = 6656 / (8240 + 4420) = 53 % NBS

FIRST FLOOR

% NBS = 2242 / 4420 BU = 51 % NBS.

- This is assuming there is no connection between the concrete walls and the floor / ceiling diaphragm.

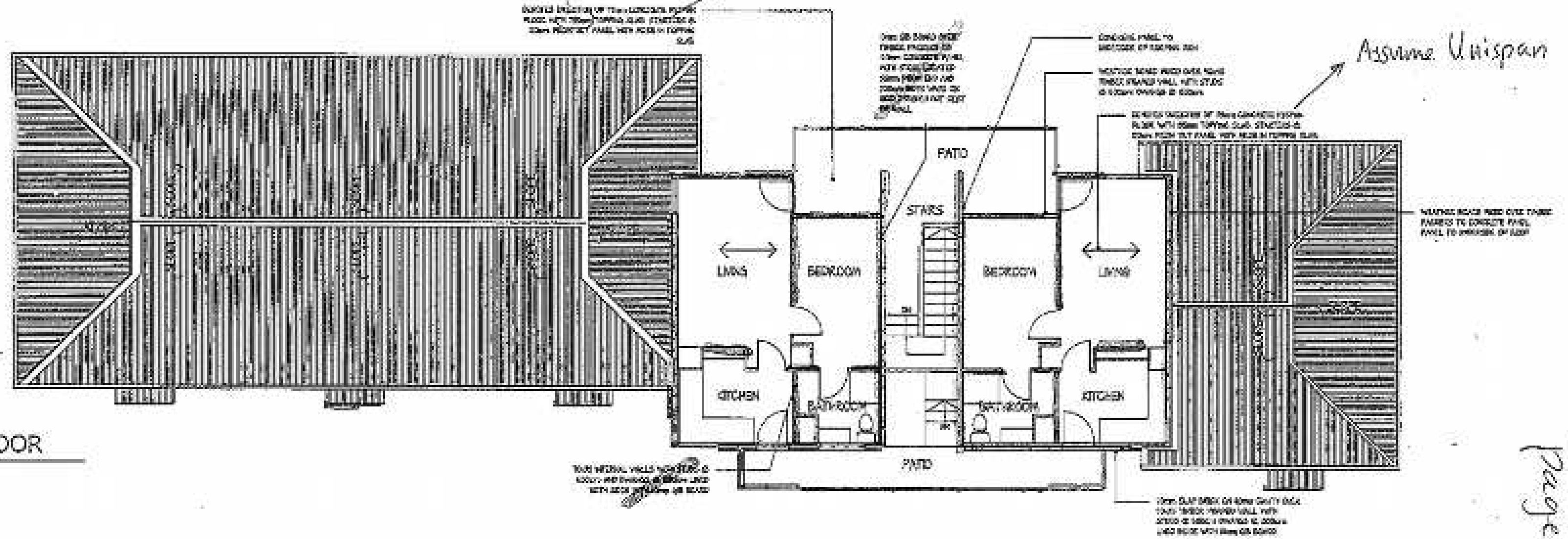
- If we check the diaphragm connection then we could use the concrete wall and recheck the transverse direction for $\mu = 1.25$ demands.



GROUND FLOOR

SCALE 1:100

Assume 75mm

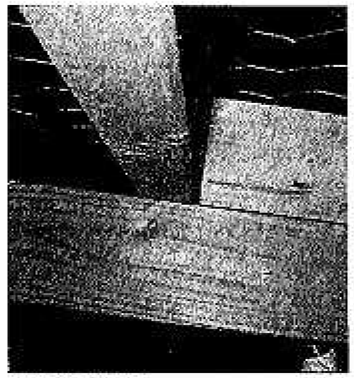


FIRST FLOOR

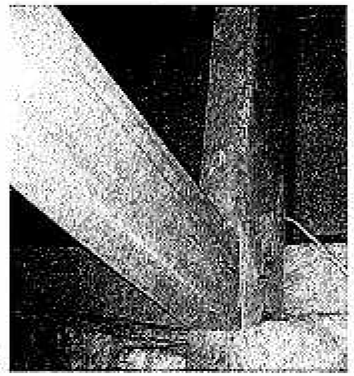
SCALE 1:100

Area 1

TRUSS TO CONCRETE PARTY WALL



PURLIN TO TRUSS DETAIL



PURLIN TO TRUSS DETAIL

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SKM

Job No. 7B01276.218

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Client

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Job Name

By KS

Calcs Title

Date 19/4/13

Roof Diaphragm

Ceiling: 13mm gibboard fixed to Roof trusses via 70x35 timber @ 600 crs

Braces: Roof angle braces

NZS 3604 Guidelines) assuming correctly nailed then by inspection roof diaphragm has sufficient capacity.

Connection to walls is unknown but assumed to be present.

Calculations on the following pages.

SKM

Client

Job Name

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Page 20

By KS

Date 19/4/13

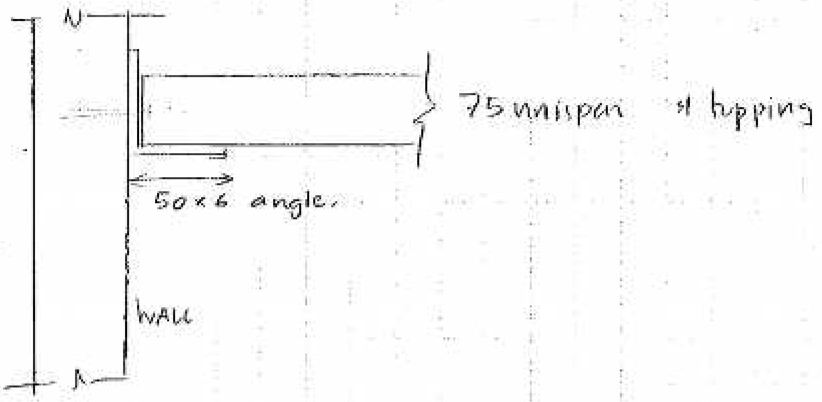
Concrete wall - Diaphragm:

- Transverse: • 2 × single storey concrete wall
  • 4 × concrete walls, 2 storeys high.

- Connection of concrete floor to concrete wall should be substantial enough to distribute the loadings.

- Conversations with Build Qual verify they could not see the connections but assume they are there

Unispan fixed to panel with steel angle



This assume walls are tied into diaphragm above.

Assuming connection is adequate to distribute the loads then the transverse loading will be ≥ 100% NBS.

In plane capacity of walls

$\phi V_c = 612kN$ $(L = 7500mm)$

$V^* = 716kN$ ∴ %NBS ≥ 100%

$\mu = 1.25$

Check connection.

Conversations with BuildQual said they were not able to see the connection

- Angle size - 50x6 bottom flange measured, assume this is an equal angle.

- Bolted to wall at 1m centres with M12 chemsets or similar.

Shear capacity of 1 M12 Chemset.

$\phi V_s = 21kN$

Therefore diaphragm connection capacity = 21 kN/m

- Demand:

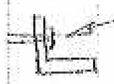
Use demand from splitting up rooms.

1st Floor transverse = 508kN

4 walls = 127 kN

- Capacity = 21kN/m × 7.5m

= 157 kN.

Assuming connection is:  ϕV = 20kN @ 1m centres

Then capacity ≥ 100 %.

Connection between concrete walls & timber ceilings:

Assume ceiling correctly nailed to timber blocking and runners.

90x35 bolted to concrete wall.

Assume M12 bolts @ 1000 crs

By inspection this connection will be ≥ 100%NBS

MUST STATE ASSUMPTION IN REPORT.

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Job Name

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Therefore;

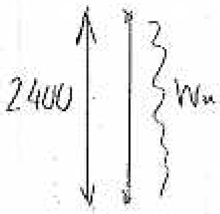
transverse demand = 1st = 790kN
Roof = 425 kN
} /4 = 1.25

Capacity of 1 x 7.5m long wall = ϕV_c = 612kN

Therefore %NBS ≥ 100%

2 walls = 1224 / (790 + 425) ≥ 100% NBS

Out of Plane Timber Wall Studs:



Wu = seismic weight of timber, lining & brick veneer.

Brick veneer weight: 1.35kPa

Timber framing & insulation : 0.17kPa Σ 1.55kPa

Seismic coefficient - $C_d(T) = 0.37$

Parts coefficient - $F_{ph} = C_p(T)\, C_{ph}\, K_p W_p \leq 3.6 W_p$

$C_p(T) = C(0)\, C_{Hi}\, C_i(T_p)$

$C(0) = 1.12 \times 0.30 \times 1.0 \times 1.0 = 0.336$

$C_{Hi} = 1 + h_i/6 = 1.4$ (For GF wall)

$C_i(T_p) = 2.0$ for $T_p \leq 0.75$ sec.

$C_p(T) = 0.336 \times 1.4 \times 2.0 = 0.941$

$K_p = 1.0$

$C_{ph} = 0.85$ for $\mu = 1.25$

$F_{ph} = 0.85 \times 0.941\ W_p \leq 3.6 W_p$

$0.80\ W_p \leq 3.6 W_p \ \& \ \geq C_d(T)$

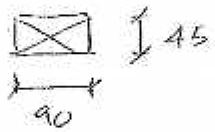
Demand:

$M^* = \frac{wl^2}{8} = \frac{(0.8 \times 1.55) \times 2.4^2}{8} = 0.89 kN\cdot m$ /m width

The studs are at 600 crs

$\therefore$ Demand on each stud $\Rightarrow$ $M^* = 0.54 kN\cdot m$

Capacity.



$\phi M = \phi k_1 k_4 k_8 f_b Z$

Assuming No 1 Grade framing,

$f_b = 10 MPa$

$Z = 45 \times 90^2 / 6 = 60,750 mm^3$

$\phi = 1.0$

$k_1 = 1.0$

$k_8 =$ based on $L_{ay}/b = 800/45 = 17.8$

$k_8 = d/b = 2$

$k_8 = 1.0$

$\phi M = 1.0 \times 1.0 \times 1.0 \times 1.0 \times 10 \times 60,750$

$\phi M = 0.60 kN\cdot m$

% NBS $\geq$ 100% NBS.

SKM

Job No. Z601276.218

Calc. Series BLK – PUBL. RENT.

Client CCC

Page 0-1

Job Name MAURICE CARTER COURTS – PUBLIC RENTAL

By TOMAS BILEK

Calcs Title QUANTITATIVE DEE

Date 7/2/2013

SCOPE

CALCULATE SEISMIC CAPACITY OF THE 2-STOREY RESIDENTIAL BUILDING ("PUBLIC RENTAL" – BU1103 – 003 EQ2) AT 16 DUNDEE PLACE / MAURICE CARTER COURTS IN CHRISTCHURCH.

THE STRUCTURE IS A TIMBER FRAME BUILDING WITH TIMBER FLOOR AND TIMBER ROOF TRUSSES. THE BUILDING CONSISTS OF 4 No UNITS DIVIDED BY CONCRETE BLOCK WALLS. THE BUILDING IS FOUNDED ON CONCRETE STRIP FOOTING SUPPORTING CONCRETE GROUND BEARING SLAB ON FILL.

LATERAL STABILITY IS PROVIDED BY COMBINATION OF CONCRETE BLOCK WALL (IN TRANSVERSE DIRECTION ONLY) AND SHEARING RESISTANCE OF GIB LINING.

ARCHITECTURAL DRAWINGS (WITH LIMITED AMOUNT OF STRUCT. INFORMATION) PRODUCED BY "CITY ARCHITECT DIVISION: CITY WORKS & PLANNING DEPARTMENT" WERE AVAILABLE FOR THE CALCULATION

FOLLOWING CALCULATIONS WERE CARRIED OUT FOR THE PURPOSE OF QUANTITATIVE DEE.

CONTENT / NBS SUMMARY

1	LOADING		
2	BRACING STRATEGY & CAPACITY	– LONGITUDINAL	42% NBS
		– TRANSVERSE	48% NBS
3	MASONRY WALLS	– OUT OF PLANE FLEXURE	83% NBS
		– IN PLANE SHEAR	> 100% NBS
4	FLOOR DIAPHRAGM		100% NBS
5	ROOF DIAPHRAGM		100% NBS
6	TIMBER WALLS	– OUT OF PLANE FLEXURE	51% NBS
7	FOUNDATIONS	– GROUND BEARING PRESSURE	47% NBS
X	MANUFACTURER'S DATA		

SKM

Job No. ZB01276.218

Calc. Series BLK - PUBL. RENT.

Client CCC

Page 1 - 1

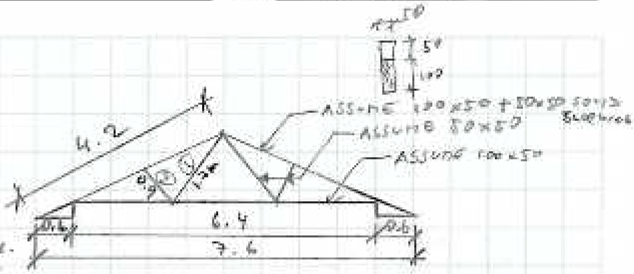
Job Name MAURICE CARTER COURTS - BL. PUBLIC REST.

By TOMAS BILEK

Calcs Title LOADING

Date 11/2/2013

LOADING - SELF WEIGHT



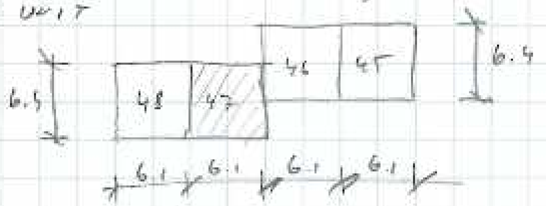
ROOF

- ROOF TRUSS (1 UNIT)

RAFTERS	$2 \times 0.05 \times 0.1 \times 4.2 \times 4.6$	=	0.193
BOTTOM CHORD	$1 \times 0.05 \times 0.1 \times 7.6 \times 4.6$	=	0.175
DIAGONALS ①	$2 \times 0.05 \times 0.05 \times 1.7 \times 4.6$	=	0.039
②	$2 \times 0.05 \times 0.05 \times 0.9 \times 4.6$	=	0.021
SOLID BLOCKING	$2 \times 0.05 \times 0.05 \times 4.2 \times 4.6$	=	0.097

ROOF TRUSS $[\Sigma = 0.53$ kN / TRUSS$]$

CALCULATE AS UNIT AREA (1ST FLOOR AREA)
- 6 No TRUSSES PER UNIT
- UNIT AREA
$6.4 \times 6.1 = \underline{39 m^2}$



$$6 \times [\text{TRUSS}] / (6.4 \times 6.1) = 6 \times 0.53 / (6.4 \times 6.1) = \underline{0.082 \text{ kN/m}^2}$$

- PURLINS @ 600 crs $\quad 0.05 \times 0.1 \times \frac{1000}{600 \cos 25} \times 4.6 = \quad 0.042$ kN/m²

- METAL SHEETING (0.4mm) $\quad 0.0004 \times 78.5 = \quad 0.032$ kN/m²
- SARKING (PLY 12mm) $\quad 0.012 \times 6 = \quad 0.072$ kN/m²
- CEILING

BATTENS @ 400 crs	$0.07 \times 0.04 \times \frac{1000}{400} \times 4.6$	=	0.032 kN/m²
INSULATION (30kg/m³)	0.1×0.3	=	0.03 kN/m²
GIB CEILING (9.5mm)	0.0095×10	=	0.095 kN/m²

CEILING $[\Sigma = 0.157$ kN/m²$]$

- 135 LITRE WATER TANK IN ROOF SPACE + PARTIAL FLOORING
SAY 2 kN

TOTAL ROOF = 0.082 + 0.042 + 0.032 + 0.072 + 0.157 + WATER TANK

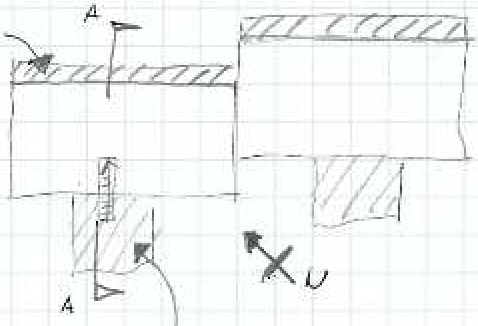
= 0.385 kN/m² + WATER TANK 2kN

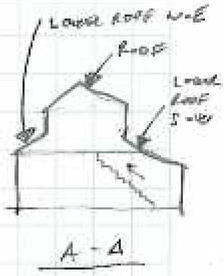
SKM

Client	CCC	Job No.	ZB01276.218
Job Name	MAURICE CARTER COURTS - PUBLIC RENTALS	Calc. Series	BLK - PURL. RECT.
Calcs Title	LOADING	Page	1-2
		By	TB
		Date	4/2/2013

LOWER ROOF - NORTH-EAST

ALLOW SAME AS PER MAIN ROOF = 0.385 kN/m²





LOWER ROOF - SOUTH-WEST

[kN/m²]

• RAFTERS @ 1.2 m crs	$0.05 \times 0.15 \times \frac{1000}{1200}$	=	0.000625
• PURLINS @ 600mm crs	$0.05 \times 0.1 \times \frac{1000}{600}$	=	0.00833
• METAL SHEETING (0.4mm)	0.0004×78.5	=	0.0314
• SARKING (12mm)	0.012×6	=	0.072

[Σ = 0.113 kN/m²]

ALLOW FOR 25° PITCH ⇒ 0.113 / cos 25° = 0.124 kN/m²

• CEILING

CEILING JOISTS @ 400 crs	$0.1 \times 0.05 \times \frac{1000}{400}$	=	0.0125
-11- ORTOGONAL DIR	-11-	=	0.0125
FIBREGLAS INSUL. (100mm)	0.1×0.3	=	0.03
GIB (9.5mm)	0.0095×10	=	0.095

[Σ 0.15 kN/m²]

TOTAL LOWER ROOF - S-W = [ROOF] + [CEILING]
= 0.124 + 0.15 = 0.274 kN/m²

FLOOR

[kN/m²]

• FLOOR JOISTS @ 400 crs	$0.2 \times 0.05 \times \frac{1000}{400} \times 4.6$	=	0.115
• CUSTOM WOOD (20mm)	0.02×8	=	0.16
• GIB CEILING (9.5mm)	0.0095×10	=	0.095
• CARPET			0.03

Σ = 0.4 kN/m²

STAIR

TREADS (30mm)	0.03×4.6	=	0.138
RISERS (30mm @ 300 crs)	$0.03 \times 0.18 \times \frac{1000}{300} \times 4.6$	=	0.083
STRINGER (300x50) 1m APART	$0.3 \times 0.05 \times \frac{1000}{1000} \times 4.6 / \cos 31°$	=	0.081
	ALLOW FOR PITCH		

Σ = 0.302 kN/m²

SKM

Job No. Z001276.218

Calc. Series BLK - PUBL. RENT.

Client CCC

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Job Name MAURICE CARTER COURTS - BL. PUBL. RENTAL

By TB

Calcs Title LOADING

Date 11/2/2013

WALL ① - TIMBER STUDWORK + BRICK VENEER EXT + GIB LINING INT.

• STUDWORK [kN/m²]

VERTICAL STUDS @600 CRS	$0.05 \times 0.1 \times \frac{1000}{600} \times 4.6$	= 0.04
HORIZONTAL STUDS @600 CRS	—— // ——	= 0.04
FIBREGLAS INS (75mm)	0.075×0.3	= 0.023

• EXT. LEAF

PLYWOOD (12mm)	0.012×6	= 0.072
BRICK VENEER	0.09×18	= 1.62

• INTERNAL LEAF

GIB (9.5mm)	0.0095×10	= 0.095

$\Sigma = 1.9$ kN/m²

WALL ② - TIMBER STUDWORK + GIB/CUSTOMWOOD/HARDIFLEX LINING INT. + + HARDI PLANKS EXT.

• STUDWORK [kN/m²]

VERTICAL STUDS @ 600 CRS	$0.05 \times 0.1 \times \frac{1000}{600} \times 4.6$ =	0.04
HORIZONTAL STUDS @ 600 CRS	—//— =	0.04
FIBRE GLAS INSUL (75mm)	0.075×0.3 =	0.023

• EXT. LEAF

HARDI PLANKS (7.5mm)	0.0075×13	= 0.098

• INTERNAL - LEAF

EITHER GIB 9.5mm	$0.0095 \times 10 =$ 0.095	USE THE HIGHEST VALUE	= 0.095
OR CUSTOMWOOD 10mm	$0.01 \times 8 = 0.08$		
OR HARDIFLEX 6mm	$0.006 \times 13 = 0.078$		

$\Sigma = 0.3$ kN/m²

SKM

Job No. Z601276.218

Calc. Series BL. - PUBL. RENT.

Client CCC

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Job Name MAURICE CARTER COURTS - BL. PUBLIC RENTALS

By TB

Calcs Title LOADING

Date 11/2/2013

WALL (3) - TIMBER STUDWORK + GIB LINING BOTH SIDE (/CUSTOMWOOD ONE SIDE ← LIGHTER THAN GIB, HENCE USE GIB WEIGHT INSTEAD)

			[kN/m²]
• STUDWORK			
VERTICAL STUDS	@600 CRS	$0.05 \times 0.1 \times \frac{1000}{600} \times 4.6$	= 0.04
HORIZONTAL STUDS	@600 CRS	— // —	= 0.04
• LINING			
2x GIB 9.5mm		$2 \times 0.0095 \times 10$	= 0.19

$\Sigma = 0.27$ kN/m²

WALL (4) - 190 mm THICK BLOCKWORK + 30 mm STYROLINER EACH SIDE

		[kN/m²]
• CONCRETE BLOCK WALL WITH CORE INFILL	0.19×24 =	4.56
• STYROLINER 30mm BOTH SIDES		
9.5mm GIB EACH SIDE	$2 \times 0.0095 \times 10$	= 0.19
30-9.5 mm STYROFOAM (33 kg/m³) EACH SIDE	$2 \times 0.0205 \times 0.33$	= 0.014

$\Sigma = 4.8$ kN/m²

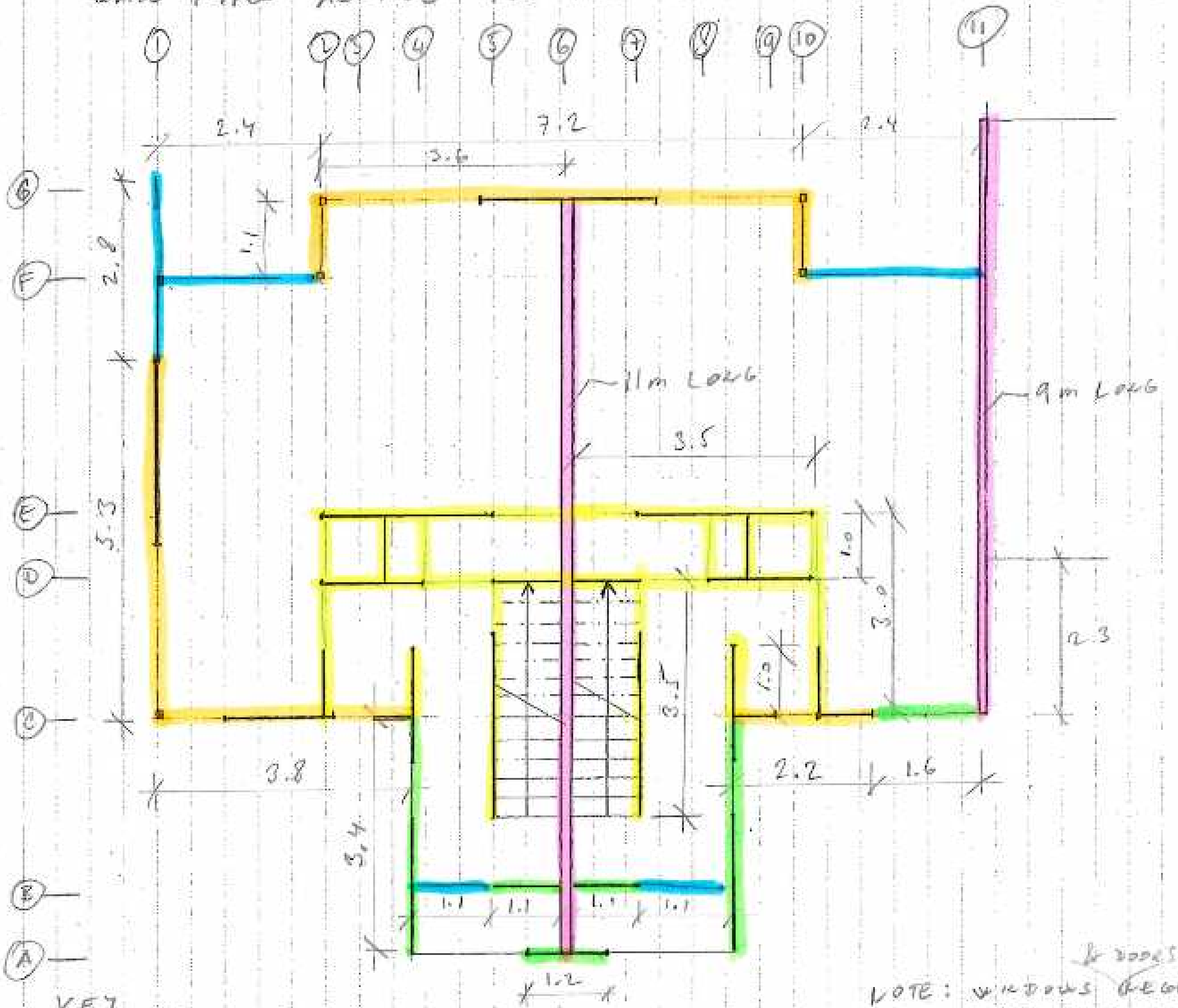
GLAZING

• ALLOW 0.3 kN/m²

SKM

Client: CCC	Job No. 28012 76.218
Job Name: MAURICE CARTER COURTS – BL. PUBLIC RECT.	Calc. Series: BL. - PUBL. - RECT
Calcs Title: WALL TYPES – GROUND FLOOR	Page: 1-5
	By: TB
	Date: 11/2/2013

WALL TYPES ASSUMED FOR THE CALCULATION OF MASS



NOTE: WINDOWS & DOORS NEGLECTED SINCE THEY ARE GENERALLY LIGHTER THAN CLADDING.

KEY

GLAZING [0.3 kg/m²]

WALL ① – TIMBER STUDWORK + BRICK [90mm] VENEER EXT + GIB [9.5mm] LINING INT [1.9 kN/m²]

WALL ② – TIMBER STUDWORK + HARDI PLANKS [7.5mm] + EITHER GIB [9.5mm] / CUSTOMWOOD [10mm] / HARDIFLEX [6mm] [0.3 kN/m²]

WALL ③ – TIMBER STUDWORK + GIB [9.5mm] LINING TO BOTH SIDES / OR GIB ONE SIDE & CUSTOMWOOD [10mm] THE OTHER SIDE [0.27 kN/m²]

WALL ④ – 100 THICK BLOCKWORK + 30mm STYROLINER [9.5mm GIB + STYROFOAM] BOTH SIDES [4.8 kN/m²]

SKM

Job No. ZB01276.218

Calc. Series BL, PUBL. RENT.

Client CCC

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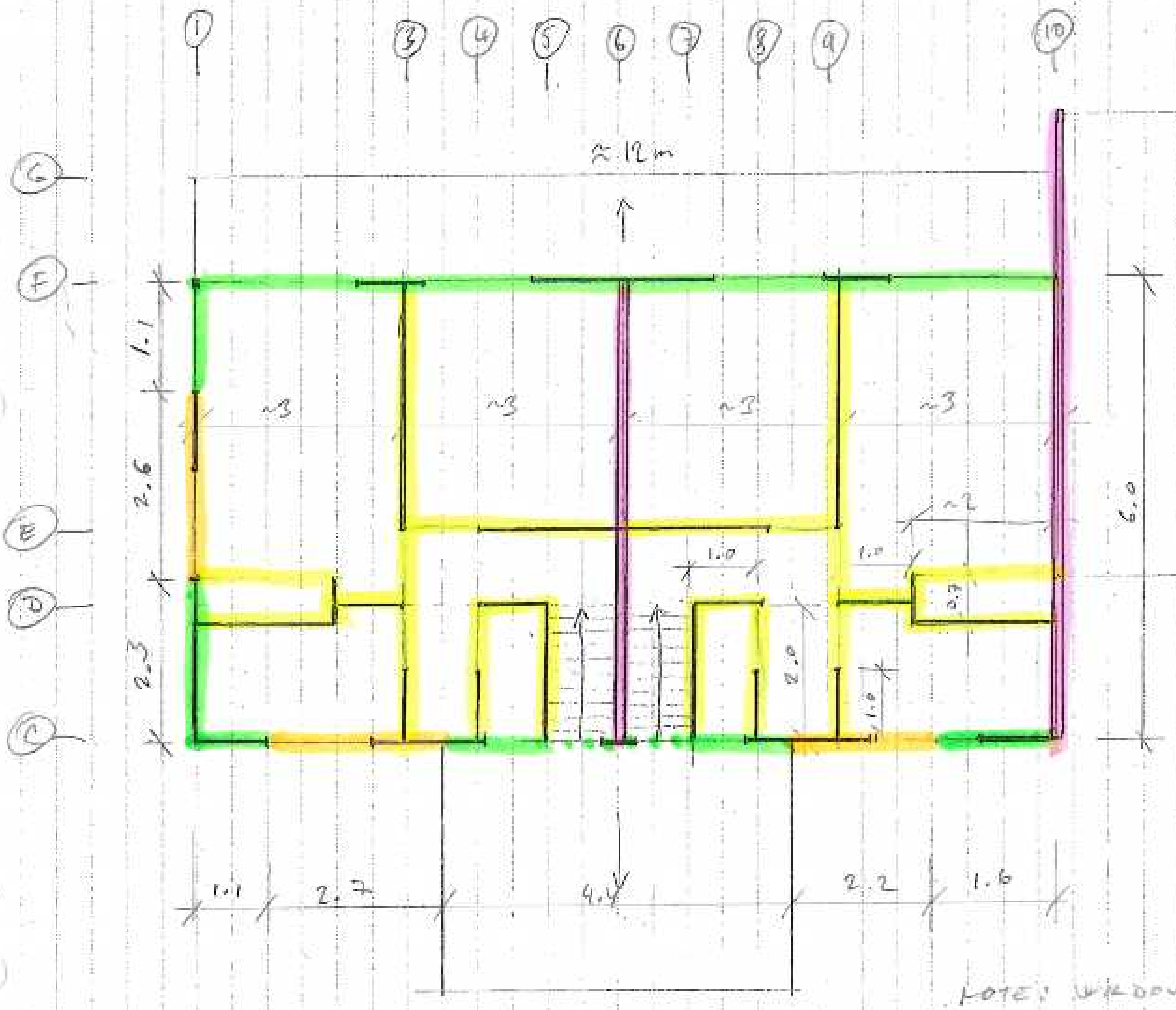
Job Name MAURICE CARTER COURTS - BL. PUBLIC RENTAL

By TB

Calcs Title WALL TYPES - FIRST FLOOR

Date 11/2/2013

WALL TYPES ASSUMED FOR THE CALCULATION OF MASS.



NOTE: WINDOWS & DOORS NEGLECTED

KEY

SEE PREVIOUS PAGE

SKM

Job No. ZB 01276.218

Calc. Series BLK - PUBL. RENT

Client CCC

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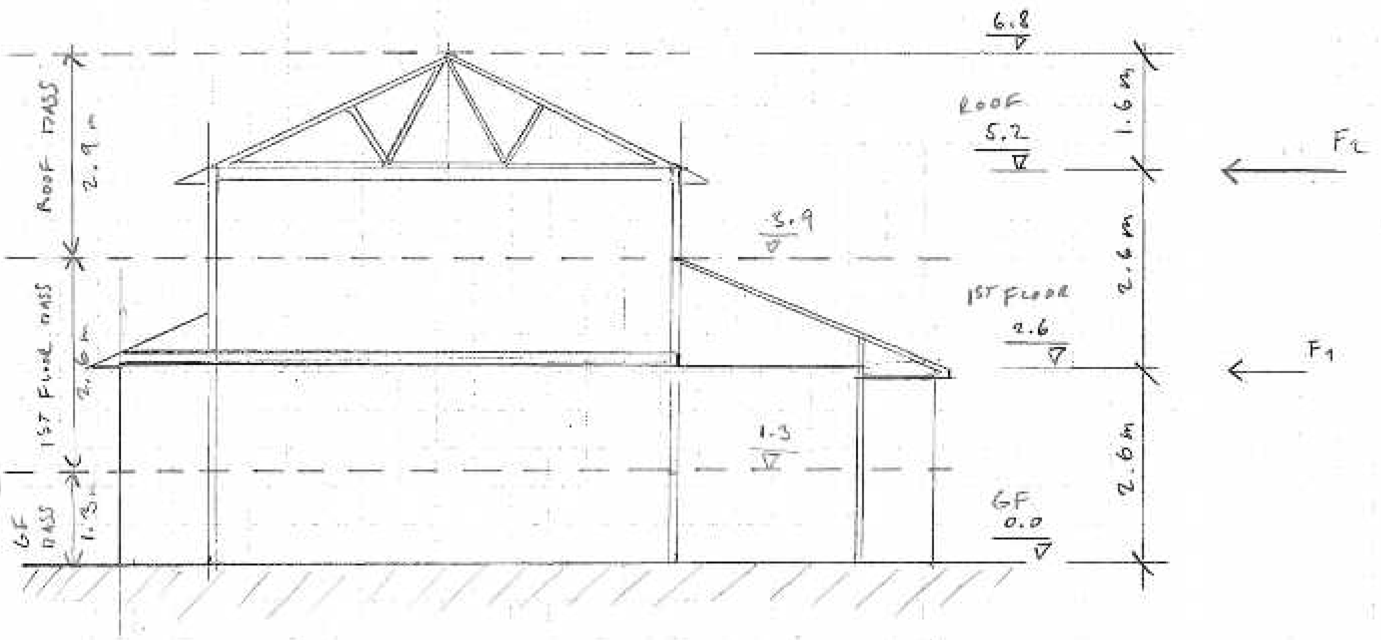
Job Name MAURICE CARTER COURTS - PUBL. RENTALS

By TOMAS BICK

Calcs Title SEISMIC LOADING - STOREY MASSES

Date 11/2/2013



INDICATIVE CROSS SECTION

SKM

	Job No. 2601276.218
	Calc. Series BLK – PUBL. RENT.
Client CCC	Page 1–8
Job Name MAURICE CARTER COURTS – PUBL. RENTALS	By TB
Calcs Title SEISMIC LOADING – STOREY MASSES	Date 11/2/2013

CALCULATE SEISMIC WEIGHTS – END UNIT

A) FIRST FLOOR ZONE

1) WALLS WITHIN 1ST FLOOR ZONE (1.3 – 3.9 ABOVE GF)

• GF WALLS (TOP HALF)

TYPE	OVERALL LENGTH [m]		HEIGHT ASSUMED	UNIT WEIGHT	SEISMIC WEIGHT
1	5.3 + 3.8 + 1.1 + 3.6	= 13.8 m	× 1.3 m	× 1.9 kN/m²	= 34.1 kN
2	3.4 + 1.1 + $\frac{1.2}{2}$	= 5.1 m	"	0.3	2.0 kN
3	3×3.5 + 3 + 3×1.0	= 16.5 m	"	0.27	5.8 kN
4	11/2 (EACH UNIT TAKES HALF)	= 5.5 m	"	4.8	34.3 kN
GLAZING	2.8 + 2.4	= 5.2 m	"	0.3	2.1 kN

Σ = 78.3 kN

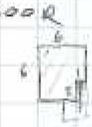
• FIRST FLOOR WALLS (BOTTOM HALF)

TYPE	OVERALL LENGTH [m]		HEIGHT ASSUMED	UNIT WEIGHT	SEISMIC WEIGHT
1	2.6 + 2.7	= 5.3 m	× 1.3 m	× 1.9 kN/m² =	13.1 kN
2	6+1.1+2.3+1.1+$\frac{4.4}{2}$	= 12.7 m	"	0.3	5.0 kN
3	6+2×2+0.7+2×3+2+1	= 19.7 m	"	0.27	6.9 kN
4	6/2 (EACH UNIT TAKES HALF)	= 3 m	"	4.8	18.7 kN

Σ = 43.7 kN

2) FLOORS / CEILINGS / ROOFS WITHIN 1ST FLOOR ZONE

	AREA		UNIT WEIGHT		SEISMIC WEIGHT
• 1ST FLOOR STRUCTURE	6×6 – 1×2 = 34 m²	×	0.4 kN/m²	=	13.6 kN
• LOWER ROOF – NE	6×1.8 = 10.8 m²	×	0.385 kN/m²	=	4.2 kN
• LOWER ROOF – SW	2.2×3.4 = 7.5 m²	×	0.274 kN/m²	=	2.1 kN
• TOP HALF OF STAIRS	1×1.75 = 1.75 m²	×	0.302 kN/m²	=	0.6 kN



SKM

Client CCC

Job Name MAURICE CARTER COURTS - BL. PUBL. RECT.

Calcs Title SEISMIC LOADING - STOREY MASSES

Job No. ZB 01276.218

Calc. Series BL. PUBL. RECT.

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By TB

Date 11/2/2013

3) LIVE LOADS WITHIN 1ST FLOOR ZONE

1ST FLOOR AREA ≈ 6x6 = 36 m²   (INCL. STAIRS)

UDL = 1.5 kN/m²   (GENERAL AREA IN SELF-CONTAINED DW.)

MASS = 36 x 1.5 = __54 kN__

__TOTAL SEISMIC WEIGHT - END UNIT - 1ST FLOOR ZONE (1.3-3.9m ABOVE GF)__

TOTAL GRAVITATIONAL FORCE $W_1 = G + \psi_E Q$

WHERE $\psi_E = 0.3$ (CL. 4.2, NZS 1170.5:2004)

$\underline{W_1} = \{$[GF WALLS (TOP HALF)] + [FIRST FLOOR WALLS (BOTTOM HALF)] + [1ST FLOOR STRUCTURE] + [LOWER ROOF - NE] + + [LOWER ROOF - SW] + [TOP HALF OF STAIRS]$\} + \psi_E$[LIVE LOAD] =

$= \{ 78.3 + 43.7 + 13.6 + 4.2 + + 2.1 + 0.6 \} + 0.3 \times 54 =$

$= 143 + 0.3 \times 54 = \underline{\underline{160\ kN}}$

SKM

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	Calc. Series BL. PUBL. RENT.
Client CCC	Page 1-10
Job Name DAVRIDE CARTER COURTS - BL. PUBL. RENT	By TB
Calcs Title SEISMIC LOADING - STOREY MASSES	Date 12/2/2013

B) ROOF ZONE

1) WALLS WITHIN ROOF ZONE (3.9 - 6.8 ABOVE GF)

• FIRST FLOOR WALLS (TOP HALF)

TYPE	OVERALL LENGTH [m]		ASSUMED HEIGHT	UNIT WEIGHT	SEISMIC WEIGHT
1	2.6 + 2.7	= 5.3 m	× 1.3 m	× 1.9 kN/m² =	13.1 kN
2	$6+1.1+2.3+1.1+\frac{4.4}{2}$	= 12.7 m	"	0.3	5.0
3	6+2×2+0.7+2×3+2+1	= 19.7 m	"	0.27	6.9
4	6/2 (EACH UNIT TAKES HALF)	= 3 m	"	4.8	18.7

Σ = 43.7 kN

• WALLS IN ROOF SPACE

4  A = 6 × 1.6/2 = 4.8 m² × 4.8 kN/m² = 23 kN

2) FLOORS/CEILINGS/ROOFS WITHIN ROOF ZONE

	AREA		UNIT WEIGHT		SEISMIC WEIGHT
• MAIN ROOF STRUCTURE	≈ 6 × 6 = 36 m²	×	0.385 kN/m²	=	14 kN
• 135L WATER TANK [INCL. FLOORING BELOW]					2 kN

3) LIVE LOADS WITHIN ROOF ZONE

• ATTIC AREA ≈ 6×6 = 36 m²

UDL = 0.5 kN/m² (NON-HABITABLE ROOF SPACES)

MASS = 36 × 0.5 = 18 kN

SKM

Job No. ZB 01276.218

Calc. Series BL. PUBL. RENT

Client CCC

Page 1 - 11

Job Name MAURICE CARTER COURTS - BL. PUBL. RENT.

