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# Haast Courts Block B
# BU 0792-002 EQ2

## Detailed Engineering Evaluation
## Quantitative Report

Version FINAL

Units 5 to 10, 43 Haast Street, Linwood

INFRASTRUCTURE | MINING & INDUSTRY | DEFENCE | PROPERTY & BUILDINGS | ENVIRONMENT

# Haast Courts Block B
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Detailed Engineering Evaluation
Quantitative Report
Version FINAL

Units 5 to 10, 43 Haast Street, Linwood

Christchurch City Council

**Prepared By**
Peter O'Brien

**Reviewed By**
David Lee

**Date**
**31 October 2012**

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# Quantitative Report Summary

**Haast Courts Block B**

**BU 0792-002 EQ2**

**Detailed Engineering Evaluation**

**Quantitative Report - SUMMARY**

**Version FINAL**

**Units 5 to 10, 43 Haast Street, Linwood**

### Background

This is a summary of the Quantitative report for the Haast Courts Block B social housing units, and is based in general on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, NZS 3101: 2006 Concrete Structures Standard and visual inspections on the 20th of July 2012, followed by intrusive investigations carried out throughout August and September 2012 by City Care. Block B consists of two 3 level similar buildings with a common external staircase.

### Brief Description

The main structural wall components are reinforced precast concrete wall panel construction supported by strip footing foundations. The roof consists of asbestos shingles adhered to a plywood substrate on timber battens which are supported by timber trusses. Exterior walls up to second floor roof level are of precast reinforced concrete panel construction with the gable walls of the buildings above second floor window level being of timber frame construction with a lightweight sheet panel cladding. Floors to the first and second floor are cast in-situ suspended reinforced concrete slabs. Foundations are strip footings under the concrete panel walls, which also support the edges of the ground floor.

### Key Damage Observed

Key damage observed includes:-

- Minor cracking to interior plasterboard wall linings throughout.
- Minor cracking to exterior of concrete wall panels around windows and openings.
- Cracking to suspended concrete floor slabs in locations where carpets were pulled back and is believed to be original shrinkage cracking that may have been widened by seismic activity.
- Cracking to asbestos ceiling linings.

### Indicative Building Strength (from Quantitative)

Based on the information available, the information obtained from a site measure and the information obtained from the intrusive investigations, analysis of the building's capacity with regards to the New Building Standards (NBS) has indicated that units 5 to 7 are in the order of 45% NBS and units 8 to 10 are in the order of 50% NBS.

As both buildings have been assessed to have a seismic capacity greater than 33% NBS and less than 67% NBS, they are therefore considered to be Potentially Earthquake Risk buildings.

### Recommendations

The recent seismic activity in Christchurch has caused minor damage to the buildings, with minor cracking in walls, wall linings and ceiling linings being the only structural damage noted. As the buildings have suffered minor damage that would not compromise the load resisting capacity of the existing structural systems, general occupancy of the buildings is permitted.

The buildings have however been assessed as being potentially Earthquake Risk buildings as they have achieved 45% and 50% NBS. As such, GHD Limited recommends that strengthening options be explored in order to increase the % NBS of the buildings to at least 67% NBS as recommended by the NZSEE Guidelines.

As the connections between the staircase and the supporting precast panel walls do not allow for adequate seismic lateral movement in the plane of the panel, it is recommended that secondary supports be provided to all landings in the form of equal angle supports fixed to the panel walls. The stairs are the only means of egress from the upper floors and this issue should be addressed immediately.

Due to the presence of cracking to the asbestos plaster ceiling lining in a number of the units throughout the complex, GHD Limited recommends that all the ceiling linings are to be inspected by asbestos specialists and dealt with according to their recommendations.

# 1. Background

GHD has been engaged by the Christchurch City Council (CCC) to undertake a detailed engineering evaluation of Haast Courts Block B social housing units.

This report is a Quantitative Assessment and is based in general on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, NZS 3101: 2006 Concrete Structures Standard and a visual inspection on the $20^{th}$ of July 2012, followed by intrusive investigations throughout August and September 2012.

The quantitative assessment to the buildings comprise of an investigation on the in-plane and out-of-plane strength of the precast concrete panel walls and the strength of the connections between the precast walls and the cast in situ suspended floors. The investigation is based on the analysis of the seismic loads that the structure is subjected to, the analysis of the distribution of these forces throughout the structure and the analysis of the capacity of existing structural elements to resist the forces applied. The capacities of the existing structural elements are compared to the demand placed on the elements to give the percentage of New Building Standard (%NBS) of each of the structural elements.

Electromagnetic scans have been carried out on site to ascertain the extent of the reinforcement throughout. Where electromagnetic scans have not revealed sufficient detail of a critical connection to be able to analyse it, intrusive investigation works were carried out.

Finite element modelling of the building structure has been carried out to ascertain the distribution of the lateral forces throughout the buildings.

# 2. Compliance

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

## 2.1 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

## 2.2 Building Act

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

**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).

**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.

### 2.2.1 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.

**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.

**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.

**Section 131 – Earthquake Prone Building Policy**

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

## 2.3 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;
- 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 33% 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.

## 2.4 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:

- Hazard Factor increased from 0.22 to 0.3 (36% increase in the basic seismic design load)
- 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)

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.

# 3. 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 new building standard load requirements have been determined in accordance with the current earthquake loading standard (NZS 1170.5:2004 Structural design actions - Earthquake actions - New Zealand).

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 1 below.

| 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 |

**Figure 1 NZSEE Risk Classifications Extracted from table 2.2 of the NZSEE 2006 AISPBE**

Table 1 compares the percentage NBS to the relative risk of the building failing in a seismic event with a 10% risk of exceedance in 50 years (i.e. 0.2% in the next year). It is noted that the current seismic risk in Christchurch results in a 6% risk of exceedance in the next year.

| 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 |

**Table 1 %NBS compared to relative risk of failure**

# 4. Building Description

## 4.1 General

Haast Courts Block B consists of two similar 3 level building units, located at 43 Haast Street, Linwood, Christchurch. The buildings were constructed in 1983 along with Blocks A, C and D on the site; all similar in building style. The use of each of the buildings is residential with one apartment on each floor of each building. No alterations have been made to the buildings since construction.

The main structural wall components are reinforced precast concrete wall panel construction supported by strip footing foundations. The roof consists of asbestos shingles adhered to plywood a substrate on timber battens which are supported by timber trusses. Exterior walls up to second floor roof level are of precast reinforced concrete panel construction with the gable walls of the buildings above second the floor window level being of timber frame construction with a lightweight sheet panel cladding. Vertical reinforcing in the concrete walls is provided by two layers of 12mm diameter steel bars at 225mm centres. Horizontal reinforcing is provided by two layers of 12mm diameter bars at 300mm centres top and bottom. Floors to the first and second floors are cast in-situ suspended reinforced concrete slabs supported by exterior concrete walls. Reinforcing is provided by 16mm diameter steel bars at 200mm centres in both directions. The suspended concrete floor slabs are tied back into the supporting precast wall panels by 12mm diameter steel starter bars coming out of the precast panels at 250mm centres and embeded approximately 500mm into the cast in situ floor slabs. Interior partition walls are timber framing with plasterboard lining.

Foundations are strip footings under the concrete panel walls. The ground floor consists of an on-grade concrete slab.

Block B consists of two separate blocks linked by a staircase. Each unit block is approximately 8m wide by 8m long, and stands 7.6m high with plan area of approximately 64$m^2$ each. The nearest waterway to the site is the Avon River, approximately 250m to the north. The site is flat.

Conceptual architectural drawings were made available for this building however no structural drawings were found. All structural details have been inferred from the electromagnetic wall scans and the opening up works that have been carried out.

## 4.2 Gravity Load Resisting System

Gravity loads in the structure are resisted by load bearing exterior concrete tilt panel external walls. Roof loads are transferred from horizontal battens to the timber roof trusses. Loads are then transferred to the external precast concrete panel walls which carry the loads to foundation level. Imposed loads on the first and second floors are transferred through the cast in situ concrete floor slab spanning to the concrete external walls which then carry the load down to the foundations. Ground floor loads are carried by the ground floor slab, which bears directly on the soil beneath.