By TB

Calcs Title SEISMIC LOADING - STOREY MASSES

Date 12/2/2017

TOTAL SEISMIC WEIGHT - END UNIT - ROOF ZONE (3.9 - 6.8m ABOVE GF)

$W_2 = G + \psi_E Q$

$\psi_E = 0.3$

W_2 = {[FIRST FLOOR WALLS (TOP HALF)] + [WALLS IN ROOF SPACE] + [MAIN ROOF STRUCTURE] +

+ [135L WATER TANK]} + ψ_E [LIVE LOAD IN ATTIC] =

= { 43.7 + 23 + 14 +

+ 2 } + 0.3 x 18 =

= 83 + 0.3 x 18 = 89 kN

TOTAL SEISMIC MASS OF THE END UNIT

$W_t = \underbrace{W_1}_{\text{[FIRST FL. ZONE]}} + \underbrace{W_2}_{\text{[ROOF ZONE]}} = 160 + 89 = 249$ kN

CALCULATIONS

SKM SINCLAIR KNIGHT MERZ

PROJECT	CCC BU 1103-003 EQ2 - Maurice Carter Courts - Blocks Public Rentals		
PART OF STRUCTURE	Earthquake Loading - Design Action Coeficients	PROJECT No. ZB01276.218	REVISION 0
	LONGITUDINAL DIRECTION	DATE 28 Feb 13	BY Tomas Bilek

Earthquake Loading to NZS 1170.5:2002

Spreadsheet Rev 0.1

This spreadsheet is for the calculation of equivalent static earthquake loads on structures. All references are to NZS 1170.5:2004 except where noted. As per NZS 1170.5 this spreadsheet is not applicable to bridges, tanks containing liquids, civil structures (dams and bunds etc) off-shore structures and soil retaining structures. It is recommended that the structure period is calculated but if not 0 seconds should be input to acheive conservative results. This spreadsheet is not applicable to parts of structures - see section 8 for design of parts.

INPUT

	Site location	Christchurch	
3.1.3	Site Subsoil class	D	
AS/NZS1170.0	Importance level	2	
AS/NZS1170.0	Design life	50 years	
AS/NZS1170.0	ULS Earthquake Annual probability of exceedance	1/500	
AS/NZS1170.0	SLS Earthquake Annual probability of exceedance	1/25	

CALCULATION

Structure period, T	0.5 s	structure period less than 0.5s
Structural Ductility Factor, μ	2.00	timber walls resisting seismic loads
ULS Structural Performance Factor from material code, S_p		
Note: Leave these S_p blank to use the values in NZS1170.5		
Spectral shape factor, $C_h(T)$	3.00	
Spectral shape factor, $C_h(0)$	3.00	
Hazard factor, Z	0.3	
ULS Return Period, R_u	1.00	
SLS Return Period, R_s	0.25	
ULS Near fault factor, N(T,D)	1.00	
SLS Near fault factor, N(T,D)	1.00	
ULS Elastic site hazard spectrum for horizontal loading, C(T)	0.90	
SLS Elastic site hazard spectrum for horizontal loading, C(T)	0.23	
ULS Elastic site hazard spectrum for vertical loading, $C_v(T)$	0.63	
SLS Elastic site hazard spectrum for vertical loading, $C_v(T)$	0.16	
ULS Structural Performance Factor, S_p	0.7	
ULS Structural Performance Factor for sliding or toppling, S_p	1.0	
SLS Structural Performance Factor, S_p	0.7	
k_μ	1.71	

ULS horizontal design action coefficient, $C_d(T_1)$	**0.37**
ULS horizontal design coefficient sliding or toppling, $C_d(T_1)$	**0.53**
ULS vertical design action coefficient, $C_{vd}(T_1)$	**0.44**

SLS horizontal design action coefficient, $C_d(T_1)$	**0.09**
SLS vertical design action coefficient, $C_{vd}(T_1)$	**0.11**

CALCULATIONS

SINCLAIR KNIGHT MERZ SKM

PROJECT	CCC BU 1103-003 EQ2 - Maurice Carter Courts - Blocks Public Rentals		
PART OF STRUCTURE	Earthquake Loading - Design Action Coeficients	PROJECT No. ZB01276.218	REVISION 0
	TRANSVERSE DIRECTION	DATE 28 Feb 13	BY Tomas Bilek

Earthquake Loading to NZS 1170.5:2002

Spreadsheet Rev 0.1

This spreadsheet is for the calculation of equivalent static earthquake loads on structures. All references are to NZS 1170.5:2004 except where noted. As per NZS 1170.5 this spreadsheet is not applicable to bridges, tanks containing liquids, civil structures (dams and bunds etc) off-shore structures and soil retaining structures. It is recommended that the structure period is calculated but if not 0 seconds should be input to acheive conservative results. This spreadsheet is not applicable to parts of structures - see section 8 for design of parts.

INPUT

	Site location	Christchurch	
3.1.3	Site Subsoil class	D	
AS/NZS1170.0	Importance level	2	
AS/NZS1170.0	Design life	50 years	
AS/NZS1170.0	ULS Earthquake Annual probability of exceedance	1/500	
AS/NZS1170.0	SLS Earthquake Annual probability of exceedance	1/25	

CALCULATION

Structure period, T	0.5 s	structure period less than 0.5s
Structural Ductility Factor, μ	1.25	masonry walls resisting seismic loads
ULS Structural Performance Factor from material code, S_p		
Note: Leave these S_p blank to use the values in NZS1170.5		
Spectral shape factor, $C_h(T)$	3.00	
Spectral shape factor, $C_h(0)$	3.00	
Hazard factor, Z	0.3	
ULS Return Period, R_u	1.00	
SLS Return Period, R_s	0.25	
ULS Near fault factor, N(T,D)	1.00	
SLS Near fault factor, N(T,D)	1.00	
ULS Elastic site hazard spectrum for horizontal loading, C(T)	0.90	
SLS Elastic site hazard spectrum for horizontal loading, C(T)	0.23	
ULS Elastic site hazard spectrum for vertical loading, $C_v(T)$	0.63	
SLS Elastic site hazard spectrum for vertical loading, $C_v(T)$	0.16	
ULS Structural Performance Factor, S_p	0.9	
ULS Structural Performance Factor for sliding or toppling, S_p	1.0	
SLS Structural Performance Factor, S_p	0.7	
k_μ	1.18	

ULS horizontal design action coefficient, $C_d(T_1)$	**0.71**
ULS horizontal design coefficient sliding or toppling, $C_d(T_1)$	**0.76**
ULS vertical design action coefficient, $C_{vd}(T_1)$	**0.58**

SLS horizontal design action coefficient, $C_d(T_1)$	**0.13**
SLS vertical design action coefficient, $C_{vd}(T_1)$	**0.11**

HORIZONTAL SEISMIC SHEAR - LONGITUDINAL DIR

$$V_L = C_d(T_1) \times W_t$$

$$= 0.37 \times 249 = 92 \text{ kN}$$

EQUIVALENT STATIC HORIZONTAL FORCES AT EACH LEVEL

- 1ST FLOOR ZONE (1.3 - 3.9 m ABOVE GF)

$$\underline{\underline{F_{L1}}} = F_t + 0.92\,V \frac{W_1 \times h_1}{W_1 \times h_1 + W_2 \times h_2} =$$

WHERE

$F_t = 0$ (ZERO FOR LOWER STOREY)

$V = 92$ kN

$W_1 = 160$ kN $\quad h_1 = 2.6$ m

$W_2 = 89$ kN $\quad h_2 = 5.2$ m

$$= 0 + 0.92 \times 92 \frac{160 \times 2.6}{160 \times 2.6 + 89 \times 5.2}$$

$$\underline{\underline{= 40 \text{ kN}}}$$

- ROOF ZONE (3.9 - 6.8 m ABOVE GF)

$$\underline{\underline{F_{L2}}} = F_t + 0.92\,V \frac{W_2 \times h_2}{W_1 \times h_1 + W_2 \times h_2} =$$

WHERE:

$F_t = 0.08\,V$ (FOR TOP MOST STOREY)

$= 0.08 \times 92 = 7.36$ kN

$$= 7.36 + 0.92 \times 92 \frac{89 \times 5.2}{160 \times 2.6 + 89 \times 5.2}$$

$$\underline{\underline{= 52 \text{ kN}}}$$

HORIZONTAL SEISMIC SHEAR - TRANSVERSE DIRECTION

$$V_T = C_d(T_1) \times W_t$$

$$= 0.71 \times 249 = 177\ kN$$

EQUIVALENT STATIC HORIZONTAL FORCES AT EACH LEVEL

- 1ST FLOOR ZONE

$$F_{T,1} = F_t + 0.92\,V \frac{W_1 \times h_1}{W_1 \times h_1 + W_2 \times h_2}$$

$$= 0 + 0.92 \times 177 \frac{160 \times 2.6}{160 \times 2.6 + 89 \times 5.2} = 77\ kN$$

- ROOF ZONE

$$F_{T,2} = F_t + 0.92\,V \frac{W_2\, h_2}{W_1 h_1 + W_2 h_2} =$$

$$= 0.08 \times 177 + 0.92 \times 177 \frac{89 \times 5.2}{160 \times 2.6 + 89 \times 5.2} = 100\ kN$$

SKM

Job No. ZB 01276.218
Calc. Series BLK – PUBL. RECTI
Client CCC
Page 1–16
Job Name DAURICE CARTER COURTS – BL PUBL. RECT.
By TB
Calcs Title SEISMIC LOADING
Date 12/2/2013

EFFECTIVE ACCELERATIONS [% g]

LONG. • 1ST FLOOR $F_{L1}/W_1 = 40/160 = 0.25 \Rightarrow$ 25%

• ROOF $F_{L2}/W_2 = 52/89 = 0.58 \Rightarrow$ 58%

TRANSL. • 1ST FLOOR $F_{T1}/W_1 = 77/160 = 0.48 \Rightarrow$ 48%

• ROOF $F_{T2}/W_2 = 100/89 = 1.12 \Rightarrow$ 112%

BRACING DEMAND

LONGITUDINAL DIRECTION [bracing units]

1kN = 20 BU

ROOF $F_{L1} = 40$ kN = 800 BUs (40×20)

1ST FLOOR $F_{L1} + F_{L2} = 40 + 52 = 92$ kN = 1840 BUs (92×20)

TRANSVERSE DIRECTION

ROOF $F_{T1} = 77$ kN = 1540 BUs (77×20)

1ST FLOOR $F_{T1} + F_{T2} = 77 + 100 = 197$ kN = 3540 BUs (177×20)

SKM

Job No. ZB01276.218

Calc. Series

Client CCC

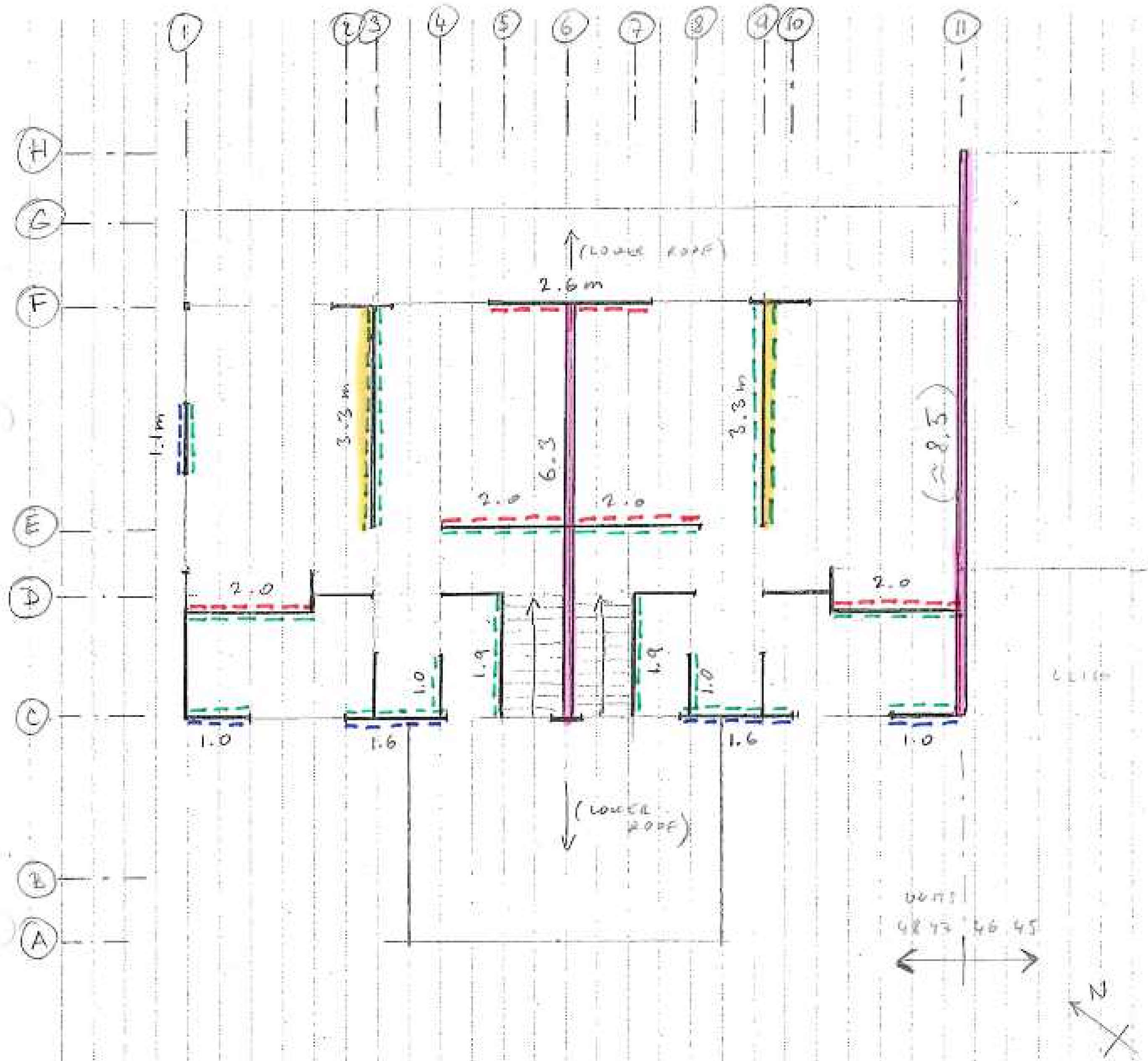
Page 2-1

Job Name MAURICE CARTER COURTS - BL. PUBLIC RENTAL

By TB

Calcs Title VERTICAL BRACING - FIRST FLOOR

Date 8/2/2013



KEY

--- 12mm PLY BRACING

--- 9.5mm GIB

--- 10mm CUSTOM WOOD

190mm THICK CONCR. BLOCK WALL WITH Ø12 @ 600 CRS VERT.

22×22×1.2 ANGLE BRACE (ACTING IN BOTH TENSION & COMPR.)

SKM

Client CCC

Job Name MAURICE CARTER COURTS - BL. PUBLIC REPT.

Calcs Title VERTICAL BRACING - GROUND FLOOR

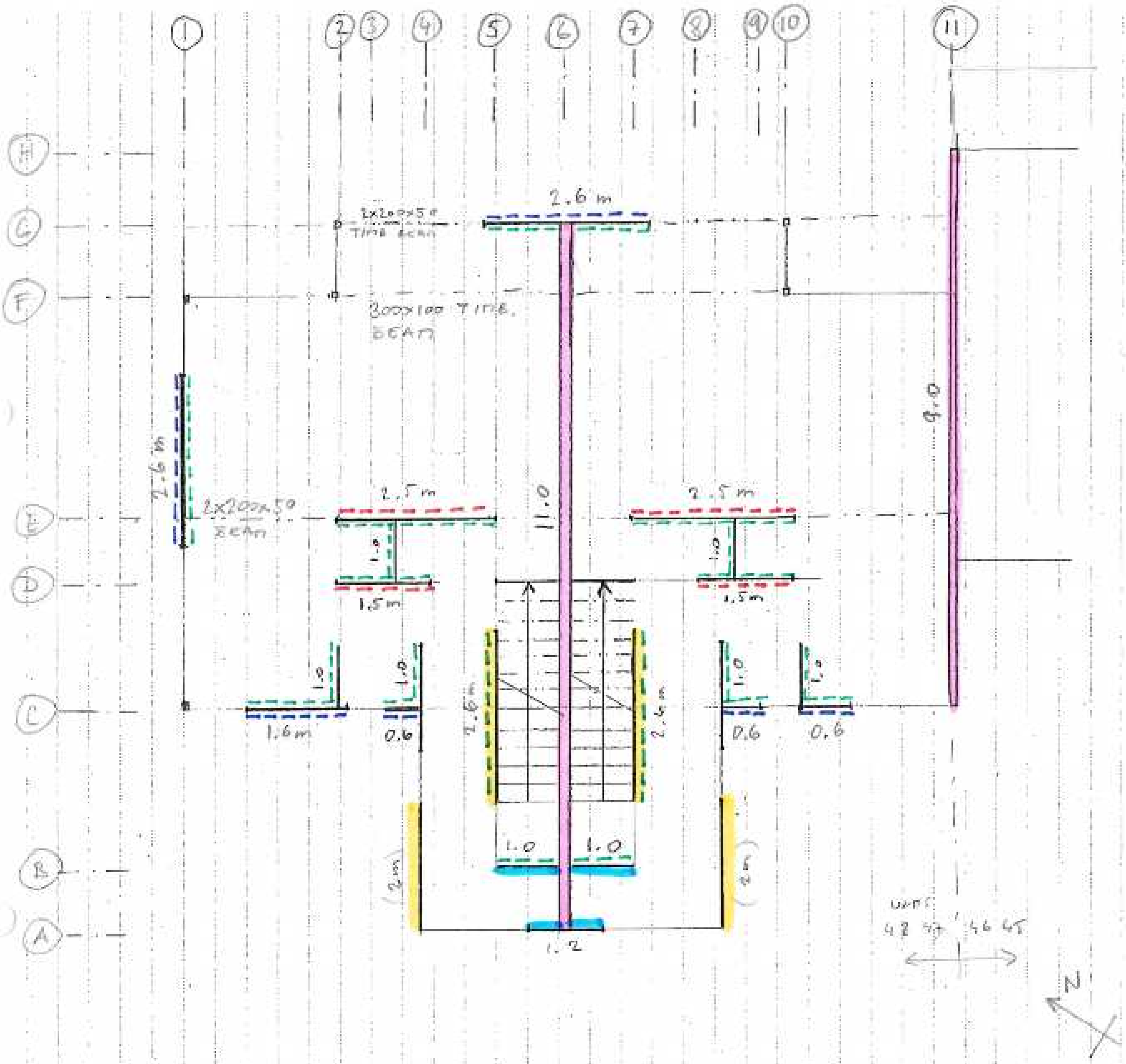
Job No. 2B01276.218

Calc. Series

Page 2-2

By TB

Date 8/2/2013



KEY

12 mm PLY BRACING

9.5 mm GIB

10 mm CUSTOM WOOD

190 mm THICK CONCR. BLOCK WALL WITH ϕ12@ 600 crs VERT

22x22x1.2 mm ANGLE BRACE (ACTING IN BOTH TENSION & COMPR.)

6 mm HARDIFLEX

SKM

Job No. ZB01276.218

Calc. Series

Client CCC

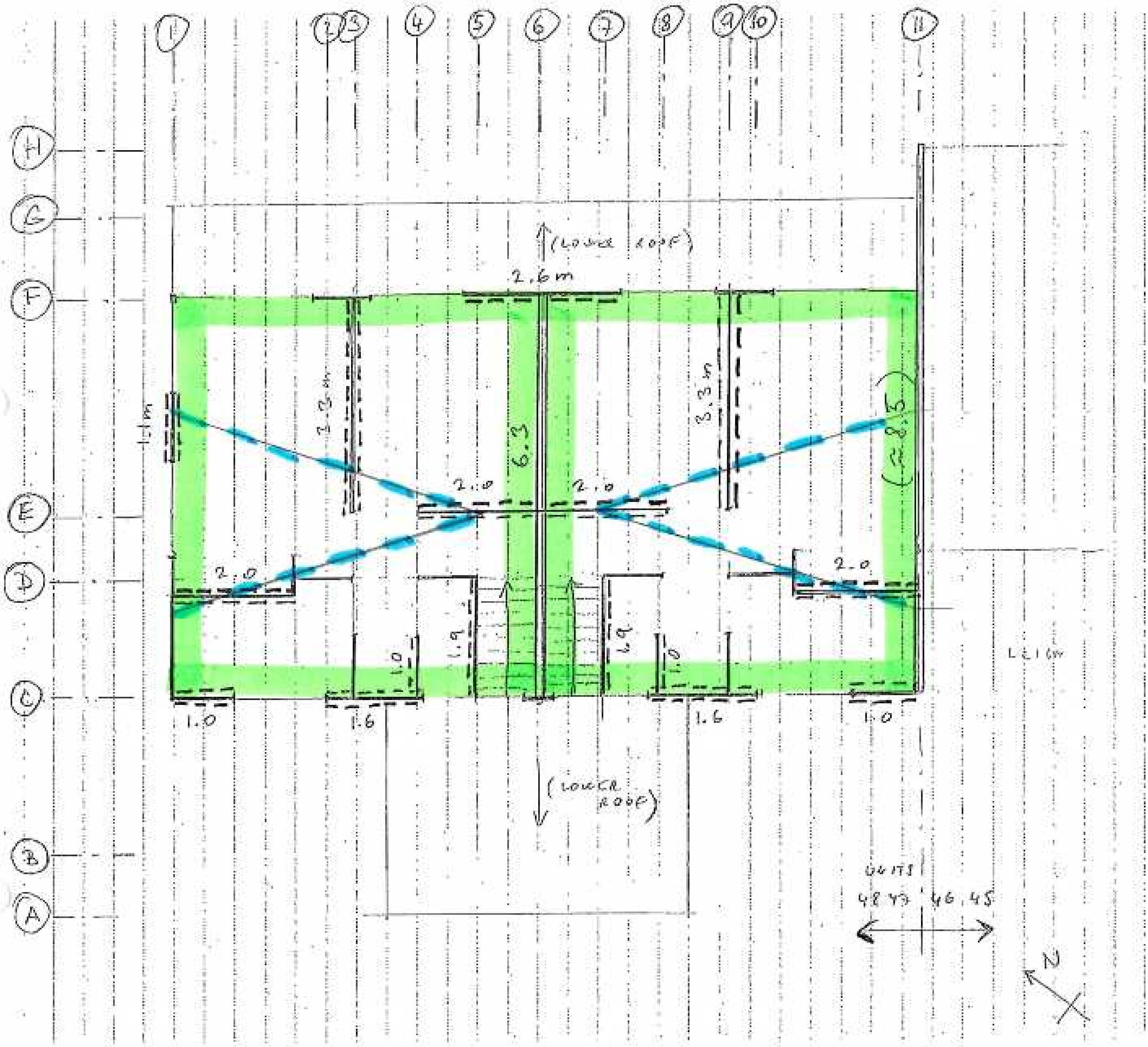
Page 2-3

Job Name MAURICE CARTER COURTS - BL. PUBLIC RENTAL

By TE

Calcs Title HORIZONTAL BRACING - FIRST FLOOR

Date 8/2/2013



KEY

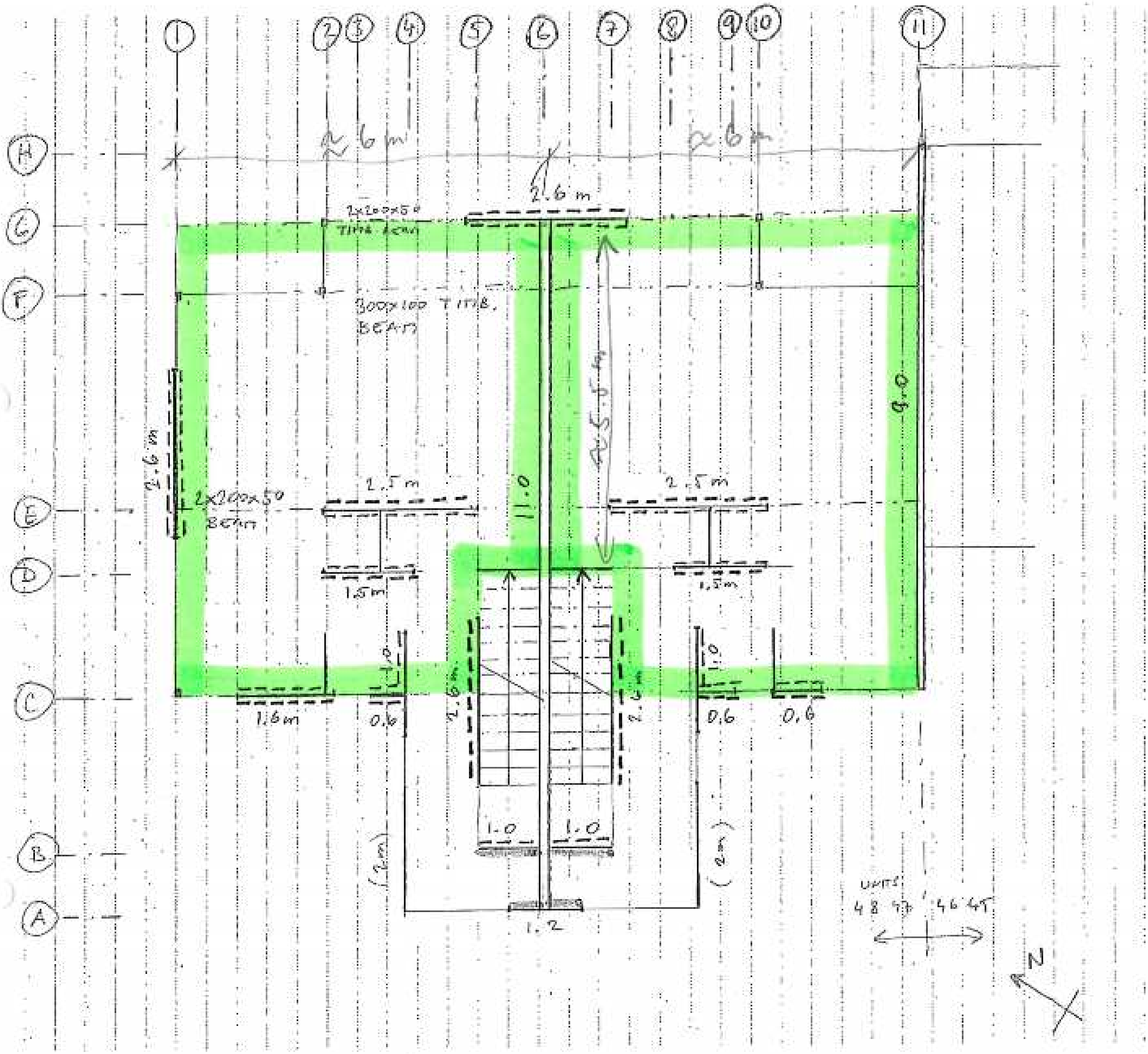
FLOOR DIAPHRAGM ABOVE FIRST FLOOR LEVEL FORMED BY 9.5mm THICK PLASTER BOARDS.

CEILING BRACES TO FIG 56 & 57 OF NZS 3604:1984 (100x50 mm TIMBER FIXED TO UPPER SIDE OF EACH ROOF TRUSS BOTTOM CHORD AND SUPPORTING WALLS AND CHEMICAL WALLS.

SKM

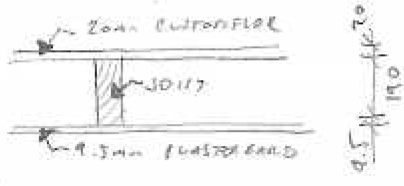
Job No. 2B01276.218
Calc. Series
Page 2-4
By TB
Date 8/2/2013

Client CCC
Job Name MAURICE CARTER COURTS - BL. PUBLIC REST.
Calcs Title HORIZONTAL BRACING - GROUND FLOOR



KEY

FLOOR DIAPHRAGM ABOVE GROUND FLOOR LEVEL
FORMED BY: 9.5mm THICK PLASTER BOARDS (CEILING)
20mm THICK CUSTOMWOOD (FLOOR FINISH)



SKM

Job No.	ZB01276.218
Calc. Series	BL. PUBL. REST.
Client	CCC
Page	2-5
Job Name	MAURICE CARTER COURTS - BL. PUBL. REST.
By	TB
Calcs Title	VERTICAL BRACING
Date	13/2/2013

CALCULATE BRACING CAPACITIES OF BRACING ELEMENTS

BR1 - PERIMETER WALL - TIMBER STUDWORK WITH 12mm PLY TO EXT. + 9.5mm GIB INTERNALLY.

SHOWN AS:

- NO TEST RESULTS OF THE ABOVE SPEC WAS AVAILABLE
- PLYWOOD WAS INSTALLED AS BRACING ELEMENT AND THEREFORE LIKELY WITH NAILING PATTERN AS PER NZS 3604:1984 (I.E. 150mm CRS TO PERIMETER, 300mm CRS INT. FRAMING)

• USING MoE TECHNICAL GUIDANCE FOR BRACING UNIT RATING
7.5mm PLYWOOD ... 95 BUs/m
(GIB OMITTED FROM CONSIDERATION - AS PER NZSEE GUIDANCE)

• BUs PER:

0.6m LONG WALL	0.6×95 =	57 BUs
1.6m	1.6×95 =	152 BUs
2.6m	2.6×95 =	247 BUs
1.0m	1.0×95 =	95 BUs
1.1m	1.1×95 =	105 BUs

BR2 - INTERNAL WALL - TIMBER STUDWORK WITH 10mm CUSTOMWOOD TO ONE SIDE + 9.5mm GIB TO THE OTHER SIDE

SHOWN AS:

- BRANZ P21 RACKING TEST FOR 9mm CUSTOMWOOD CLAD TIMBER (MAY 2004) IS AVAILABLE FOR EQ:
(PAGE X-1 TO X-5)

		BU	BU/m
• 0.6m LONG WALL ON	- CONCR. FOUND.	67	112
	- TIMBER FOUND	63	105
• 2.4m LONG WALL ON	- CONCR. FOUND	209	87
	- TIMBER FOUND.	210	87

- ADOPT FOLLOWING BRACING CAPACITY

• GF (CONCR. FOUND.)	- 1.5m LONG WALL	1.5×87 =	131 BUs
	- 2.5m LONG WALL	2.5×87 =	218 BUs
• FIRST FL. (TIMBER FOUND)	- 2m LONG WALL	2×87 =	174 BUs
	2.6m LONG WALL	2.6×87 =	226 BUs

SKM

Client: CCC
Job Name: MAURICE CARTER COURTS – BL. PUBL. RECT.
Calcs Title: VERTICAL BRACING

Job No. Z601276.218
Calc. Series: BL. PUBL. RECT.
Page: 2-6
By: TB
Date: 25/2/2013

BR 3 – INTERNAL WALL – TIMBER STUD WALL WITH 9.5mm GIB ONE SIDE (OTHER NOT CONSIDERED)

SHOWN AS ---- 9.5mm GIB

• USING NZSEE GUIDANCE (TAB 11.1 ON PAGE 11-4)

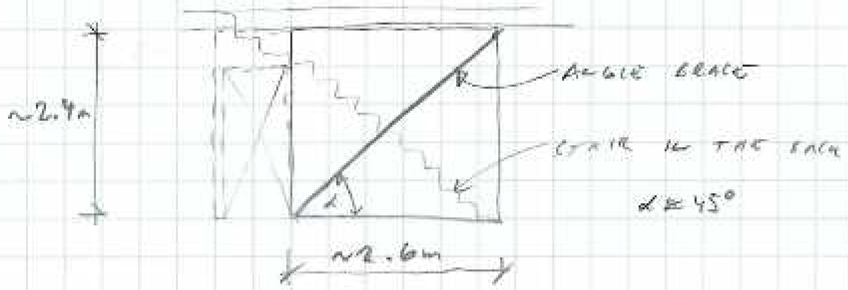
$3 kN/m \times 0.7 \times 20 = 42$ BU/m

THEREFORE: 1m LONG WALL $1.0 \times 42 = 42$ BU
1.9m $1.9 \times 42 = 80$ BU
3.3m $3.3 \times 42 = 138$ BU

BR 4 – INTERNAL WALL – TIMBER STUD WALL WITH 9.5mm GIB ONE SIDE + ANGLEBRACE

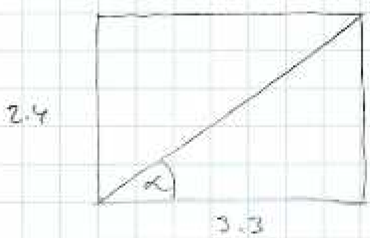
• ANGLEBRACE 22×22×1.2 SHOWN AS 9.5mm GIB / ANGLE BRACE

GF
GRID ~5/C



ONLY AVAILABLE SPECIFICATION OF ANGLE BRACE FROM PRYDA (PAGE X-6)
FOR $\alpha = 45°$ ⇒ 60 BU PER BRACE
INCLUDING GIB $= 60 + 2.6 \times 42 = 169$ BU PER WALL

FIRST FLOOR



$\alpha = 36° < 45°$ ⇒ 60 BU PER BRACE

INCLUDING GIB $= 60 + 3.3 \times 42 = 199$ BU PER WALL

SKM

Job No. 2101276.718

Calc. Series BL. PUBL. RENT.

Client CCC

Page 2-7

Job Name MAURICE CARTER COURTS - BL. PUBLIC RENT.

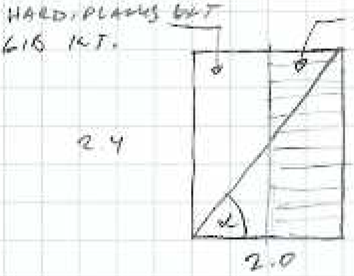
By TB

Calcs Title VERTICAL BRACING

Date 25/2/2013

BR5 - EXTERNAL WALL - HARDIPLANKS EXT + GIB/HARDIPL. INTERNALLY WITH ANGLEBRACE

SHOWN AS [orange] ANGLEBRACE



$\alpha = 50°$ >45, $<55°$ ⇒ 60 BU, 40 BU } SAY 50 BU/BRACE

EXCLUDE HARDIPLANKS/GIB FROM CONSIDERATION

BR6 - EXTERNAL WALL - TIMBER STUDWORK WITH HARDIPLANKS EXT + 6mm HARDIFLEX INTERNALLY

SHOWN AS: [blue] HARDIFLEX

- NO DATA FOR NZ AVAILABLE

- USING MANUFACTURER'S DESIGN MANUAL (JAMES HARDIE, AUSTRALIA, 2006 - (LATEST AVAILABLE) - TABLE 1, PAGES DESIGN BRACING CAPACITY (PAGE X-7 TO X-11):
  6mm JHFC SHEETS, SINGLE SIDED ... 2.8 kN/m

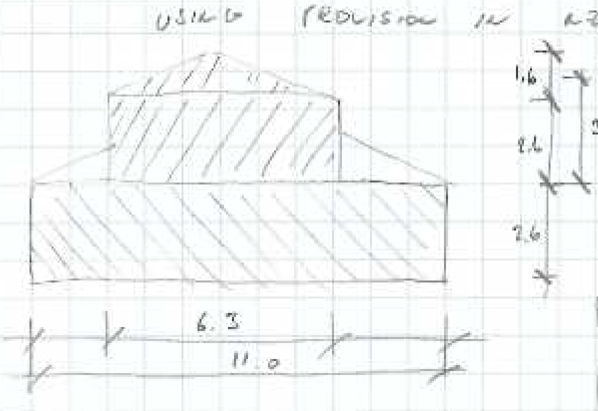
IN BU: $2.8 \times 20 = 56$ BU/m

FOR: 1m LONG WALL $1 \times 56 = 56$ BU
1.2m LONG WALL $1.2 \times 56 = 67$ BU

BR7 - INTERNAL PARTY WALL - (190mm THICK CONCR. BLOCK WALL WITH VERTICAL REINFORCEMENT ϕ12 @ 600 CRS
HORIZONTAL -//- ϕ12 @ 1200 CRS

SHOWN AS [pink] 190mm BLOCK WORK WITH REINFORCEMENT

USING PROVISION IN NZS 3604:2011



WALL ON GRID 6 PER WALL

FIRST FL: $\frac{6.3}{3.4} = 1.85$ → 100 BU/m → 630 BU (6.3×100)

GF: $\frac{11}{2.6} = 4.23$ → 200 BU/m → 2200 BU (11×200)

(WALL ON GRID 11) - FOR UNIT 47

FIRST FL. $8.5/3.4 = 2.5$ → 100 BU/m → 850 BU (8.5×100)

GF $9/2.6 = 3.46$ → 200 BU/m → 1800 BU (9×200)

SKM

Job No. ZB01276.218

Calc. Series

Client CCC

Page 2-8

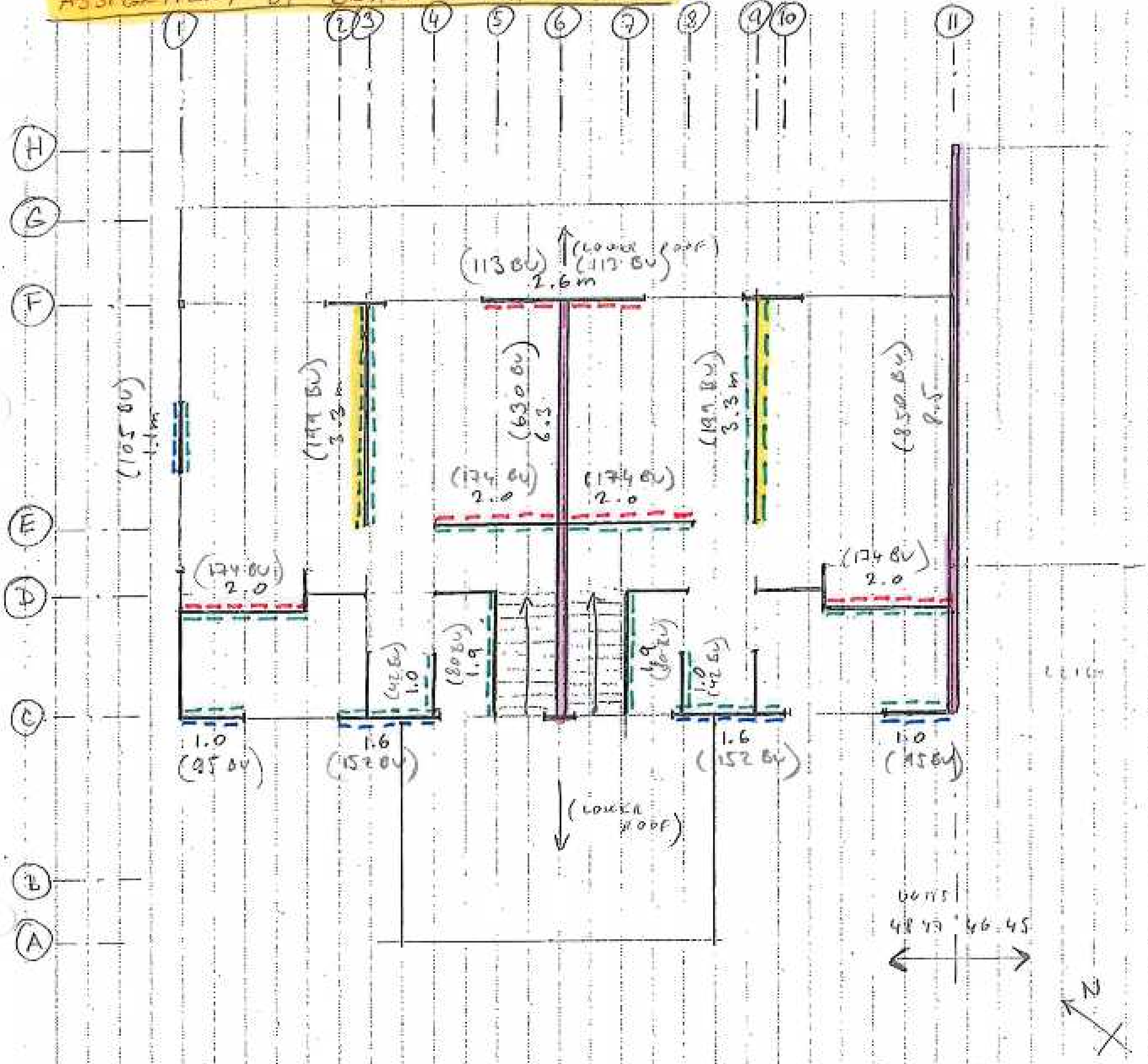
Job Name MAURICE CARTER COURTS - BL. PUBLIC RENTAL

By TB

Calcs Title VERTICAL BRACING - FIRST FLOOR

Date 8/2/2013

ASSIGNMENT OF BRACING CAPACITIES



KEY

- - - 12mm PLY BRACING

- - - 9.5mm GIB

- - - 10mm CUSTOM WOOD

190 mm THICK CONCR. BLOCK WALL WITH ~~D~~ 12 @ 600 CRS VERT.