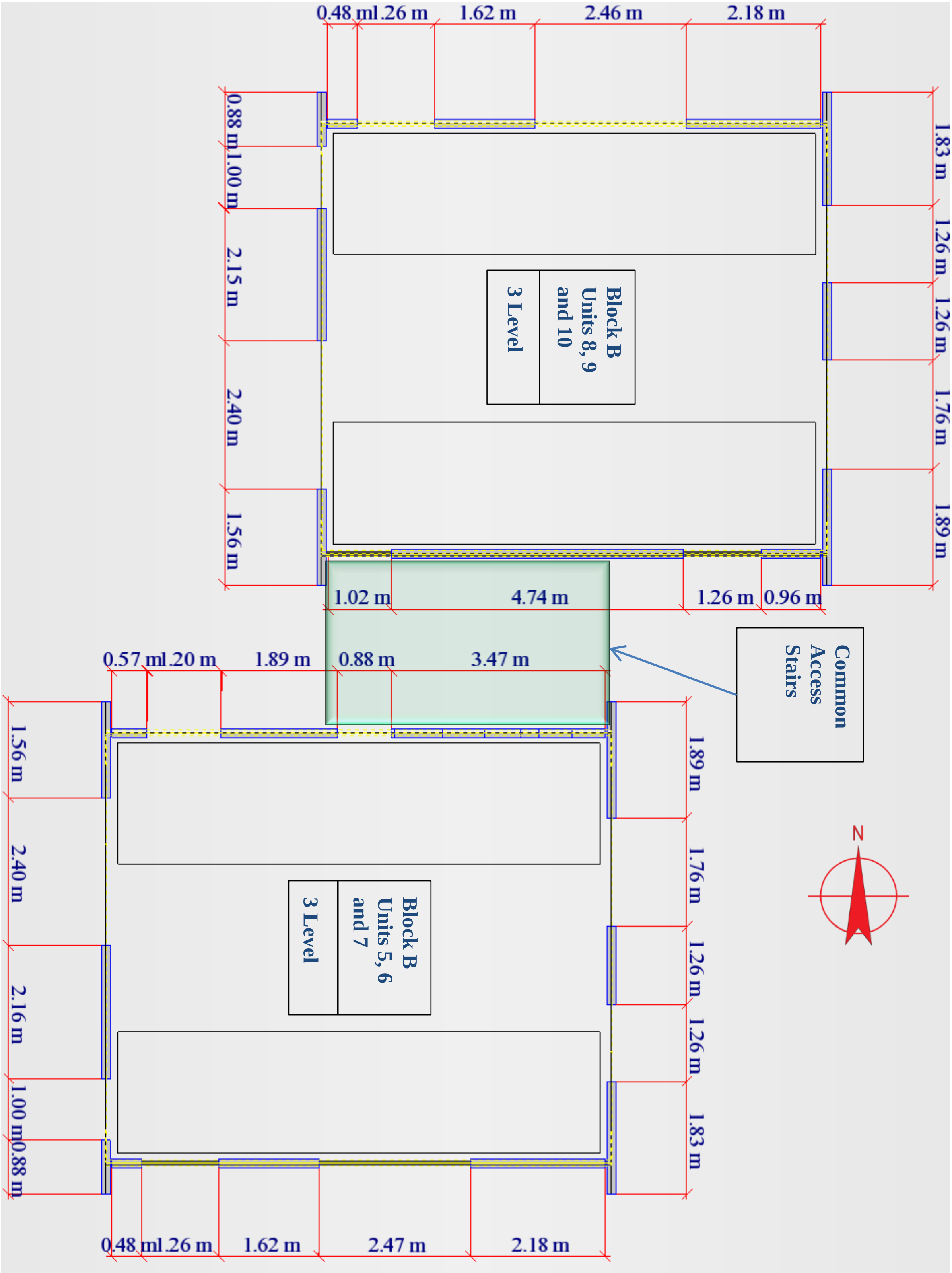


**Figure 2 Plan sketch showing key structural elements of each building unit**

## 4.3 Lateral Load Resisting System

This housing block is divided structurally into two separate three storey buildings consisting of units 5, 6 & 7 to one structure and units 8, 9 & 10 to the other. The two structures share precast concrete stairs which are connected to the walls by 20mm dowel bars with a seismic gap of approximately 30mm.

Lateral loads from the roof are transferred through diaphragm action of the plasterboard lined timber framed ceilings to the external concrete tilt panel walls. Each of the two structures relies on the in-plane shear capacity of the concrete panel walls to carry lateral seismic load through to the foundations. The suspended floors between the panels act as a brace for out of plane forces acting on the panels. Lateral loads generated by these floors are resisted by the panels through in- plane action.

# 5. Damage Assessment

## 5.1 Surrounding Buildings

Haast Courts Block B is located in a residential complex with 7 other residential blocks and 3 blocks of garages. Some of the older masonry residential units have suffered damage with the collapse of a portion of the gable end of Block G being the most noticeable. The only damage noted to the garage blocks is minor cracking to the floor slabs. Structural damage to these surrounding buildings has however no direct bearing on the performance of Block B units.

## 5.2 Residual Displacements and General Observations

No residual displacements of the structure were noticed during our inspection of the building.

Cracking was noted to the internal plasterboard linings in several locations throughout the building, primarily above window and door openings. The plasterboard lining is not considered as part of the lateral load resisting system and as such these cracks are not considered to be significant.

Minor cracking was evident in the asbestos plaster finish to the underside of the first floor slab. It was not clear if these cracks penetrate into the concrete slab.

Minor cracking was noted to the exterior of the building. These cracks may be original shrinkage cracks that may have opened up slightly during the recent seismic activity.

During the opening up works to determine the connection detail between the exterior walls and suspended slabs, diagonal cracking of the suspended slab at the corner of the building was noted upon removal of the carpet. These cracks are believed to be old shrinkage cracks that have opened up as a result of the seismic activity.

No damage was noted to the roof structure and all fixings are believed to be adequate.

## 5.3 Ground Damage

There was no evidence of ground damage on the property or surrounding neighbours land.

## 5.4 Floor Levels

No level or verticality surveys have been carried out as part of this assessment.

# 6. Geotechnical Consideration

## 6.1 Site Description

The site is located in the suburb of Linwood, in eastern Christchurch. It is relatively flat, with an elevation in the order of 5m above mean sea level. The site is approximately 250m south of the Avon River, and 6km west of the coast (Pegasus Bay).

## 6.2 Published Information on Ground Conditions

### 6.2.1 Published Geology

The geological map of the area[1] indicates that the site is underlain by:

- Holocene alluvial soils of the Yaldhurst Member, sub-group of the Springston Formation, comprising alluvial sand and silt overbank deposits.

Figure 72 (Brown & Weeber) indicates that groundwater levels are likely to be within 1m of the surface.

### 6.2.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that one borehole with a lithographic log (Ref. M35/2119) is located 150m north of the site. This indicates that the area is silt/clay to 1.8m bgl, overlying gravels to ~10m bgl, which is shown to be underlain by alternating layers of sand/clay, and gravels.

It should be noted that the boreholes were sunk for groundwater extraction and not for geotechnical purposes. Therefore, the amount of material recovered and available for interpretation and recording will have been variable at best and may not be representative. The logs have been written by the well driller and not a geotechnical professional or to a standard. In addition strength data is not recorded.

### 6.2.3 EQC Geotechnical Investigations

The Earthquake Commission has undertaken geotechnical testing in the area of the site. Information pertaining to this investigation is included in the Tonkin & Taylor Report for Linwood[2]. Three investigation points were considered, as summarised below in Table 2.

---

[1] Brown, L. J. & Weeber, J.H. (1992): *Geology of the Christchurch Urban Area*. Institute of Geological and Nuclear Sciences 1:25,000 Geological Map 1. IGNS Limited: Lower Hutt.

[2] Tonkin & Taylor Ltd (2011): Christchurch Earthquake Recovery, *Geotechnical Factual Report, Linwood*

**Table 2 EQC Geotechnical Investigation Summary Table**

| Bore Name | Grid Reference | Depth (m bgl) | Log Summary |
|---|---|---|---|
| CPT – LWD - 02 | 2481936.2 mE | 0 – 4.5 | Soft Silts and Clays |
| | 5742258.3 mN | 4.5 – 24.5 | Dense Sand |
| CPT – LWD - 03 | 2482276.3 mE | 0 – 2.0 | Loose Sands |
| | 5472317.3 mN | 2.0 – 2.5 | Soft Silt and Clay |
| | | 2.5 – 4.0 | Dense Sand |
| CPT – LWD - 17 | 2481825.2 mE | 0 – 5.0 | Silts and Clays |
| | 5472012.7 mN | 5.0 – 26.0 | Sand |

Initial observations of the CPT results indicate the soils are composed predominantly of soft silt and clay underlain by dense sands. This would infer that liquefaction is possible in a significant seismic event.

### 6.2.4 Land Zoning

Canterbury Earthquake Recovery Authority (CERA) indicates the site is within the Green Zone, meaning repair and rebuild may take place.

CERA has published areas showing the Green Zone Technical Category in relation to the risk of future liquefaction and how these areas are expected to perform in future earthquakes.

Categorised residential properties adjacent to the site are indicated to be TC2 (yellow). This means that minor to moderate land damage from liquefaction is expected in future significant earthquakes.

### 6.2.5 Post February Aerial Photography

Aerial photography taken following the 22 February 2011 earthquake shows no signs of liquefaction outside the building footprints or adjacent to the site, as shown in Figure 3.

**Figure 3 Post February 2011 Earthquake Aerial Photography**[3]

## 6.3 Seismicity

### 6.3.1 Nearby Faults

There are many faults in the Christchurch region, however only those considered most likely to have an adverse effect on the site are detailed below.

**Table 3 Summary of Known Active Faults**[4,5]

| Known Active Fault | Distance from Site | Direction from Site | Max Likely Magnitude | Avg Recurrence Interval |
|---|---|---|---|---|
| Alpine Fault | 120 km | NW | ~8.3 | ~300 years |
| Greendale (2010) Fault | 23 km | W | 7.1 | ~15,000 years |
| Hope Fault | 110 km | N | 7.2~7.5 | 120~200 years |
| Kelly Fault | 110km | NW | 7.2 | 150 years |
| Porters Pass Fault | 63 km | NW | 7.0 | 1100 years |

[3] Aerial Photography Supplied by Koordinates sourced from http://koordinates.com/layer/3185-christchurch-post-earthquake-aerial-photos-24-feb-2011/

[4] Stirling, M.W, McVerry, G.H, and Berryman K.R. (2002) A New Seismic Hazard Model for New Zealand, Bulletin of the Seismological Society of America, Vol. 92 No. 5, pp 1878-1903, June 2002.

[5] GNS Active Faults Database

Recent earthquakes since 22 February 2011 have identified the presence of a new active fault system /zone underneath Christchurch City and the Port Hills. Research and published information on this system is in development and not generally available. Average recurrence intervals are yet to be estimated.

#### 6.3.2 Ground Shaking Hazard

This seismic activity has produced earthquakes of Magnitude 6.3 with peak ground accelerations (PGA) up to twice the acceleration due to gravity (2g) in some parts of the city. This has resulted in widespread liquefaction throughout Christchurch.

New Zealand Standard NZS 1170.5:2004 quantifies the Seismic Hazard factor for Christchurch as 0.30, being in a moderate to high earthquake zone. This value has been provisionally upgraded recently (from 0.22) to reflect the seismicity hazard observed in the earthquakes since 4 September 2010.

### 6.4 Field Investigations

In order to further understand the ground conditions at the site, intrusive testing comprising two piezocone CPT investigations were conducted at the site on 28 June 2012. The locations of the tests are indicated on Figure 4 below.


**Figure 4 Aerial Photograph depicting CPT Investigation Locations [3]**

The coordinates of the test locations are tabulated in Table 4.

| Investigation | Depth (m bgl) | Easting (NZMG) | Northing (NZMG) |
|---|---|---|---|
| CPT 1 | 23.07 | 2482216 | 5742185 |
| CPT 2 | 27.89 | 2482259 | 5742157 |

**Table 4 Coordinates of Investigation Locations**

The CPT investigations were undertaken by McMillans Drilling Ltd on 28 June 2012, typically to a target depth of 20m below ground level. However, testing was continued to depths of 23m bgl and 27.9m bgl due to the presence of soft silts and loose sands at 20m. Please refer to Appendix D for CPT logs.

### 6.4.1 Ground Conditions Encountered

Interpretation of output graphs[6] from the investigation showing Cone Tip Resistance ($q_c$), Friction Ratio (Fr), Inferred Lithology and Inferred Liquefaction Potential are summarised in Table 5 and Table 6.