22 x 22 x 1.2 ANGLE BRACE (ACTING IN BOTH TENSION & COMPR.)

SKM

Job No. 2B01276.218

Calc. Series

Page 2-9

Client CCC

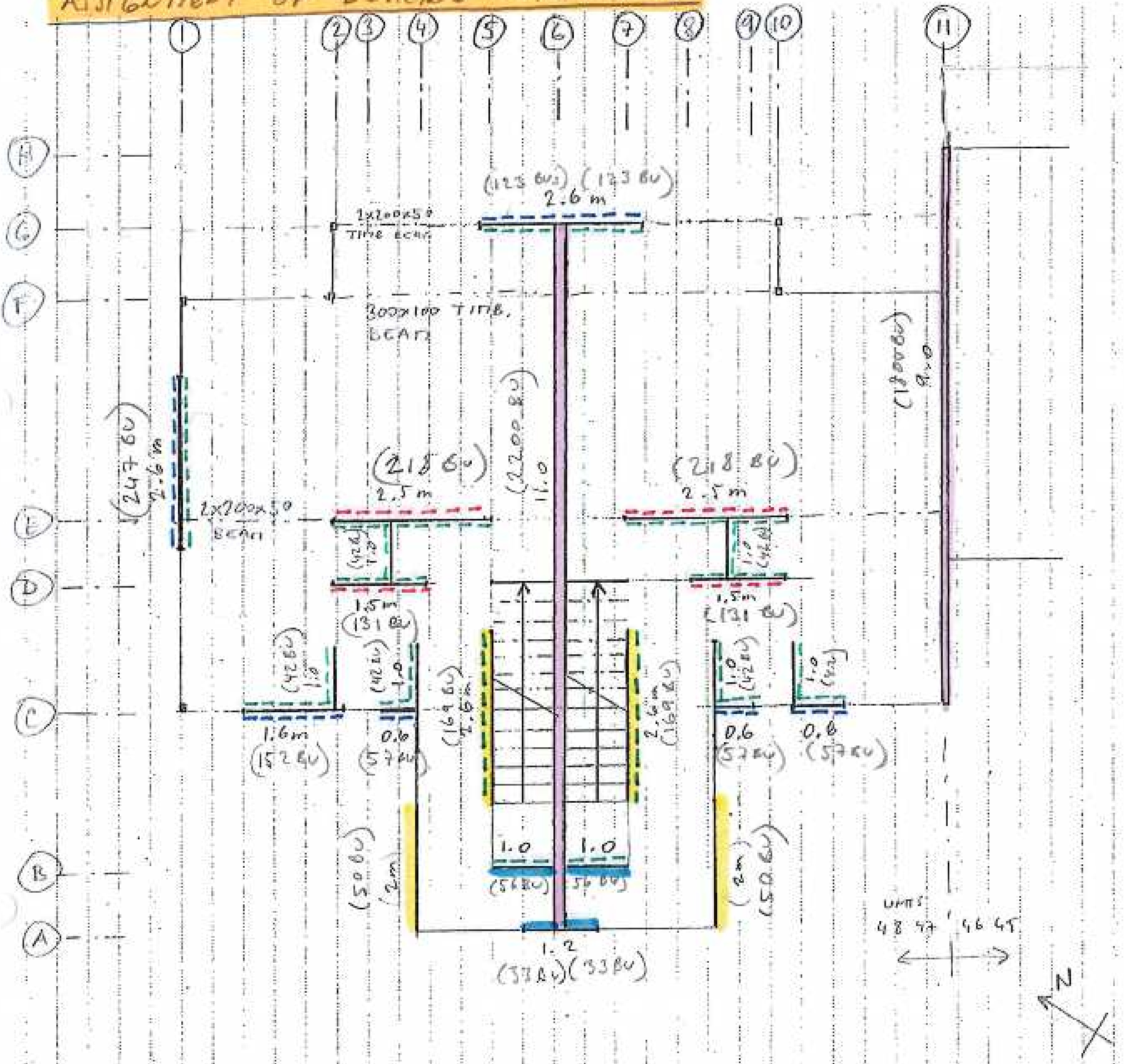
By TB

Job Name MAURICE CARTER COURTS - BL. PUBLIC REST.

Date 8/2/2013

Calcs Title VERTICAL BRACING - GROUND FLOOR

ASSIGNMENT OF BRACING CAPACITIES



KEY

- 12 mm PLY BRACING
- 9.5 mm GIB
- 10 mm CUSTOM WOOD
- 190 mm THICK CONCR. BLOCK WALL WITH φ12@ 600 crs VERT
- 22x22x1.2 mm ANGLE BRACE (ACTING IN BOTH TENSION & COMPR.)
- 6 mm HARDIFLEX

SKM

Client: CCC	Job No. ZB01276.218
Job Name: MAURICE CARTER COURTS – PUBL. RENT.	Calc. Series: BL PUBL. RENTAL
Calcs Title: VERTICAL BRACING – SUMMARY	Page: 2-10
	By: TB
	Date: 25/2/2013

COMPARE CAPACITY VS DEMAND – UNIT 48 (END UNIT)

	TRANSVERSE DIR	LONGITUDINAL DIR
FIRST FLOOR 	CAPACITY = $105 + 199 + 42 + 80 + \frac{630}{2}$ = 741 BU DEMAND = 1540 BU %NBS = $\frac{741 \times 100}{1540}$ = **48% NBS**	CAPACITY = $113 + 174 + 174 + 95 + 152$ = 708 BU DEMAND = 800 BU %NBS = $\frac{708}{800} \times 100$ = **88% NBS**
GROUND FL. 	CAPACITY = $247 + 3 \times 42 + 169 + 50 + \frac{2200}{2}$ = 1692 BU DEMAND = 3540 BU %NBS = $\frac{1692}{3540} \times 100$ = **48% NBS**	CAPACITY = $123 + 218 + 131 + 152 + 57 + 56 + 33$ = 770 BU DEMAND = 1840 BU %NBS = $\frac{770}{1840}$ = **42% NBS**

THE LOWEST BRACING CAPACITY @ GROUND FLOOR IN LONGITUDINAL DIRECTION ⇒ **42% NBS**

UNIT 45 (END UNIT AT THE OTHER END)

- SAME RESULTS AS PER UNIT 48 APPLY

UNIT 46 & 47

TRANSVERSE DIRECTION
- FIRST FL. CAP = $199 + 42 + 80 + \frac{630}{2} + \frac{830}{2} = 1061$ BUs
  %NBS = $1061 \times 100 / 1540$ = **69% NBS**
- GROUND F. CAP = $3 \times 42 + 169 + 50 + \frac{2200}{2} + \frac{1800}{2} = 2345$ BUs
  %NBS = $2345 \times 100 / 3540$ = **66% NBS**

LONGITUDINAL DIRECTION – SIMILAR TO UNIT 48 (MARGINAL DIFFERENCE IN CAPACITY & DEMAND)

SKM

Job No. 2001276.218

Calc. Series PUBL. RENT

Client CCC

Page 3-1

Job Name MAURICE CARTER'S COURT – PUBL. RENT.

By TB

Calcs Title MASONRY WALLS

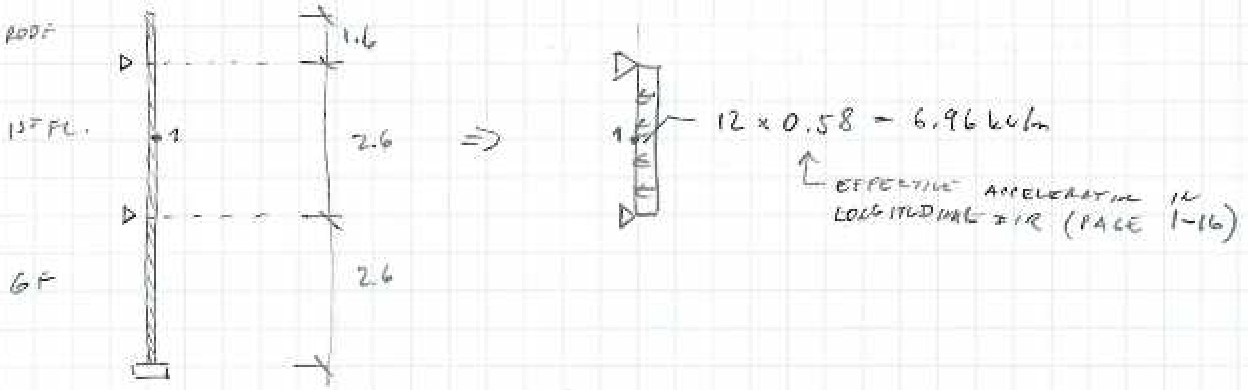
Date 11/03/2013

CHECK CAPACITY OF MASONRY PARTITION WALLS

A) OUT-OF-PLANE FLEXURE

APPLY SEISMIC MASS IN PERPENDICULAR DIRECTION ONTO THE WALL

TAKE CONSERVATIVELY 1 m OF WALL AT 1ST FLOOR (PINNED TOP & BOTTOM)



WALL MASS
ASSUME 0.2 m THICK CONCRETE .... $0.2 \times 24 = 12\ kN/m^2$

LATERAL LOADING ON WALL $= 12 \times 0.58 = 6.96\ kN/m$

AXIAL LOADING ON WALL $= 12\ kN/m$ (TAKE WEIGHT OF WALL ONLY)

$M_1^* = \frac{1}{8} \times 6.96 \times 2.6^2 = 5.9\ kNm$ (AT POINT 1)

$N_1^* = \frac{2.6}{2} \times 12 = 15.6\ kN$ (AT POINT 1)

SKM

Client: CCC

Job Name: MAURI OF CARTER'S COURT – PUBL. RELT

Calcs Title: MASONRY WALLS

Job No. ZB 01276.218

Calc. Series: PUBL. RELT

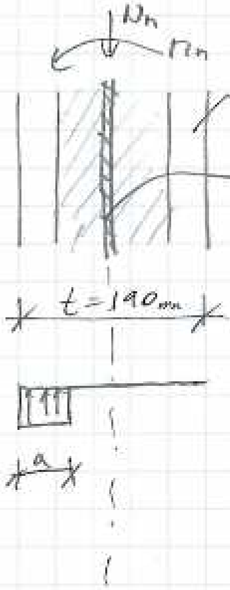
Page: 3-2

By: TB

Date: 11/3/2013

CALCULATE OUT-OF-PLANE BENDING CAPACITY OF THE WALL

- USE EXAMPLE "3.3 OUT-OF-PLANE FLEXURE" OF "USER'S GUIDE TO NZS 4230:2004" BY NEW ZEALAND CONCRETE MASONRY ASSOCIATION INC". (2004)



$f'_m = 12\,MPa$ (OBSERVATION TYPE B)

$f_y = 250\,MPa$ (CONSERVATIVELY) D12 @ 600 crs

$As = 188.5\ mm^2/m$

FOR $N_n = 15.6\ kN$ AND $\phi = 0.85$ (FLEXURE)

$$a = \frac{N_n + As\,f_y}{0.85\,f'_m \times 1.0} = \frac{15.6\times10^{-3} + 188.5\times10^{-6}\times 250}{0.85\times 12\times 1.0}$$

$$= 0.00615\ m = \underline{6.15\ mm}$$

$$\phi M_n = 0.85\left(N_n + As\,f_y\right)\left(\frac{t}{2} - \frac{a}{2}\right)$$

$$= 0.85\left(15.6\times10^{-3} + 188.5\times10^{-6}\times 250\right)\left(\frac{190}{2} - \frac{6.15}{2}\right)\times10^{-3}$$

$$= 0.85\,(0.062725) \times 0.091925$$

$$= 0.0049\ MNm$$

$$= \underline{4.9\ kNm}$$

COMPARE CAPACITY Vs DEMAND

$\phi M_n = 4.9\ kNm < M_1^* = 5.9\ kNm$ FAILS

$$\%\ NBS = \frac{4.9\times100}{5.9} = \boxed{83\%\ NBS}$$

SKM

Job No. ZB01276.218

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Job Name MAURICE CARTER COURTS – PUBL. RENT

By TB

Calcs Title MASONRY WALLS

Date 11/03/2013

B) IN PLANE SHEAR

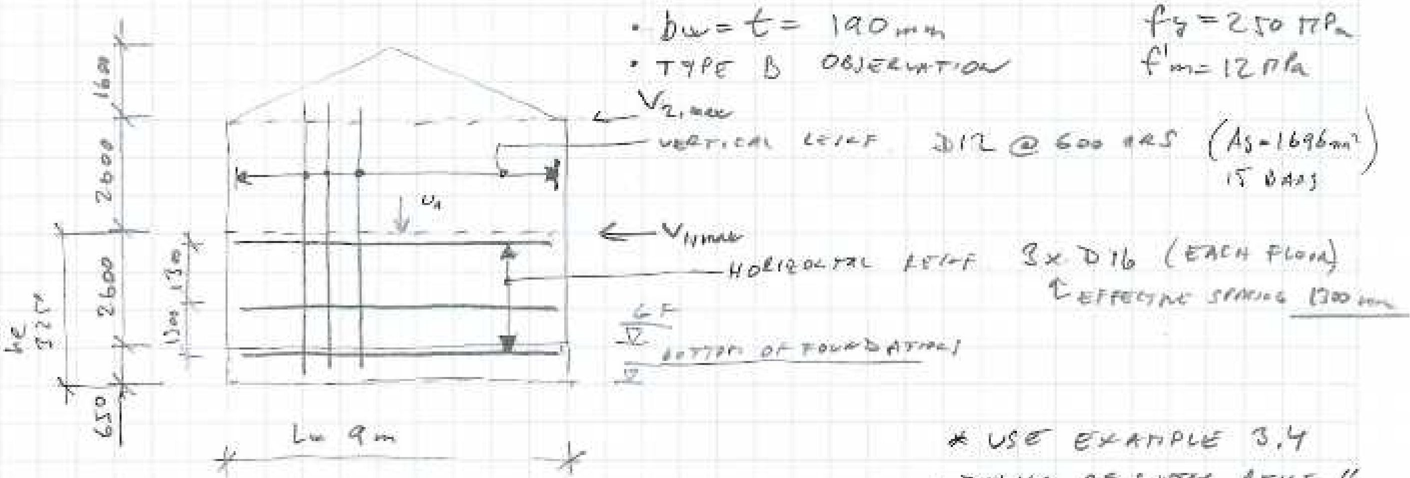
MASONRY WALLS CONTRIBUTE TO THE BRACING CAPACITY OF THE BUILDING IN THE TRANSVERSE DIRECTION AS SHOWN ON PAGES 2-1 & 2-2

CALCULATE CAPACITY OF WALL ON GRID 11

- $b_w = t = 190$ mm
- TYPE B OBSERVATION

$f_y = 250$ MPa

$f'_m = 12$ MPa



* USE EXAMPLE 3.4 "DESIGN OF SHEAR REINF." IN USER'S GUIDE TO NZS 4230:2004.

CALCULATE $V_{1,max}$

$$V_{1,max} = v_{n,1}\ b_w\ d = (v_m + v_p + v_s)\ b_w d$$

- v_m (CONTRIBUTION OF MASONRY)

  - $p_w = A_s/(b_w d) = 1696/(190 \times 0.8 \times 9000) = 0.0012$
  - $C_1 = 33\ p_w \frac{f_y}{300} = 33 \times 0.0012 \times \frac{250}{300} = 0.034$
  - $h_e/L_w = 3250/9000 = 0.361 \rightarrow C_2 = 0.42(4 - 1.75(h_e/L_w))$

$$= 0.42(4 - 1.75 \times 0.361)$$

$$C_2 = 1.415$$

  - $v_{bm} = 0.7$ MPa (TYPE B OBSERVATION)

$\underline{v_m} = (C_1 + C_2)\ v_{bm} = (0.034 + 1.415) \times 0.7 = \underline{1.014 \text{ MPa}}$

SKM

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Job Name MAURICE CARTER'S COURTS - PUBL RENT

By TB

Calcs Title MASONRY WALLS

Date 11/03/2013

- V_P (CONTRIBUTION OF AXIAL LOAD)

AXIAL LOAD - ASSUME WEIGHT OF WALL ABOVE ONLY

SAY $N_1 = 0.19 \times 9 \times 2.6 \times 20 = 88$ kN

↑ LIGHTER THAN RC

$- p = \dfrac{15 \text{ BARS D12}}{b_w \times L_w} = \dfrac{1696}{190 \times 9000} = 0.001$

$- p \dfrac{f_y}{f'_m} = 0.001 \dfrac{250}{12} = 0.021$

$- \dfrac{N_n}{f'_m \, L_w \, t} = \dfrac{88 \times 1000}{12 \times 9000 \times 190} = 0.004$

TABLE 6

$C/L_w = 0.0326$

(INTERPOLATED)

↓

$C = 0.0326 \times L_w$

$= 0.0326 \times 9000$

$= 293.4$ mm

UNREINFORCED MASONRY ($\beta = 0.85$)

$- a = \beta \times C = 0.85 \times 293.4 = 249.4$ mm

$- \tan\alpha = \dfrac{L_w/2 - a/2}{h} = \dfrac{9000/2 - 249.4/2}{3250} = 1.346$

$\underline{v_P} = 0.9 \dfrac{N_1}{b_w \times d} \times \tan\alpha = 0.9 \dfrac{88 \times 1000}{190 \times 0.8 \times 9000} \times 1.346 = \underline{0.0779 \text{ MPa}}$

- v_S (CONTRIBUTION OF HORIZONTAL REINFORCEMENT)

$- C3 = 0.8$ ($C3 = 0.8$ FOR WALLS)

$- A_v = 1 \times D16 = 201 \text{ mm}^2$ (AREA OF ONE BAR)

$- S = 1300$ mm (SPACING)

$\underline{v_S} = C_3 \dfrac{A_v \times f_y}{b_w \times S} = 0.8 \dfrac{201 \times 250}{190 \times 1300} = \underline{0.163 \text{ MPa}}$

- $v_n = 1.014 + 0.0779 + 0.163 = \underline{1.255 \text{ MPa}}$

- OVERALL CAPACITY

$V_{1,max} = v_n \times b_w \times d = 1.255 \times 10^3 \times 0.19 \times 0.8 \times 9$

$= 1716$ kN

$\phi V_{1,max} = 0.85 \times 1716 = 1460$ kN ($\approx 29\,000$ BUs) → $\gg$ $\underline{\underline{100\% \text{ NBS}}}$

SKM

Job No. Z601276.218

Calc. Series PUBL. RENT

Client CCC

Page 4-1

Job Name MAURICE CARTER'S COURTS – PUBLIC RENTAL

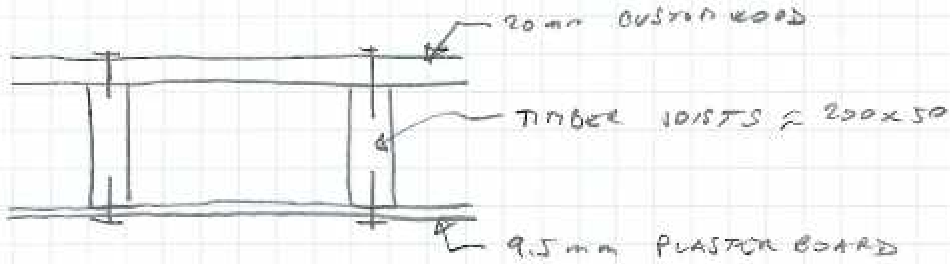
By TB

Calcs Title FLOOR DIAPHRAGM

Date 12/3/2013

FLOOR DIAPHRAGM – PHILOSOPHY / CAPACITY

- FLOOR DIAPHRAGM IS FORMED BY 20mm DEEP CUSTOM WOOD PANELS ATTACHED TO THE TIMBER JOISTS. ADDITIONAL CAPACITY CAN BE CONSIDERED DUE TO EXISTANCE OF PLASTERBOARD CEILING ATTACHED DIRECTLY TO THE UNDERNEATH OF FLOOR JOISTS.



- SEE PAGE 2-4 FOR PLAN OF FLOOR DIAPHRAGM.

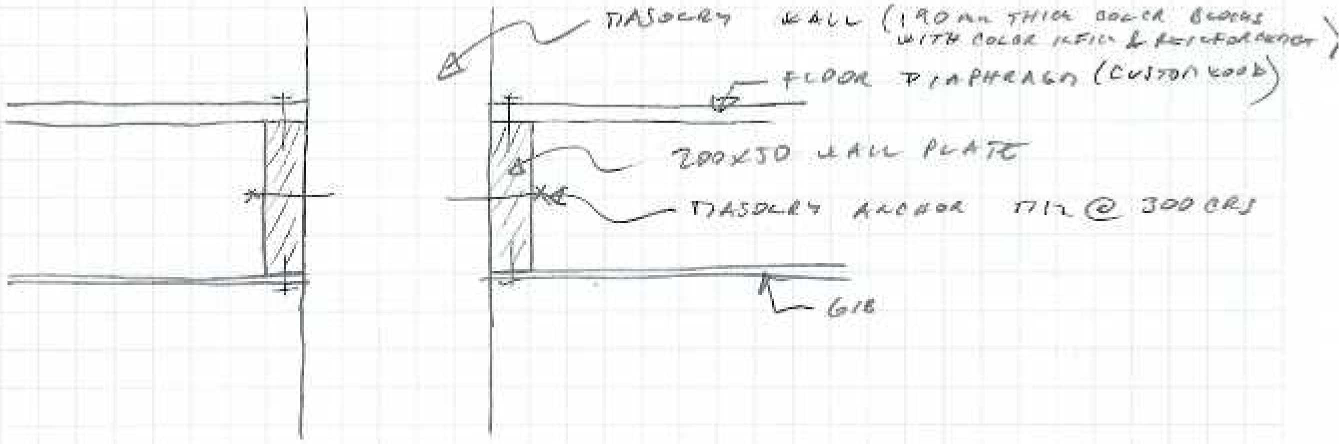
- BY INSPECTION, FLOOR DIAPHRAGM MEETS REQUIREMENTS OF NZS 3604:2011 PROVIDED IT IS SUFFICIENTLY FIXED TO THE SUPPORTING WALLS.

$\boxed{= 100\%\ \text{NBS}}$

Job No.	ZB 01276.218
Calc. Series	PUBL. REPT.
Client	CCC
Page	4-2
Job Name	MAURICE CARTER'S COURTS – PUBL. RPT
By	TB
Calcs Title	FLOOR DIAPHRAGM
Date	12/3/2013

CHECK CONNECTIVITY BETWEEN FLOOR DIAPHRAGM AND MASONRY WALL.

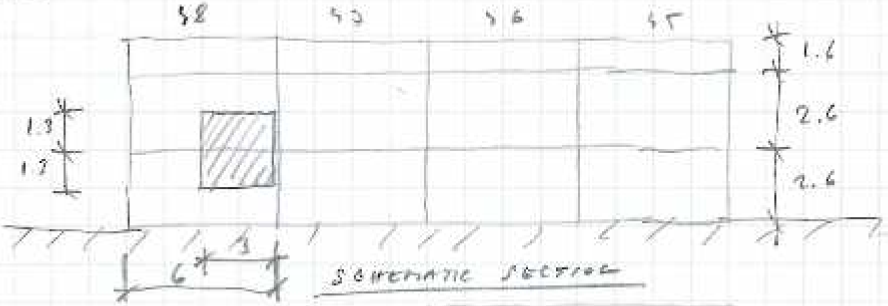
CONFIGURATION



- MASONRY ANCHORS PRIMARILY LOADED IN SHEAR
  - VERTICAL (DUE TO WEIGHT OF THE FLOOR & FLOOR LOADS)
    - CAN BE NEGLECTED SINCE VERTICAL LOADS ARE PRIMARILY TAKEN BY FLOOR JOIST
  - HORIZONTAL (DUE TO SEISMIC LOAD – I.E. FLOOR WEIGHT & FLOOR LOADS)

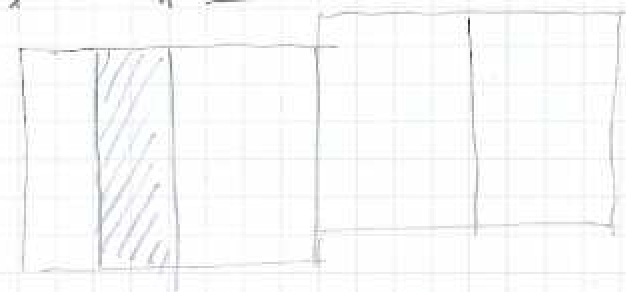
• CALCULATE HORIZONTAL SHEAR ALONG THE EDGE OF FLOOR DIAPHRAGM

SEISMIC MASS CONSIDERED



SCHEMATIC SECTION

THIS SECTION IS NOT REQUIRED. SEE FOLLOWING PAGE



SCHEMATIC PLAN

SKM

	Job No. ZO 01276.219
	Calc. Series PUBLIC RENTAL
Client CCC	Page 4-3
Job Name MAURICE CARTER'S COURTS – PUBLIC RENTAL	By TB
Calcs Title FLOOR DIAPHRAGM	Date 12/3/2013

CAPACITY OF BOLT

- ASSUME TIMBER GROUP J5

$/\!/ Q_{KL}$ = LESS OF: a) $k_{11} \times f_{cj} \times d_a^2 = 2 \times 36.1 \times 0.012^2 \times 10^3 = \underline{10.4\ kN}$

b) $0.5 b_e \times f_{cj} \times d_a = 0.5 \times 0.05 \times 36.1 \times 0.012 \times 10^3 = \underline{10.83\ kN}$

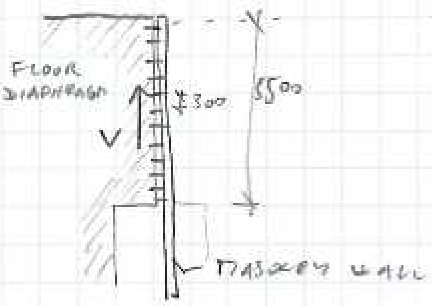
$\underline{/\!/ Q_{KL} = 10.4\ kN}$

(SHEAR STRENGTH OF M12 BOLT GRADE 4.6 = 15.1 kN)

CAPACITY OF THE WHOLE SYSTEM

$V = \dfrac{5500}{300} \times 10.4 = 198\ kN$ (= 3971 BU$_s$)

5500 – OVERAL LENGTH
300 – SPACING



THE ABOVE CAPACITY IS GREATER THAN OVERAL DEMAND FOR WHOLE BUILDING ⇒ % NBS $\boxed{\gg 100\%\ NBS}$

SKM

Job No. ZB 01276.218

Calc. Series PUBL. RENT.

Client CCC

Page

Job Name MAURICE CARTER'S COURTS – PUBLIC RENTAL

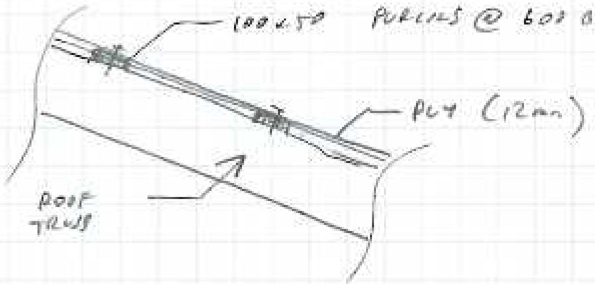
By TB

Calcs Title ROOF DIAPHRAGM.

Date 12/3/2013

ROOF DIAPHRAGM – PHILOSOPHY / CAPACITY

- ROOF DIAPHRAGM IS FORMED BY 12mm PLY WOOD BOARDS PLACED OVER THE WHOLE ROOF (ALL ROOF PLANES) AND ATTACHED TO ROOF TRUSSES VIA 100x50 mm PURLINS



- AT CEILING PLANE, THE DIAPHRAGM IS FORMED BY 9.5mm THICK PLASTERBOARD CEILING FIXED TO THE UNDERNEATH OF THE ROOF TRUSSES VIA 40x70 BATTENS @ 400 CRS.

- ADDITIONAL STABILITY IS PROVIDED BY PROVISION OF CEILING BRACES AS DETAILED IN FIG 55 & 56 OF NZS 3604:1984. ALTHOUGH THIS IS NOT MANDATORY BY CURRENT STANDARD UNLESS SPECIFICALLY REQUIRED BY DESIGN.

• BY INSPECTION, ROOF DIAPHRAGM MEETS REQUIREMENTS OF NZS 3604:2011.

• BY INSPECTION ROOF DIAPHRAGM IS SUFFICIENTLY CONNECTED TO SUPPORTING ELEMENTS

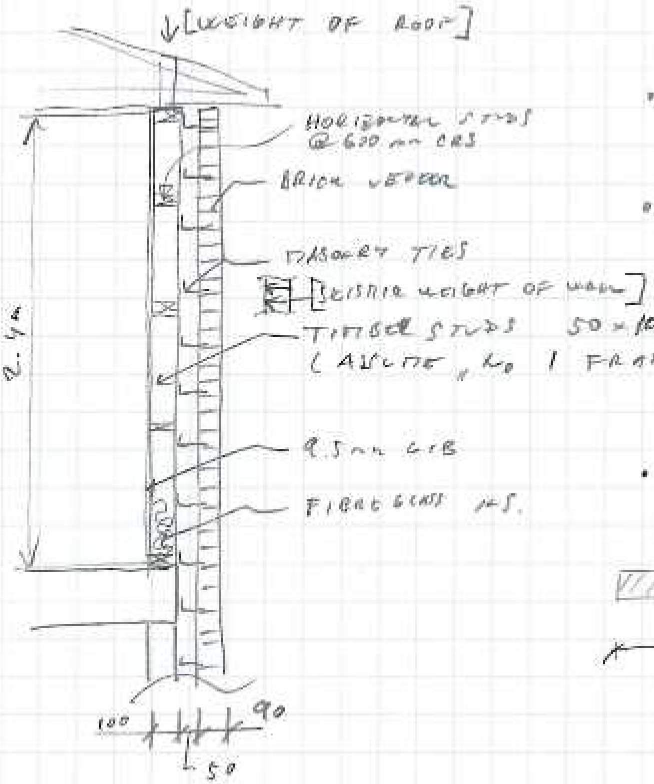
$\boxed{= 100\ \%\ NBS}$

SKM

Job No.	ZB 01276.218
Calc. Series	PUBL. RENTAL
Client	CCC
Page	6-1
Job Name	MAURICE CARTER'S COURTS - PUBLIC RENTAL
By	TR
Calcs Title	TIMBER WALLS
Date	12/3/2013

CHECK OUT OF PLANE BENDING CAPACITY OF TIMBER STUD WALLS

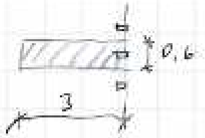
- CHECK WALL ON GRID ① AT FIRST FLOOR

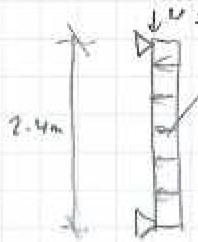


- WEIGHT OF WALL = 1.9 kN/m² AS PER PAGE 1-3.
- TAKE 600 mm WIDE STRIP (S)

$w_S = 1.9 \times 0.6 = 1.14$ kN/m

- WEIGHT OF ROOF (AXIAL FORCE IN STUD) = 0.385 kN/m² AS PER PAGE 1-1

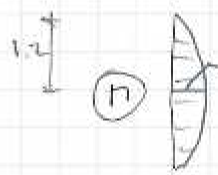




$w_S \times C_d(T_1) = 1.14 \times 1.12 = 1.277$ kN/m

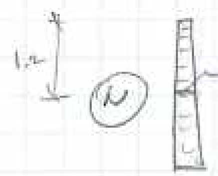
EFFECTIVE ACCELERATION IN TRANSVERSE DIRECTION (CONSERVATIVELY)

VERTICAL BENDING



$M^* = \frac{1}{8} \times 1.277 \times 2.4^2 = \underline{0.92 \text{ kNm}}$

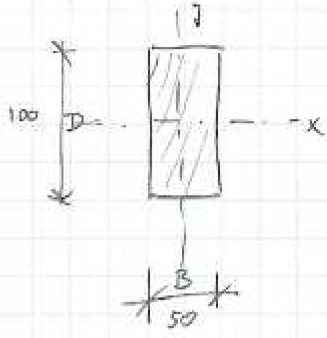
AXIAL COMPRESSION



N^* = [WEIGHT OF ROOF] + [WEIGHT OF WALL]
$= 3 \times 0.6 \times 0.385 + 1.2 \times 1.14$
$= 0.693 + 1.368 = \underline{2.06 \text{ kN}}$

(SHEAR AT BASE = 1.277 × 1.2 = 1.54 kN)

CHECK TIMBER ELEMENT FOR COMBINATION OF BENDING & AXIAL.



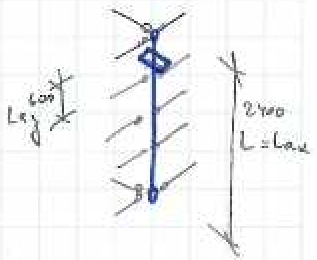
$A = 50 \times 100 = 5000 \ mm^2$

$Z_x = \frac{1}{6} \times 50 \times 100^2 = 83\,333 \ mm^3$

MATERIAL PROPERTIES:

GRADE: No 1 FRAMING

$f_b = 10$ MPa
$f_c = 15$ MPa



• CALCULATE STRENGTH OF MEMBER IN BENDING

$k_1 = 1.0$ (DURATION OF LOAD: BRIEF)
$k_4 = 1.0$ (PARALLEL SUPPORT FACTOR)
$k_5 = 1.0$ (GRID SYSTEM FACTOR)

$S_{1x} = 1.35\left[\frac{L_{ay}}{b}\left[\left(\frac{d}{b}\right)^2 - 1\right]^{0.5}\right]^{0.5} = 1.35\left[\frac{600}{50}\left[\left(\frac{100}{50}\right)^2 - 1\right]^{0.5}\right]^{0.5} = \underline{6.155 < 10}$

$k_8 = 1.0$ (STABILITY FACTOR ; $k_8 = 1$ SINCE $S_{1x} < 10$)

$\underline{M_{nx}} = k_1 k_4 k_5 k_8 f_b Z_x = 1 \times 1 \times 1 \times 1 \times 10 \times 83\,333 \times 10^{-6} = \underline{0.833 \ kNm}$

• CALCULATE STRENGTH OF MEMBER IN COMPRESSION

$k_1 = 1.0$
$k_{10} = 1.0$ (EFFECTIVE LENGTH FACTOR)

$S_2 =$ LESS OF $\frac{k_{10} L}{d} = \frac{1 \times 2400}{100} = 24$ OR $\frac{L_{ax}}{d} = \frac{2400}{100} = 24$; $S_2 = 24 < 85$ OK

$S_3 =$ LESS OF $\frac{k_{10} L}{b} = \frac{1 \times 2400}{50} = 48$ OR $\frac{L_{ay}}{b} = \frac{600}{50} = 12$; $S_3 = 12 < 85$ OK

$$k_{8,x} = a_1 + a_2 S_2 + a_3 S_2^2 + a_4 S_2^3$$
$$= 0.21 + 0.175 \times 24 + (-0.0116) \times 24^2 + 1/5000 \times 24^3$$
$$= 0.4932$$

$$k_{8,y} = a_1 + a_2 S_3 + a_3 S_3^2 + a_4 S_3^3$$
$$= 0.21 + 0.175 \times 12 + (-0.0116) \times 12^2 + 1/5000 \times 12^3$$
$$= 0.9852$$

$$\underline{N_{ncx}} = k_1 k_{8,x} f_c A = 1.0 \times 0.4932 \times 15 \times 10^3 \times 5000 \times 10^{-6} = \underline{37 \text{ kN}}$$
$$\underline{N_{cy}} = k_1 k_{8,y} f_c A = 1.0 \times 0.9852 \times 15 \times 10^3 \times 5000 \times 10^{-6} = \underline{73.9 \text{ kN}}$$

• CHECK COMBINED BENDING & COMPRESSION

UTILISATION

$\boxed{M_x \& N_{cx}}$ $$\frac{M_x^*}{\phi M_{nx}} + \frac{N^*_c}{\phi N_{ncx}} = \frac{0.92}{0.8 \times 0.833} + \frac{2.06}{0.8 \times 37} = 1.38 + 0.07 = \underline{1.45}$$

$\boxed{M_x \& N_{cy}}$ $$\left(\frac{M_x^*}{\phi M_{nx}}\right)^2 + \frac{N_c^*}{\phi N_{ncy}} = \left(\frac{0.92}{0.8 \times 0.833}\right)^2 + \frac{2.06}{0.8 \times 73.9} = 1.905 + 0.034 = \underline{1.94}$$

CONVERT 70 % NBS $\frac{100}{1.94}$ = $\boxed{51\% \text{ NBS}}$

6-4

CALCULATIONS

VERIFICATION OF 6-2 & 6-3 USING SPREADSHEET (DOUBLE CHECK)

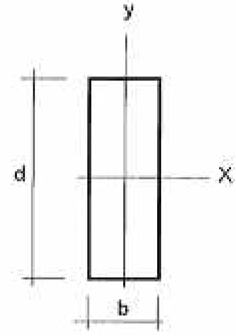
SINCLAIR KNIGHT MERZ
SKM

PROJECT	CCC Maurice Carter Courts - Public rentals		
PART OF STRUCTURE	Timber stud	PROJECT No. ZB01276.218	REVISION 0
	Perimeter wall subject to out-of-plane bending and compression	DATE 12 Mar 13	BY TB

Timber Beam and column Design

Spreadsheet Rev 0.3

This calculation is for undertaking combined design checks of rectangular timber beams using NZS 3603:1993 Timber Structure Standard. This calculation only covers single members, built-up members need specific considersation regarding slenderness and effective length.
Ensure that connections are always checked as these often govern timber design.