A summary of the lithology inferred from the CPT results is outlined in Table 5 below.

| **Depth (m)** | **Inferred Lithology** | **Cone Tip Resistance** $q_c$ **(MPa)** | **Friction Ratio Fr (%)** | **Relative Density Dr (%)** |
|---|---|---|---|---|
| 0 – 6.5 | SILT mixtures (with sand lenses) | 1 to 8 | 1 to 6 | (Su ≥ 30kPa) |
| 6.5 – 10 | SANDS | 14 to 25 | 0.5 | 80 to 100 |
| 10 – 16 | SANDS | 2 to 18 | 0.5 to 2 | 50 to 80 |
| 16 – 19 | SANDS | 12 to 30 | 0.5 | 70 to 90 |
| 19 – 27 | Layers of: | | | |
| | • SILT mixtures; and, | 1 | ~3 | (Su ≥ 50kPa) |
| | • SANDS | 15 to 30 | 0.5 | 60 to 80 |

**Table 5 Summary of CPT-Inferred Lithology**

From the results above, the ground conditions at the site are understood to be predominantly silts to 6.5m, overlying sands to 19m, and layers of sands and silts to depth.

This is considered consistent with the published geology and EQC investigations for the area, from the desktop information reviewed in Sections 7.1.1 and 7.1.3.

Please refer to Appendix D for further detail.

During the CPT investigations, groundwater was inferred to be at 1.2m below ground level. This is slightly lower than, but still consistent with, the inference by Brown & Weeber of groundwater being within 1m of the surface. It is also consistent with site levels in relation to the Avon River.

### 6.4.2 Liquefaction Analysis

As the subsoils encountered consisted of sand and silt beneath the site, a more comprehensive liquefaction assessment has been undertaken.

#### 6.4.2.1 Parameters used in Analysis

Assumptions made for the analysis process are as follows:

- $D_{50}$ particle sizes for the site soil (sands) from CPT soil analysis;

[6] McMillans Drilling CPT data plots, Appendix D.

- Importance Level 2, post seismic event (50-year design life); and,
- Peak ground acceleration (PGA) 0.35g.

The following equation has been used to approximate soil unit weight from the CPT investigation data: [7]

$$\gamma = \frac{\gamma_w Gs}{2.65}\left(0.27\log Fr + 0.36\log\left(\frac{q_C}{p_{atm}}\right) + 1.236\right)$$

This typically gave values ranging between 16 and 20 kN/m$^3$ (saturated).

The liquefaction analysis process has been conducted using the methodology from Robertson & Wride[8], and from the NZGS Guidelines[9].

#### 6.4.2.2 Results of Liquefaction Analysis

The results of the liquefaction analysis, as outlined in Table 6, indicate that depths to 6.5m, and 10m to 19m, are considered highly liquefiable.

| Depth (m) | Inferred Lithology | Triggering Factor $F_L$ | Liquefaction Susceptibility[10] |
|---|---|---|---|
| 0 – 6.5 | SILT mixtures (with sand lenses) | 0.3 to 0.8 | High (Bands) |
| 6.5 – 10 | SANDS | >> 1 | Negligible |
| 10 – 16 | SANDS | 0.4 to 2 | Severe |
| 16 – 19 | SANDS | 0.3 to 1 | High (Bands) |
| 19 – 27 | Layers of:<br>• SILT mixtures; and,<br>• SANDS. | <br>-<br>0.5 to 1.8 | <br>*Not Liquefiable*<br>High |

**Table 6 Summary of Liquefaction Susceptibility**

(Bands) means that only some bands of soil are indicated to be susceptible within this layer.

While layers at 19m to 27m are indicated to be highly susceptible by the analysis, the severity of liquefaction at this depth is considered significantly reduced due to the greater levels of vertical overburden stress.

Settlement estimates for the CPT points are between 150mm and 270mm for ULS conditions.

Please refer to Appendix D for further details.

[7] Robertson P.K., & Cabal K.L. (2010): *Estimating soil unit weight from CPT*. Gregg Drilling & Testing Inc.: Signal Hill, California, USA.

[8] Robertson P.K. & Wride C.E. (1998): *Evaluating cyclic liquefaction potential using the cone penetration test.* Canadian Geotechnical Journal, 35: pp. 442–459.

[9] Cubrinovski M., McManus K.J., Pender M.J., McVerry G., Sinclair T., Matuschka T., Simpson K., Clayton P., Jury R. (2010): Geotechnical earthquake engineering practice: *Module 1 – Guideline for the identification, assessment and mitigation of liquefaction hazards*. NZ Geotechnical Society

[10] Table 6.1, NZGS Guidelines Module 1 (2010)

### 6.4.3 Interpretation of Ground Conditions

#### 6.4.3.1 Liquefaction Assessment

Overall, the site is considered to be highly susceptible to liquefaction. This is based on:

- Limited evidence of liquefaction at the surface in the post-earthquake aerial photography;
- Estimated settlements from the CPT results (150mm to 270mm) are well in excess of the 100mm limit for TC2 classification, indicating the site should be considered in line with TC3 guidelines; and,
- The layers of 1m to 6m and 9m to 17m are indicated to be highly susceptible, as outlined in Table 6.

#### 6.4.3.2 Slope Failure and/or Rockfall Potential

The site is located within Linwood, a flat suburb in eastern Christchurch. Global slope instability is considered negligible. However, any localised retaining structures and/or embankments should be further investigated to determine the site-specific slope instability potential.

#### 6.4.3.3 Foundation Recommendations

Based on the information presented above, we recommend the following for the subject site:

- The soil class of **D** (in accordance with NZS 1170.5:2004) recommended in Section 8 of the Qualitative DEE/IEP is still believed to be appropriate; and,
- Any remedial works to foundations (or proposed new structures) be undertaken in accordance with DBH's guidelines for **TC3** land, due to the high levels of estimated settlement. Due to the multi-storey nature of the structures in questions, specifically-designed foundations may be required.

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# 7. Assessment Methodology

Visual inspections of the buildings were undertaken on the 20th of July 2012, with further intrusive investigations of the building carried out throughout August and September 2012. Both the interior and exterior of the buildings were inspected. The foundations were not inspected.

The inspection consisted of a review of the available drawings and observing the building to determine the structural systems and likely behaviour of the buildings during an earthquake. The site was assessed for damage, including examination of the ground conditions, checking for damage in areas where damage would be expected for the type of structure and noting general damage observed throughout the building in both structural and non-structural elements.

## 7.1 Quantitative Assessment

A 3D structure analysis using Robot Structure Analysis Professional engineering software was undertaken to model the buildings structure for 100% NBS loads. Additional design requirements for earthquake effects were taken into account when modelling the buildings. To obtain realistic predictions for the internal actions in the statically indeterminate structure, and to estimate the periods of vibration and particularly lateral deflections, allowance was made for the effects of cracking on member stiffness. This is in accordance with C6.9.1 of the commentary to NZS 3101: 2006. The loads from the analysis were then checked against the structural member capacities derived from NZS 3101: 2006 Concrete Structures Standard.

## 7.2 Seismic Coefficient

The elastic site hazard spectrum for horizontal loading, C(T), for the building was derived from Equation 3.1(1) of NZS 1170.5: 2004;

$$\mathrm{C(T) = C_h\ (T) Z\ R\ N(T.D)}$$

Where

$C_h(T)$ = the spectral shape factor determined from CL 3.1.2

Z = the hazard factor from CL 3.1.4 and the subsequent amendments which increased the hazard factor to 0.3 for Christchurch

R = 1.0, the return period factor from Table 3.5 for an annual probability of exceedance of 1/500 for an Importance Level 2 building

N(T,D) = the near-fault scaling facto from CL 3.1.6

The structural performance factor, $S_P$, was calculated in accordance with CL 4.4.2

$$S_P = 1.3 - 0.3\mu$$

Where µ is the structural ductility factor. A structural ductility factor of 1.25 has been taken for lateral loading in both directions of the building; this is due to the use of reinforced precast panels to provide lateral load resistance. These walls lack the robustness to resist significant damage while retaining lateral load stability.

The seismic weight coefficient was then calculated in accordance with Cl 5.2.1.1 of NZS 1170.5: 2011. For the purposes of calculating the seismic weight coefficient a period, $T_1$, of less than 0.4s was taken for the structure. The coefficient was then calculated using Equation 5.2(1);

$$C_d(T_1) = \frac{C(T_1)S_P}{k_\mu}$$

Where

$$k_\mu = \frac{(\mu - 1)T_1}{0.7} + 1$$

## 7.3 Shear Capacity of the Reinforced Concrete Walls

The shear capacity of the concrete shear walls was calculated using Sections 10.3.10.3 (columns) and 11.3.10 (walls) of NZS 3101:2006.

Shear capacity comprises two components; that from the concrete, and that from the steel reinforcing. These are calculated separately, and added together.

This first involved calculating the shear capacity of the concrete, $V_C$, based on the following equations:

For concrete columns (equations 10-11 and 10-12);

$$v_b = (0.07 + 10\rho_w)\sqrt{f_c'}$$

$$V_C = k_a k_n v_b A_{CV}$$

And for concrete walls (equation 11-12);

here

- $k_a$ = Shape factor = 1.0 for fixed-pinned;
- $k_n$ = Compression force factor (taken as 1.0);
- $f'_C$ = characteristic compressive strength of the concrete; and,
- $A_{CV}$ = cross-sectional area resisting shear, being the narrowest width of the element multiplied by its length.

The shear capacity component from the reinforcing steel, $V_S$, was calculated using equation 11-18;

$$V_S = A_V\, f_{yt}\frac{d}{s}$$

Where

- $A_V$ = area of transverse (horizontal) reinforcing at spacing s;
- $f_{yt}$ = characteristic yield strength of the transverse steel;
- d = depth from compression end of wall to centre of reinforcing, approximated as 0.8 times the wall length.

These two components were then added together, and factored by the strength reduction factor, Ø, which for shear capacity is taken as 0.75 (Clause 2.3.2.2, NZS 3101:2006), as follows:

$$\emptyset \mathrm{V_n} = \emptyset(\mathrm{V_S} + \mathrm{V_C})$$

## 7.4 Moment Capacity of the Reinforced Concrete Walls

The moment capacity of the shear walls has been calculated using Section 7.4 of NZS 3101:2006, in conjunction with first principles for strain compatibility and force equilibrium (Clause 11.3.9). The maximum allowable concrete compressive strain is 0.003 (Clause 7.4.2.3).

It was assumed that all steel within the wall yields at the wall's ultimate capacity.