INPUT

Section 100 x 50, No 1 Framing Radiata Pine, Dry (m/c = 16 %)

Beam Depth (nominal)	100	mm
Beam Breadth (nominal)	50	mm
Timber Species	Radiata Pine	
Grade of Timber	No 1 Framing	
Timber Moisture content	Dry (m/c = 16 %)	

Note: The timber moisture content should be matched closely to the nevironmental conditions the timber will be in. For example for internal timber it is recommended to use Dry.

Number of members effectively connected		1		
Span of member/column height	L	2400	mm	
Dist betw restraints against lat. movement about X	L_{ax}	2400	mm	(i.e.restraints in dir Y at....mm)
Dist betw restraints against lat. movement about Y	L_{ay}	600	mm	(i.e.restraints in dir X at.....mm)
Lateral restraint provided at the tension edge?		No		→ S1 according to Cl. 3.2.5.2

Note: Effectively connected means that they are constrained to the same deformation

Tbls 2.2 & 2.3	Bending strength	fb	10.0 MPa	
Tbls 2.2 & 2.3	Compression Strength	fc	15.0 MPa	
Tbls 2.2 & 2.3	Tension Strength	ft	4.0 MPa	
Tbls 2.2 & 2.3	Shear Strength	fs	3.8 MPa	
Tbls 2.2 & 2.3	Elastic Modulus	E	6.0 GPa	
Tbls 2.2 & 2.3	Lower Bound Elastic modulus	E_{lb}	4.0 GPa	
	Final Modulus of Elasticity	E	4.0 GPa	
	Actual Depth	d	100 mm	(check if same as nominal above)
	Actual Breadth	b	50 mm	(check if same as nominal above)

LOADING

Major axis bending moment	M*x	0.92	kNm	(bending about x)
Minor Axis bending moment	M*y	0	kNm	(bending about y)
Axial load (+ve compression)	N*	2.06	kN	(+compression, - tension)
Shear Force major direction	V^*_x	1.54	kN	(shear in dir x)
Shear Force minor direction	V^*_y	0	kN	(shear in dir y)
Loading condition		Brief (wind, earthquake, impact etc)		
Strength reduction factor	ø	0.8		

SECTION PROPERTIES

Section Area	A	5000 mm^2		
Self weight		0.025 kg/m		
Second moment of area about major axis	Ix	4.17 $\times 10^6 mm^4$		(about x)
Second moment of area about minor axis	Iy	1.04 $\times 10^6 mm^4$		(about y)
Section modulus about major axis	Zx	83.33 $\times 10^3 mm^3$		(about x)
Section modulus about minor axis	Zy	41.67 $\times 10^3 mm^3$		(about y)
Shear Area (loaded about major axis)	A_s	3333 mm^2		
Slenderness coefficient (for bending about X - major)	S_{1x}	6.155	< 85 OK	as Cl. 3.2.5.2
Slenderness coefficient (for bending about Y - minor)	S_{1y}	0.000	< 85 OK	(always 0)
Slenderness coefficient (compr. buckling about X)	S_2	24.000	< 85 OK	
Slenderness coefficient (compr. buckling about Y)	S_3	12.000	< 85 OK	

B-5

CALCULATIONS

SINCLAIR KNIGHT MERZ

PROJECT	CCC Maurice Carter Courts - Public rentals		
PART OF STRUCTURE	Timber stud	PROJECT No. ZB01276.218	REVISION 0
	Perimeter wall subject to out-of-plane bending and compression	DATE 12 Mar 13	BY TB

Duration of load factor	k_1	1.0	
Parallel support factor	k_4	1.00	
Grid system factor (assumed to be 1.0)	k_6	1.00	
Stability factor, Bending about x	$k_{8,bend,x}$	1.00	(lat tors buckling about Y)
Stability factor, Bending about y	$k_{8,bend,y}$	1.00	(lat tors buckling about X)
Stability factor, Compress. - buckling about x axis	$k_{8,comp,x}$	0.49	(buckling about X - derived form S2)
Stability factor, Compress. - buckling about y axis	$k_{8,comp,y}$	0.99	(buckling about Y - derived form S3)
Effective length factor	$k_{10,x}$	1	(buckling about X)
	$k_{10,y}$	1	(buckling about Y)

STRENGTH OF MEMBER

Flexural Shear Strength	Vn	12.7 kN
Bending strength about x axis	Mnx	0.83 kNm
Bending strength about y axis	Mny	0.42 kNm
Compression strength about x axis	N_{ncx}	37.0 kN
Compression strength about y axis	N_{ncy}	73.9 kN
Tension strength	N_{nt}	20.0 kN

3.2.3.1		**Flexural Shear Strength**							**Utilisation Ratio**		
	Vx	V^*_x	≤	ϕV_n							
		1.5	<	10.1					0.15		
	Vy	V^*_y	≤	ϕV_n						max	
		0.0	<	10.1					N/A	0.15	
3.2.4		**Strength in Bending**									
	Mx	M^*_x	≤	ϕM_{nx}							
		0.9	>	0.7					1.38		
	My	M^*_y	≤	ϕM_{ny}						max	
		0.0	<	0.3					N/A	1.38	
3.3.4		**Compressive Strength**									
	Ncx	N^*_c	≤	ϕN_{ncx}							
		2.1	<	29.6					0.07		
	Ncy	N^*_c	≤	ϕN_{ncy}						max	
		2.1	<	59.1					0.03	0.07	
3.4.2		**Tension Strength**									
	Nt	N^*_t	≤	ϕN_{nt}						max	
		0.0	<	16.0					N/A	0.00	
3.5.1		**Combined Bending and Compression**									
	My & Ncy	$\frac{M^*_y}{\phi M_{ny}}$	+	$\frac{N^*_c}{\phi N_{ncy}}$	≤	1.0					
		0.0	+	0.0	=	0.0	<	1.0	N/A		
	Mx & Ncx	$\frac{M^*_x}{\phi M_{nx}}$	+	$\frac{N^*_c}{\phi N_{ncx}}$	≤	1.0					
		1.4	+	0.1	=	1.4	>	1.0	1.45		
	Mx & Ncy	$\left(\frac{M^*_x}{\Phi M_{nx}}\right)^2$	+	$\frac{N^*_c}{\phi N_{ncy}}$	≤	1.0					$\frac{100}{1.94} = 51\%$
		1.9	+	0.0	=	1.9	>	1.0	1.94		
	My & Ncx	$\left(\frac{M^*_y}{\Phi M_{ny}}\right)^2$	+	$\frac{N^*_c}{\phi N_{ncx}}$	≤	1.0				max	
		0.0	+	0.1	=	0.1	<	1.0	N/A	1.94	
3.6		**Combined Bending and Tension**									
	Mx & Nt	$\frac{M^*_x}{\phi M_{nx}}$	+	$\frac{N^*_t}{\phi N_{nt}}$	≤	1.0					
		1.4	+	0.0	=	1.4	>	1.0	N/A		
	My & Nt	$\frac{M^*_y}{\phi M_{ny}}$	+	$\frac{N^*_t}{\phi N_{nt}}$	≤	1.0				max	
		0.0	+	0.0	=	0.0	<	1.0	N/A	0.00	

SKM

	Job No. ZB 01276.218
	Calc. Series PUBL. RENT.
Client CCC	Page 7 - 1
Job Name MAURICE CARTERS COURTS - PUBLIC RENTAL	By TB
Calcs Title FOUNDATIONS	Date 12/3/2013

CONSIDER FOUNDATIONS IN TERMS OF SEISMIC RESISTANCE

BUILDING FOUND TYPICALY ON THE TOP OF SLAB ON FILL (WITH STRIP FOOTING TO THE PERIMETER) APART FROM MASONRY WALLS THAT ARE FOUND ON STRIP FOOTING.

BY INSPECTION - OVERTURNING NOT LIKELY TO OCCUR
- LATERAL SPREAD OF GROUND UNLIKELY TO OCCURE, HENCE FOUNDATIONS DON'T HAVE TO BE DESIGNED FOR THIS PURPOSE
- LIQUEFACTION OF SUPPORTING GROUND UNLIKELY TO OCCURE ....(AS ABOVE)
- SLIDING UNLIKELY TO OCCUR (SHEAR KEY WITHIN MASONRY WALL FOUNDATIONS)

MAXIMUM GROUND PRESSURE WILL LIKELY DEVELOP UNDER MASONRY WALLS UNDER SEISMIC LOADING IN TRANSVERSE DIRECTION

- CHECK MAX GROUND PRESSURE (FOLLOWING PAGE)

Job No. ZB 01276.218

Calc. Series PUBL. RECT

Client CCC

Page 7-2

Job Name MAURICE CARTER'S COURTS – PUBL. RECT.

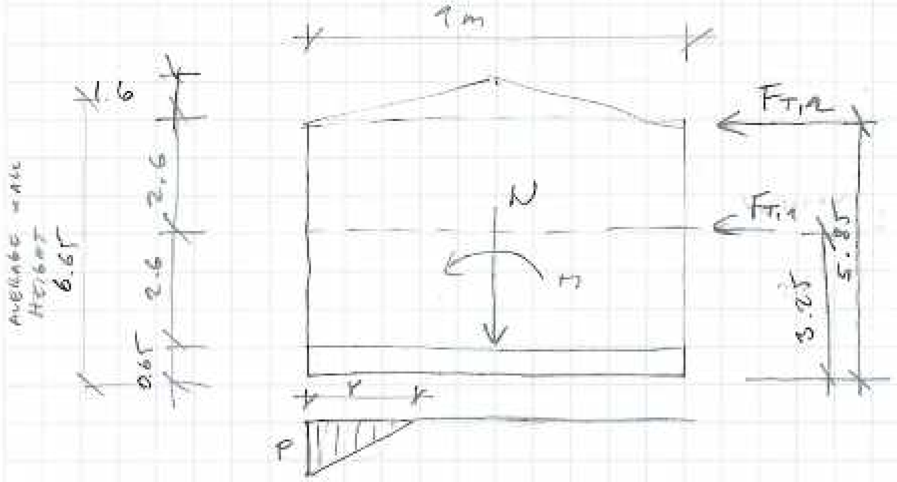
By TB

Calcs Title FOUNDATIONS

Date 12/3/2013

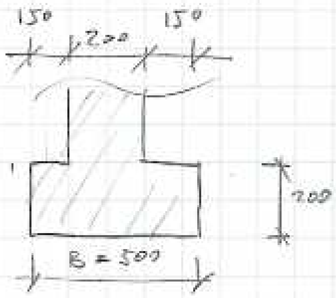
CHECK GROUND PRESSURE UNDER MASONRY WALL

– CHECK WALL ON GRID (11), TAKING HALF OF EACH ADJACENT UNIT FROM PAGE 1-15 '.



$F_{T_2} = 100$ kN

$F_{T,1} = 77$ kN



$\underline{M} = F_{T,1} \times 3.25 + F_{T,2} \times 5.85 = 77 \times 3.25 + 100 \times 5.85 = \underline{835 \text{ kNm}}$

$\underline{N}$ = WEIGHT OF STRUCTURE

- MASONRY WALL $\quad 0.19 \times 9 \times 6.65 \times 24 = 273$ kN
- ROOF $\quad 0$ (SINCE SPANNING TRANSVERSELY)

$\Sigma N = 273$ kN

- $e = M/N = 835/273 = \underline{3.06 \text{ m}} > D/6 = 9/6 = 1.5$ m

HIGH ECCENTRICITY

- $Y = 3\left(\frac{D}{2} - e\right) = 3\left(\frac{9}{2} - 3.06\right) = 4.32$ m

- $\underline{P} = \frac{2N}{BY} = \frac{2 \times 273}{0.5 \times 4.32} = \underline{253 \text{ kPa}}$ (MAX GROUND PRESSURE)

ALLOWABLE BEARING CAPACITY

$\underline{ABP} = GBP \times 0.6 = 200 \times 0.6 = \underline{120 \text{ kPa}}$

GBP = 200 kPa (FROM GEOTECHNICAL REPORT BY SKM ON 19 DEC 2012)

SAFETY FACTOR = 0.6

% NBS $\quad 120/253 \times 100 = \boxed{47 \text{ \% NBS}}$

X-1

BRANZ

BUILDING ON KNOWLEDGE

ST0571 Issue 2

P21 Racking Tests on Customwood Clad Timber Framing

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CONDITIONS OF ISSUE OF BRANZ REPORTS

P21 Racking Tests on Customwood Clad Timber Framing

1. CLIENT

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Private Bag 1001
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Christchurch
New Zealand

2. OBJECTIVE

To derive bracing ratings to be used with NZS 3604:1999 for bracing walls constructed on both timber and concrete floors, P21 racking tests were conducted on specimens which consisted timber framed walls clad on one side with 9 mm Customwood MDF sheets. Wall lengths tested were nominal 0.6 m and 2.4 m.

3. DESCRIPTION OF SPECIMENS

3.1 Product Description

The sheet material is known as 9 m thick Customwood MDF. The sheets come in size 2.44 m high by 1.22 m wide. These were cut to 2.40 m high and either 1.2 m wide or 0.6 m wide and were fixed to one side of the wall framing only. The sheets had a mean thickness of 9.2 mm and density of 6.83 kg/m^2. This report pertains to the samples provided only.

3.2 Construction of the Specimens

Four specimens were constructed for each of the two specimen lengths. For reasons discussed in Section 4.2, three of the 0.6 m long walls were used for simulated timber floor construction and one for simulated concrete floor construction. One of the 2.4 m long walls were used for simulated timber floor construction and three for simulated concrete floor construction.

All timber frames were constructed from 90 x 45 mm machine stress graded F5 kiln dried radiata pine. Figures 1 and 2 show the framing for the nominal 0.6 m and 2.4 m long walls respectively. (Note: These drawings do not show the 'P21 end restraint' described in Section 4.1.) The 'dummy' end stub studs shown in Figure 1 were omitted for the construction on the simulated concrete floor. The frames were 2.420 m high and either 0.645 m long or 2.445 m long for the nominal 0.6 m and 2.4 m long walls respectively. A single nog was used at wall mid-height. Two 90 x 3.33 mm gun nails were used to connect studs to the top and bottom plates and the nogs to the studs. Three 90 x 3.33 mm gun nails were used to connect the 'dummy' end stub studs to the studs.

Sheets were fastened to the timber framing using Hurricane nominal 30 x 2.5 mm flat head galvanised steel clouts as shown in Figures 3 and 4 for the nominal 0.6 m and 2.4 m long walls respectively. Around the perimeter of each sheet clouts were used at a maximum spacing of 150 mm with these fasteners being placed using a 10 mm edge distance. Fasteners along the top and bottom plate commenced at 75 mm from the vertical edge. For the 2.4 m wide wall, clouts were used at 300 mm centres to the intermediate studs. The sheets were not directly fixed to the nogs.

SJT RHS

3.2.1 Simulated Timber Floors

The wall base construction is shown in Figures 5 for nominal 0.6 m wide walls. The nominal 2.4 m wide walls did not used end straps or 'dummy' end stub studs.

The bottom plates of the frames were nailed through a strip of 20 mm thick particle board floor to the 150 x 100 mm timber foundation beam using pairs of 100 x 4 mm nails at nominal 600 mm centres. For the nominal 2.4 m wide walls these commenced at 50 mm from the inside face of end studs and were thereafter at nominal 600 mm centres. For the nominal 0.6 m wide walls these were at 50 mm from the inside face of the 'dummy' end stub studs.

At each end of the 0.6 m long walls, a 300 mm length of 25 mm x 1 mm galvanised mild steel strap was fixed between the studs and the foundation beam and also between the 'dummy' end stub studs and foundation beam as shown in Figure 5. Each strap was fixed to both the foundation beam and studs with six 30 mm x 2.5 mm diameter flat head galvanised clouts to each. The steel straps were installed before the cladding was fixed in place. No straps were used for construction of the nominal 2.4 m wide walls.

3.2.2 Simulated Concrete Floors

The wall base construction is shown in Figure 6. The nominal 2.4 m wide walls did not use end straps.

The bottom plate was fixed to the foundation beam with a 200 x 12 mm coach screw located 50 mm from the inside face of the end studs to simulate a concrete hold-down bolt. A 50 x 50 x 3 mm washer was used beneath the head of the coach screw. For the nominal 2.4 m wide walls there was also a bolt and washer 50 mm one side of the face of the stud at mid-wall length.

(a) Nominal 0.6 m wide wall

At each end of the bracing element, a 25 mm (wide) x 1 mm mild steel strap was wrapped under the bottom plate and fixed to the studs with six 30 mm x 2.5 mm diameter flat head galvanised clouts on each side.

(b) Nominal 2.4 m wide wall

No end straps were used on the 2.4 m wide wall.

SJT RHS

X-2

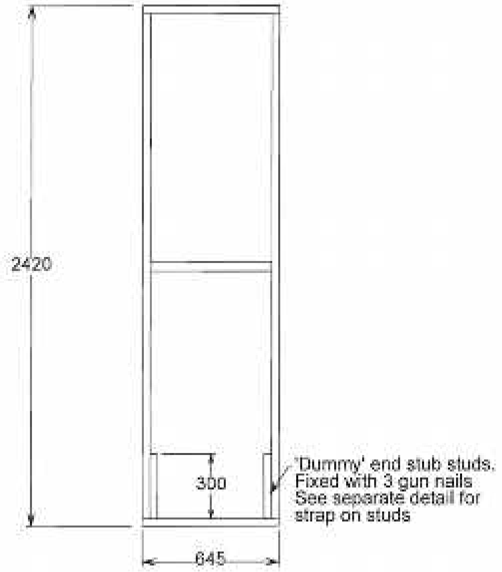


Figure 1 - Framing Used for Nominal 0.6 m long walls

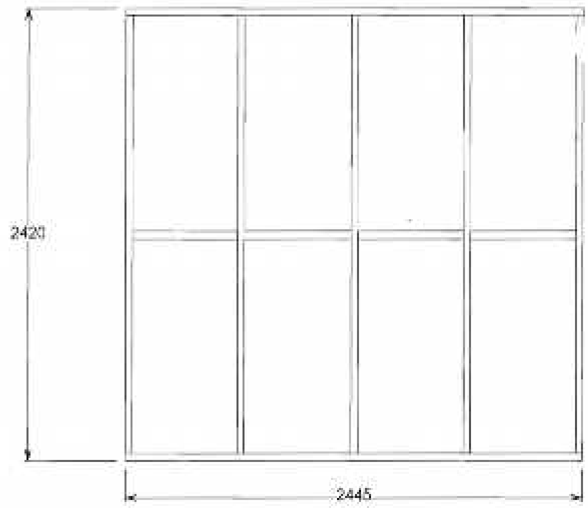
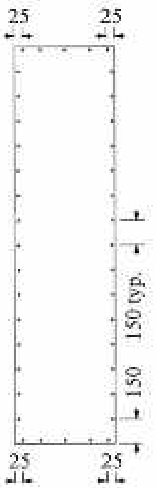
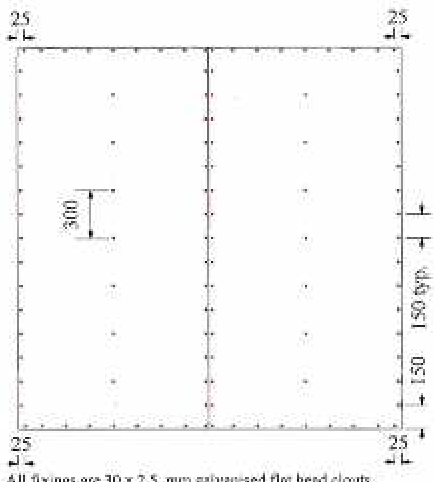


Figure 2 - Framing Used for Nominal 2.4 m long walls



All fixings are 30 x 2.5 mm galvanised flat head clouts
Edge distance is 10 mm except 25 mm at corners as shown.
Clouts around the perimeter are at 150 mm centres
No clouts to mid-height nogs.

Figure 3 – Nailing Pattern for the nominal 0.6 m long walls



All fixings are 30 x 2.5 mm galvanised flat head clouts
Edge distance is 10 mm except 25 mm at corners as shown.
Clouts around the perimeter continue at 150 mm centres
Clouts on studs at middle of sheets are at 300 mm centres starting 300 m
from sheet edge.
No clouts to mid-height nogs.

Figure 4 - Nailing Pattern for the nominal 2.4 m long walls

X-3

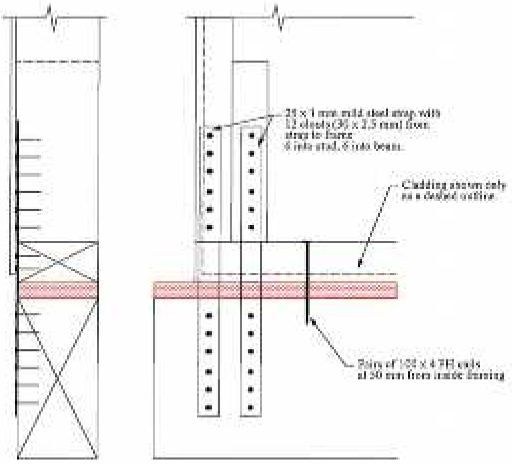


Figure 5 Connection of 0.6 m Long Wall to Timber Foundation

(Fixing 2.4 m long walls to timber foundations was similar but omitted the steel straps and 'dummy' end stub studs.)

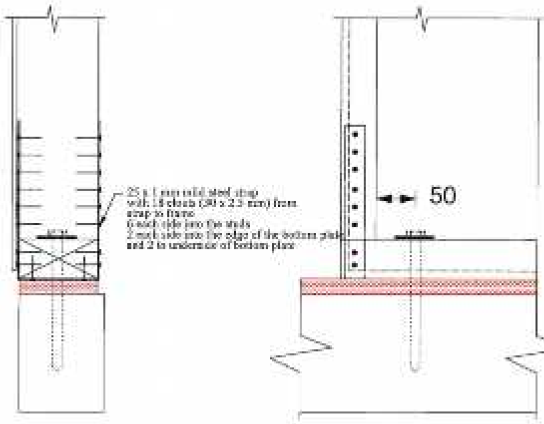


Figure 6 Fixing 0.6 m Long Walls to Simulated Concrete Foundations
(Fixing 2.4 m long walls to simulated concrete foundations was similar but omitted the steel straps.)

4. DESCRIPTION OF TESTS

4.1 Test Arrangement and Equipment

The rocking test specimens were installed in a rigid steel loading frame. P21 end restraints were installed in accordance with the recommendations of BRANZ P21:1998. "A Wall Bracing Test and Evaluation Procedure".

Horizontal load was applied to the centre of the specimen top plate with a 30 kN closed loop electro-hydraulic ram and measured with a 25 kN load cell.

Nylon rollers were used to prevent out-of-plane movement of the top plate.

Linear potentiometers were used to measure the horizontal displacement of the top plate, vertical uplift of the studs at either end of the specimen, and horizontal displacement of the bottom plate.

The test load and displacement measurements were recorded using a PC running a software program to record the data. The load cell was calibrated to International Standard EN ISO 7500-1 1999 Grade 1 accuracy and the linear potentiometers were calibrated to an accuracy of ±0.2 mm.

4.2 Test Procedure

The first two specimens of each test length were representative of a timber foundation and a concrete foundation respectively. The foundation condition that yielded the lowest indicative bracing result was then used for the remaining two specimens. After analysis of data after the first two tests it was concluded that the timber foundation case gave the lowest bracing ratings for the nominal 0.6 m long wall length and the concrete foundation gave the lowest bracing ratings for the nominal 2.4 m long wall length, although the difference between the concrete and timber foundation evaluated bracing ratings was small. Therefore, two more timber foundation cases were tested for the 0.6 m wall length and two more concrete foundation cases were tested for the 2.4 m wall length.

The loading sequence consisted of 3 displacement controlled cycles of the top plate to displacements of ±8, ±15, ±20, ±25, ±30, ±35, ±45 and ±55 mm except in some tests the cycling stopped after the cycles to ±35. In these instances the end restraint was then changed to an 'EM3' end restraint and the full test regime repeated. This was to facilitate possible future reduction of test results to the new proposed EM3 test and evaluation procedure. The results of this portion of the testing have been retained by BRANZ and are not reported herein. The actual top plate displacement regime used can also be seen in the hysteresis plots given in the appendix.

4.3 Date and Location of Tests

The tests were carried out in February 2004 at the Structural Engineering Laboratory of BRANZ Ltd, Judgeford, New Zealand.

5. OBSERVATIONS AND RESULTS

5.1 Observations

All timber floor simulation tests deformed in the same manner. Approximately 50% of the top plate movement was attributed to 'rocking' action of the whole specimen and 50% due to slip between sheet and framing. Sheet damage at clout locations was almost undetectable with the naked eye. The 'rocking action' was due to two mechanisms, both of which required 'slip' between the P21 end restraint and end studs and (for the 0.6 m long walls) 'give' in the nailed joint between end straps and timber. These were:

- vertical movement of the end studs; and
- nail pullout between bottom plate and foundation beam and hence upwards movement of the bottom plate.

The concrete floor simulation test specimens deformed in a similar manner except that the bottom plate only had small uplift at the bolt locations and curved upwards between bolt and end studs.

5.2 Results

Calculation sheets and typical test hysteresis loops are given in the appendix. P21 results based on three replicate test specimens are summarised in Table 1.

Table 1 –Bracing ratings for 2.4 m high walls lined with nominal 9 mm thick Customwood based on three replicate specimens

Foundation Type	Nominal Wall Length	Earthquake		Wind	
		Specimen Rating (BU)	Rating per metre (BU)	Specimen rating (BU)	Rating per metre (BU)
Timber	0.6 m	63	105	66	110
Concrete	2.4 m	209	87	234	97

Indicative bracing ratings on construction for which only a the single specimen was tested are summarised in Table 2. Thus, Table 2 gives indicative bracing ratings for 0.6 m long walls on concrete foundations whereas Table 1 gives bracing ratings for 0.6 m long walls on timber foundation. The P21 test procedure requires that bracing ratings be determined from three replicates and thus the values in Table 2 are indicative ratings only.

Table 2 –Indicative bracing ratings for 2.4 m high walls lined with nominal 9 mm thick Customwood based on a single test specimen only

Foundation Type	Nominal Wall Length	Earthquake		Wind	
		Specimen Rating (BU)	Rating per metre (BU)	Specimen rating (BU)	Rating per metre (BU)
Concrete	0.6 m	67	112	65	108
Timber	2.4 m	210	87	237	99

X-4

6. ASSESSMENT OF BRACING RATING OF CUSTOMWOOD WALLS.

Note that (with the exception of the wind bracing rating for the 0.6 m long walls) for each wall length that the average results for the full three specimen test series given in Table 1 is less than or equal to the indicative result for the corresponding wall length for the other floor simulation case in Table 2. The exception is expected to be due to the variability in the results for replicate construction. The closeness of the derived bracing values for the two foundation cases in corresponding construction indicates that the results from Table 1 can be applied to both construction lengths.

Thus, based on the results given in Section 5 of this report, and the previously published P21 test results for 1.2 m long bracing panels with a single end strap given in BRANZ Report No. STR 227/2, BRANZ considers that the wall bracing ratings given in Table 3 may be used for lateral load resistance determination of buildings using the design procedures of Section 5 of NZS 3604:1999. The results are applicable for walls on either timber or concrete foundations. Sheathing fixing, bottom plate fixing and end straps must be as per the figure numbers noted in Table 4.

Table 3 – Bracing ratings for 2.4 m high walls lined with nominal 9 mm thick Customwood

Bracing Element Type	Bracing Element Length	Bracing Rating BU's/m		Hold-down at each end
		Earthquake	Wind	
A	0.6 m to 1.2 m	105	110	2 straps
B	Longer than 1.2 m	121	128	1 strap
C	Longer than 2.4 m	87	97	No strap

Table 4 – Figure numbers for construction details

Part	Bracing Element Type	Timber Foundations	Concrete Foundations
Sheathing Fixing	A	Figure 3	Figure 3
Sheathing Fixing	B and C	Figure 4	Figure 4
Bottom Plate	A, B and C	Figure 5	Figure 6
End Straps	A	Figure 5	Figure 6
End Straps	B*	Figure 5	Figure 6
End Straps	C	No end straps or dummy studs required	

Legend

* the 'dummy' end stud and associated steel strap is omitted.

Appendix

- **Load-displacement plots**
- **P21 Calculation Sheets**

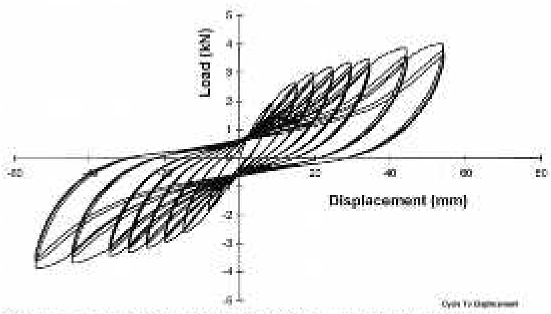
Data is in the following order:

1. Load-displacement plot for Specimen 2 (0.6 m long wall on timber foundation)
2. Load-displacement plot for Specimen 3 (0.6 m long wall on timber foundation)
3. Load-displacement plot for Specimen 4 (0.6 m long wall on timber foundation)
4. Analysis spreadsheet of results from Specimens 2, 3 and 4.

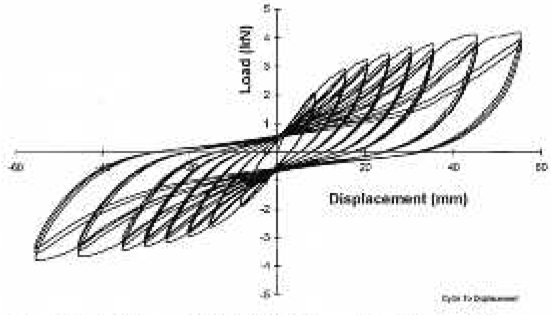
5. Load-displacement plot for Specimen 5 (2.4 m long wall on concrete foundation)
6. Load-displacement plot for Specimen 7 (2.4 m long wall on concrete foundation)
7. Load-displacement plot for Specimen 8 (2.4 m long wall on concrete foundation)
8. Analysis spreadsheet of results from Specimens 5, 7 and 8.

9. Load-displacement plot for Specimen 1 (0.6 m long wall on concrete foundation)
10. Indicative analysis spreadsheet of results from Specimen 1.

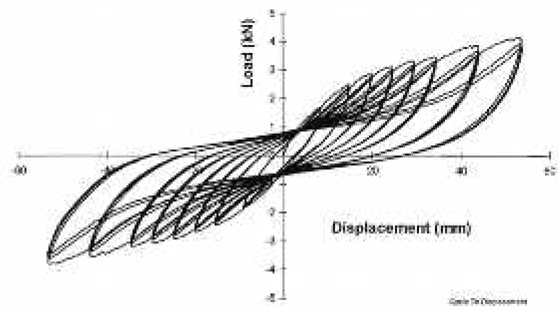
11. Load-displacement plot for Specimen 6 (2.4 m long wall on timber foundation)
12. Indicative analysis spreadsheet of results from Specimen 6.



Specimen 2. Load-displacement plot. Nominal 0.6 m long wall on a timber foundation



Specimen 3. Load-displacement plot. Nominal 0.6 m long wall on a timber foundation



Specimen 4. Load-displacement plot. Nominal 0.6 m long wall on a timber foundation

	Serviceability Cycles Cycle To Displacement x = 8 (mm)		Ultimate Cycles Cycle To Displacement y = 36 (mm)			
Specimen No	Load S (kN)	Residual Displacement C (mm)	Maximum Load F(kN)	Calculated F(0)(kN)	Displacement @F(0) (mm)	5th Cycle Load R (kN)
1	+ 1.81	+ 2.30	+ 3.03	+ 1.73	+ 6.70	+ 3.11
	- 1.95	- 2.55	- 3.52			- 3.11
2	+ 1.88	+ 2.22	+ 2.98	+ 1.73	+ 7.90	+ 3.11
	- 1.99	- 2.40	- 3.18			- 3.48
3	+ 1.70	+ 1.98	+ 3.22	+ 1.58	+ 6.90	+ 2.98
	- 1.70	- 2.13	- 3.09			- 3.13
Averages	Sa 1.86	Ca 2.30	Fu 3.48		pe 7.45	Ra 3.13

K1 = 1.4 - C/X = 1.03

F = K1 x S = 1.88

The "Asymmetry Of Performance" criterion in the last paragraph of Section 8B shall be followed.

a = p/d = 4.84

a	1.00	2.00	3.00	4.00	5.00	6.00
K4	0.50	0.60	0.67	0.74	0.87	1.00

For other values of a, linear interpolation is used to determine K4

Therefore K4 = 1.00

EVALUATION : EARTHQUAKE PERFORMANCE

BU(EQ) = 20 x the lesser of K4R or 2.1 x F/K4

K4 x R = 3.13 2.1 x F/K4 = 3.9

Therefore BU(EQ) = 20 x 3.13 **BU(EQ) = 63 Bracing Units**

EVALUATION : WIND PERFORMANCE

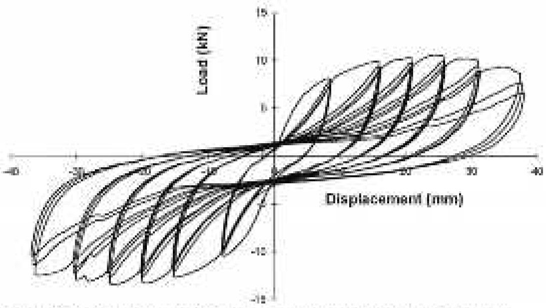
BU(wind) = 20 x the lesser of F or 1.75 x F

F = 3.48 1.75 x F = 3.3

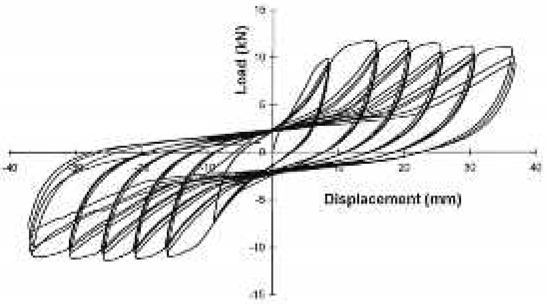
Therefore BU(WIND) = 20 x 3.30 **BU(WIND) = 66 Bracing Units**

600 mm long Customwood sheathed wall on a timber foundation.

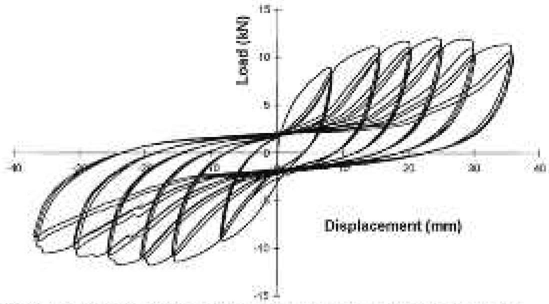
X-5



Specimen 5. Load-displacement plot. Nominal 2.4 m long wall on a concrete foundation



Specimen 7. Load-displacement plot. Nominal 2.4 m long wall on a concrete foundation



Specimen 8. Load-displacement plot. Nominal 2.4 m long wall on a concrete foundation

	Serviceability Cycles Cycle To Displacement ± 8 (mm)		Ultimate Cycles Cycle To Displacement ± 24 (mm)			
Specimen No	Load δ (kN)	Residual Displacement C (mm)	Maximum Load P(kN)	Calculated F(kN)	Displacement @F/Ks (mm)	4th Cycle Load at y mm R (kN)
1	+ 8.74	+ 1.00	+ 12.03	+ 6.33	+ 3.49	+ 11.91
	- 8.13	- 4.13	- 10.03			- 9.87
2	+ 7.74	+ 4.00	+ 11.20	+ 5.90	+ 4.20	+ 10.26
	- 8.07	- 2.13	- 11.19			- 10.96
3	+ 8.22	+ 3.40	+ 11.55	+ 5.96	+ 2.30	+ 10.28
	- 8.80	- 3.80	- 12.03			- 10.90
Average	δ= 8.52	C= 3.47	P= 11.69		Δ= 3.33	R= 10.47

K1 = 1.4 - C/X = 0.87

F = K1 x δ = 6.33

The "Asymmetry Of Performance" criterion in the last paragraph of Section 6.5 shall be followed.

μ = y/Δ = 8.01

μ	1.00	2.00	2.50	3.00	3.50	4.00
K4	0.35	0.60	0.67	0.74	0.87	1.00

For other values of μ, linear interpolation is used to determine K4

Therefore K4 = 1.00

EVALUATION : EARTHQUAKE PERFORMANCE

BU(EQ) = 20 x the lesser of K4 x R or 2.1 x F/K4

K4 x R = 10.47 2.1 x F/K4 = 17.5

Therefore BU(EQ) = 20 x 10.47 **BU(EQ) = 209 Bracing Units**

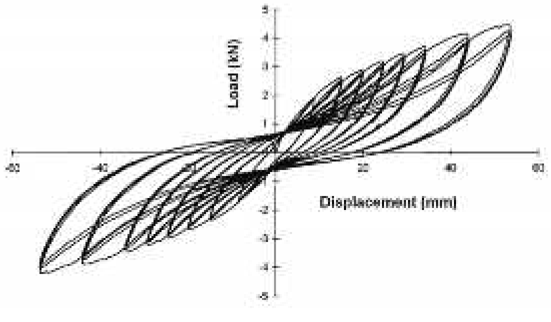
EVALUATION : WIND PERFORMANCE

BU(wind) = 20 x the lesser of P or 1.78 x F

P = 11.69 1.78 x F = 14.83

Therefore BU(WIND) = 20 x 11.69 **BU(WIND) = 234 Bracing Units**

2400 mm long Customwood sheathed walls on a concrete foundation.