The forces involved in the shear wall can be summarised by equating the net internal compressive force to the net external compressive force:

$$C_C + C_S - T_S = N^*$$

Where

$C_C$ = strength of concrete in compression;

$C_S$ = strength of steel reinforcement in compression;

$T_S$ = strength of steel reinforcement in tension; and,

$N^*$ = axial load acting on the top of the shear wall.

The reinforcing steel is considered to experience forces calculated by the cross-sectional area of steel bars multiplied by the characteristic yield strength, in line with the "all steel yields" assumption.

The capacity of concrete in compression can be calculated using the following equation:

$$C_C = \alpha_1 \beta_1 f_C' \, t \, c$$

Where

$\alpha_1$ = concrete stress block adjustment factor (Clause 7.4.2.7(c));

$\beta_1$ = neutral axis depth factor (Clause 7.4.2.7(d));

t = thickness of the wall; and,

c = depth from compression edge to neutral axis of the wall.

Equating the two expressions gives the equation:

$$\alpha_1 \beta_1 f_C' \, t \, c - \sum_{\mathrm{T-C}} A_S f_y = N^*$$

Where

$\Sigma A_S f_y$ = sum of tension steel strength *less* sum of compression steel strength

This equation involves an iterative process, where the number of bars in each tension and compression is initially estimated, and the force equilibrium solved for the neutral axis depth, 'c'. Once 'c' is between the last compression steel and first tension steel, then the equation is solved.

Internal moments for the concrete and steel force components are then taken about the neutral axis to calculate the moment capacity of the wall, as follows:

$$\emptyset \mathrm{M_n} = \emptyset \left[ C_C \left(1 - \frac{\alpha_1}{2}\right) c + \sum A_S f_y (l - c) + N^* \left(\frac{l}{2} - c\right)\right]$$

Where

$l$ = depth of each steel reinforcing element from the compression edge

$\emptyset$ = strength reduction factor = 0.85 for flexure (NZS 3101:2006)

## 7.5 Calculation of %NBS

The shear and moment capacity of the panel walls in both the along and across directions are then compared to their respective demands to asses which are the most critical and thus determine the overall %NBS for the building.

$$\%\mathrm{NBS} = \frac{\emptyset \mathrm{V_n}}{\mathrm{V^*}} \times 100\%$$

$$\%\mathrm{NBS} = \frac{\emptyset \mathrm{M_n}}{\mathrm{M^*}} \times 100\%$$

## 7.6 Staircase Assessment

Visual assessment of the staircase connections and support system has been undertaken.

## 7.7 Timber Framed Roof Assessment

The timber framed roof members were not included in the model but the weight was accounted for. The roof is lightweight and fixings have been assumed to be adequate based on visual inspections. Connections between the roof trusses and the precast concrete panel walls appear to be adequate based on visual inspections (See photographs 11 and 12).

# 8. Critical Structural Weaknesses

## 8.1 Staircase

The dowel connection between the common landings and the building panel walls allows for differential movement of the staircase and wall panel in one direction only (out-of-plane of the wall panel). At the doors of apartments, the seismic gap between the landings and walls is filled but this seal does not appear to be flexible which would cause the transfer of loads between the two blocks of the building. Should differential seismic movement in the plane of the supporting panels occur, there is the potential for shear failure of the steel dowels which could cause failure of the staircase. This has the potential to cause serious injury to residents who might be using the stairs during a severe seismic event and may also prevent egress following the event.

## 8.2 Liquefaction Potential

Although liquefaction has not occurred as a result of previous seismic events, the geotechnical conditions on site indicate that the site is considered to be highly susceptible to liquefaction. The effects of soil liquefaction has been assessed as minor in the analysis of the seismic capacity of the buildings as the foundations appear to have adequate strength to be able to spread the building loads around the perimeter of the buildings.

# 9. Seismic Capacity Assessment

## 9.1 Seismic Loading Investigation

A 3D structure analysis using Robot Structure Analysis Professional engineering software was undertaken to model the building structure for 100% NBS loads. Loads were applied in both the along and across directions of the building. The loads from the analysis were then checked against the structural member capacities derived from NZS3101: 2006 Concrete Structures.

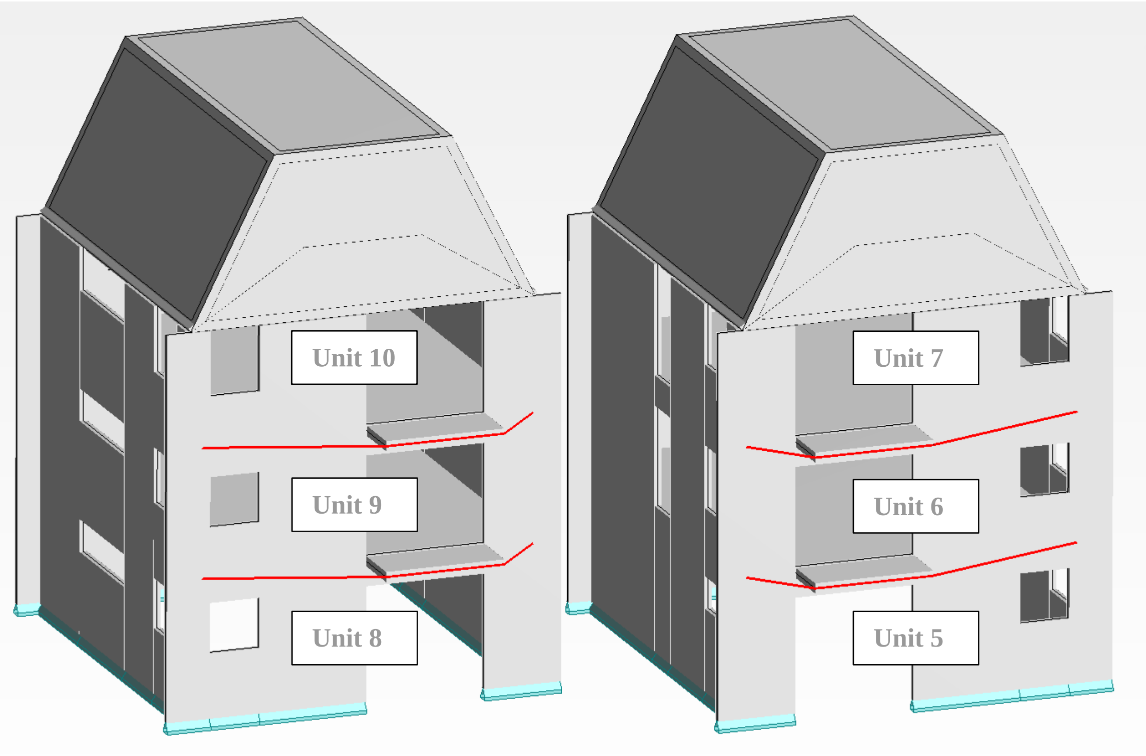



**Figure 5 3D structural model of Haast Courts Block B apartments**

## 9.2 Building Analysis and Results

For the purposes of analysing the demand, each block was considered to be separate. Each precast wall was divided into a combination of smaller panels (See Figure 6). Forces acting on the edges and centrelines of each of the smaller panels were extracted for comparison with their individual capacities. The balconies were assessed as being part of the internal floor slabs cantilevering from the face of the wall. As each floor is tied into all four exterior walls they effectively act as rigid diaphragms, spreading loads evenly between the walls.

The capacity of each wall element is taken as the capacity of the weakest part of the panel making up that wall element. Panels spanning vertically were analysed as columns for shear and both in-plane and out-of-plane flexure (Table 7 and Table 8) and spandrel beam panels spanning horizontally between columns were analysed for shear capacity and in-plane flexure (Table 9 and Table 10). A map of panel locations for each wall is included in Appendix F.

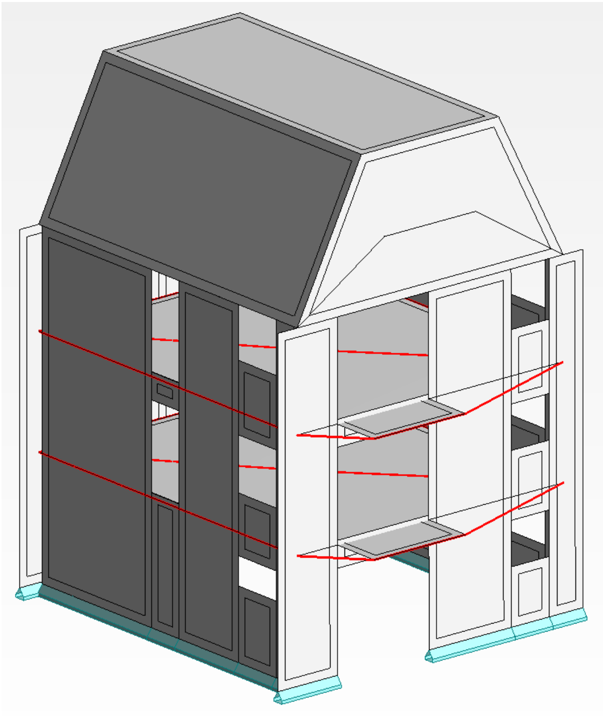


**Figure 6 Walls of structure reduced to smaller panels for analysis**

| Robot Panel No. | Robot Design Forces | | | Out of Plane Bending | | Out of Plane Shear | | In Plane Bending | | In Plane Shear | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | N* | V* In Plane | M* Out of Plane | Moment Capacity | % NBS | ΦVn | % NBS | ΦMn | %NBS | ΦVn | % NBS |
| | kN | kN | kN | kNm | | kN | % | kN | % | kN | % |
| 1 | 17.71 | 109.55 | 1.34 | 31.18 | 2327 | 117.53 | 1920 | 239.70 | 143 | 473.79 | 432 |
| 400 | 53.03 | 142.1 | 3.28 | 24.11 | 735 | 76.50 | 609 | 132.43 | 416 | 330.97 | 233 |
| 402 | 208.92 | 42.13 | 3.28 | 28.50 | 869 | 67.32 | 632 | 143.14 | 596 | 270.90 | 643 |
| 415 | 41.64 | 76.12 | 1.89 | 20.20 | 1069 | 63.07 | 1267 | 65.96 | 463 | 266.65 | 350 |
| 418 | 35.89 | 74.17 | 1.08 | 27.34 | 2532 | 91.80 | 1645 | 178.93 | 1031 | 397.16 | 535 |
| 420 | 52.16 | 18.41 | 0.81 | 15.46 | 1909 | 43.61 | 2850 | 48.11 | 771 | 196.29 | 1066 |
| 423 | 85.6 | 194.55 | 7.32 | 74.48 | 1017 | 256.72 | 718 | 1146.40 | 515 | 1071.02 | 551 |
| 426 | 90.66 | 125.05 | 2.9 | 45.01 | 1552 | 135.34 | 1052 | 325.81 | 1253 | 593.38 | 475 |
| 427 | 77.41 | 126.74 | 1.5 | 39.61 | 2641 | 116.03 | 764 | 248.29 | 507 | 523.18 | 413 |
| 428 | 60.81 | 119.84 | 1.31 | 28.97 | 2212 | 96.39 | 2340 | 199.75 | 522 | 401.75 | 335 |
| 430 | 60.87 | 20.57 | 0.11 | 15.39 | 13989 | 36.72 | 1316 | 40.89 | 433 | 189.40 | 921 |
| 438 | 129.93 | 148.5 | 3.37 | 50.19 | 1489 | 166.77 | 683 | 592.37 | 335 | 675.71 | 455 |
| 439 | 56.52 | 237.61 | 8.33 | 51.19 | 615 | 182.42 | 791 | 564.83 | 468 | 742.26 | 312 |
| 442 | 25.8 | 207.87 | 2.06 | 36.00 | 1748 | 132.01 | 2264 | 304.90 | 323 | 539.16 | 259 |
| 444 | 55.69 | 110.46 | 1.13 | 29.64 | 2623 | 90.71 | 732 | 146.20 | 346 | 396.07 | 359 |
| 447 | 20.23 | 164.04 | 3.4 | 26.93 | 792 | 94.33 | 817 | 163.29 | 276 | 399.69 | 244 |
| 449 | 237.88 | 131.37 | 4.76 | 51.33 | 1078 | 140.00 | 537 | 517.40 | 332 | 598.04 | 455 |
| 453 | 274.68 | 126.92 | 0.77 | 53.59 | 6960 | 144.59 | 1721 | 558.79 | 318 | 602.63 | 475 |

**Table 7 %NBS of panels spanning vertically of Block B Units 5, 6 and 7**

| Robot Panel No. | Robot Design Forces | | | Out of Plane Bending | | Out of Plane Shear | | In Plane Bending | | In Plane Shear | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | N* | V* In Plane | M* Out of Plane | Moment Capacity | % NBS | ΦVn | % NBS | ΦMn | %NBS | ΦVn | % NBS |
| | kN | kN | kN | kNm | | kN | % | kNm | % | kN | % |
| 690 | 93.64 | 114.69 | 5.93 | 48.09 | 811 | 166.77 | 1543 | 565.34 | 401 | 636.09 | 555 |
| 692 | 236.91 | 114.91 | 4.54 | 51.57 | 1136 | 144.59 | 1325 | 535.84 | 368 | 551.47 | 480 |
| 696 | 182.66 | 216.51 | 1.15 | 53.12 | 4619 | 165.24 | 788 | 624.92 | 290 | 630.25 | 291 |
| 697 | 44.54 | 130.6 | 1.52 | 23.62 | 1554 | 76.50 | 2898 | 129.37 | 481 | 291.78 | 223 |
| 698 | 186.82 | 38.22 | 0.88 | 27.40 | 3113 | 67.32 | 1352 | 137.02 | 570 | 256.77 | 672 |
| 699 | 70.91 | 100.92 | 5.77 | 39.11 | 678 | 116.70 | 1320 | 252.28 | 688 | 464.49 | 460 |
| 700 | 46.7 | 95.74 | 1.64 | 28.16 | 1717 | 96.39 | 2152 | 193.46 | 673 | 367.65 | 384 |
| 701 | 50.94 | 14.68 | 1.33 | 14.90 | 1120 | 36.72 | 1716 | 39.27 | 583 | 140.06 | 954 |
| 704 | 29.09 | 188.03 | 3.14 | 36.25 | 1155 | 131.67 | 1721 | 302.86 | 351 | 510.18 | 271 |
| 705 | 58.4 | 100.22 | 4.72 | 29.85 | 632 | 90.43 | 1396 | 144.93 | 330 | 360.90 | 360 |
| 708 | 24.09 | 149.19 | 0.96 | 27.22 | 2836 | 93.93 | 3239 | 161.25 | 321 | 364.86 | 245 |
| 718 | 198.44 | 119.36 | 3.43 | 49.22 | 1435 | 140.00 | 3617 | 494.02 | 329 | 533.96 | 447 |
| 721 | 61.54 | 186.25 | 4.65 | 51.57 | 1109 | 181.91 | 1734 | 562.62 | 596 | 710.67 | 382 |
| 772 | 229.78 | 603.41 | 3.76 | 110.96 | 2951 | 339.94 | 705 | 1949.39 | 321 | 1359.41 | 225 |
| 773 | 130.52 | 59.87 | 1.52 | 28.13 | 1851 | 73.44 | 1268 | 152.32 | 634 | 280.11 | 468 |
| 775 | 260.74 | 95.71 | 3.35 | 35.08 | 1047 | 78.80 | 1779 | 205.79 | 583 | 300.54 | 314 |
| 776 | 54.38 | 140.5 | 3.21 | 28.60 | 891 | 96.39 | 1550 | 196.86 | 429 | 367.65 | 262 |

**Table 8 %NBS of panels spanning vertically of Block B Units 8, 9 and 10**

| Robot Panel No. | N* | M* | V* | ΦVn | % NBS | ΦMn | %NBS |
|---|---|---|---|---|---|---|---|
| | kN | kNm | kN | kN | % | kNm | % |
| 406 | 17.76 | 31.3 | 73.73 | 495.84 | 673 | 184 | 587 |
| 407 | 122.32 | 129.13 | 70.24 | 174.13 | 248 | 58 | 45 |
| 409 | 128.43 | 119.42 | 74.27 | 174.13 | 234 | 58 | 49 |
| 421 | 13.13 | 35.43 | 63.92 | 416.52 | 652 | 129 | 365 |
| 424 | 9.9 | 23.16 | 36.83 | 495.84 | 1346 | 184 | 793 |
| 431 | 19.31 | 80.12 | 104.76 | 419.44 | 400 | 144 | 180 |
| 432 | 19.05 | 41.22 | 48.15 | 419.44 | 871 | 144 | 350 |
| 440 | 60.34 | 179.45 | 190.29 | 576.45 | 303 | 271 | 151 |
| 441 | 12.5 | 83.92 | 99.74 | 576.45 | 578 | 271 | 322 |
| 445 | 32.34 | 162.17 | 218.91 | 498.61 | 228 | 203 | 125 |
| 446 | 39.71 | 83.4 | 109.23 | 498.61 | 456 | 203 | 244 |
| 450 | 21.64 | 116.67 | 202.64 | 420.41 | 207 | 144 | 124 |
| 457 | 33.57 | 10.44 | 51.71 | 174.13 | 337 | 58 | 557 |

**Table 9 % NBS of spandrel panels of Block B Units 5, 6 and 7**

| Robot Panel No. | N* | M* | V* | ΦVn | % NBS | ΦMn | %NBS |
|---|---|---|---|---|---|---|---|
| | kN | kNm | kN | kN | % | kNm | % |
| 703 | 18.54 | 26.39 | 28.17 | 419.44 | 1489 | 144 | 547 |
| 706 | 27.8 | 148.49 | 192.98 | 498.61 | 258 | 203 | 137 |
| 707 | 35.94 | 78.83 | 97.51 | 498.61 | 511 | 203 | 258 |
| 715 | 119.09 | 115.85 | 60.35 | 174.13 | 289 | 58 | 50 |
| 716 | 23.65 | 65.39 | 149.57 | 419.44 | 280 | 144 | 221 |
| 717 | 115.61 | 114.61 | 56.88 | 174.13 | 306 | 58 | 51 |
| 719 | 20.64 | 108.05 | 175.63 | 420.41 | 239 | 149 | 138 |
| 720 | 38.57 | 58.91 | 85.6 | 420.41 | 491 | 149 | 254 |
| 722 | 41.35 | 138.91 | 152.46 | 576.45 | 378 | 149 | 108 |
| 723 | 12.38 | 61.45 | 78.06 | 576.45 | 738 | 270 | 440 |
| 724 | 16.25 | 30.89 | 75.84 | 495.84 | 654 | 184 | 595 |
| 774 | 61.02 | 8.59 | 38.86 | 174.13 | 448 | 58 | 676 |
| 778 | 14.29 | 55.06 | 94.27 | 419.44 | 445 | 144 | 262 |

**Table 10 %NBS of spandrel panels of Block B Units 8, 9 and 10**

Following the detailed assessment, units 5 to 7 have been assessed as achieving 45% NBS (New Building Standard). Units 8 to 10 have been assessed as achieving 50% NBS. Under the New Zealand Society for Earthquake Engineering (NZSEE) guidelines the building is considered to be an Earthquake Risk building as it achieves greater than 33% NBS but less than 67% NBS.

## 9.3 Discussion of Results

The two residential building units were constructed in 1983 and likely designed for the loading standard current at the time: NZS 4203: 1976. The design loads used in this code are less than those required by the current loading standard. In addition, the detailing requirements for ductile seismic behaviour that are present in the current codes are unlikely to have been considered in the design of this building. As a result, it would be expected that the building would not achieve 100% NBS. The increase in the hazard factor for Christchurch to 0.3 would be expected to further reduce the %NBS score.

The out-of-plan flexural capacities of the wall panels are greater than those required for the New Building Standard as the reinforcement present was more than likely designed for gravity loads acting on the structure during construction, which are greater than those for out-of-plane flexural forces due to seismic events. In-plane flexural forces however, vary throughout the building with some of the horizontal spandrel panels with smaller cross sectional areas and less reinforcement achieving less than 67% NBS.

Should any of the low % NBS value spandrel elements fail during a severe seismic event, it is not expected that the building will collapse as a result. Other elements have sufficient strength to provide redundancy within the structure.

# 10. Recommendations

The recent seismic activity in Christchurch has only caused minor damage to the buildings, with minor cracking in concrete walls, wall linings and ceiling linings the only damage noted. As the buildings have suffered minor damage that would not compromise the load resisting capacity of the existing structural systems, general occupancy of the buildings is permitted.

Both buildings have however been assessed as being potentially Earthquake Risk buildings as they have achieved less than 67% NBS to localised spandrel panels. As such, GHD Limited recommend that strengthening options be explored in order to increase the % NBS of the buildings to 67% NBS as recommended by the NZSEE Guidelines.