Specimen 1. Load-displacement plot. Nominal 0.6 m long wall on a concrete foundation

	Serviceability Cycles Cycle To Displacement x = 6 (mm)		Ultimate Cycles Cycle To Displacement y = 36 (mm)			
Specimen No	Load δ (kN)	Residual Displacement C (mm)	Maximum Load P(kN)	Calculated F(kN)	Displacement @F/K4 (mm)	4th Cycle Load at y mm R (kN)
1	+ 1.72	+ 1.07	+ 3.48	+ 1.73	+ 8.13	+ 3.30
	- 1.91	- 2.49	- 3.71			- 3.50
Average	δ= 1.82	C= 1.98	P= 3.60		Δ= 8.13	R= 3.35

K1 = 1.4 - C/X = 1.00

F = K1 x δ = 1.82

The "Asymmetry Of Performance" criterion in the last paragraph of Section 6.5 shall be followed.

μ = y/Δ = 4.41

μ	1.00	2.00	2.50	3.00	3.50	4.00
K4	0.35	0.60	0.67	0.74	0.87	1.00

For other values of μ, linear interpolation is used to determine K4

Therefore K4 = 1.00

EVALUATION : EARTHQUAKE PERFORMANCE

BU(EQ) = 20 x the lesser of K4 x R or 2.1 x F/K4

K4 x R = 3.35 2.1 x F/K4 = 3.81

Therefore BU(EQ) = 20 x 3.35 **BU(EQ) = 67 Bracing Units**

EVALUATION : WIND PERFORMANCE

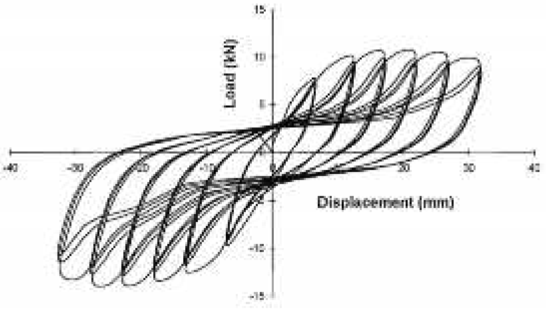
BU(wind) = 20 x the lesser of P or 1.78 x F

P = 3.60 1.78 x F = 3.23

Therefore BU(WIND) = 20 x 3.23 **BU(WIND) = 65 Bracing Units**

Indicative test only.

0.6 m long Customwood wall on a concrete foundation



Specimen 6. Load-displacement plot. Nominal 2.4 m long wall on a timber foundation

	Serviceability Cycles Cycle To Displacement x = 8 (mm)		Ultimate Cycles Cycle To Displacement y = 22 (mm)			
Specimen No	Load δ (kN)	Residual Displacement C (mm)	Maximum Load P(kN)	Calculated F(kN)	Displacement @F/K4 (mm)	4th Cycle Load at y mm R (kN)
1	+ 6.94	+ 2.40	+ 12.80	+ 6.96	+ 2.81	+ 11.45
	- 7.18	- 3.32	- 10.75			- 9.54
Average	δ= 6.96	C= 2.61	P= 11.83		Δ= 2.81	R= 10.49

K1 = 1.4 - C/X = 1.00

F = K1 x δ = 6.96

The "Asymmetry Of Performance" criterion in the last paragraph of Section 6.5 shall be followed.

μ = y/Δ = 7.83

μ	1.00	2.00	2.50	3.00	3.50	4.00
K4	0.35	0.60	0.67	0.74	0.87	1.00

For other values of μ, linear interpolation is used to determine K4

Therefore K4 = 1.00

EVALUATION : EARTHQUAKE PERFORMANCE

BU(EQ) = 20 x the lesser of K4 x R or 2.1 x F/K4

K4 x R = 10.49 2.1 x F/K4 = 15

Therefore BU(EQ) = 20 x 10.49 **BU(EQ) = 210 Bracing Units**

EVALUATION : WIND PERFORMANCE

BU(wind) = 20 x the lesser of P or 1.78 x F

P = 11.83 1.78 x F = 15.2

Therefore BU(WIND) = 20 x 11.83 **BU(WIND) = 237 Bracing Units**

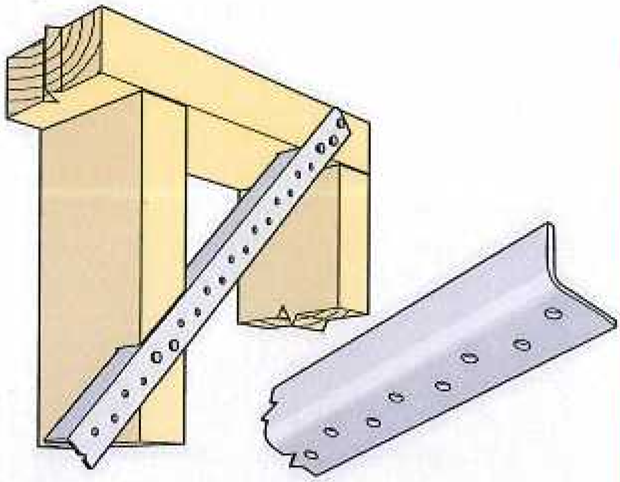
Indicative test only.

2.4 m long Customwood wall on timber foundation

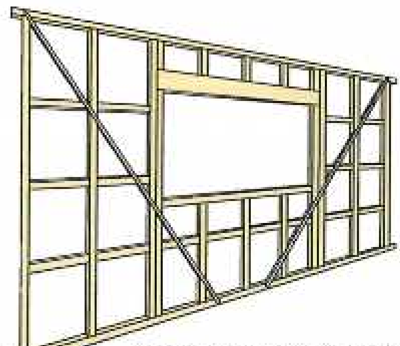
X-6

Angle Brace

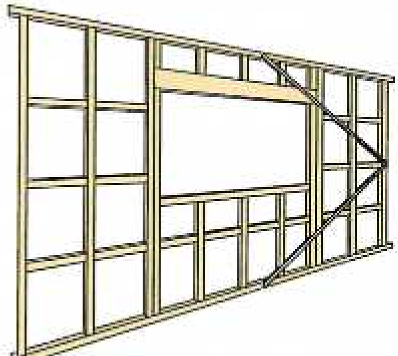
A fast, effective brace for timber frames



Applications



Angle Brace with Timber Nogging



Angle Brace use for Dog-Leg Bracing

Features

Pryda Angle Brace is the fast effective way to brace interior or exterior timber framing. It is fitted by making a single saw cut into the studs, inserting the brace, then nailing.

Because Pryda Angle Brace is power punched, it features clean, fully punched holes (no nails are bent or wasted by trying to force them through the brace).

Pryda Angle Brace utilises the tension and compression strength of steel with the properties of timber. It holds studs straighter, allows better air circulation and makes it easier to install wiring, plumbing and insulation.

"Checking in" Pryda Angle Brace flush with the surface of the timber can be done easily with the Pryda Angle Brace Checka fitted to an ordinary power saw. This attachment makes the saw cut and removes the timber to "check in" the brace in one operation.

Installation

After squaring up wall or temporary frame ready for bracing:-

1. Use the edge of the steel brace to draw a straight line where the brace is to go.
2. Cut the studs 20mm deep on this line with either a Pryda Angle Brace Checka, power saw or a hand saw.
3. Slide plain leaf of the angle into the sawcut. For safety reasons the punched leaf of the angle must point downwards. Nail punched leaf to the stud through the holes provided using 30 x 3.15 Pryda Product Nails, two per stud and minimum of three per end.
4. Brace is to be 150mm minimum from end of plate.

Specifications

Size:
20 X 20 X 1.00mm

Material:
G300 Z275 galvanised steel coil.

Product Code:
AB30 (3.0m long), AB33 (3.3m long), AB36 (3.6m long), AB42 (4.2m long), AB48 (4.2m long).

Packing:
Bundles of 10 lengths

Loads (Wind Only)

Steel: Characteristic Strength = 11.2 kN Design Capacity (LSD) = 10.0 kN			**Note:** These steel tensile loads cannot be achieved through normal nailing in 1 leg of the angle.		
Tension: Nails in one leg only			**Compression:** Studs @ 600mm centres		
Number of nails each end	Characteristic Strength	Design Load (LSD) Brief		Brace at 45°	Brace at 55°
			Clear Brace Length	780mm	980mm
3	4.7 kN	3.7 kN	Characteristic Buckling Load	4.6 kN	3.1 kN
4	6.2 kN	5.0 kN	Design Load	3.7 kN	2.5 kN
			Bracing Units per brace	60 BU	40 BU

X-7

STRUCTURAL BRACING
HARDIBRACE® SHEET BRACING, HARDIFLEX® SHEET, HARDIPLANK® WEATHERBOARD, LINEA™ WEATHERBOARD, HARDITEX® BASE SHEET, PANELCLAD® SHEET, PINERIDGE® LINING, VERSILUX® LINING AND VILLABOARD® LINING

AUSTRALIA
APRIL 2003

DESIGN MANUAL

James Hardie®

CONTENTS

WE VALUE YOUR FEEDBACK

To continuously improve the development of our products and systems, we value your input. Please send any suggestions, including your name, contact details and relevant illustrations to:

Ask James Hardie™
Fax 02 9638 9550
literaturefeedback@jameshardie.com.au

1 INTRODUCTION

1.1 GENERAL

All buildings require bracing against lateral forces due to wind (and in some instances earthquake). Due to current design trends (open plan spaces, wider window openings etc), the role of bracing is becoming more critical. This is particularly so for houses built on the coastal fringe of Australia north of latitude 30° (Coffs Harbour in the east, and Green Head in WA) where designs have to cater for cyclonic winds.

This manual describes how to achieve the necessary bracing requirements in timber- and steel-framed houses using HardiBrace® sheet bracing and other James Hardie fibre cement cladding products.

When using products other than HardiBrace sheet bracing ensure that this manual is read in conjunction with the relevant technical specification or installation manual for that product.

For ease of use, the core of this manual has been divided into three parts:

- Section 2 explores AS 1684-1999 'Residential Timber Framed Construction' ("the code"), and gives fixing details and bracing capacities for HardiBrace sheet bracing and other James Hardie fibre cement cladding products in accordance with the design requirements of this standard.
- Section 3 gives fixing details and bracing capacities for HardiBrace sheet bracing and other James Hardie fibre cement cladding products fixed with anchor rods (normally used in high wind and cyclonic areas) to timber frames;
- Section 4 provides fixing details and bracing capacities for James Hardie fibre cement cladding products fixed to steel frames.

Unlike previous James Hardie literature on this subject, this manual does not contain design aids for calculating wind forces and bracing units. AS 1684-1999 provides adequate information on fixing of bracing panels to the structural frame.

This manual must be used as a guide only to, but not as a substitute for, AS 1684-1999 because it may be subject to regular amendments and individual designs in this manual may vary from these.

All design capacities quoted are Ultimate Limit State (ULS) figures and have been certified by consulting engineers, Cardno (NSW) Pty Ltd. Permissible stress capacity may be obtained by dividing the ULS value by 1.5.

The specifier or other party responsible for the project must ensure the details in this specification are appropriate for the intended application and that additional detailing is performed for specific design or any areas that fall outside the scope and specifications of this manual.

Make sure your information is up to date
When specifying or installing James Hardie products, ensure you have the current manual. If you're not sure you do, or if you need more information, visit www.jameshardie.com.au or Ask James Hardie™ on 13 11 03

1.2 BRACING WITH FIBRE CEMENT CLADDING PRODUCTS

Fibre cement (FC) cladding on double-sided or single-sided wall systems can provide resistance against lateral forces or racking stress.

When fixed in accordance with this manual, and properly coated in external applications, these cladding products can provide bracing capacity to buildings as well as serving as a wall cladding.

Apart from HardiBrace sheet bracing, the design tables in this manual provide bracing values for other James Hardie cladding products of 6mm or greater thickness. These are:

1. 6mm HardiFlex® sheets;
2. 6mm Villaboard® and Versilux® linings;
3. 6mm PanelClad® sheets and PineRidge® lining;
4. 7.5mm HardiTex® base sheets;
5. All thickness and widths of HardiPlank® and Linea™ weatherboards, provided that fasteners pass through both planks (see note below).

IMPORTANT NOTE
For simplicity, items 1 to 4 will be referred to in the design tables as 6mm JHFC sheets and item 5 as JHFC planks and weatherboards.

2 BRACING FOR TIMBER FRAMING IN ACCORDANCE WITH AS 1684-1999

2.1 TIMBER FRAMING CODE

AS 1684-1999 'Residential Timber Framed Construction' ("the code"), is an extensive revision of the earlier code of practice. It has been issued in four parts:

Part 1: Design criteria
Part 2: Non-cyclonic areas
Part 3: Cyclonic areas
Part 4: Simplified non-cyclonic areas

The main change was the move to Limit State design. With regard to structural bracing, the former Type A and Type B bracing units have been placed into AS 1684.4, the simplified design procedure, which is covered in Clause 2.3 of this manual. In the simplified method, the number of bracing units is determined directly from tables relating to the shape of the building and bracing units are then assigned according to the rules of the code.

Structural bracing using the conventional Limit State design method is covered in Section 8 "Racking and Shear Forces" of both Part 2 and Part 3 of the code. This is covered in Clause 2.4 of this manual. In this method, the total racking force is determined from tabulated data and bracing walls are designed on the basis of their actual kN/m bracing capacity.

Note that throughout the code the wind classifications of AS 4055 "Wind Loads for Housing" have been used:

- In Part 2, the pressures have been tabulated for non-cyclonic wind classifications N1 to N4 (with N5 and N6 ignored);
- In Part 3, the pressures have been tabulated for cyclonic wind classifications C1 to C3 (with C4 ignored).

2.2 TYPES OF BRACING

The code describes two types of bracing against lateral load:

1. Nominal bracing
Nominal bracing is defined as (a) any wall framing lined with fibre cement sheets (or other material) not fixed in accordance with this manual, and/or (b) with the frames nominally fixed to the floor and the roof or ceiling frame (ie not tied down in accordance with this manual). The capacity depends on whether the simplified design method (Section 2.3 of this manual) or the conventional design method (Section 2.4 of this manual) is used. For framing, fixing and installation of nominal bracing, refer to Clause 8.1.
2. Structural bracing
Also known as "designated" bracing, structural bracing is purpose-fitted bracing such as the James Hardie systems detailed in this manual. Fixing must be as per the instructions given in this manual.

2.3 SIMPLIFIED DESIGN METHOD

2.3.1 Limitations, procedure and other rules
The simplified method given in AS 1684.4 applies only to Class 1 and Class 10 Buildings as defined by the Building Code of Australia (BCA). Clause 1.6 of AS 1684.4 describes these limitations as follows:

- single- and two-storey dwellings only;
- a maximum wind classification of N2 (ie non cyclonic);
- a maximum width of building of 12m excluding eaves;
- a maximum wall height of 2700mm;
- a maximum rafter overhang of 750mm;
- a maximum roof pitch of 30°;
- a maximum rafter spacing of 900mm for tile roofs and 1200mm for sheet roofs;
- spacing of bracing elements not to exceed 9m;
- there are certain maximum building masses for floor framing, wall framing and roof framing.

This would cover the vast majority of homes in urban areas south of 30° latitude.

The design procedure can then be as follows:

(a) Determine wind classification using Clause 1.5 of AS 1684.4;
(b) Determine the appropriate house elevation option for single or upper storey or the lower storey of a two storey building for both wind directions (use the code Figure 8.3);
(c) Determine the number of bracing units required for each wind direction (use the code Table 8.2);
(d) Allocate the required number of structural bracing units in conjunction with the amount of nominal bracing if necessary;
(e) Distribute the bracing units evenly (use the code Figures 8.4 and 8.5).

Other rules and allowances that need to be considered include the following (refer to the code Clause 8.3.2.3 for full details):

- Bracing may be a combination of Type A and/or Type B structural bracing units and/or nominal bracing;
- Nominal bracing shall not constitute more than 50% of the required bracing for each wind direction or in each storey;
- Where structural bracing occurs in the same section of wall as nominal bracing, the nominal bracing in that section of wall shall not be considered as contributing to the house bracing requirements;
- Generally a minimum of two structural bracing units (Type A or Type B) shall be provided in each overall length of external wall in each storey, located as closely as possible to the external corners (see the code for rules of exceptions);
- One Type B unit equals two Type A units;
- Bracing units need to be installed at right angles to the wall area or elevation (ie parallel to wind direction) for which the bracing was defined.

Clause 1.7(f) of AS 1684.4 states that the design capacities are [illegible] per 900mm for Type A bracing units and [illegible] per 900mm for Type B. These are Ultimate Limit State (ULS) figures.

2.3.2 Nominal bracing
Cladding not fixed in accordance with this manual, or wall frames not connected to the structure in accordance with this manual, is nominal bracing. Respectively a 1m length of single-sided nominal bracing or a 4m length of double sided nominal bracing constitutes one Type A bracing unit.

x - 8

2.3.3 Structural Bracing
Apart from using HardiBrace sheet bracing as structural bracing as per Clauses 2.3.6 and 2.3.7 below, Type A and B units can also be achieved with other James Hardie JHFC sheets as detailed in Clause 2.3.8 of this manual. In this simplified method, bracing units must not be less than 900mm wide.

2.3.4 Bracing panels wider than 900mm
Bracing units are generally based on a standard width of 900mm. For wider walls than this, the bracing capacity is increased in direct proportion to the required width divided by 900. For example, a 1200mm wall section is equivalent to 1200/900 or 1.33 times the bracing resistance of the 900mm unit.

2.3.5 Tie-down requirements
In order to provide structural bracing resistance, the bracing panels must be adequately tied-down to the floor system. For tie-down requirements, refer to AS 1684.4 Clause 8.3.6.7 (bottom) and Clause 8.3.6.9 (top).

2.3.6 Type A bracing units
To achieve Type A bracing capacity (3.4kN/900mm), fix the HardiBrace sheet bracing in accordance with Figure 1, Section 6 and Clause 2.3.5 of this manual.

2.3.7 Type B bracing units
To achieve Type B bracing capacity (5kN/900mm), fix the HardiBrace sheet bracing in accordance with Figure 2, Section 6 and Clause 2.3.5 of this manual.

2.3.8 Other James Hardie cladding products
Type A or Type B bracing capacities may be achieved with other James Hardie cladding products:

- To achieve Type A bracing capacity with 6mm JHFC sheets as defined at Clause 1.2 of this manual, fix sheets in accordance with Figure 1, Section 6 and Clause 2.3.5 of this manual.
- To achieve Type B bracing capacities with 6mm JHFC sheets as defined at Clause 1.2 of this manual, fix sheets in accordance with Figure 2, Section 6 and Clause 2.3.5 of this manual.

The bracing rules and methods of determining the required number of bracing units remain the same as previously described.

2.4 CONVENTIONAL LIMIT STATE DESIGN

2.4.1 Design procedure
For a building outside the scope of the simplified method, use the procedure given in both AS 1684.2 and AS 1684.3. In both parts of the code, Clause 8.3.1 states that bracing shall be designed and provided for each storey of the house (and subfloor where required) in accordance with the following procedure:

(a) Determine the wind classification (see the code Clause 1.5 and AS 1170.2 or AS 4055);
(b) Determine the wind pressure (see the code Clause 8.3.2);
(c) Determine the area of elevation (see the code Clause 8.3.3 and Figure 8.2);
(d) Calculate racking force (see the code Clause 8.3.4);
(e) Design bracing systems (for walls, see the code Clause 8.3.5 and continue with the code Clause 8.3.6);
(f) Check even distribution and spacing (see the code Clauses 8.3.6.6 and 8.3.6.7 and the code Tables 8.18 and 8.19);
(g) Check connection of bracing to rooftop/ceilings and floors (see the code Clauses 8.3.6.9 and 8.3.6.10).

Instead of proportioning bracing units required versus those provided, the actual racking shear capacities of the bracing panels are added up and made to exceed the total racking force calculated. All pressures and forces are Ultimate Limit State (ULS) figures.

2.4.2 Nominal bracing
The two categories, structural wall bracing and nominal wall bracing, exist in this method too and the same rules apply in that nominal bracing (as defined at Clause 2.2 of this manual) may provide no more than 50% of the total required bracing capacity.

The ULS capacity of nominal bracing walls is given by the code as 0.45kN/m for single-sided walls and 0.75kN/m for double-sided walls. The minimum length for which nominal bracing capacity may be claimed is 450mm.

TABLE 1

ULS DESIGN BRACING CAPACITY OF JAMES HARDIE FIBRE CEMENT CLADDING ON TIMBER FRAMES (kN/m)	
HardiBrace sheet bracing fixed with standard nail pattern (see Figure 1)	3.8
HardiBrace sheet bracing fixed with close-nailed pattern (see Figure 2)	6.6
6mm JHFC sheets, single-sided, fixed vertically as per standard HardiBrace sheet bracing (see Figure 1)	3.8
6mm JHFC sheets, single-sided, fixed vertically as per close nailed HardiBrace sheet bracing (see Figure 2)	6.6
6mm JHFC sheets, single-sided, fixed vertically (see Figure 3, 4 or 5b), or horizontally with set joints (see Figure 5a)	2.8
6mm JHFC sheets double-sided, fixed vertically (see Figure 3, 4 or 5b)	4.5
6mm JHFC sheets double-sided, fixed horizontally with set joints (see Figure 5a)	4.5
6mm JHFC sheets, fixed vertically (see Figure 3, 4 or 5b) or horizontally with joints set (see Figure 5a) + JHFC planks or weatherboards on other side (see Figures 6 and 7)	5.5
JHFC planks or weatherboards on one side only (see Figures 6 and 7)	2.5
Linea weatherboards on one side of frame only using face fixing method (see Figure 8)	2.4
Linea weatherboards on one side of frame using face fixing method (see Figure 8) + 6mm JHFC sheets, fixed vertically (see Figure 3, 4 or 5b), or horizontally with joints set (see Figure 5a)	4.1

For permissible shear capacity divide by 1.5

NOTES FOR TABLE 1
1. If the bracing panel occurs in isolation within a length of wall and is not connected to any cross-wall, then the capacity given in Table 1 must be reduced by 20%.
2. For definition of 6mm JHFC sheets see the note of Clause 1.2 of this manual. Thicker sheets are assumed to provide at least the tabulated value.
3. If JD5 grade timber is used in the framing, then the capacity given in Table 1 must be reduced by 12.5%.
4. Butt joints are permitted in vertical sheets provided that both sheet edges are fixed to a nogging with fasteners at the same spacing as nominated for the top and bottom edges.
5. For horizontally fixed sheets, if edges at a butt joint are not fixed to a nogging behind the joint, then the joint needs to be properly tape-set in order to claim the tabulated design bracing capacity.

2.4.3 Structural Bracing
Table 1 provides the ULS design capacities for the James Hardie fibre cement products that may be used as designated structural bracing in this procedure.

Where greater bracing capacities are required, anchor rods may be used and the values in Table 4 in Section 3 of this manual claimed.

2.4.4 Wall height and capacity modification
The capacity of bracing walls is given for a standard wall height of 2700mm and decreases as the height increases. Refer to Clause 8.3.6.4 of both Parts 2 and 3 of the code, interpolated as in Table 2.

TABLE 2

REDUCTION FACTORS FOR HEIGHT	
WALL HEIGHT (mm)	BRACING REDUCTION FACTOR
2700	1.0
3000	0.9
3300	0.8
3600	0.75

Intermediate values may be interpolated.

2.4.5 Panels less than 900mm wide
Generally the minimum width of a designated bracing panel is 900mm, although exceptions are permitted with reference to Clause 8.3.6.5 of both Parts 2 and 3 of the code. This is interpreted in Table 3.

TABLE 3

REDUCTION FACTORS FOR HEIGHT	
LENGTH OF NARROW BRACING PANEL (mm)	BRACING REDUCTION FACTOR
900	1.0
850	0.83
750	0.75
700	0.68
600	0.50
500	0.33
450	0.20

Ensure that an intermediate stud is used for bracing panels over 600mm in width.

2.4.6 Location, distribution and spacing of bracing walls
Refer to Clause 8.3.6.6 of both Parts 2 and 3 of the code for required location and distribution and Clause 8.3.6.7 for spacing rules.

2.4.7 Tie-down requirements
In order to achieve structural bracing resistance (as defined at Clause 2.1.2 of this manual) the bracing panel needs to be tied into the structure. For tie-down requirements, refer to Clause 8.3.6.9 (top of wall) and Clause 8.3.6.10 (bottom of wall) in both Parts 2 and 3 of the code.

2.4.8 Other James Hardie cladding products
The 6mm JHFC sheet products (as defined at Clause 1.2 of this manual) as well as the plank and weatherboard range provide structural bracing capacity as given in Table 1. Fixing details for the different products are given below.

(a) HardiFlex sheets, FlexiRidge lining and PanelClad sheets
- Non-cyclonic areas: sheets fixed vertically in accordance with Figure 3, Section 6 and Clause 2.4.7 of this manual will achieve the bracing capacities stated in Table 1.
- Cyclonic areas: sheets fixed vertically along with anchor rods in accordance with Figure 3, Section 6 and Clause 3.3 of this manual will achieve the values stated in Table 4.

(b) HardiTex base sheets
- Non-cyclonic areas: sheets fixed vertically in accordance with Figure 4, Section 6 and Clause 2.4.7 of this manual will achieve the bracing capacities stated in Table 1.
- Cyclonic areas: sheets fixed vertically along with anchor rods in accordance with Figure 4, Section 6 and Clause 3.3 of this manual will achieve the values stated in Table 4.

(c) Villaboard and Versilux linings
- Non-cyclonic areas: sheets fixed vertically or horizontally in accordance with Figure 5, Section 6 and Clause 2.4.7 of this manual will achieve the bracing capacities stated in Table 1.
- Cyclonic areas: sheets fixed vertically or horizontally along with anchor rods in accordance with Figure 5, Section 6 and Clause 3.3 of this manual will achieve the values stated in Table 4.

(d) Planks and weatherboards external cladding
- Non-cyclonic areas: The bracing capacity stated in Table 1 applies to all JHFC planks and weatherboards, when fixed in accordance with Figure 6, Section 6 and Clause 2.4.7 of this manual.
- Cyclonic areas: The bracing capacity stated in Table 4 applies to all JHFC planks and weatherboards, when fixed along with anchor rods in accordance with Figure 6, Section 6 and Clause 3.3 of this manual.
- In both the above cases, JHFC planks and weatherboards must be fixed at 150mm maximum centres along top and bottom plates as shown in Figure 7. For Linea weatherboard however, this figure must be 75mm.
- For Linea weatherboard fixing, see Figure 8.

3 BRACING FOR TIMBER FRAMING WITH ANCHOR RODS

3.1 INTRODUCTION

This section details James Hardie fibre cement sheet cladding used for bracing with timber framing and anchor rods, specifically for winds in cyclonic areas. These rods provide resistance against uplift forces and add to the racking capacity of the wall panels.

Bracing capacities quoted in this section were proved by testing in conjunction with the James Cook Cyclone Structural Testing Station.

3.2 BRACING RESISTANCE CAPACITIES

Table 4 provides the ULS design bracing capacities of HardiBrace sheet bracing, 6mm JHFC sheets, JHFC planks and weatherboards used with anchor rods when fixed in accordance with the relevant parts of this manual.

TABLE 4

ULS DESIGN BRACING CAPACITY OF JAMES HARDIE FIBRE CEMENT CLADDING ON TIMBER FRAMES WITH ANCHOR RODS (kN/m)	
HardiBrace sheet bracing fixed with standard nail pattern (see Figure 1)	6.8
HardiBrace sheet bracing fixed with standard nail pattern (see Figure 1) + 6mm JHFC sheets other side, fixed vertically (see Figure 3, 4 or 5b), or horizontally with set joints (see Figure 5a)	10.0
6mm JHFC sheets, single-sided, fixed vertically (see Figure 3, 4 or 5b) or horizontally with set joints (see Figure 5a)	5.5
6mm JHFC sheets, double-sided, fixed vertically (see Figure 3, 4 or 5b) or horizontally with set joints (see Figure 5a)	7.2
JHFC planks or weatherboards (see Figures 6 and 7) + 6mm JHFC sheets other side, fixed vertically (see Figure 3, 4 or 5b) or horizontally with set joints (see Figure 5a)	6.6
JHFC planks or weatherboards, single-sided (see Figure 6 and Figure 7)	2.4
Linea weatherboards on one side of frame only using face fixing method (see Figure 8)	5.8
Linea weatherboards on one side of frame using face fixing method (see Figure 8) + 6mm JHFC sheets, fixed vertically (see Figure 3, 4 or 5b), or horizontally with joints set (see Figure 5a)	6.8

For permissible shear capacity divide by 1.5

NOTES FOR TABLE 4
1. For definition of 6mm JHFC sheets see Clause 1.2 of this manual. Thicker sheets are assumed to provide at least the tabulated value.
2. The tabulated bracing strengths relate to 600mm maximum stud centres.
3. The capacities apply to bracing sheets up to 2700mm high and not less than 900mm wide. If different, refer respectively to Clauses 2.4.4 and 2.4.5 of this manual.
4. For horizontally fixed sheets, if edges at a butt joint are not fixed to a nogging, then the joint needs to be properly tape-set in order to claim the tabulated design bracing capacity.

3.3 ANCHOR ROD DETAIL AND TIE-DOWN

Anchor rods must be 12mm diameter, full-length mild steel (M12) rods tying the wall top plate through the frame cavity to the sub-structure.

A standard 38mm diameter flat round washer must be used under each nut.

Anchor rods must be placed at both ends of each section of the bracing wall and at not more than 2.4m centres.

Anchor rods must be located within 100mm of the adjacent face of the stud ends. See Figure 9.

Between anchor rods, one M10 hold-down bolt must be provided at a maximum of 1.2m centres to further fix the bottom plate to the sub-structure.

Fixing of the timber frames into the structure (ie 'tie-down') must comply with government building regulations and/or AS 1684.3-1999. Refer to the code Clauses 8.3.6.9 (top) and 8.3.6.10 (bottom).

4 BRACING FOR STEEL FRAMING

4.1 INTRODUCTION

Extensive testing conducted in the James Hardie R&D laboratory and at the James Cook Cyclone Structural Testing Station has formed the basis of the information and the design capacities quoted in this section.

4.2 BRACING RESISTANCE CAPACITY

Table 5 shows the bracing capacity of HardiBrace sheet bracing and other James Hardie cladding products when fixed to 0.55mm and 0.75mm light gauge steel frames or 1.2mm and 1.6mm medium gauge, welded steel frames.

NOTE
Only the values tested are shown in Table 5. Other values may, in certain instances, be interpolated with experience and due diligence.

The bracing capacities are achieved by using the fixing methods outlined in Clause 4.4 and Section 6 of this manual.

Design capacities were determined in accordance with AS 3623-1993 for 2700mm high panels unless noted otherwise.

The minimum length of a bracing wall or panel must be 900mm. The maximum wall length to which the capacities apply is 3600mm.

4.3 FRAMING CONNECTIONS

0.55mm to 0.95mm light gauge steel frames:
The studs need to be fixed to the top and bottom tracks by screws, rivets, bolts or mechanical crimping.

1.0mm to 1.6mm medium gauge steel frames:
The connections may be welded or riveted, noting that the design bracing capacity is 20% lower for the riveted frames.

4.4 TIE-DOWN REQUIREMENTS

0.55mm to 0.95mm light gauge steel frames:
Provide M10 minimum hold-down bolts with 50 x 50 x 3mm distribution washers at the two outside frame studs and M8 minimum hold-down bolts with 32mm diameter 2.5mm thick round washers at the interior studs. All bolts to be placed within 40mm of the stud.

0.95mm to 1.6mm medium gauge steel frames:
Provide M12 minimum hold-down bolts with 75 x 75 x 6mm distribution washers at 900mm centres and within 75mm of the face of studs.

NOTES FOR TABLE 5
1. HardiBrace sheet bracing must not be used as exposed, finished, external cladding.
2. Bracing capacity can only be claimed for JHFC plank or weatherboard cladding if screws pass through both planks. See Figure 6.
3. For riveted frames of 1.2 and 1.6mm gauge, the tabulated bracing capacities must be multiplied by a factor of 0.8.
4. For definition of 6mm JHFC sheets see Clause 1.2 of this manual. Thicker sheets are assumed to provide at least the tabulated value.
5. Butt joints are permitted in vertical sheets provided that both sheet edges are fixed to a nogging with fasteners at the same spacing as nominated for the top and bottom edges.
6. For horizontally fixed sheets, if edges at a butt joint are not fixed to a nogging, then the joint needs to be properly tape-set in order to claim the tabulated design bracing capacity.
7. For external sides of walls, 6mm Villaboard lining must be replaced by properly coated 6mm HardiFlex or 7.5mm HardiTex sheets.

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TABLE 8

ULS BRACING CAPACITY OF JAMES HARDIE FIBRE CEMENT CLADDING ON STEEL FRAMES

MATERIAL	FIXING DETAILS	STUD SPACING (mm)	SCREW SPACING (mm) A/B/C (See Figure 10)	BRACING CAPACITY (kN/m)			
				Light gauge steel frames		Welded steel frames (see Note 3)	
				0.55mm BMT studs	0.75mm BMT studs	1.2mm BMT studs	1.6mm BMT studs
6mm HardiBrace sheet bracing (see Note 1)	Single-sided, fixed vertically, joints (if any) not set	600	100/100/150	5.4	6.0*	-	-
		450	100/100/150	6.0	6.2	-	6.8
		300	100/100/150	-	-	-	6.8
7.5mm HardiFlex base sheet	Single-sided, fixed vertically, joints (if any) not set	600	200/200/200	3.0	3.6	-	-
		450	200/200/200	-	-	-	5.1
6mm JHFC sheets (see Note 4) (see Note 7)	Single-sided, fixed vertically, joints (if any) not set	600	200/200/200	3.0	3.3	-	-
		450	200/200/200	-	-	5.4	6.0
		300	200/200/200	-	-	-	7.5
7.5mm JHFC planks or weatherboards	Single-sided	600	See Note 2 and Figures 6 and 7	2.1	2.2	-	-
		450	See Note 2 and Figures 6 and 7	-	-	-	3.4
		300	See Note 2 and Figures 6 and 7	-	-	-	3.6
	Double-sided. Internal lining 6mm JHFC sheets (see Note 4) fixed vertically (joints not set, see Note 5) or horizontally (joints tape set, see Note 6)	600	See Note 2 and Figures 6 and 7. For 6mm JHFC sheets 200/200/200	3.8	4.0	-	-
		450	See Note 2 and Figures 6 and 7. For 6mm JHFC sheets 200/200/200	-	-	-	5.7
6mm JHFC sheets (see Note 4) (see Note 7)	Single-sided, fixed horizontally, joints not set	600	200/200/200	2.4	2.7	-	-
	Single-sided, fixed horizontally, joints tape set (see Note 6)	600	200/200/200	4.2	4.5	-	-
	Double-sided, fixed vertically, joints (if any) not set	600	200/200/200	5.6	5.7	-	-
		450	200/200/200	-	-	6.0	7.6
		300	200/200/200	-	-	-	11.0
	Double-sided, fixed horizontally, joints not set	600	200/200/200	3.0	4.2	-	-
	Double-sided, fixed horizontally, joints tape set (see Note 6)	600	200/200/200	5.6	5.8	-	-
16mm Linea weatherboard	Single-sided	600	See Note 2 and Figure 8	3.8	3.6	-	-
	Double-sided, internal lining 6mm JHFC sheets (see Note 4), fixed vertically (joints not set, see Note 5) or horizontally (joints tape set, see Note 6)	600	See Note 2 and Figure 8. For 6mm JHFC sheets 200/200/200	4.6	4.5	-	-

* For 2700mm high frames, this figure is 5.4 kN/m.

5 SAFE WORKING PRACTICES

NOTE
The information in this section is relevant for HardiBrace sheet bracing only. For other products refer to the relevant technical specification or installation manual.

WARNING - DO NOT BREATHE DUST AND CUT ONLY IN WELL VENTILATED AREA

James Hardie products contain sand, a source of respirable crystalline silica which is considered by some international authorities to be a cause of cancer from some occupational sources. Breathing excessive amounts of respirable silica dust can also cause a disabling and potentially fatal lung disease called silicosis, and has been linked with other diseases. Some studies suggest smoking may increase these risks. During installation or handling: (1) work in outdoor areas with ample ventilation; (2) minimise dust when cutting by using either 'score and snap' knife, fibre cement shears or, where not feasible, use a HardiBlade® saw blade and dust reducing circular saw attached to a HEPA vacuum; (3) warn others in the immediate area to avoid breathing dust; (4) wear a properly-fitted, approved dust mask or respirator (e.g. P1 or P2) in accordance with applicable government regulations and manufacturer instructions to further limit respirable silica exposures. During clean-up, use HEPA vacuums or wet clean-up methods - never dry sweep. For further information, refer to our installation instructions and Material Safety Data Sheets available at www.jameshardie.com.au. FAILURE TO ADHERE TO OUR WARNINGS, MATERIAL SAFETY DATA SHEETS, AND INSTALLATION INSTRUCTIONS MAY LEAD TO SERIOUS PERSONAL INJURY OR DEATH.

JAMES HARDIE RECOMMENDED SAFE WORKING PRACTICES

CUTTING OUTDOORS
1. Position cutting station so wind will blow dust away from the user or others in working area.
2. Use one of the following methods based on the required cutting rate:
Best
• Score and snap
• Hand guillotine
• Fibreshear
Better
• Dust reducing circular saw equipped with HardiBlade® saw blade and HEPA vacuum extraction.
Good
• Dust reducing circular saw equipped with HardiBlade® saw blade.

CUTTING INDOORS
• Cut only using score and snap, hand guillotine or fibreshears (manual, electric or pneumatic).
• Position cutting station in a well-ventilated area.

DRILLING/OTHER MACHINING
When drilling or machining you should always wear a P1 or P2 dust mask and warn others in the immediate area.

IMPORTANT NOTES
1. For maximum protection (lowest respirable dust production), James Hardie recommends always using "Best" - level cutting methods where feasible.
2. NEVER use a power saw indoors.
3. NEVER use a circular saw blade that does not carry the HardiBlade® logo.
4. NEVER dry sweep - Use wet suppression or HEPA vacuum.
5. NEVER use grinders.
6. ALWAYS follow tool manufacturers' safety recommendations.

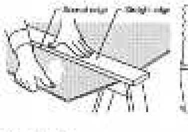
P1 or P2 respirators should be used in conjunction with above cutting practices to further reduce dust exposures. Additional exposure information is available at www.jameshardie.com.au to help you determine the most appropriate cutting method for your job requirements. If concern still exists about exposure levels or you do not comply with the above practices, you should always consult a qualified industrial hygienist or contact James Hardie for further information.