As a result with the lack of seismic separation between the staircases and the supporting precast panel walls, it is recommended that secondary support be provided to all stair landings in the form of equal angle supports fixed to the supporting panel walls. As the stairs are the only means of egress from the upper floors, this issue should be addressed immediately.

Due to the presence of cracking to the asbestos plaster ceiling lining in a number of the units throughout the complex, GHD Limited recommends that all the ceiling linings are inspected by asbestos specialists and dealt with according to their recommendations.

# 11. Limitations

## 11.1 General

This report has been prepared subject to the following limitations:

- No level or verticality surveys have been undertaken.
- No material testing has been undertaken other than asbestos testing of the ceiling lining prior to intrusive investigations.
- No calculations, other than those described in the report, have been undertaken.

It is noted that this report has been prepared at the request of Christchurch City Council and is intended to be used for their purposes only. GHD accepts no responsibility for any other party or person who relies on the information contained in this reportrite a specific limitations section.

## 11.2 Geotechnical Limitations

The data and advice provided herein relate only to the project and structures described herein and must be reviewed by a competent geotechnical engineer before being used for any other purpose. GHD Limited (GHD) accepts no responsibility for other use of the data by third parties.

Where drill hole or test pit logs, cone tests, laboratory tests, geophysical tests and similar work have been performed and recorded by others under a separate commission, the data is included and used in the form provided by others. The responsibility for the accuracy of such data remains with the issuing authority, not with GHD.

The advice tendered in this report is based on information obtained from the desk study investigation location test points and sample points. It is not warranted in respect to the conditions that may be encountered across the site other than at these locations. It is emphasised that the actual characteristics of the subsurface materials may vary significantly between adjacent test points, sample intervals and at locations other than where observations, explorations and investigations have been made. Subsurface conditions, including groundwater levels and contaminant concentrations can change in a limited time. This should be borne in mind when assessing the data.

It should be noted that because of the inherent uncertainties in subsurface evaluations, changed or unanticipated subsurface conditions may occur that could affect total project cost and/or execution. GHD does not accept responsibility for the consequences of significant variances in the conditions and the requirements for execution of the work.

The subsurface and surface earthworks, excavations and foundations should be examined by a suitably qualified and experienced Engineer who shall judge whether the revealed conditions accord with both the assumptions in this report and/or the design of the works. If they do not accord, the Engineer shall modify advice in this report and/or design of the works to accord with the circumstances that are revealed.

An understanding of the geotechnical site conditions depends on the integration of many pieces of information, some regional, some site specific, some structure specific and some experienced based. Hence this report should not be altered, amended or abbreviated, issued in part and issued incomplete in any way without prior checking and approval by GHD. GHD accepts no responsibility for any circumstances which arise from the issue of the report which have been modified in any way as outlined above.

Appendix A

# Photographs

**Photograph 1 North elevation of Haast Courts Block B (Units 5-10).**

**Photograph 2 Cracking to exterior walls at window corners.**

**Photograph 3 Cracking to plaster finish on the underside of the suspended floor slab.**

**Photograph 4 Connection of stairs to first floor landing.**

**Photograph 5 location of Hilti Ferroscan of wall at ground floor.**

**Photograph 6 Scan of a connection between a flight of stairs and the supporting landing.**

**Photograph 7 Seismic gap between stairs landing and precast concrete wall panel.**

**Photograph 8 Seismic gap between stairs landing and precast concrete wall panel showing steel dowel pin connection.**

Photograph 9 Internal view of roof space showing inclined members of the timber truss and timber framing of the roof/gable wall.

Photograph 10 Connection of timber roof truss members.

**Photograph 11 Flat steel strap roof cross bracing.**

**Photograph 12 End connection of flat steel strap roof cross bracing.**

**Photograph 13 Removal of wall linings to expose concrete panel wall to allow scanning for reinforcement details at second floor level.**

**Photograph 14 Connection between cast in situ floor slab and precast panel wall at second floor level.**

**Photograph 15 Angle cleat connection between precast wall panels at second floor.**

**Photograph 16 Wall linings removed to expose angle cleat connection and precast concrete panel at second floor level.**

**Photograph 17 Removal of wall linings in first floor apartment to expose precast concrete wall panel to scan for reinforcement.**

**Photograph 18 Scan locations looking for details of connection between the precast panel and the cast in-situ suspended floor slab.**

**Photograph 19 Black lines indicating where reinforcement is expected to be found and red lines indicating area to be exposed.**

**Photograph 20 Suspended slab steel reinforcement cage found not to be tied into precast wall from the top.**

**Photograph 21 Further opening up of the suspended slab showed the steel reinforcement cage not to be tied into precast wall from the top.**

**Photograph 22 Opening up to the underside of the second floor suspended slab showed the steel cage reinforcement not tied into the precast wall.**

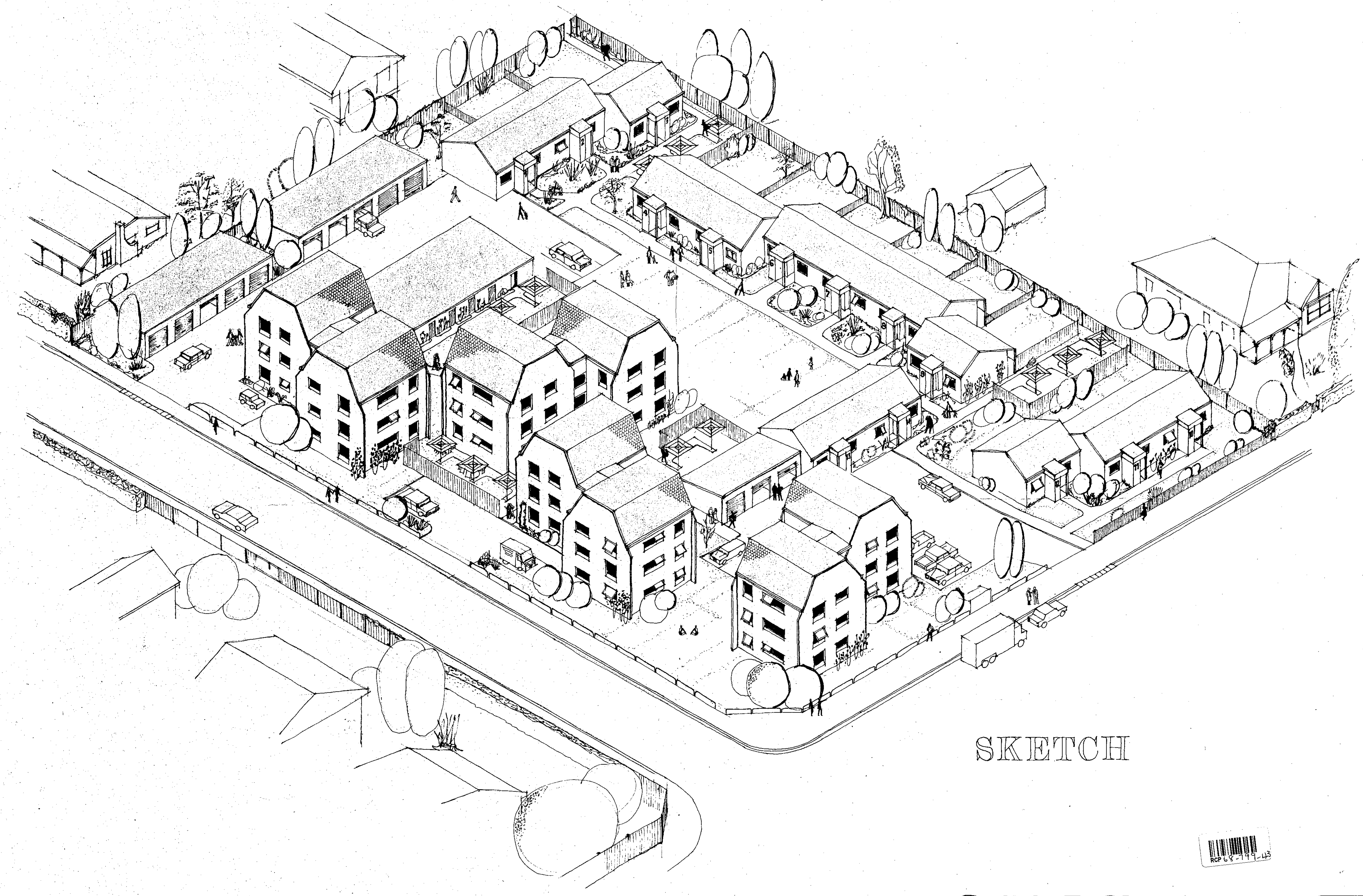
Photograph 23 Opening up in another location revealed a 10mm diameter steel bar tying the suspended slab back into the precast concrete wall panels.

Photograph 24 Further opening up and reinforcement scanning revealed regular spacing of bars tying the slab into the precast panels at 250mm centres along all walls.

Appendix B

# Existing Drawings

43 Haast



SKETCH

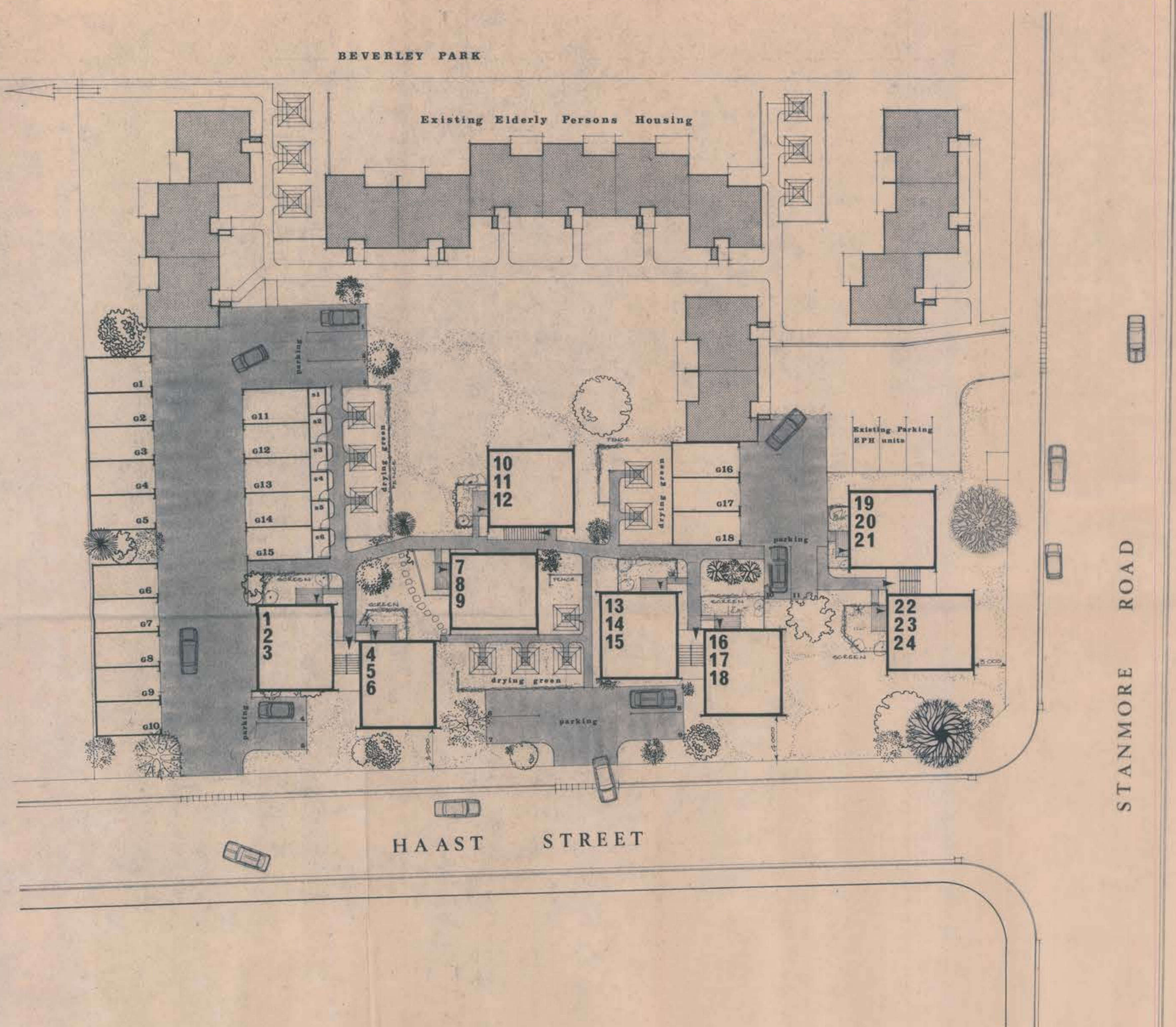
# STANMORE ROAD HOUSING
# CHRISTCHURCH CITY COUNCIL


**M.L. PAYNTER LTD.**
design construction management
11 dollans lane christchurch ph 67812

**IAN KRAUSE ASSOCIATES**
REGISTERED ARCHITECTS Urban Design & Environment Consultants
P.O. Box 1766 Christchurch Phone 60323


Date: SEPTEMBER 1982
Scale: —
Job No: 571

1




# STANMORE ROAD HOUSING
# CHRISTCHURCH CITY COUNCIL

| | |
|---|---|
| Date: | SEPTEMBER 1982 |
| Scale: | 1 : 200 |
| Job No: | 571 |

**2**

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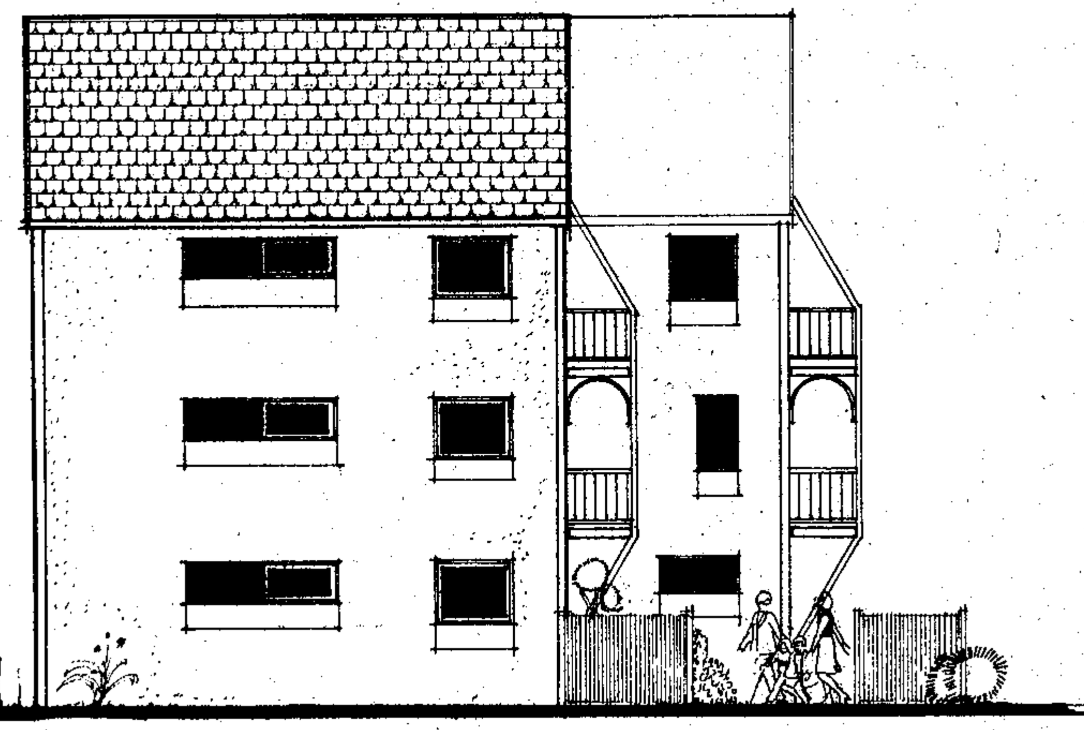
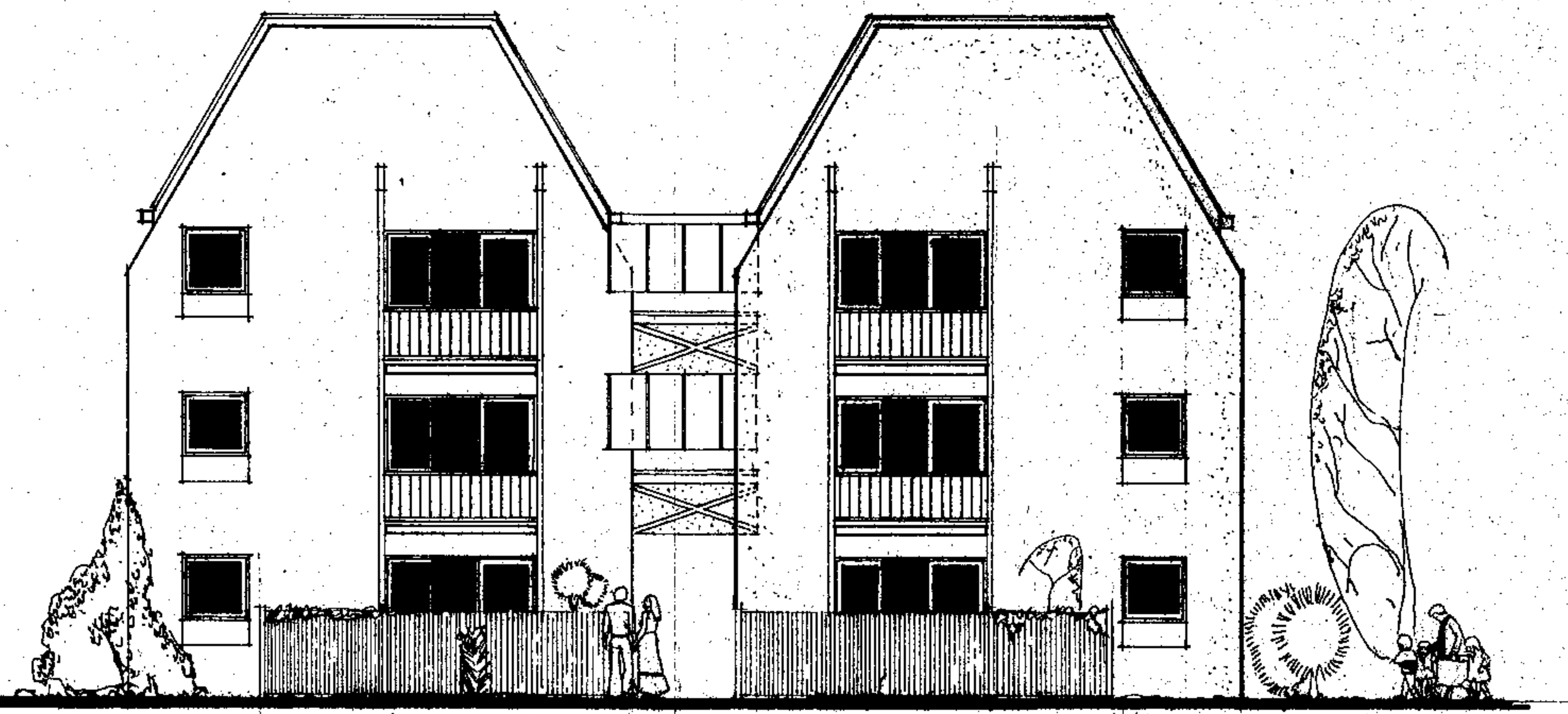
11 dollans lane christchurch ph 67812