WORKING INSTRUCTIONS

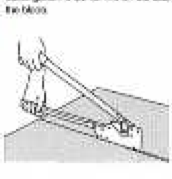
Refer to recommended safe working practices before starting any cutting or machining of product.

Score and snap
Score and snap is a fast and efficient method of cutting James Hardie building products using James Hardie's special tungsten tipped score and snap knife.

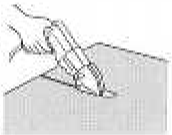
Preferably score on the face side of the product. Score against a straight edge and repeat the action to obtain adequate depth for clean break - normally one third of sheet thickness. Snap upwards to achieve break. Smooth any rough edges with a rasp.



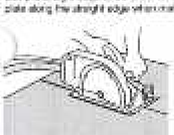
Hand guillotine
Make guillotine cut on the off-cut side of line to allow for the thickness of the blade.



Fibreshear
An electrically powered, fast, clean and effortless way of cutting James Hardie building products, especially around curves such as archways. Make fibreshear cut on the 'off-cut' side of the line to allow for the thickness of the shear.



HardiBlade® saw blade
The HardiBlade® saw blade used with a dust-reducing saw and HEPA vacuum extraction allows for fast, clean cutting of James Hardie fibre cement products. A dust-reducing saw uses a dust deflector or a dust collector which can be connected to a vacuum system. When sawing, clamp a straight edge to the sheet as a guide and run the saw base plate along the straight edge when making the cut.



HOLE-FORMING

For smooth clean cut circular holes:
• Mark the centre of the hole on the sheet.
• Pre-drill a pilot hole.
• Using the pilot hole as a guide, cut the hole to the appropriate diameter with a hole saw fitted to a heavy duty electric drill.

For irregular holes:
• Small rectangular or circular holes can be cut by drilling a series of small holes around the perimeter of the hole then tapping out the waste piece from the sheet face.
• Tap carefully to avoid damage to sheets, ensuring the sheet edges are properly supported.

STORAGE AND HANDLING

To avoid damage, all James Hardie building products should be stored with edges and corners of the sheets protected from chipping.

James Hardie building products must be installed in a dry state and protected from rain during transport and storage. The product must be laid flat under cover on a smooth level surface clear of the ground to avoid exposure to water, moisture, etc.

QUALITY

James Hardie conducts stringent quality checks to ensure any product manufactured falls within our quality spectrum. It is the responsibility of the builder to ensure the product meets aesthetic requirements before installation. James Hardie will not be responsible for rectifying obvious aesthetic surface variations following installation.

6 PRODUCT INFORMATION

NOTE
The information in this section is relevant for HardiBrace sheet bracing only. For other products refer to the relevant technical specification or installation manual.

6.1 GENERAL

HardiBrace sheet bracing is a cellulose fibre reinforced cement building product. The basic composition is Portland cement, ground sand, cellulose fibre and water.

HardiBrace sheet bracing is manufactured to AS/NZS 2908.2 'Cellulose-Cement Products Part 2: Flat Sheets' (ISO 8336 'Fibre Cement Flat Sheets').

HardiBrace sheet bracing is classified Type A, Category 2 in accordance with AS/NZS 2908.2 'Cellulose Cement Products'.

For Material Safety Data Sheets (MSDS) visit www.jameshardie.com.au or Ask James Hardie® on 13 11 03.

6.2 PRODUCT MASS

Based on equilibrium moisture content the approximate mass of HardiBrace sheet bracing is [illegible] kg/m².

6.3 DURABILITY

6.3.1 Resistance to moisture/rotting
HardiBrace sheet bracing has demonstrated resistance to permanent moisture induced deterioration (rotting) by passing the following tests in accordance with AS/NZS2908.2:

• Water permeability (Clause 8.2.2)
• Warm water (Clause 8.2.4)
• Heat rain (Clause 6.5)
• Soak dry (Clause 8.2.5)

6.3.2 Resistance to fire
HardiBrace sheet bracing is suitable where non-combustible materials are required in accordance with C1.12 of the Building Code of Australia.

HardiBrace sheet bracing has been tested by CSIRO and is classified as a Group 1 material in accordance with Specification C1.10a of the BCA.

HardiBrace sheet bracing has the following early fire hazard indices (tested to AS 1530 Part 3):

EARLY FIRE HAZARD INDICES (TESTED TO AS1530 PART 3)	
Ignition index	0
Flame spread index	0
Heat evolved index	0
Smoke developed index	0-1

6.3.3 Resistance to termite attack
Based on testing completed by CSIRO Division of Forest Products Report Numbers FP349 and FP274 James Hardie fibre cement has demonstrated resistance to termite attack.

6.4 ALPINE REGIONS

In regions subject to freeze/thaw conditions, fibre cement external cladding must be painted. In addition, fibre cement cladding must not be in direct contact with snow and/or ice build up for extended periods, e.g. external walls in alpine regions subject to snow drifts over winter.

Tested for resistance to frost in accordance with AS/NZS2908.2 Clause 8.2.3.

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7 COMPONENTS

The following checklist describes the components required to install HardiBrace sheet bracing.

Timber or steel framing may be used, but must comply with relevant building regulations and standards and the requirements of this manual.

Mass includes timber packing for transport.

HARDIBRACE SHEET BRACING SIZES (4mm THICKNESS)			
	Description: square edge, drill sheet.		
	Mass: 6.7kg/m²		
	Length (mm)	Width (mm)	Thickness (mm)
	2440	900 1200	5 5
	2755	900 1200	5 5

Not available in WA.
Selling unit: square metre (m²)

JAMES HARDIE HARDIBRACE SHEET BRACING COMPONENTS		
	HardiDrive® screw For light gauge steel frames 0.75mm to 1.6mm thick NOTE: Contact your fastener manufacturer for fasteners with suitable corrosion resistance.	8g x 20mm 8g x 30mm
	HardiBlade® saw blade 185mm poly diamond blade, for fast, clean cutting of James Hardie fibre cement.	HardiBlade® saw blade Selling unit: each
	JH score and snap knife Tungsten tipped scoring tool for easy cutting.	
	Fibreshear Electric cutting tool.	
COMPONENTS NOT SUPPLIED BY JAMES HARDIE		
Fibre cement nail Galvanised nails for timber.		2.8mm x 30mm, 2.8mm x 40mm and 2.8mm x 50mm
Buildex FibreTEKS® For light gauge steel frames 0.55 to 0.75mm thick * denotes a registered mark of Buildex		10-6 x 20mm

8 FRAMING, FIXING AND INSTALLATION DETAILS FOR HARDIBRACE SHEET BRACING

8.1 GENERAL

This section sets out the framing, fixing and installation recommendations for HardiBrace sheet bracing.

For framing, fixing and installation recommendations for other James Hardie cladding products mentioned in this manual, refer to the following manuals:

- Villaboard lining, Versilux lining, FineRidge lining:
  James Hardie Internal lining range Fixing Manual
- HardiTex base sheets:
  James Hardie HardiTex system Technical Specification
- HardiFlex sheets, PanelClad sheets:
  James Hardie external cladding Technical Specification
- HardiPlank® cladding:
  James Hardie external cladding Technical Specification

NOTE
To achieve structural bracing using these products, you must use the fastener spacing and hold down recommendations of this manual instead of those set out in the individual James Hardie fixing manuals.

8.2 FRAMING

8.2.1 General
HardiBrace sheet bracing can be fixed to either timber or domestic steel framing. The framing used must comply with the relevant building regulations and standards and the requirements of this manual.

Frames must be straight and true to provide a flush face to receive the sheeting.

8.2.2 Timber
Use only seasoned timber. Unseasoned timber must not be used as it is prone to shrinkage and can cause sheets to move.

"Timber used for house construction must have the level of durability appropriate for the relevant climate and expected service life and conditions, that is exposure to insect attack or to moisture, which could cause decay".

Reference AS1684.2 "Residential Timber Framed Construction".

8.2.3 Steel
The base metal thickness (BMT) of a steel frame must be between 0.55mm and 1.6mm.

TABLE 6

GUN NAILING NOTES				
COIL NAILER		NAILS		
Manufacturer	Name	Manufacturer	Name	Size
Hitachi	VH650 coil nailer	Otter	Galv. coil nail	30mm x 2.5mm dia. 40mm x 2.5mm dia. 50mm x 2.2mm dia.
Paslode	Impulse compact nailer (B20544)	Paslode	Impulse DrivFast® 50mm Value Pack (B20531V)	50mm x 2.87mm dia.
			Impulse DrivFast® 50mm Handy Pack (B20507)	50mm x 2.87mm dia.
Duo-Fast	XCN50A coil nailer (Part No. D4004S)	Duo-Fast	C27/32 RD coil nail (D4180S)	32mm x 2.7mm dia.
Stanley Bostitch	N50C coil nailer	Stanley Bostitch	AC45P250 gal coil nail	45mm x 2.5mm dia.
Senco	SCN-50 coil nailer (fitted with adjustable depth of drive)	Senco	BTH 45 ADB Weathertex Gal and Sheraton	45mm x 2.5mm dia.

NOTES
1. Fasteners with equivalent dimensions, ie head size and shape, shank diameter and length to those shown are acceptable for use.
2. All fasteners are to be galvanised or suitably coated for intended external application.
3. Nailing guns must be fitted with flush drive attachments.
4. Some nailing guns incorporate an adjustable head set to control nail depth (eg Duo-Fast coil nailer and Senco coil nailer).
5. When gun nailing, apply pressure to the face of the cladding by holding the cladding against the stud to reduce blow out at back of the sheet.

Nail gun manufacturers have supplied the information contained in this table. Should a nail gun model or nail shown in the table not be available, please contact the relevant nail gun manufacturer for advice.
If the nail gun overdrives sheets, contact nail gun manufacturer for advice.

8.3 FIXING

8.3.1 General
You must select a fastener that is suitable for the type of frame you are using.

8.3.2 Fastener corrosion protection
Fasteners must have the appropriate level of durability required for the intended project. This is of particular importance in coastal areas, areas subject to salt spray and other corrosive environments.

Fasteners must be fully compatible with all other materials that they are in contact with to ensure the durability and integrity of the assembly.

Contact fastener manufacturers for more information.

NOTE
Fasteners must be at least Class 3 external grade finish.

8.3.3 Fixing depth
Nail sheets and boards in accordance with the nailing details shown in this manual. Do not overdrive the nails. Proud nailing is desirable, but flush head nailing is acceptable. See Figure 11.

8.3.4 Fastening to timber
Use 2.8 x 30mm hot dipped galvanised fibre cement nails when hand nailing.

HardiBrace sheet bracing can be gun nailed onto timber frames using flat head nails. Suitable combinations are shown in Table 6.

8.3.5 Fastening to steel
For steel framing of thickness 0.55mm to 0.75mm BMT, 20mm Buildex FibreTEKS self drilling screws.

For steel framing of thickness 0.75mm to 1.6mm, use 8g - 20mm or 8g - 30mm HardiDrive® grey external grade screws.

Fasteners should be screwed as close as possible to the stud corners to avoid deflection of the stud flange. See Figure 12.

8.3.6 Screw gun specification
Use variable speed screw guns with high torque, a maximum speed of 2500rpm, fitted with a depth control attachment.

Set the depth control attachment to avoid overdriving. As the screw thread begins to pull into the steel frame, drop the revs back to bed the head flush with the surface of the sheet.

8.4 INSTALLATION DETAILS

8.4.1 General
HardiBrace sheet bracing can be used for cavity bracing in brick veneer construction or internally in locations such as behind built-in full-height cupboards or robes.

For fastener spacings and hold-down recommendations, refer in this manual to:

- Clause 2.2 for the simplified design method or Clause 2.3 for the conventional Limit State design method for timber framing;
- Clauses 3.2 and 3.3 for timber framing with cyclone rods;
- Figure 10 and Clause 4.4 for steel framing.

NOTE
HardiBrace sheet bracing must not be used as exposed, finished, external cladding.

8.4.2 Brick ties
Brick ties can be installed through HardiBrace sheet bracing. Simply refer to the hole forming recommendations in Section 6. Ensure the hole is not greater than 50mm diameter through the sheet to allow insertion of brick ties. See Figure 13.

9 DETAILS

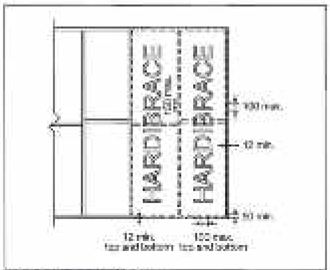

FIGURE 1 TYPE A NAILING DETAIL

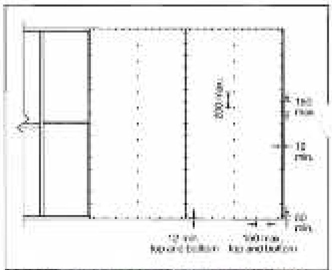

FIGURE 3 NAILING DETAIL FOR HARDIFLEX SHEETS, FINERIDGE LINING OR PANELCLAD SHEETS

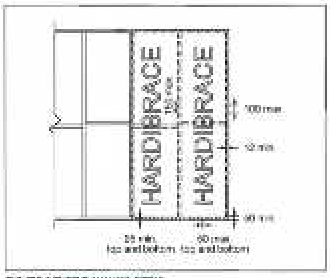

FIGURE 2 TYPE B NAILING DETAIL

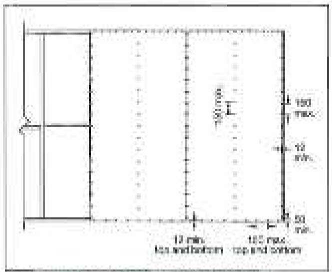

FIGURE 4 NAILING DETAIL FOR HARDITEX BASE SHEETS

NOTE
Nails along the top and bottom plates should be 25mm from the edge of the sheet for 50mm thick plates. When 38mm nominal thick plates are used, reduce edge distance to 20mm.

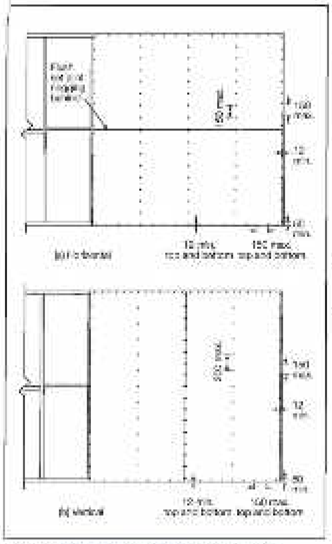

FIGURE 5 NAILING DETAIL FOR VILLABOARD SHEETS OR VERSILUX LINING

NOTE
For details of lining over these Villaboard linings, refer to the James Hardie Villaboard lining Installation Manual.

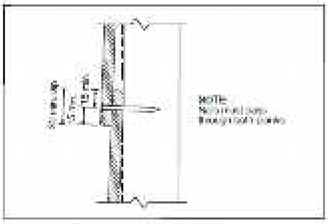

FIGURE 6 NAILING DETAIL FOR HARDIPLANK

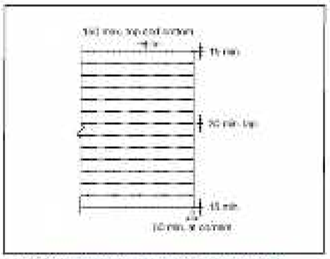

FIGURE 7 NAILING SPACING DETAIL FOR JHFC PLANKS OR WEATHERBOARDS

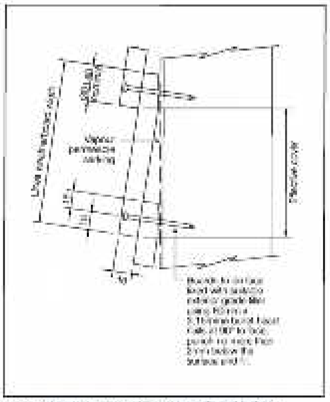

FIGURE 8 NAILING DETAIL FOR LINEA WEATHERBOARDS

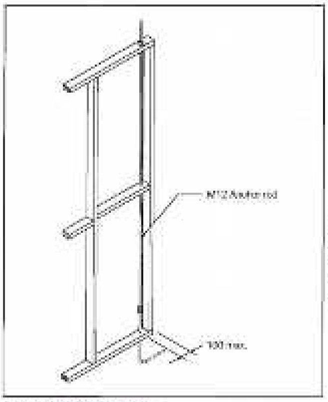

FIGURE 9 ANCHOR ROD DETAIL

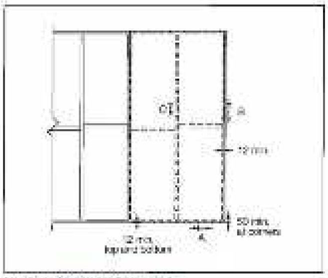

FIGURE 10 SCREW SPACING DETAIL

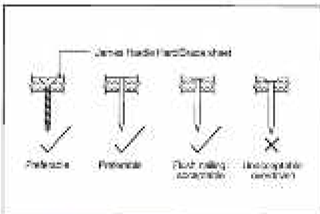

FIGURE 11 NAIL FASTENER DEPTH

NOTE
HardieBrace sheet bracing will not straighten excessively warped or distorted frames and any warping may be visible after the internal lining is applied.

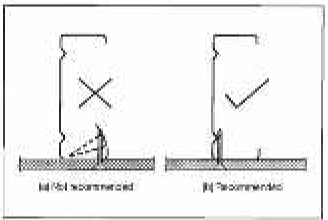

FIGURE 12 SCREW FASTENING TO STEEL FRAME

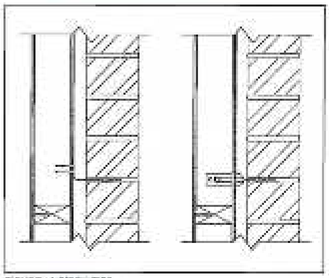

FIGURE 13 BRICK TIES

10 WARRANTY

James Hardie Australia Pty Limited ("James Hardie") warrants to the first purchaser of the product and the last purchaser prior to installation of the product for a period of 10 years from the date of purchase that the HardieBrace® sheet bracing (the "Product"), will be free from defects due to defective factory workmanship or materials and, subject to compliance with the conditions below, will be resistant to cracking, rotting, fire and damage from termite attacks to the extent set out in James Hardie's relevant published literature current at the time of installation. James Hardie warrants for a period of 12 months from the date of purchase that the accessories supplied by James Hardie will be free from defects due to defective factory workmanship or materials.

Nothing in this document shall exclude or modify any legal rights a customer may have under the Trade Practices Act or otherwise which cannot be excluded or modified at law.

CONDITIONS OF WARRANTY
The warranty is strictly subject to the following conditions:

a) James Hardie will not be liable for breach of warranty unless the claimant provides proof of purchase and makes a written claim either within 30 days after the defect would have become reasonably apparent or, if the defect was reasonably apparent prior to installation, then the claim must be made prior to installation;

b) this warranty is not transferable;

c) the Product must be installed and maintained strictly in accordance with the relevant James Hardie literature current at the time of installation and must be installed in conjunction with the components or products specified in the literature. To obtain copies of such literature contact Ask James Hardie on 13 11 03. Further, all other products, including coating and jointing systems, applied to or used in conjunction with the Product must be applied or installed and maintained strictly in accordance with the relevant manufacturer's instructions and good trade practice;

d) the project must be designed and constructed in strict compliance with all relevant provisions of the current BCA, regulations and standards;

e) the claimant's sole remedy for breach of warranty is (at James Hardie's option) that James Hardie will either supply replacement product, rectify the affected product or pay for the cost of the replacement or rectification of the affected product;

f) James Hardie will not be liable for any losses or damages (whether direct or indirect) including property damage or personal injury, consequential loss, economic loss or loss of profits, arising in contract or negligence or howsoever arising. Without limiting the foregoing James Hardie will not be liable for any claims, damages or defects arising from or in any way attributable to poor workmanship, poor design or detailing, settlement or structural movement and/or movement of materials to which the Product is attached, incorrect design of the structure, acts of God including but not limited to earthquakes, cyclones, floods or other severe weather conditions or unusual climatic conditions, efflorescence or performance of paint/coatings applied to the Product, normal wear and tear, growth of mould, mildew, fungi, bacteria, or any organism on any Product surface or Product (whether on the exposed or unexposed surfaces);

g) All warranties, conditions, liabilities and obligations other than those specified in this warranty are excluded to the fullest extent allowed by law.

h) If meeting a claim under this warranty involves re-coating of Products, there may be slight colour differences between the original and replacement Products due to the effects of weathering and variations in materials over time.

DISCLAIMER
The recommendations in James Hardie's literature are based on good building practice, but are not an exhaustive statement of all relevant information and are subject to conditions (c), (d), (f) and (g) above. Further, as the successful performance of the relevant system depends on numerous factors outside the control of James Hardie (eg quality of workmanship and design) James Hardie shall not be liable for the recommendations in that literature and the performance of the relevant system, including its suitability for any purpose or ability to satisfy the relevant provisions of the Building Code of Australia ("BCA"), regulations and standards.

SKM

Job No. 2B01276-218

Calc. Series

Client CCC

Page 1

Job Name Maurice Courts

By KS

Calcs Title Resident lounge

Date 18/4/13

Hall // Residents lounge

Assumptions:

- μ = 2, timber framed & lined
- Cd(T) = 0.37
- No live loads in roof voids.
- Ignore verandahs out front & over entrance. because negligable

Results:

Transverse %NBS = 47%

Longitudinal %NBS 62%

SKM

Client	Job No. ZB01276.218
Job Name	Calc. Series
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	By KS
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Seismic Weight:

Roof Buildup:

- Roofing iron ⇒ 0.15kPa
- Building paper & netting & insulation ⇒ 0.03kPa
- 12mm plywood ⇒ 0.07kPa
- 90 × 45 timber purlins @ 800 crs ⇒ 0.025kPa
- Timber trusses ⇒ 0.10kPa

Area : 10m × 15m = 150m² ⇒ 56kN

Ceiling ①

- 13mm gib baord ⇒ 0.07kPa
- 150 × 45 timber joists @ 600crs ⇒ 0.05kPa

Area : (3.2 × 3.9) + (5.0 × 2.0) = 23m² ⇒ 5kN

- Insulation & Services ⇒ 0.05kPa

Ceiling ②

- 13mm gib bard ⇒ 0.07kPa
- 90 × 45 battens @ 600 crs ⇒ 0.03kPa
- Insulation & services ⇒ 0.05kPa

Area : (9.6m × 15m) = 143m² ⇒ 22kN

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Client

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Job Name

By KS

Calcs Title

Date 18/4/13

- Walls: Timber

Internal - 0.4kPa

length = $(3.2 \times 3) + 3.9 + 2.0 + 1.5 + (0.6 \times 3)$

length = 19m

Height = 3.6m

Weight = $\Rightarrow$ 27kN

- External - 0.4kPa

length = $10 \times 2 + 15 \times 2$ = 50m

Height = 2.5m

Weight = $\Rightarrow$ 50kN

- Wall Cladding - Brick Veneer

- 1.35kPa
- length = $15 + (1.2 \times 2) + (6.5 \times 2)$

length = 30m

height = 2.5m $\Rightarrow$ 103kN

- Wall Cladding - Weatherboard

- 25kg/m² or 0.25kPa
- Area = $(\frac{1}{2} \times 3.25 \times 2.1) \times 2 \times 2$ ends

$+ (4m \times 2.5)$ = 25m² $\Rightarrow$ 6 kN

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Calc. Series

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Job Name

By KS

Calcs Title

Date 18/4/13

Total Weight

$W_T = 56 + 5 + 22 + 27 + 50 + 103 + 6$

$W_T = 270$ kN

$V_b = 0.37 \times 270$ kN

$V_b = 100$ kN

Load to be resisted is half wall height plus ceilings

Therefore :

$F_1 = 56 + 5 + 22 + 27/2 + 50/2 + 103/2 + 6/2$

$F_1 = 175$ kN

$F_1 = 175 \times 0.37$

$F_1 = 65$ kN

ZB01276.218
pages

Working instructions

Refer to recommended Safe Working Practices before starting any cutting or machining of product.

HardieBlade™ Saw Blade

The HardieBlade™ Saw Blade used with a dust-reducing saw connected to a HEPA vacuum is ideal for fast, clean cutting of James Hardie fibre cement products. A dust-reducing saw uses a dust deflector or a dust collector connected to a vacuum system. When sawing, clamp a straight-edge to the sheet as a guide and run the saw base plate along the straight edge when making the cut.

Hole-forming

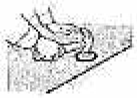
For smooth clean cut circular holes:

Mark the centre of the hole on the sheet.

Pre-drill a pilot hole.

Using the pilot hole as a guide, cut the hole to the appropriate diameter with a hole saw fitted to a heavy duty electric drill.

For irregular holes:



Small rectangular or circular holes can be cut by drilling a series of small holes around the perimeter of the hole then tapping out the waste piece from the sheet face.

Tap carefully to avoid damage to sheets, ensuring that the sheet edges are properly supported.

Storage and handling

All James Hardie building products should be stored to avoid damage, with edges and corners of the sheets protected from chipping.

James Hardie building products must be installed in a dry state and be protected from rain during transport and storage. The product must be laid flat under cover on a smooth level surface clear of the ground to avoid exposure to water or moisture, etc.

Quality

James Hardie conducts stringent quality checks to ensure that any product manufactured falls within our quality spectrum. It is the responsibility of the builder to ensure that the product meets aesthetic requirements before installation. James Hardie will not be responsible for rectifying obvious aesthetic surface variations following installation.

12 Product sizes

Table 6

Scyon Linea Weatherboard and Scyon Axent Trim sizes

						Coverage Information			
Product	Length (mm)	Width (mm)	Thickness (mm)	End Details	Effective Cover (mm)	No. of planks/ metre height (approx.)	Mass kg/lineal m (approx. at EMC)	Mass kg/m² (approx. at EMC)	Weight/ packs (60 units/ pack)
Scyon Linea Weatherboard 135	4200*	135	16	T & G	105	9.5	2.62	24.93	660.00
Scyon Linea Weatherboard 150	4200*	150	16	T & G	120	8.3	3.1	25.70	781.00
Scyon Linea Weatherboard 180	4200*	180	16	T & G	150	6.7	3.57	23.92	899.00
Scyon Axent Trim 84mm	2600	84	16	Square	N/A	N/A	1.6	N/A	N/A
Scyon Axent Trim 100mm	2600	100	16	Square	N/A	N/A	1.9	N/A	N/A

*Length is 4200mm plus 5mm for the tongue and groove making overall length 4205mm

*The effective thickness of finished Scyon Linea Weatherboard on the wall at the lap is approximately 33 to 35mm

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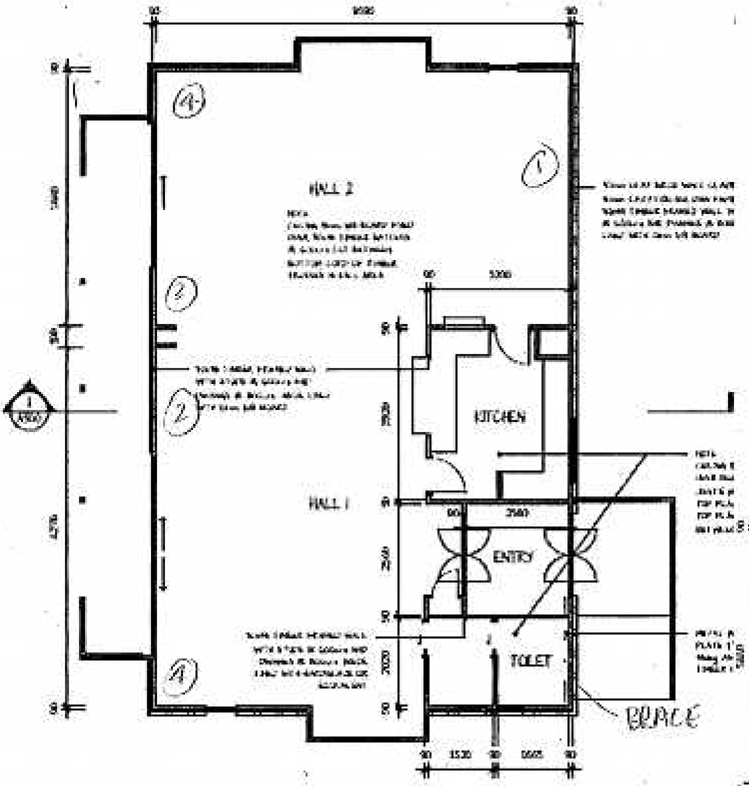
Date 18/4/13

Lateral load Resisting structure.

- Along : timber lined walls & angle brace
- Across : timber lined walls.

F_1 = 65kN Each direction.

F_1 = 1300 BU req'd.

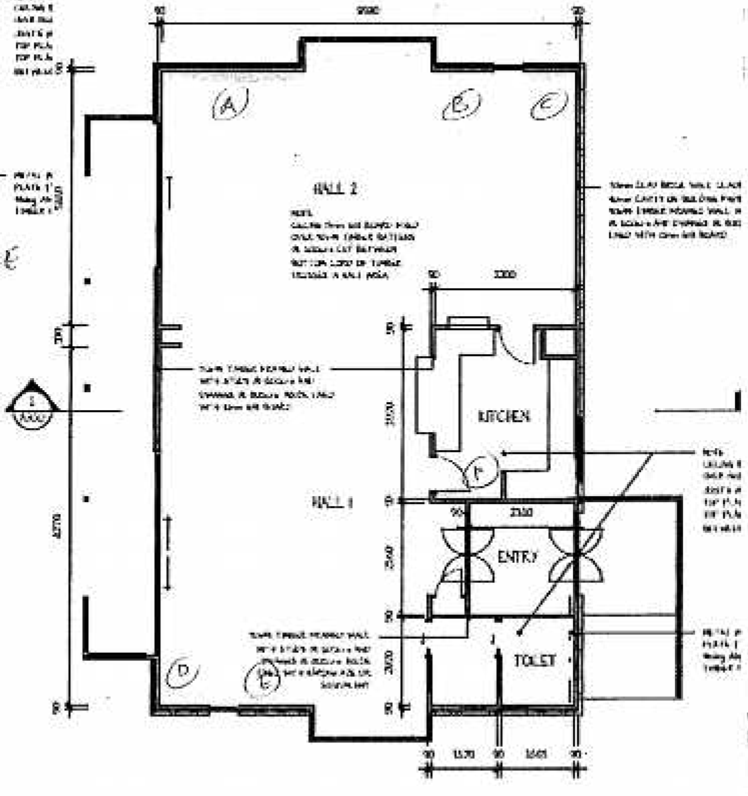


Along:

10mm Gib board single side + Angle brace

Across:

10mm Gib. single side



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Bracing Capacity:

10mm Gibboard

GIB EzyBrace® Systems

Bracing Resistance

GIB

JUNE 2011

Table 1: GIB® Standard Plasterboard Bracing Unit ratings

Type	Minimum Length (m)	Lining	Other Requirements	BU/m W	BU/m EQ
GS1-N	0.4	GIB® Standard Plasterboard one side	N/A	50	55
	1.2			70	60
GS2-N	0.4	GIB® Standard Plasterboard both sides	N/A	70	65
	1.2			95	85
GSP-H	0.4	GIB® Standard Plasterboard one side plywood the other	Panel hold-down fixings	100	115
	1.2			150*	150*

Table 2: GIB Braceline® Bracing Unit ratings

Type	Minimum Length (m)	Lining	Other Requirements	BU/m W	BU/m EQ
BL1-H	0.4	GIB Braceline® one side	Panel hold-down fixings	90	100
	1.2			125*	105
BLG-H	0.4	GIB Braceline® one side GIB® Standard Plasterboard the other	Panel hold-down fixings	110	115
	1.2			150*	145*
BLP-H	0.4	GIB Braceline® one side plywood the other	Panel hold-down fixings	135*	135*
	1.2			150*	150*

Note: The BU/m ratings for GIB EzyBrace® systems are responsibly conservative. Using the GIB EzyBrace® software will deliver higher ratings than using the manual tables.

* ***Timber Floors** – A limit of 120 BU/m for NZS 3604:2011 timber floors applies unless specific engineering ensures that uplift forces generated by elements rated higher than 120 BU/m can be resisted by floor framing.*

Wall Heights other than 2.4m

The published Bracing Unit ratings are based on a 2.4 metre height. For greater heights, the ratings must be multiplied by a factor f = 2.4 divided by the actual wall height. The Bracing Unit ratings for walls higher than 2.4 metres will reduce.

For example:

The Bracing Unit rating of a 2.7 metre high wall is obtained by multiplying the values in Tables 1 and 2 by f = 2.4/2.7 = 0.89

The Bracing Unit rating of a 3.6 metre high wall is obtained by multiplying the values in Tables 1 and 2 by f = 2.4/3.6 = 0.67

The height of walls with a sloping top plate can be taken as the average height.

Walls lower than 2.4 metres shall be rated as if they were 2.4 metres high.

Along:

Length of walls:

1		-	6.0m	×	60 BU/m		=	360 BU
2		-	2.3m	×	60 BU/m		=	138 BU
3		-	1.3m	×	60 BU/m		=	78 BU
4	(×2)	-	1.2m	×	60 BU/m	×2	=	144 BU

Angle Brace

- 1 - tension - 5.6kN capacity = 112 BU
- 1 - Compression - 4.1kN capacity = 82 BU

↳ Add compression only because compression < tension

Σ 802 BU

% NBS - 802 / 1300 = 62% NBS.

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Across :

Length of walls :

A	-	3.3m	x	60BU/m	=	198	BU
B	-	1.4m	x	60 BU/m	=	84	BU
C	-	1.2m	x	60 BU/m	=	70	BU
D	-	1.2m	x	60 BU/m	=	70	BU
E	-	1.8m	x	60 BU/m	=	108	BU
F	-	1.4m	x	60 BU/m	=	84	BU

Σ 614 BU

% NBS = 614 / 1300 = 47 % NBS

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By KS
Date 19/4/13

Blocks B/C/D Summary

Check that Block A - minus one unit produces answers similar

Using Block A weights

Assume:

- $\mu = 2$ & $\mu = 1.25$
- $Cd(T) = 0.37$ & 0.71 (Z = 2, $T \leq 0.5$sec, Soil Class D)
- No live load on Roof, 2kPa at 1st Floor.

Results:

Longitudinal	%NBS	GF	=	44 %	$\mu = 2.0$
		1st	≥	100 %	
Transverse	%NBS	GF	≥	100 %	$\mu = 1.25$
		1st	≥	100 %	

Other checks same as A. Block.

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Seismic Weight:

Roof weights:	Area 1	(5.8×2) × 7.5 = 87m²	⇒	26kN
	Area 2		⇒	13kN
	Area 3		⇒	32kN
Ceiling	Area 1	- 87m²	⇒	13kN
	Area 2		⇒	7kN
	Area 3	- level 1	⇒	13kN
		Roof	⇒	13kN
1st Floor			⇒	383kN
Stair			⇒	60kN
Patio			⇒	59kN
Concrete walls:	Yellow ×1 wall		⇒	78kN
	Green		⇒	220kN
	Pink		⇒	220kN
	Roof		⇒	43kN
	GF		⇒	36kN
Internal timber walls		GF (5units)	⇒	17kN
		1st	⇒	7kN
Cladding - weatherboard		GF (5units)	⇒	20.5kN
		1st	⇒	5kN
Cladding - Brick veneer		GF (5units)	⇒	144kN
		1st	⇒	40kN
External timber walls		GF (5units)	⇒	53kN
		1st	⇒	21kN
Live load		1st	⇒	51kN
			Σ	1575kN

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Job No. ZB01176.218

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By KS

Calcs Title

Date 19/4/13

Total Weight:

$F_1 = 26 + 13 + 13 + 7 + 13 + 383 + 60 + 59 + 78/2 + 220/2 + 220/2$
$+ 36 + 17/2 + 7/2 + 20.5/2 + 5/2 + 144/2 + 40/2$
$+ 53/2 + 21/2 + 51$

$F_1 = 1074\text{kN}$

$F_R = 32 + 13 + \quad + 220/4 + 220/4 + 43 + 7/2 + 5/2$
$+ 40/2 + 21/2$

$F_R = 234\text{kN}$

$F_a = 76/2 + 220/4 + 220/4 + 17/2 + 20.5/2 + 144/2 + 53/2$

$F_a = 266\text{kN}$

$F_T = 1074 + 234 + 266 = \underline{1575\text{kN}}$

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Job No. 2301276.218

Calc. Series

Client

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Job Name

By KS

Calcs Title

Date 19/4/13

Equivalent Static:

Level	h_i	W_i	Wh_i	Force (L)	Force (T)
GF	0	266	0		
1	2.5	1074	2685	373kN	716 kN
Roof	5.0	234	1170	209kN	406 kN
		1575	3855	583kN	1118 kN

$V_b = 0.37 \times 1575 = 583$ kN Longitudinal $\mu = 2$

$V_b = 0.71 \times 1575 = 1118$ kN Transverse $\mu = 1.25$

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Job No. 7301276.218

Calc. Series

Client

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Job Name

By KS

Calcs Title

Date 19/4/13

Longitudinal Demand vs Capacity

Ground floor Demand = 373 kN or 7460 BU

First floor Demand = 209 kN or 4180 BU

(From page 29)

Capacity of 1 unit = 483 BU (From A Block Calc) Page 15

5 × 483 = 2415 BU

GROUND FLOOR

% NBS = 2415 /(7460 - 4180) = 21% NBS

FIRST FLOOR

% NBS = 483 × 2 / 4180 = 23% NBS

Using out of plane capacity of concrete walls gives:

Gf = %NBS = 44%

1st = %NBS ≥ 100%

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Job No. Z 1301276.2 ZTC

Calc. Series

Client

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Job Name

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Transverse Demand vs Capacity

Ground Floor Demand = 716 kN or 14320 BU

1st Floor Demand = 406 kN or 8120 BU

V_b = 1118 kN

From page 29

In plane capacity of concrete walls.

ϕV_c = 612 kN × 2 walls = 1224 kN > V_b

Conservative

% NB >> 100 % NBS

SKM

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Block Summary:

		Block A	Block B, C, D
Longitudinal	GF	44 %	44 %
	Roof	≥ 100%	≥ 100%
Transverse	GF	≥ 100%	> 100%
	Roof	> 100%	> 100%

Therefore use : Longitudinal GF : 44%
Roof ≥ 100%.

SKM

Job No. ZB01276-216

Calc. Series

Client CCC

Page 1

Job Name Maurice Courts

By KS

Calcs Title Block H & I

Date 15/4/13

Garages - Block H & I.

Assumptions:

- $\mu = 2$ - timber framed & lined building
- $Cd(T) = 0.37$ (From TB calcs, IL 2, DL 50years)
- No live loads

Questions:

- Is there a lintel beam above garage door?
- Could I check it as timber portal along front?

Results:

Transverse	% NBS	=	≥ 100%
Longitudinal	% NBS	=	22%
Roof Bracing		≥	100%

Seismic Weight:

Roof Build up:

- 150 x 50 timber rafters @ 800 crs - 0.05 kPa
- Netting, Building paper - 0.03 kPa
- Blocking - 0.05 kPa
- Metal cladding - 0.15 kPa
- Contingency - 0.02 kPa

Σ 0.30 kPa

Area = 6.1m x 16.4m ⇒ 30kN

Walls:

- Timber framed & lined - - 0.4 kPa

length = (6.1m x 6) + (16.4 x 2) = 70m

Area = $\frac{2.1 + 2.4}{2}$ x 6.1 x 6 ⇒ 33kN

= 16.4 x 2.1 x 1 ⇒ 14 kN

= 16.4 x 2.4 x 1 ⇒ 16kN

Σ 63kN

- 70mm clay brick cladding - 1.35 kPa

Area = $\frac{2.1 + 2.4}{2}$ x 6.1 x 2 x 1.35 ⇒ 37kN

= 16.4 x 2.1 x 1 x 1.35 ⇒ 46kN

Σ 83 kN

Total Weight:

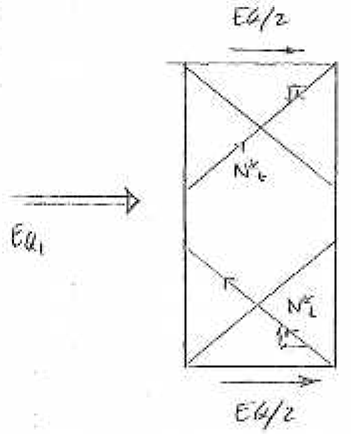
$W_T = 30 + 63 + 37 + 46 = 176\,kN$

$V_b = C_d(T_1)\,W_t$

$V_b = 176\,kN \times 0.37$

$V_b = 65\,kN$ in both directions.

Roof Bracing: Accross:



EQ_1 = Roof load only

EQ_1 = 30 kN

EQ_1 = 30 × 0.37 = 11 kN

$EQ/2$ = 6 kN

N^U_t = 10 kN

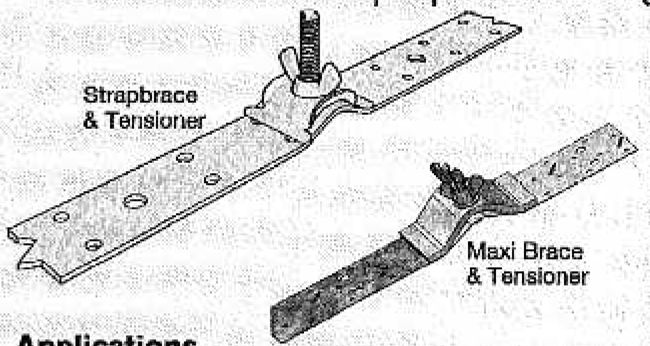
Capacity of 25mm wide diagonal brace.

Multi brace: ϕN_t = 16.4 kN

% NBS ≥ 100% NBS

Strapbrace & Maxi Strap

Convenient multi-purpose bracing for roofs, ceilings and walls



Applications

Pryda Strapbrace is suitable for bracing walls and truss/ rafter roof construction (spans up to 12m) in residential buildings. Use Pryda Maxi Strap for larger spans and commercial and industrial buildings. Pryda Tensioners provide a fast, reliable and simple method of tensioning long lengths of bracing strap.

Pryda Strapbrace complies with NZS3604:1999 Light Timber Frame Buildings, requirements for metal bracing strip with 8kN capacity, but is **not** suitable for use as holding down straps on braced wall panels: use Pryda Sheet Brace Straps for this application.

Pryda Strapbrace and Maxi Strap act in tension only; braces must be applied in pairs as illustrated. Holes are pre-punched for 3.15mm nails and 6mm tensioner bolts.

Specifications

Product Code, Size & Packaging:

SB10	- Strapbrace 10m coil (25 x 0.8mm)
SB10T	- Strapbrace 10m coil (25 x 0.8mm) plus 5 tensioners
SB30	- Strapbrace 30m coil (25 x 0.8mm)
SB30T	- Strapbrace 30m coil (25 x 0.8mm) plus 5 tensioners
SBT	- Strapbrace Tensioners - Box of 40
SB15/S	- Stainless Steel Strapbrace 15m coil (25 x 0.8mm)
SBT/SS316	- Stainless Steel Strapbrace Tensioners (Bag of 6)

SBI/10	- Maxi Strap 10m coil (50 x 0.8mm)
SBI/15	- Maxi Strap 15m coil (50 x 0.8mm)
SBI	- Maxi Strap 30m coil (50 x 0.8mm)
SBI/T	- Maxi Strap Tensioner - each
SBI/S	- Stainless Steel Maxi Strap 30m coil (50 x 0.8mm)

Material:

0.8mm G550 Z275 galvanised steel coil or stainless steel

Loads (Wind Only)

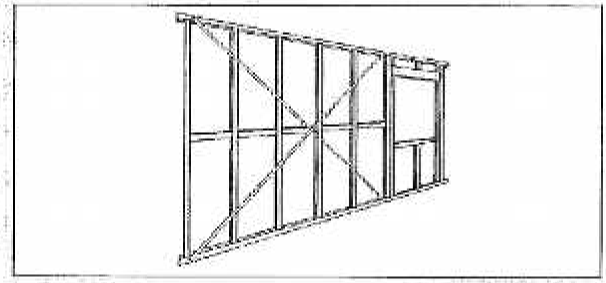
Strapbraces to be arranged in a diagonal pair and fully tensioned.

		Characteristic Strength	Wind Loading
Steel Strength	- Strapbrace	8.2	6.6
(kN)	- Maxi Strap	16.4	13.2

Wall Bracing

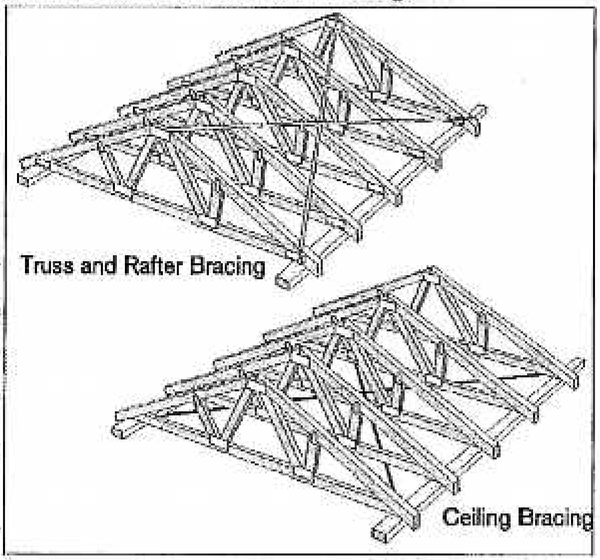
Make sure wall frame is approximately square. Nail end of brace to the top wall plate within 150mm of a stud, using 3/30 x 3.15mm Pryda Product Nails. Unroll brace coil at angle of approximately 45° and cut to length. Tighten by pulling down onto bottom wall plate. Nail within 150mm of stud with 3 nails.

Fix another brace in the same way diagonally opposite the first length. The two braces must cross to form a strong rigid brace. Fit tensioners (usually one per 3.6m length of brace) and plumb frame. Nail braces to intermediate studs with 2 nails after tensioning braces.



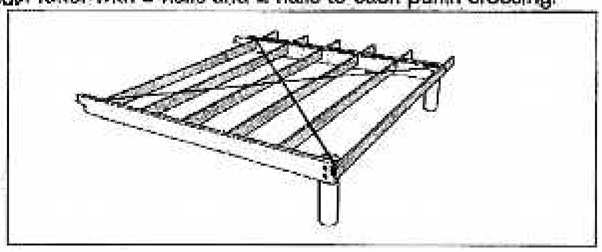
Roof and Ceiling Bracing

Use in crossed pairs as for wall braces. For residential construction in accordance with NZS3604:1999, secure braces with 6 nails at each end, and 2 nails (after tensioning braces) at truss/ rafter or Purlin crossing.



Pole & Rafter Buildings

Roof brace every second bay: Pryda Strapbrace fixed to face of rafter with 6 nails and 2 nails to each purlin crossing.



SKM

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Calc. Series

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Job Name

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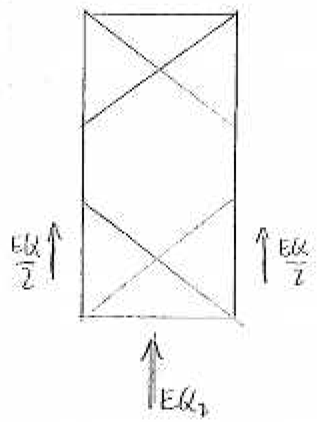
Date 15/4/13

Roof Bracing   Along:



EQ_x = Roof only = 11kN from previous page

N^*_t = 7.5kN

Multibrace = 16.4kN

Strap brace = 8.2kN   % NBS ≥ 100%

SKM

Client
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Calcs Title

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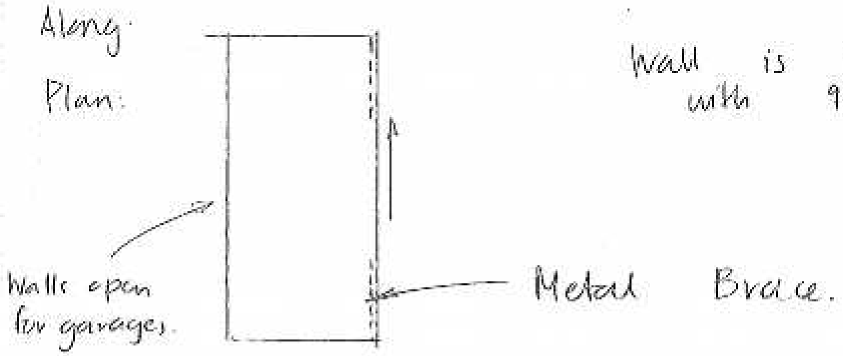
Calc. Series
Page 7
By KS
Date 15/9/13

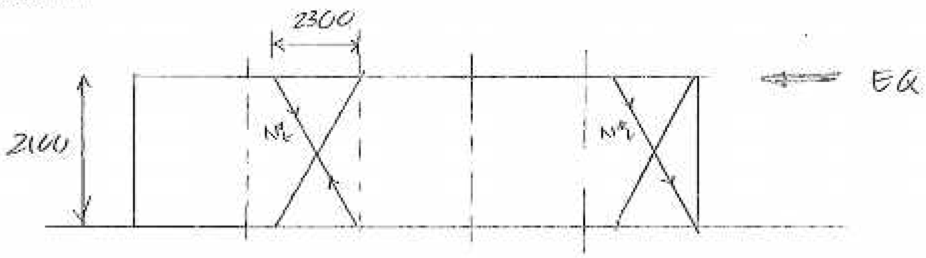
WALLS:

Along:
Plan:



Wall is timber framed with 90 x 45 studs @ 600crs

Elevation:



Assume bracing is the full height of the wall

EQ = Roof + half wall height

$EQ = 30kN + \frac{65}{2} + \frac{83}{2} = 103kN$

$EQ = 0.37 \times 103kN = 38kN$

$N^*_t = N^*_c = 13kN$ in each brace.

Angles act in tension & compression

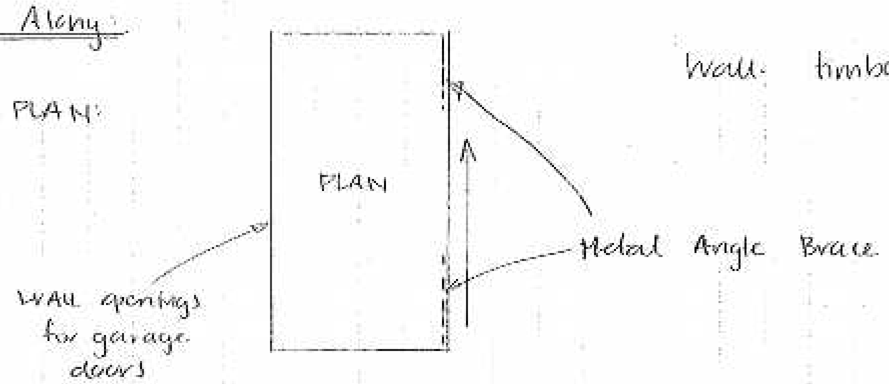
Angle brace capacity from prada catalogue

WALLS

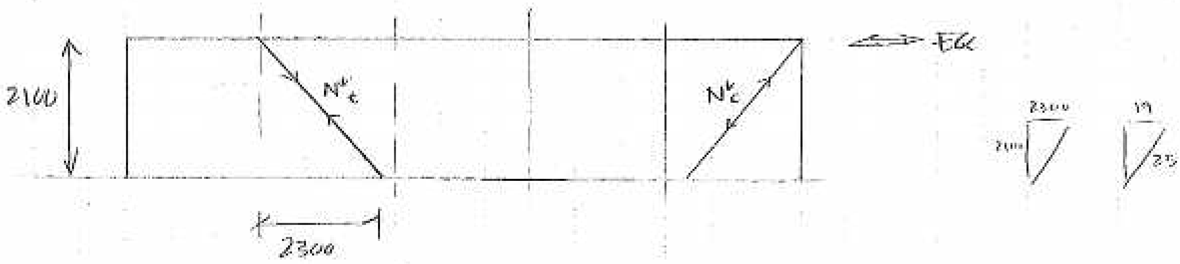
Along:

PLAN:

Wall timber framed & unlined



ELEVATION



Assuming bracing is the full height of the wall.

E_k = Roof + half wall height

E_k = 30 kN + 63/2 + 83/2 = 103 kN

E_k = 0.37 × 103 kN = 38 kN

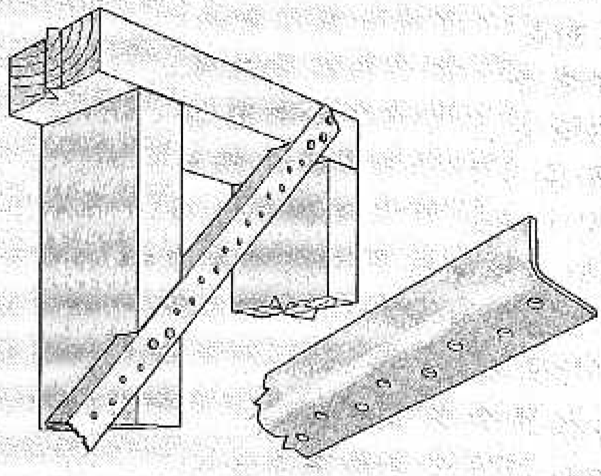
$N^*_t = N^*_c$ = 25 kN in each brace

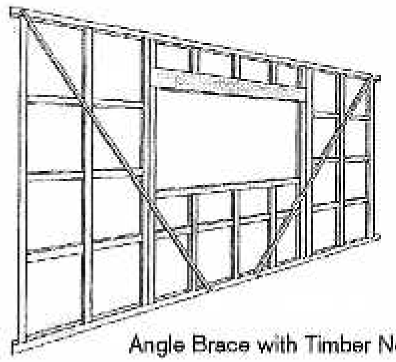
Angles act in both tension & compression

Angle Brace

page 9

A fast, effective brace for timber frames





Features

Pryda Angle Brace is the fast effective way to brace interior or exterior timber framing. It is fitted by making a single saw cut into the studs, inserting the brace, then nailing.

Because Pryda Angle Brace is power punched, it features clean, fully punched holes (no nails are bent or wasted by trying to force them through the brace).

Pryda Angle Brace utilises the tension and compression strength of steel with the properties of timber. It holds studs straighter, allows better air circulation and makes it easier to install wiring, plumbing and insulation.

"Checking in" Pryda Angle Brace flush with the surface of the timber can be done easily with the Pryda Angle Brace Checka fitted to an ordinary power saw. This attachment makes the saw cut and removes the timber to "check in" the brace in one operation.

Installation

After squaring up wall or temporary frame ready for bracing:-

1. Use the edge of the steel brace to draw a straight line where the brace is to go.
2. Cut the studs 20mm deep on this line with either a Pryda Angle Brace Checka, power saw or a hand saw.
3. Slide plain leaf of the angle into the sawcut. For safety reasons the punched leaf of the angle must point downwards. Nail punched leaf to the stud through the holes provided using 30 x 3.15 Pryda Product Nails, two per stud and minimum of three per end.
4. Brace is to be 150mm minimum from end of plate.

Specifications

Size:
20 X 20 X 1.00mm

Material:
G300 Z275 galvanised steel coil.

Product Code:
AB30 (3.0m long), AB33 (3.3m long), AB36 (3.6m long), AB42 (4.2m long), AB48 (4.8m long).

Packing:
Bundles of 10 lengths

Loads (Wind Only)

Steel: Characteristic Strength = 11.2 kN
Design Capacity (LSD) = 10.0 kN

Tension: Nails in one leg only

Number of nails each end	Characteristic Strength	Design Load (LSD) Brief
3	4.7 kN	4.2 kN
4	6.2 kN	5.6 kN

Note: These steel tensile loads cannot be achieved through normal nailing in 1 leg of the angle.

Compression: Studs @ 600mm centres

	Brace at 45°	Brace at 55°
Clear Brace Length	780mm	980mm
Characteristic Buckling Load	4.6 kN	3.1 kN
Design Load	4.1 kN	2.8 kN

SKM

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Tension Capacity

$\phi N_t = 6 \cdot 2\,kN$

Compression Capacity

$\phi N_c = 4 \cdot 6\,kN$

$\%\,MBS = (6 \cdot 2 + 4 \cdot 6)/(25 + 25) = 22\%\,MBS$

SKM

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Date 15/4/13

Across:

EQ →

EQ = 38 kN

Each wall lined on one side with 9mm timber plywood.

Each wall = 6 m

BU req'd: 38 kN / 6 walls × 20 BU = 127 BU.

9mm plywood capacity on one side

BU provd = 60 BU/m × 6m = 360 BU

% NBS > 100 % NBS

SKM

Client CCC	Job No. ZB 01276.218
Job Name MAURICE CARTER COURT – BL. J&K	Calc. Series BL. J&K
Calcs Title	Page 0-1
	By TOMAS BILEK
	Date 4/2/2017

SCOPE

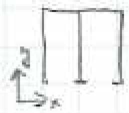
CALCULATE SEISMIC CAPACITY OF THE TWO GARAGES (BLOCKS J&K) AT 16 DUNDE PLACE / MAURICE CARTER COURTS IN CHRISTCHURCH.

STRUCTURE IS A TIMBER FRAME BUILDING WITH TIMBER TRUSSES, FOUNDED ON CONCRETE SLAB ON FILL.

ARCHITECTURAL DRAWINGS (SHOWING SOME STRUCTURAL INFORMATION) PRODUCED BY „CITY ARCHITECTS DIVISION : CITY WORKS & PLANNING DEPARTMENT" WERE AVAILABLE FOR THE CALCULATION.

FOLLOWING CALCULATIONS WERE CARRIED OUT FOR THE PURPOSE OF QUANTITATIVE ENGINEERING EVALUATION:

CONTENT



PLAN

1 LOADING

2 BRACING STRATEGY & CAPACITY — DIR X 22% NBS; DIR Y 124% NBS

3 PERIMETER WALLS = OUT-OF-PLANE FLEXURE — 143% NBS

X MANUFACTURER'S DATA

D DRAWINGS

SKM

Job No. ZB01276.218

Calc. Series BL J-K

Client CCC

Page 1-1

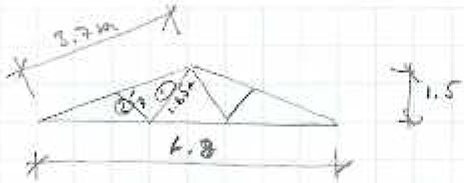
Job Name MAURICE CARTER COURTS – BL. J&K

By TB

Calcs Title LOADING

Date 4/2/2013

LOADING – SELF WEIGHT



ROOF

• ROOF TRUSS (1 UNIT)

RAFTERS	$2 \times 0.045 \times 0.09 \times 3.7 \times 4.6$	$= 0.14$ kN
BOTTOM CHORD	$1 \times 0.045 \times 0.09 \times 6.8 \times 4.6$	$= 0.13$ kN
DIAGONALS 1	$2 \times 0.045 \times 0.045 \times 1.65 \times 4.6$	$= 0.03$ kN
2	$2 \times 0.045 \times 0.045 \times 1.0 \times 4.6$	$= 0.02$ kN

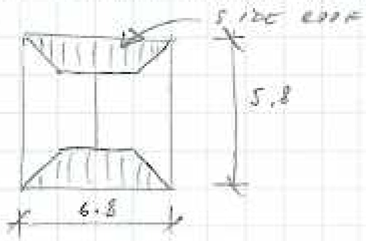
(TIMBER DENSITY)

SUB Σ = 0.32 kN

∴ CALCULATE AS UNIT AREA

SAY 4 UNITS + 2 EXTRA UNITS TO ALLOW FOR SIDE ROOFS

⇒ 6 UNITS TOTAL



$$6 \times 0.32 / (6.8 \times 5.4) = 0.053 \text{ kN/m}^2$$

• PURLINS @ 750mm c/c $\quad 0.075 \times 0.05 \times \frac{1000}{750} \times 4.6 = 0.023$ kN/m²

• METAL SHEETING (0.4mm) $\quad 0.0004 \times 78.5 = 0.032$ kN/m²

TOTAL ROOF = $0.053 + 0.023 + 0.032 = 0.11$ kN/m²

WALL – TIMBER STUDWORK + BRICK VENEER

• STUDWORK

VERTICAL STUDS @ 600mm c/c $\quad 0.05 \times 0.1 \times \frac{1000}{600} \times 4.6 = 0.04$ kN/m²

HORIZONTAL STUDS @ 600mm c/c $\quad$ — " — $= 0.04$ kN/m²

SUB Σ = 0.08 kN/m²

• BRICK VENEER $\quad 0.09 \times 18$ (BRICK DENSITY) $= 1.62$ kN/m²

Σ = 1.7 kN/m²

SKM

Job No. ZB 01276.218
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Job Name MAURICE CARTER COURTS – BL JAK
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WALL – TIMBER STUDWORK + GIB LINING (12.5) TO BOTH SIDES

STUDWORK

VERTICAL STUDS @ 600 mm c/c $\quad 0.05 \times 0.1 \times \frac{1000}{600} \times 4.6 = 0.04$ kN/m²

HORIZONTAL — // — $\quad$ — // — $= 0.04$ kN/m²

LINING

GIB DENSITY

GIB – 2 LAYERS $\quad 2 \times 0.0125 \times 10 \quad = 0.25$ kN/m²

$\Sigma = 0.33$ kN/m²

WALL – TIMBER STUDWORK + GIB INTERNALLY + WEATHER BOARDS EXTERNALLY

STUDWORK

VERTICAL STUDS @600mm c/c $\quad 0.05 \times 0.1 \times \frac{1000}{600} \times 4.6 = 0.04$ kN/m²

HORIZONTAL — // — $\quad$ — // — $= 0.04$ kN/m²

INTERNAL GIB LINING $\quad 1 \times 0.0125 \times 10 \quad = 0.125$ kN/m²

WEATHER BOARDS (7.5mm HARDI FLAXUS) $\quad 1 \times 0.0075 \times 13 \quad = 0.098$ kN/m²

$\Sigma = 0.31$ kN/m²

SKM

Job No. Z501276.218

Calc. Series BL J-K

Client CCC

Page 1-3

Job Name MAURICE CARTER COURTS - BL J-K

By TB

Calcs Title LOADING

Date 5/2/2013

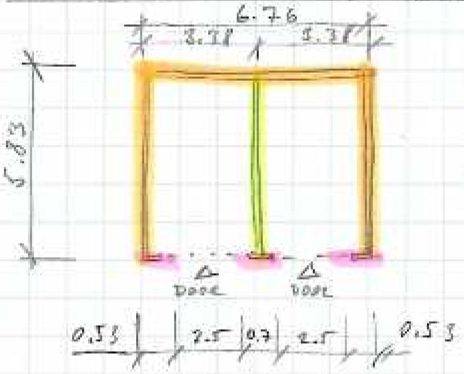
TOTAL WEIGHT OF ROOF

ROOF AREA = 6.8 × 5.4 = 36.72 m^2 (PROJECTED AREA)

WEIGHT OF ROOF = 0.11 kN/m^2

TOTAL WEIGHT OF ROOF = 36.72 × 0.11 = 4.04 kN

TOTAL WEIGHT OF WALLS



① TIMBER STUDWORK + BRICK VENEER 1.7 kN/m^2

② TIMBER STUDWORK + 2× GIB LINING 0.33 kN/m^2

③ TIMBER STUDWORK + GIB + HARDI PLANKS 0.31 kN/m^2

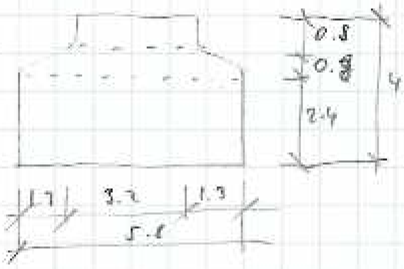
* TAKE DOORS AS ③

① $L_1 = 2 \times 5.83 + 6.78 = 18.44$ m

$W_1 = 18.44 \times \underbrace{2.4}_{\text{WALL HEIGHT}} \times 1.7 = 75.3$ kN

② $Area_2 = 5.8 \times 2.4 + 3.2 \times 1.6 + 1.3 \times 0.8 = 20.08 m^2$

$W_2 = 20.08 \times 0.33 = 6.6$ kN



③ $L_3 = 6.76$ m

$W_3 = 6.76 \times 2.4 \times 0.31 = 5.1$ kN

TOTAL WEIGHT OF WALLS = 75.3 + 6.6 + 5.1 = 87 kN

TOTAL WEIGHT OF THE BUILDING (ABOVE GROUND LEVEL)

W_b = [ROOF] + [WALLS]

W_b = 4.04 + 87 ≐ 91 kN

1-4

CALCULATIONS

SINCLAIR KNIGHT MERZ SKM

PROJECT	CCC BU 1103-10 & 11 EQ2 - Maurice Carter Courts - Blocks J & K		
PART OF STRUCTURE	Earthquake Loading - Design Action Coeficients	PROJECT No. ZB01276.218	REVISION 0
		DATE 28 Feb 13	BY Tomas Bilek

Earthquake Loading to NZS 1170.5:2002

Spreadsheet Rev 0.1

This spreadsheet is for the calculation of equivalent static earthquake loads on structures. All references are to NZS 1170.5:2004 except where noted. As per NZS 1170.5 this spreadsheet is not applicable to bridges, tanks containing liquids, civil structures (dams and bunds etc) off-shore structures and soil retaining structures. It is recommended that the structure period is calculated but if not 0 seconds should be input to acheive conservative results. This spreadsheet is not applicable to parts of structures - see section 8 for design of parts.

INPUT

	Site location	Christchurch	
3.1.3	Site Subsoil class	D	
AS/NZS1170.0	Importance level	2	
AS/NZS1170.0	Design life	50 years	
AS/NZS1170.0	ULS Earthquake Annual probability of exceedance	1/500	
AS/NZS1170.0	SLS Earthquake Annual probability of exceedance	1/25	

CALCULATION

Structure period, T	0.4	s	structure period less than 0.5s
Structural Ductility Factor, μ	1.50		timber walls & angle braces resisting seismic loads (disipation possible in case of braces failing)
ULS Structural Performance Factor from material code, S_p			
Note: Leave these S_p blank to use the values in NZS1170.5			
Spectral shape factor, $C_h(T)$	3.00		
Spectral shape factor, $C_h(0)$	3.00		
Hazard factor, Z	0.3		
ULS Return Period, R_u	1.00		
SLS Return Period, R_s	0.25		
ULS Near fault factor, N(T,D)	1.00		
SLS Near fault factor, N(T,D)	1.00		
ULS Elastic site hazard spectrum for horizontal loading, C(T)	0.90		
SLS Elastic site hazard spectrum for horizontal loading, C(T)	0.23		
ULS Elastic site hazard spectrum for vertical loading, $C_v(T)$	0.63		
SLS Elastic site hazard spectrum for vertical loading, $C_v(T)$	0.16		
ULS Structural Performance Factor, S_p	0.85		
ULS Structural Performance Factor for sliding or toppling, S_p	1.0		
SLS Structural Performance Factor, S_p	0.7		
k_μ	1.29		

ULS horizontal design action coefficient, $C_d(T_1)$	**0.60**
ULS horizontal design coefficient sliding or toppling, $C_d(T_1)$	**0.70**
ULS vertical design action coefficient, $C_{vd}(T_1)$	**0.54**

SLS horizontal design action coefficient, $C_d(T_1)$	**0.12**
SLS vertical design action coefficient, $C_{vd}(T_1)$	**0.11**

SKM

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Client	CCC
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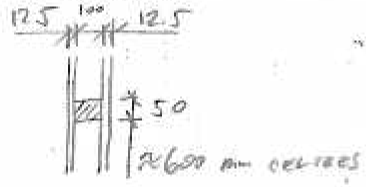
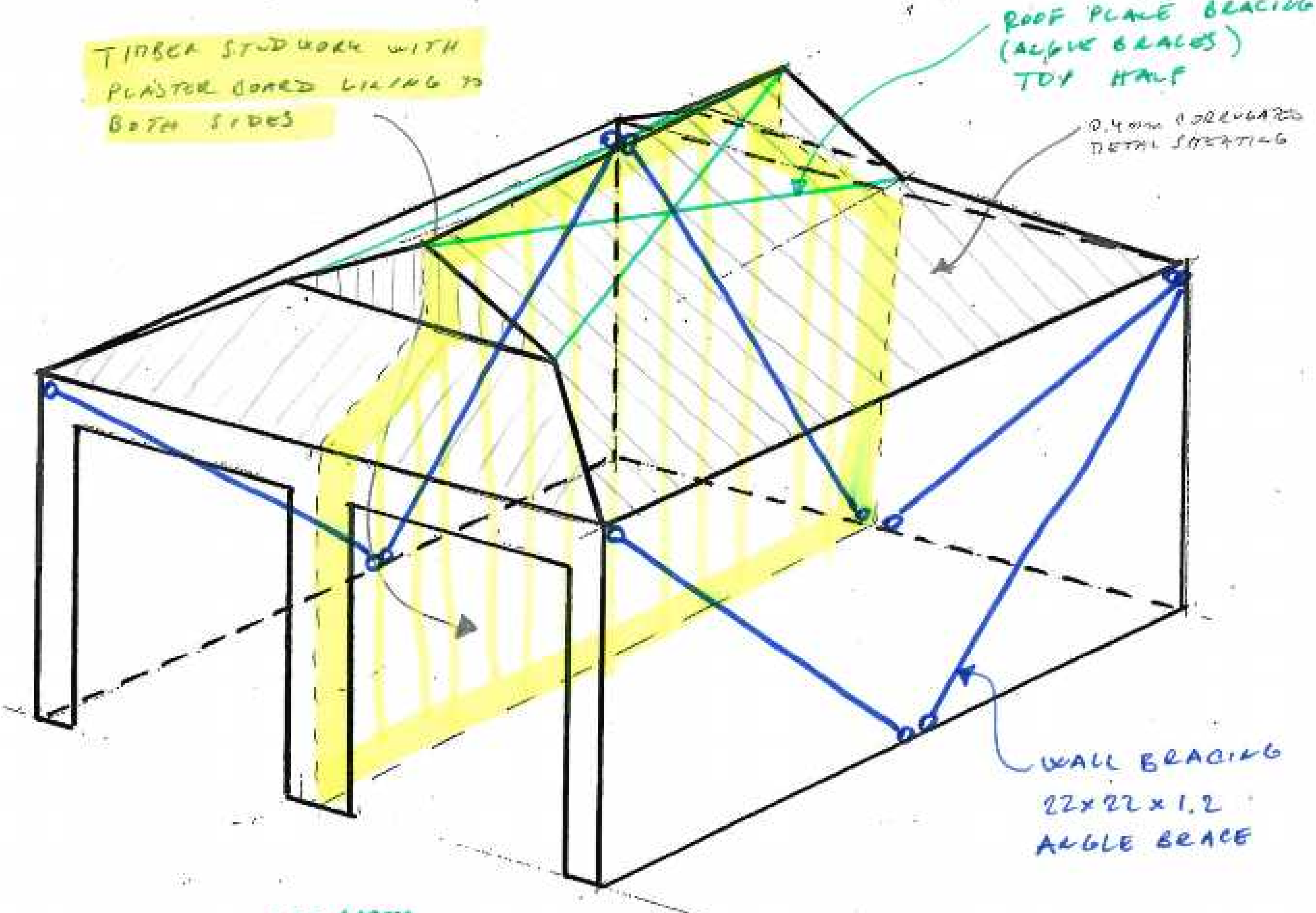
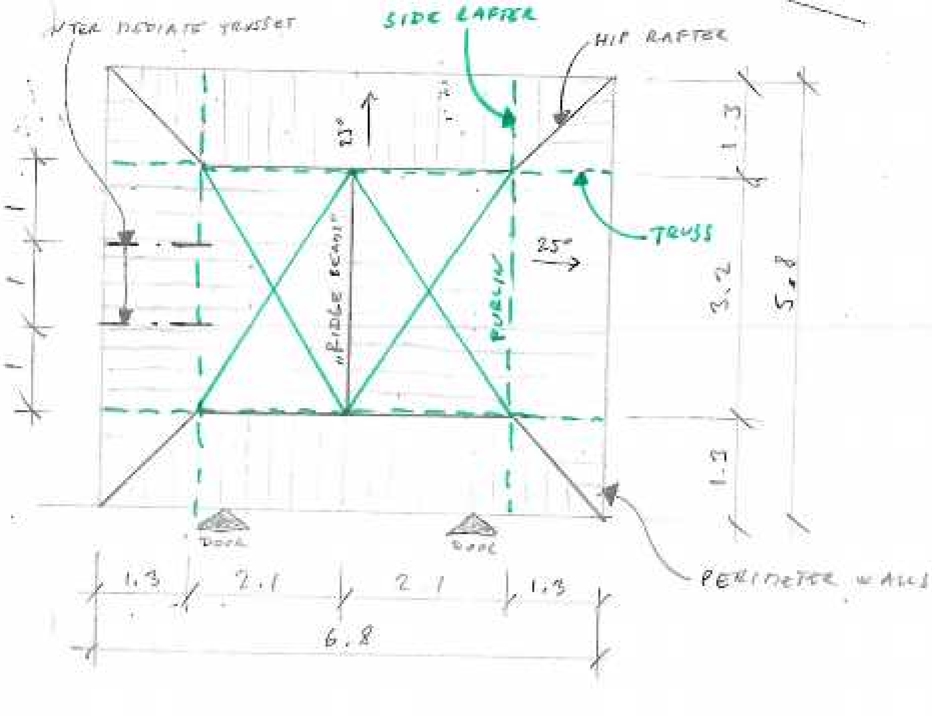
Job Name	MAURICE CARTER COURTS - BL. J & K
By	TOMAS BOLEK
Calcs Title	BRACING STRATEGY
Date	4/2/2013







→ LIGHTWEIGHT ROOF WITH SUFFICIENT TRIANGULATION.

→ ROOF BRACING OK BY INSPECTION.

PLAN ON ROOF BRACING

SKM

Job No. ZB01276.218

Calc. Series BLOCK J-K

Client CCC

Page 2-2

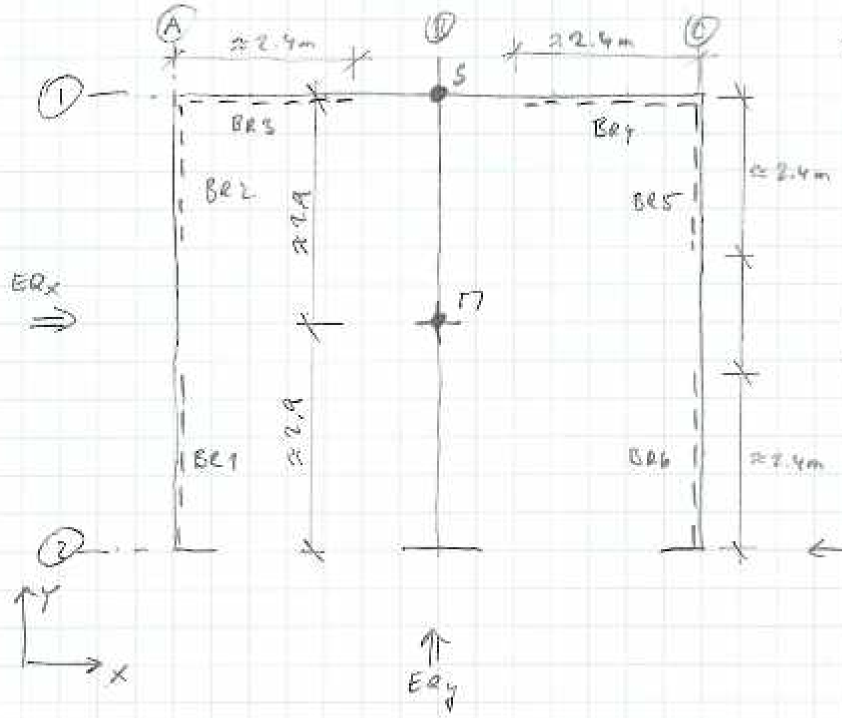
Job Name MAURICE CARTER COURTS - BLOCK J&K

By TB

Calcs Title PERIMETER WALLS

Date 5/2/2013

CHECK DIAGONAL BRACING IN WALLS



BR.. DIAGONAL BRACING
ANGLE BRACE 22x22x1.2

WORK IN BOTH TENSION & COMPRESSION (SEE MANUFACTURERS SPEC.)

• S CENTRE OF SHEAR

• M CENTRE OF MASS

← NO BRACING IN WALL ON GRID 2

• THE WORST CASE SCENARIO OCCURES FOR EARTHQUAKE LOAD IN DIRECTION X. ALL LOAD IS TAKEN BY BRACE B4 AND TORSIONAL EFFECT (DUE TO ECCENTRICITY OF CENTRE OF SHEAR) BY BRACES BR1 & BR5

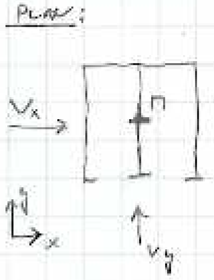
HORIZONTAL SEISMIC SHEAR (ONLY WEIGHT ABOVE 1.2m ABOVE GL)

$$V = C_d(T_1) \times \left[[ROOF] + [WALLS]/2\right] = 0.60 \times \left[4.04 + \frac{87}{2}\right] =$$

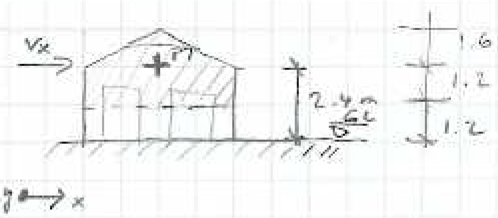
$$= 30 \text{ kN}$$

APPLY AT CENTRE OF MASS @ 2.4m ABOVE GROUND LEVEL

PLAN:



ELEVATION



$V_x = V_y = V = 30$ kN

SKM

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Page 2-3
Job Name MAURICE CARTER COURTS - BL J&K
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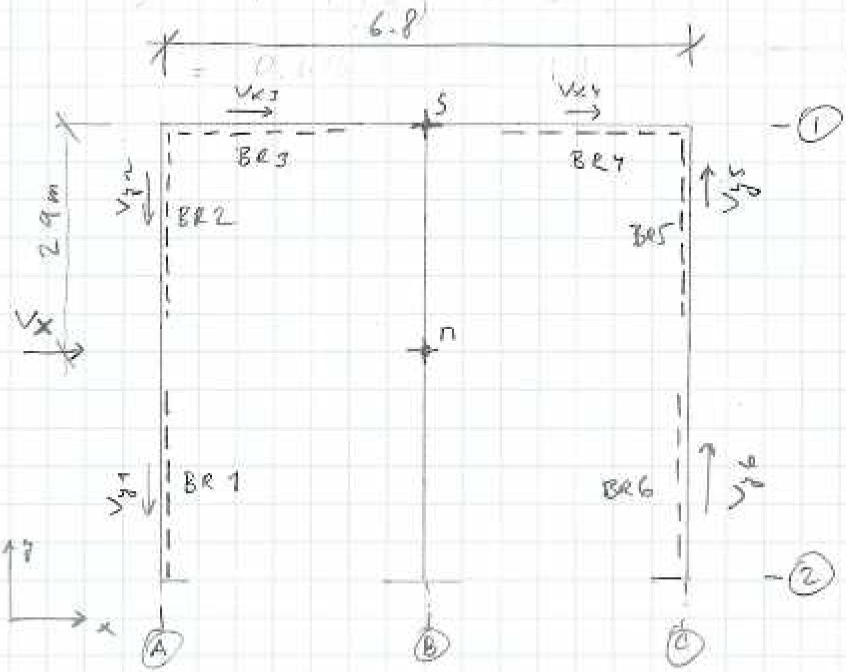
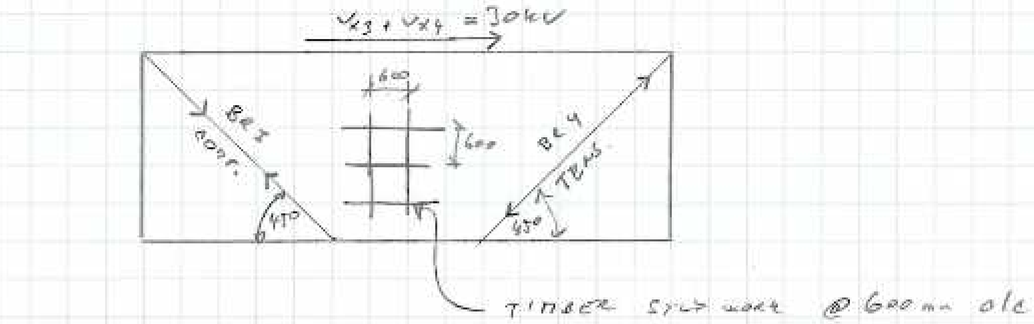
CALCULATE SHEAR DISTRIBUTION FOR V_x



$$V_{x3} + V_{x4} = V_x = 30\text{ kN}$$

$$V_{y1} + V_{y2} = V_{y5} + V_{y6} = V_x \frac{2.9}{6.8} = 30 \times \frac{2.9}{6.8} = 12.8\text{ kN}$$

CHECK WALL ON GRID 1



COMPRESSION CAPACITY OF BR3
- USING PRYDA'S SPECIFICATION SHEET FOR "20×20×1.0 G300 Z275 GALVANISED STEEL COIL" (PAGE X-1)
- CONSIDERING EQ SCENARIO, ULS : CHAR BUCKLING LOAD = 4.6 kN

TENSION CAPACITY OF BR4
- 3 NAILS AT EACH END, ULS : CHAR STRENGTH IN TENSION = 4.7 kN

FROM THE ABOVE, THE BRACES BR3 & BR4 SHARE THE LOAD AT RATIO APPROX 1:1. ⇒ $V_{x3} = 30/2 = 15$ kN
$V_{x4} = 30/2 = 15$ kN

SKM

Job No. ZB 01276.218

Calc. Series BLOCK J-K

Client CCC

Page 2-4

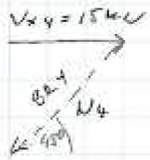
Job Name MAURICE CARTER COURTS - BLOCK J&K

By TB

Calcs Title PERIMETER WALLS

Date 5/2/13

- CHECK AXIAL LOAD IN BR4

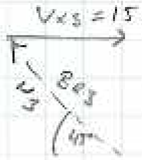


$$N_4 = \frac{V_{x4}}{\cos 45^\circ} = \frac{15}{0.707} = \underline{21.21\ kN}$$

BRACE 4 TENSION CAPACITY = 4.7 kN

$$\%NBS = \frac{4.7}{21.21} \approx \underline{22\%\ NBS}$$

- CHECK AXIAL LOAD IN BR3



$$N_3 = \frac{V_{x3}}{\cos 45^\circ} = \frac{15}{0.707} = 21.21\ kN$$

BRACE 3 COMPRESSION CAPACITY = 4.6 kN

$$\%NBS = \frac{4.6}{21.21} \boxed{\approx 22\%\ NBS}$$

DISCUSSION:

- OVERALL IMPROVEMENT CAN BE ACHIEVED BY INSTALLATION OF INTERNAL LINING (PLYWOOD OR SIMILAR) ATTACHED TO TIMBER STUDWORK.

NOTE: THE STEEL GARAGE DOORS PROVIDE CERTAIN SHEARING CAPACITY ON GRID LINE 2 WHEN IN CLOSED POSITION. THE CRITICAL SITUATION OCCURS WHEN BOTH DOORS ARE OPEN.

SEISMIC PERFORMANCE IN DIRECTION Y

1) CONTRIBUTION OF CENTRAL WALL (TIMBER STUDWORK + 12.5mm GIB LINING TO BOTH SIDES) ON GRID B

LENGTH OF WALL = 5.8 m

$$\text{SHEARING CAPACITY} = \underbrace{3 \times 0.7}_{\text{CAPACITY OF GYPSUM BOARD PER METER (NZSEE GUIDANCE - TABLE 11.1)}} \times 5.8 \times \underbrace{2}_{\text{2 FACES}} = 24.36\ kN$$

SKM

Client: CCC	Job No. 8601276.218
Job Name: MAURICE CARTER COURTS – BL J&K	Calc. Series: BL. J–K
Calcs Title: PERIMETER WALLS	Page: 2–5
	By: TB
	Date: 5/2/2017

2) CONTRIBUTION OF 4NO BRACES ON GRID LINES A & C (BR 1,2,5 &6)

– CAPACITY OF EACH ≈ $4.6 \times \cos 45° = 3.25$ kN

– OVERALL CAPACITY = $4 \times 3.25 = 13$ kN

COMBINED CAPACITY OF WALL ON GRID 5 AND WALLS ON GRIDS A & C

$= 24.36 + 13 = 37.36$ kN

DEMAND
$V_y = 30$ kN

$\% \text{ NBS} = \frac{37.36}{30} = \boxed{124\% \text{ NBS}}$

SKM

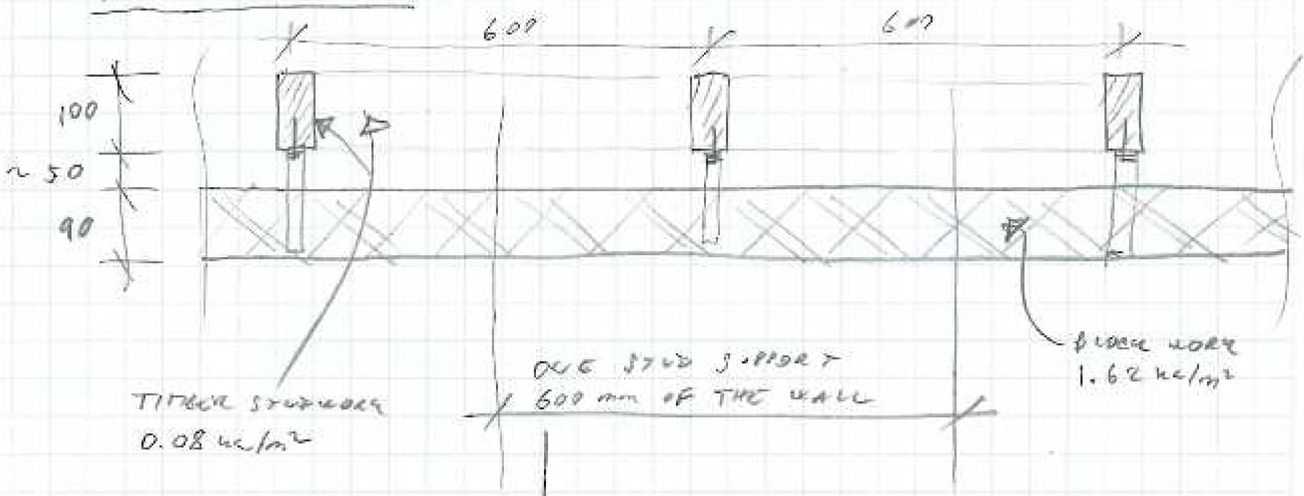
Client: CCC	Job No. ZB01276.218
Job Name: MAURICE CARTER COURTS – BL J&K	Calc. Series: BL J-K
Calcs Title: PERIMETER WALLS	Page: 3-1
	By: TB
	Date: 5/2/2013

CHECK OUT-OF-PLANE CAPACITY OF PERIMETER WALL

- CHECK WALL ON GRID A – TIMBER STUDWORK WITH BRICK VENEER

VERTICAL BENDING IS RESISTED BY VERTICAL TIMBER STUDS @ 600 mm c/c. BRICK IS CONNECTED TO TIMBER FRAME VIA WALL TIES AND IS ASSUMED TO HAVE ZERO FLEXURAL CAPACITY.

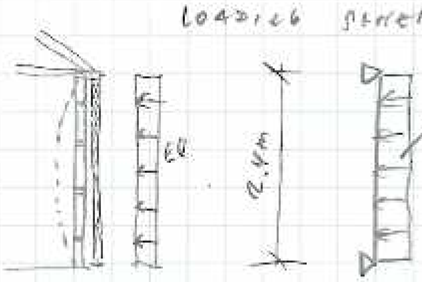
PARTIAL PLAN



WEIGHT OF THE 600mm WIDE STRIP

$W_S = 0.6(0.08 + 1.62) = 1.02$ kN/m

LOADING SCHEMATIC ON TIMBER STUD

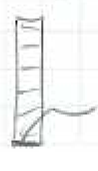


$W_S \times C_d(T_1) = 1.02 \times 0.60 = 0.62$ kN/m

VERTICAL BENDING

$M_{MAX} = \frac{1}{8} \times 0.62 \times 2.4^2 = 0.45$ kNm

AXIAL LOAD



[WEIGHT OF ROOF]

$N_{max} = 0.6 \times 3.4 \times 0.11 = 0.225$ kN $\rightarrow$ SAY 0.3 kN

SKM

Client CCC

Job Name MAURICE CARTER COURTS - BL J&K

Calcs Title PERIMETER WALLS

Job No. ZB01276.218

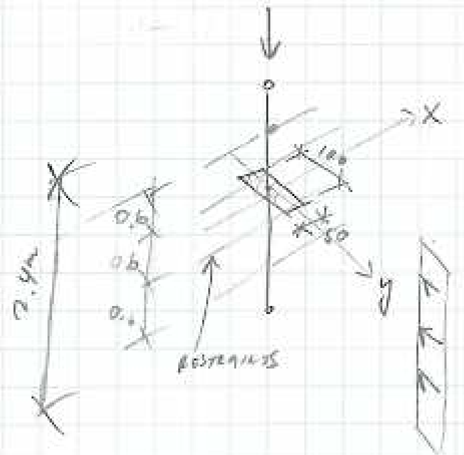
Calc. Series BL J-K

Page 3-2

By TB

Date 5/2/2013

CHECK SECTION USING EXCEL SPREADSHEET



EFFECTIVE LENGTH FOR BENDING ABOUT AXIS:

x $L_{EFF} = 2.4m$

y $L_{EFF} = 0.6m$

UTILISATION RATIO = 0.7 (COMBINED BENDING & COMP.)

$\%NBS = \frac{1}{0.7} \times 100 = \boxed{143\% \text{ NBS}}$

3-3

CALCULATIONS

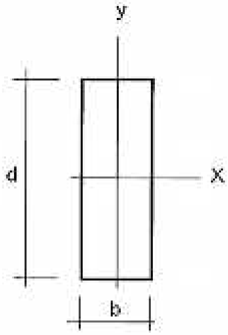
SINCLAIR KNIGHT MERZ

PROJECT	CCC Maurice Carter Courts - Block J&K (Garages)		
PART OF STRUCTURE	Timber stud	PROJECT No. ZB01276.218	REVISION 0
	Perimeter wall subject to out-of-plane bending and compression	DATE 12 Mar 13	BY TB

Timber Beam and column Design

Spreadsheet Rev 0.3

This calculation is for undertaking combined design checks of rectangular timber beams using NZS 3603:1993 Timber Structure Standard. This calculation only covers single members, built-up members need specific considersation regarding slenderness and effective length.
Ensure that connections are always checked as these often govern timber design.



INPUT

Section		100 x 50, No 1 Framing Radiata Pine, Dry (m/c = 16 %)		
Beam Depth (nominal)		100	mm	
Beam Breadth (nominal)		50	mm	
Timber Species		Radiata Pine		
Grade of Timber		No 1 Framing		
Timber Moisture content		Dry (m/c = 16 %)		

Note: The timber moisture content should be matched closely to the nevironmental conditions the timber will be in. For example for internal timber it is recommended to use Dry.

Number of members effectively connected		1		
Note: Effectively connected means that they are constrained to the same deformation				
Span of member/column height	L	2400	mm	
Dist betw restraints against lat. movement about X	L_{ax}	2400	mm	(i.e.restraints in dir Y at....mm)
Dist betw restraints against lat. movement about Y	L_{ay}	600	mm	(i.e.restraints in dir X at.....mm)
Lateral restraint provided at the tension edge?		No		→ S1 according to Cl. 3.2.5.2

Tbls 2.2 & 2.3	Bending strength	fb	10.0 MPa	
Tbls 2.2 & 2.3	Compression Strength	fc	15.0 MPa	
Tbls 2.2 & 2.3	Tension Strength	ft	4.0 MPa	
Tbls 2.2 & 2.3	Shear Strength	fs	3.8 MPa	
Tbls 2.2 & 2.3	Elastic Modulus	E	6.0 GPa	
Tbls 2.2 & 2.3	Lower Bound Elastic modulus	E_{lb}	4.0 GPa	
	Final Modulus of Elasticity	E	4.0 GPa	
	Actual Depth	d	100 mm	(check if same as nominal above)
	Actual Breadth	b	50 mm	(check if same as nominal above)

LOADING

Major axis bending moment	M*x	0.45	kNm	(bending about x)
Minor Axis bending moment	M*y	0	kNm	(bending about y)
Axial load (+ve compression)	N*	0.3	kN	(+compression, - tension)
Shear Force major direction	V^*_x	0	kN	(shear in dir x)
Shear Force minor direction	V^*_y	0	kN	(shear in dir y)
Loading condition		Brief (wind, earthquake, impact etc)		
Strength reduction factor	ø	0.8		

SECTION PROPERTIES

Section Area	A	5000 mm^2		
Self weight		0.025 kg/m		
Second moment of area about major axis	Ix	4.17 $x10^6mm^4$	(about x)	
Second moment of area about minor axis	Iy	1.04 $x10^6mm^4$	(about y)	
Section modulus about major axis	Zx	83.33 $x10^3mm^3$	(about x)	
Section modulus about minor axis	Zy	41.67 $x10^3mm^3$	(about y)	
Shear Area (loaded about major axis)	A_s	3333 mm^2		
Slenderness coefficient (for bending about X - major)	$S_{1,x}$	6.155	< 85 OK	as Cl. 3.2.5.2
Slenderness coefficient (for bending about Y - minor)	$S_{1,y}$	0.000	< 85 OK	(always 0)
Slenderness coefficient (compr. buckling about X)	S_2	24.000	< 85 OK	
Slenderness coefficient (compr. buckling about Y)	S_3	12.000	< 85 OK	

CALCULATIONS

SINCLAIR KNIGHT MERZ
SKM

PROJECT	CCC Maurice Carter Courts - Block J&K (Garages)		
PART OF STRUCTURE	Timber stud	PROJECT No. ZB01276.218	REVISION 0
	Perimeter wall subject to out-of-plane bending and compression	DATE 12 Mar 13	BY TB

Duration of load factor	k_1	1.0	
Parallel support factor	k_4	1.00	
Grid system factor (assumed to be 1.0)	k_5	1.00	
Stability factor, Bending about x	$k_{8,bend,x}$	1.00	(lat tors buckling about Y)
Stability factor, Bending about y	$k_{8,bend,y}$	1.00	(lat tors buckling about X)
Stability factor, Compress. - buckling about x axis	$k_{8,comp,x}$	0.49	(buckling about X - derived form S2)
Stability factor, Compress. - buckling about y axis	$k_{8,comp,y}$	0.99	(buckling about Y - derived form S3)
Effective length factor	$k_{10,x}$	1	(buckling about X)
	$k_{10,y}$	1	(buckling about Y)

STRENGTH OF MEMBER

Flexural Shear Strength	Vn	12.7 kN
Bending strength about x axis	Mnx	0.83 kNm
Bending strength about y axis	Mny	0.42 kNm
Compression strength about x axis	N_{ncx}	37.0 kN
Compression strength about y axis	N_{ncy}	73.9 kN
Tension strength	N_{nt}	20.0 kN

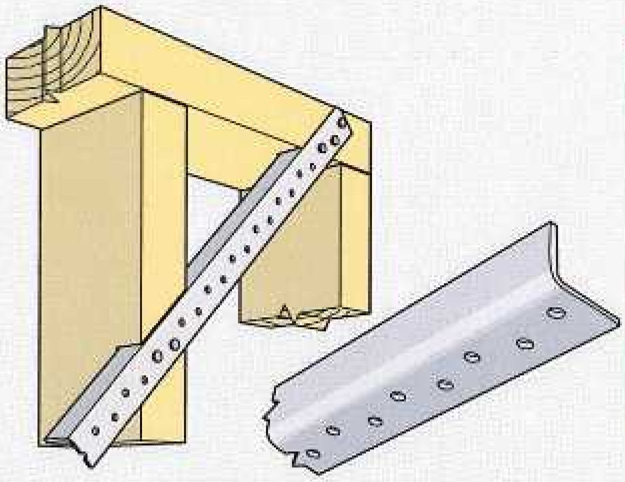
Clause		Check								Utilisation Ratio	
3.2.3.1		**Flexural Shear Strength**									
	Vx	V^*_x	≤	ϕV_n							
		0.0	<	10.1						N/A	
	Vy	V^*_y	≤	ϕV_n							max
		0.0	<	10.1						N/A	0.00
3.2.4		**Strength in Bending**									
	Mx	M^*_x	≤	ϕM_{nx}							
		0.5	<	0.7						0.68	
	My	M^*_y	≤	ϕM_{ny}							max
		0.0	<	0.3						N/A	0.68
3.3.4		**Compressive Strength**									
	Ncx	N^*_c	≤	ϕN_{ncx}							
		0.3	<	29.6						0.01	
	Ncy	N^*_c	≤	ϕN_{ncy}							max
		0.3	<	59.1						0.01	0.01
3.4.2		**Tension Strength**									
	Nt	N^*_t	≤	ϕN_{nt}							max
		0.0	<	16.0						N/A	0.00
3.5.1		**Combined Bending and Compression**									
	My & Ncy	$\frac{M^*_y}{\phi M_{ny}}$	+	$\frac{N^*_c}{\phi N_{ncy}}$	≤	1.0					
		0.0	+	0.0	=	0.0	<	1.0		N/A	
	Mx & Ncx	$\frac{M^*_x}{\phi M_{nx}}$	+	$\frac{N^*_c}{\phi N_{ncx}}$	≤	1.0					
		0.7	+	0.0	=	0.7	<	1.0		0.69	
	Mx & Ncy	$\left(\frac{M^*_x}{\Phi M_{nx}}\right)^2$	+	$\frac{N^*_c}{\phi N_{ncy}}$	≤	1.0					
		0.5	+	0.0	=	0.5	<	1.0		0.46	
	My & Ncx	$\left(\frac{M^*_y}{\Phi M_{ny}}\right)^2$	+	$\frac{N^*_c}{\phi N_{ncx}}$	≤	1.0					max
		0.0	+	0.0	=	0.0	<	1.0		N/A	0.69
3.6		**Combined Bending and Tension**									
	Mx & Nt	$\frac{M^*_x}{\phi M_{nx}}$	+	$\frac{N^*_t}{\phi N_{nt}}$	≤	1.0					
		0.7	+	0.0	=	0.7	<	1.0		N/A	
	My & Nt	$\frac{M^*_y}{\phi M_{ny}}$	+	$\frac{N^*_t}{\phi N_{nt}}$	≤	1.0					max
		0.0	+	0.0	=	0.0	<	1.0		N/A	0.00

pryda

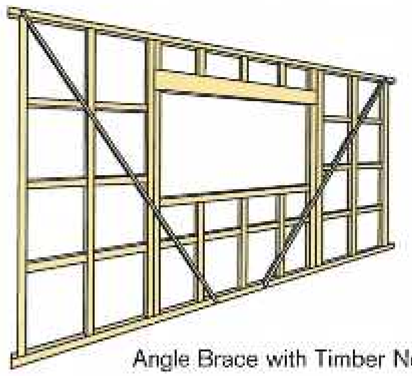
Angle Brace

X-1

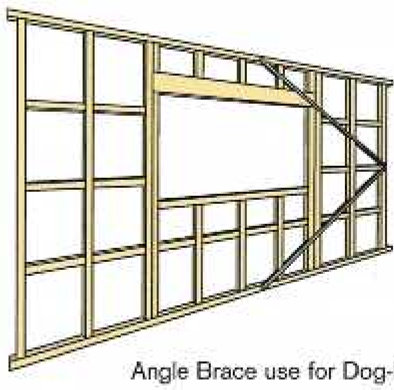
A fast, effective brace for timber frames



Applications



Angle Brace with Timber Nogging



Angle Brace use for Dog-Leg Bracing

Features

Pryda Angle Brace is the fast effective way to brace interior or exterior timber framing. It is fitted by making a single saw cut into the studs, inserting the brace, then nailing.

Because Pryda Angle Brace is power punched, it features clean, fully punched holes (no nails are bent or wasted by trying to force them through the brace).

Pryda Angle Brace utilises the tension and compression strength of steel with the properties of timber. It holds studs straighter, allows better air circulation and makes it easier to install wiring, plumbing and insulation.

"Checking in" Pryda Angle Brace flush with the surface of the timber can be done easily with the Pryda Angle Brace Checka fitted to an ordinary power saw. This attachment makes the saw cut and removes the timber to "check in" the brace in one operation.

Installation

After squaring up wall or temporary frame ready for bracing:-

1. Use the edge of the steel brace to draw a straight line where the brace is to go.
2. Cut the studs 20mm deep on this line with either a Pryda Angle Brace Checka, power saw or a hand saw.
3. Slide plain leaf of the angle into the sawcut. For safety reasons the punched leaf of the angle must point downwards. Nail punched leaf to the stud through the holes provided using 30 x 3.15 Pryda Product Nails, two per stud and minimum of three per end.
4. Brace is to be 150mm minimum from end of plate.

Specifications

Size:

20 X 20 X 1.00mm

Material:

G300 Z275 galvanised steel coil.

Product Code:

AB30 (3.0m long), AB33 (3.3m long), AB36 (3.6m long), AB42 (4.2m long), AB48 (4.2m long).

Packing:

Bundles of 10 lengths

Loads (Wind Only)

Steel: Characteristic Strength = 11.2 kN
Design Capacity (LSD) = 10.0 kN

Note: These steel tensile loads cannot be achieved through normal nailing in 1 leg of the angle.

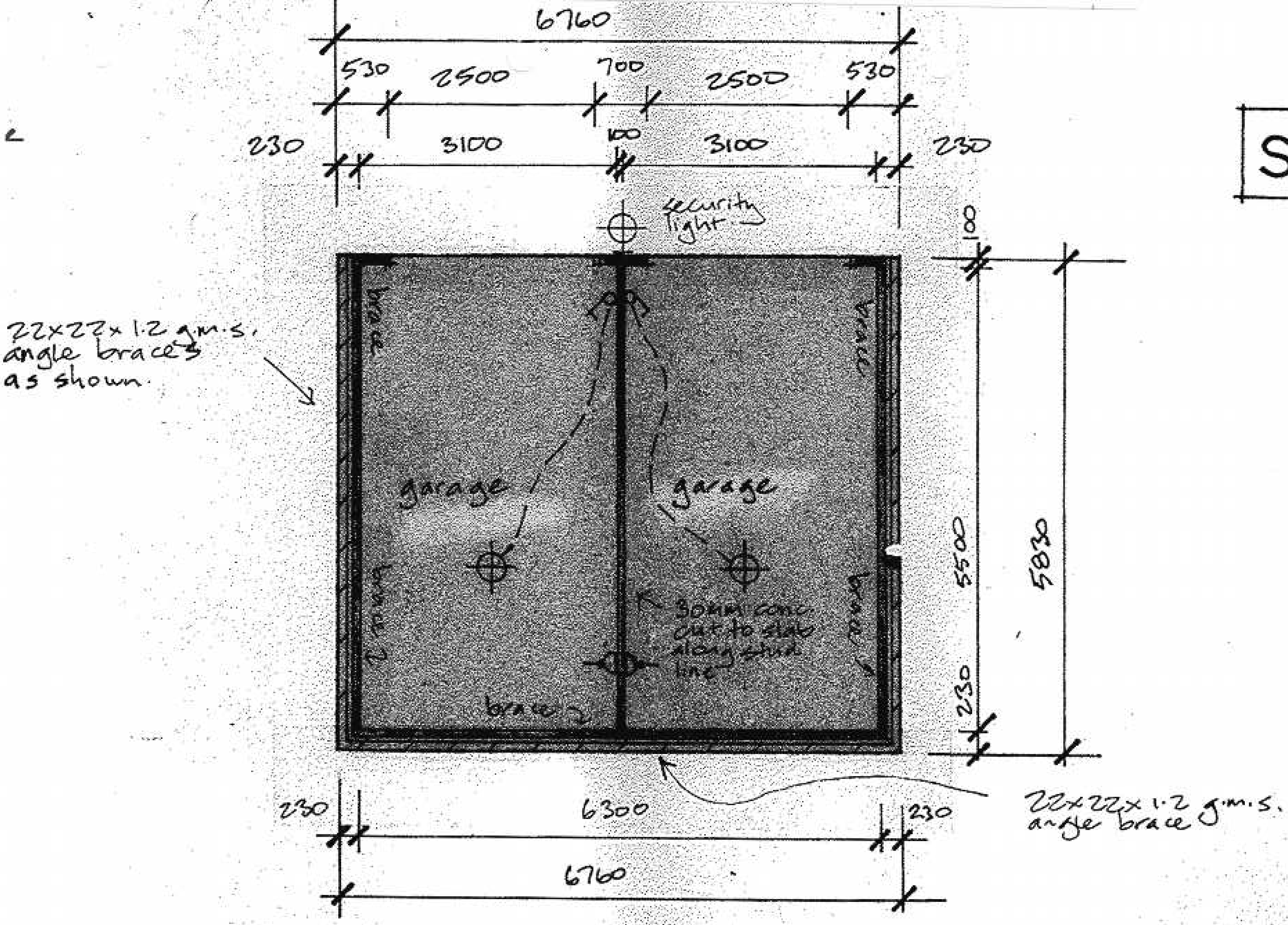
Tension: Nails in one leg only

Number of nails each end	Characteristic Strength	Design Load (LSD) Brief
3	4.7 kN	3.7 kN
4	6.2 kN	5.0 kN

Compression: Studs @ 600mm centres

	Brace at 45°	Brace at 55°
Clear Brace Length	780mm	980mm
Characteristic Buckling Load	4.6 kN	3.1 kN
Design Load	3.7 kN	2.5 kN
Bracing Units per brace	60 BU	40 BU

SECT

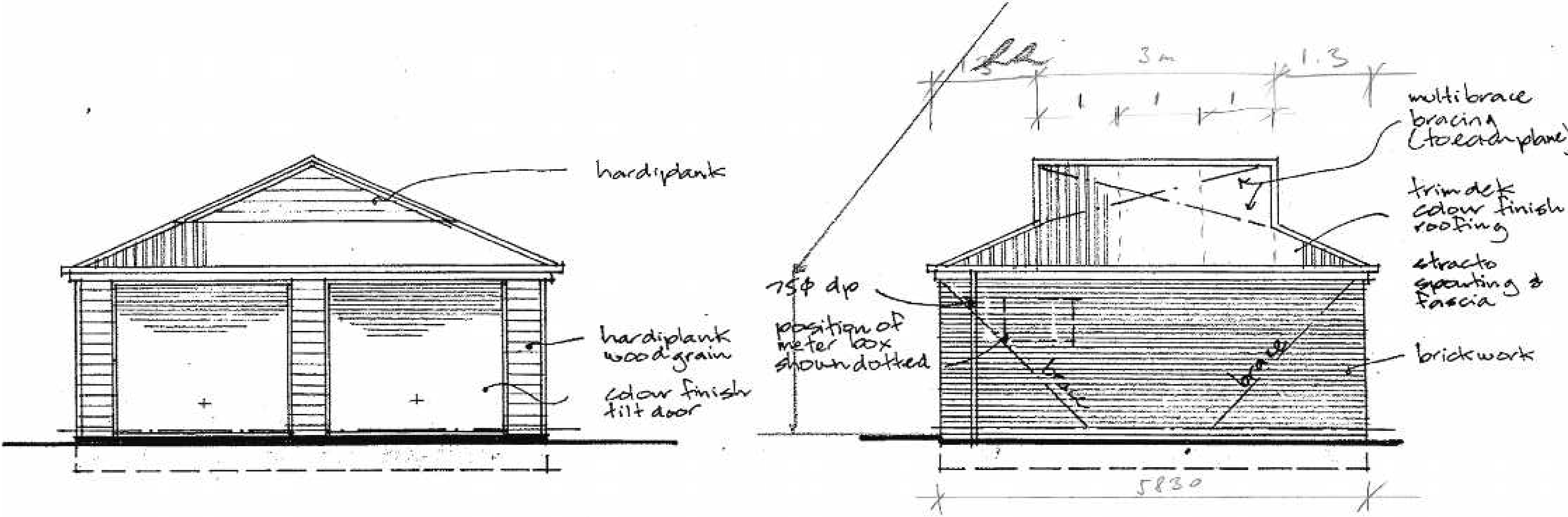


GARAGE PLAN 1:100

2 OFF total see location plan sht. A1.

BLOCK F GAR. EXTRACT OF A1 GARAGES

D-1



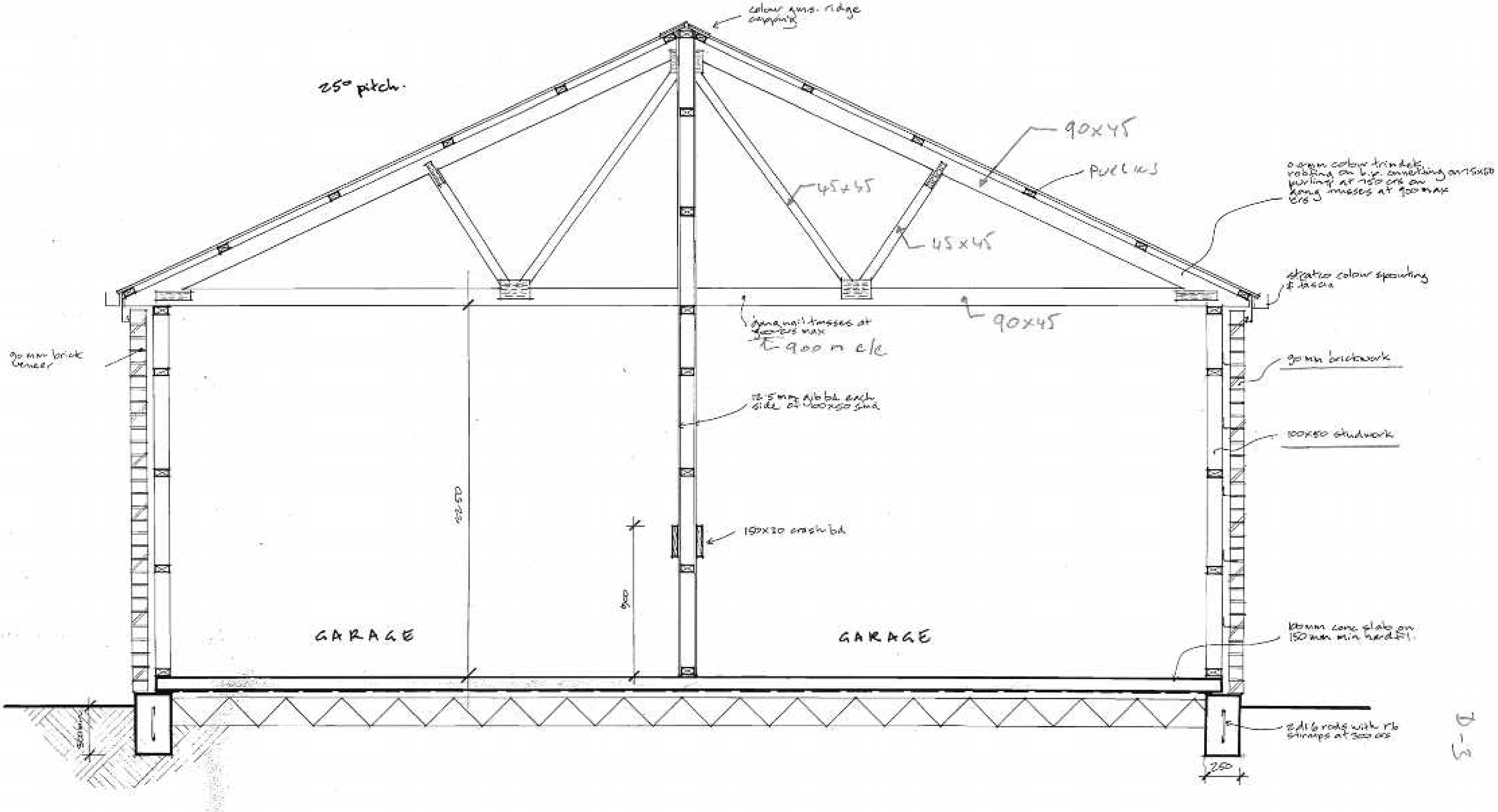
GARAGE ELEVATIONS

D-2

(1:73 @ A4)

BLOCK F GARAGE
EXTRACT OF A1

D-3

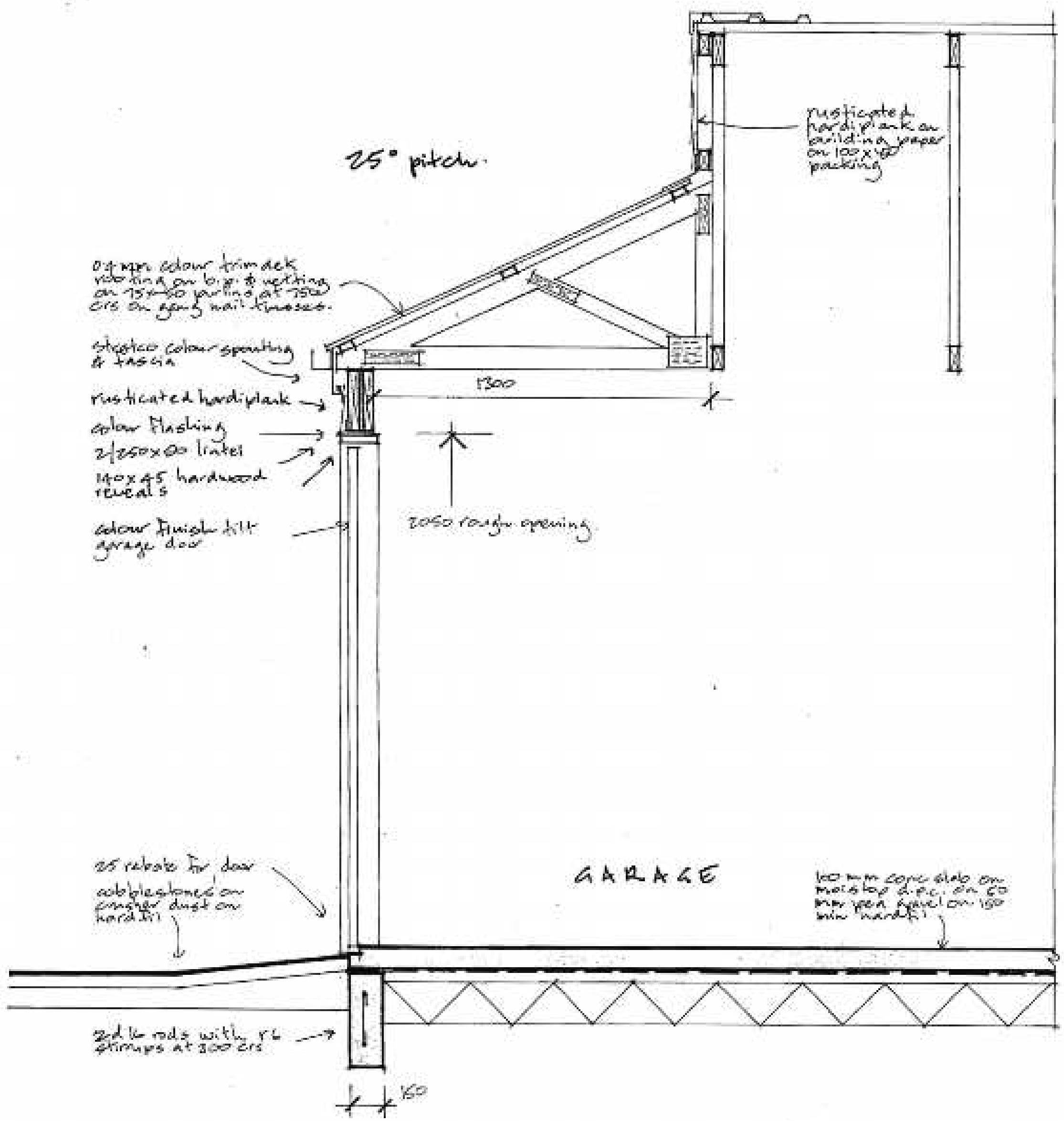


TYPICAL GARAGE CROSS SECTION 1:20

SCALE
1:30.6
@A4

BLOCK F GARAGE
EXTRACT OF A6

D-4



TYPICAL GARAGE DOOR SECTION 1:20

1:30 @ A4

BLOCK F GARAGE
EXTRACT OF A6