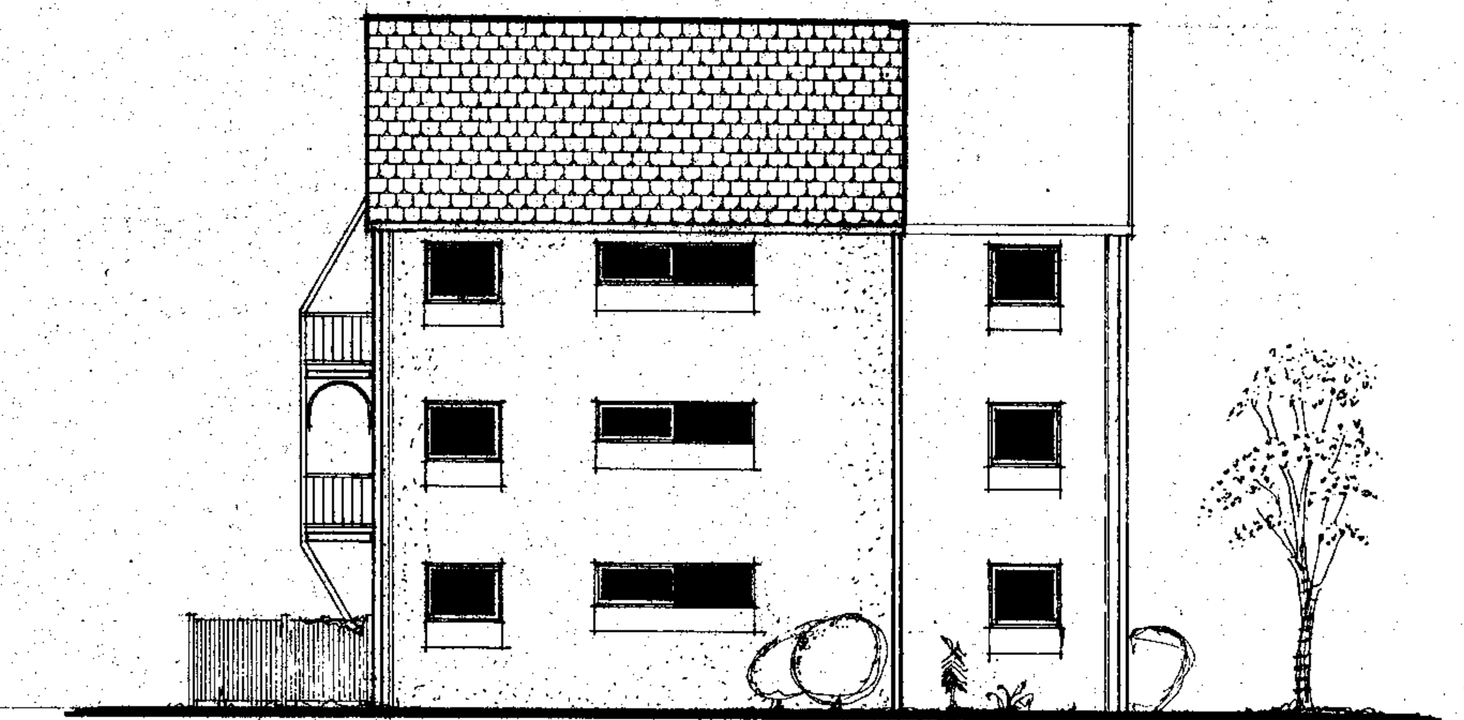
**IAN KRAUSE ASSOCIATES**
REGISTERED ARCHITECTS Urban Design & Environment Consultants
P.O. Box 1766 Christchurch Phone 60323



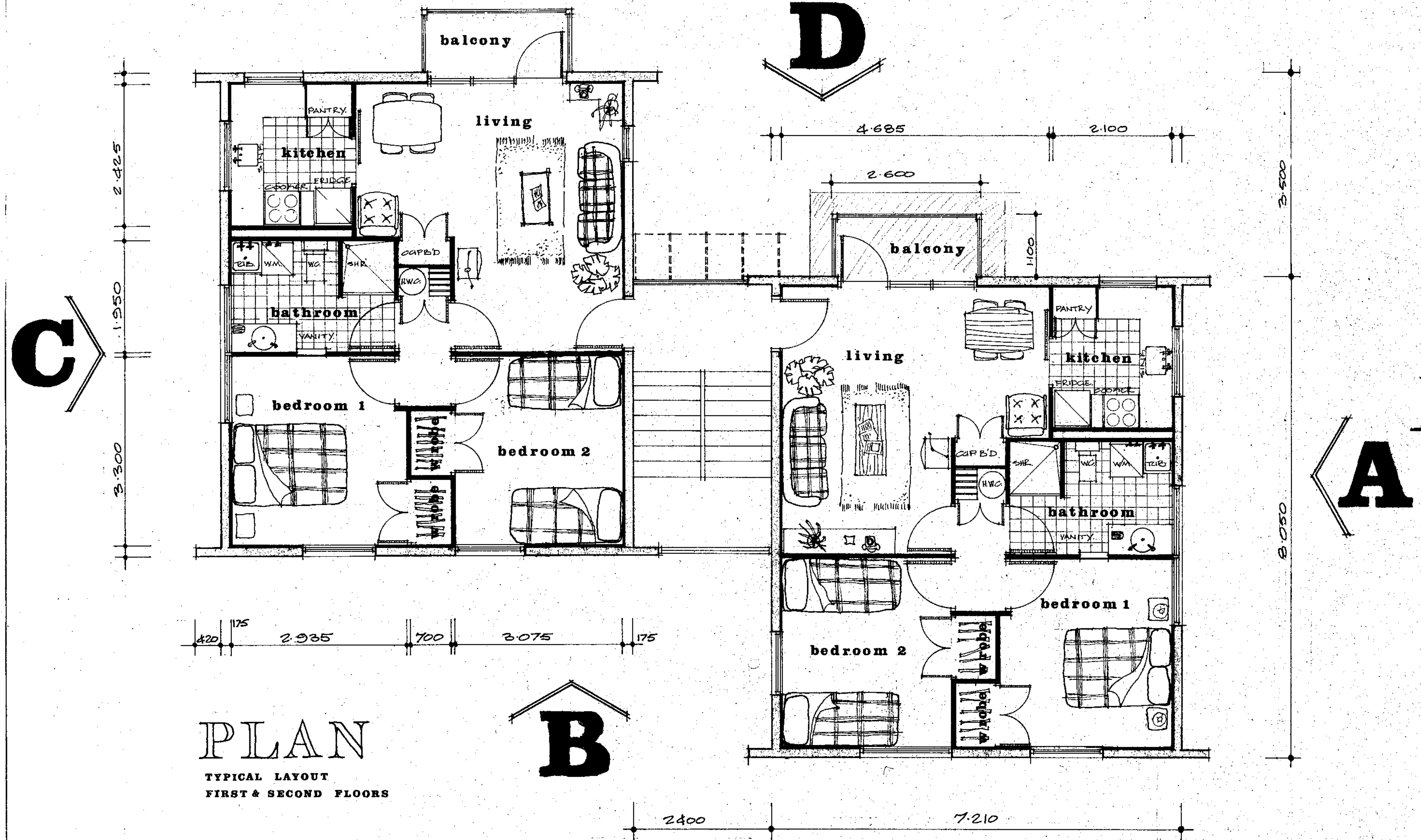
ELEVATION A



ELEVATION D



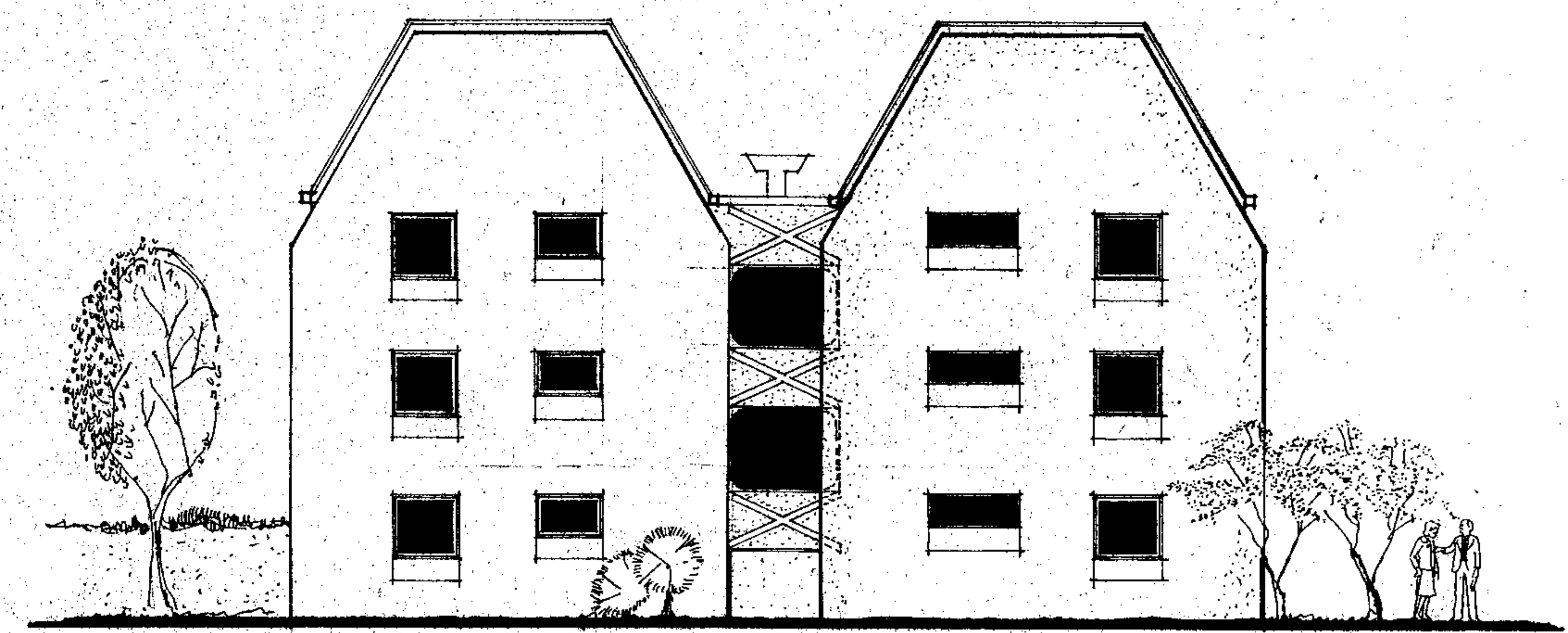
ELEVATION C




PLAN

TYPICAL LAYOUT

FIRST & SECOND FLOORS



ELEVATION B

# STANMORE ROAD HOUSING
# CHRISTCHURCH CITY COUNCIL

Date: SEPTEMBER 1982
Scale: 1:50 1:100
Job No: 571

3

M.L. PAYNTER LTD.
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11 dollans lane christchurch ph 67812

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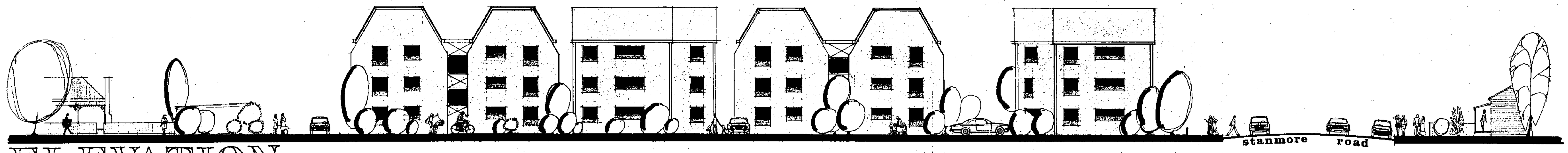
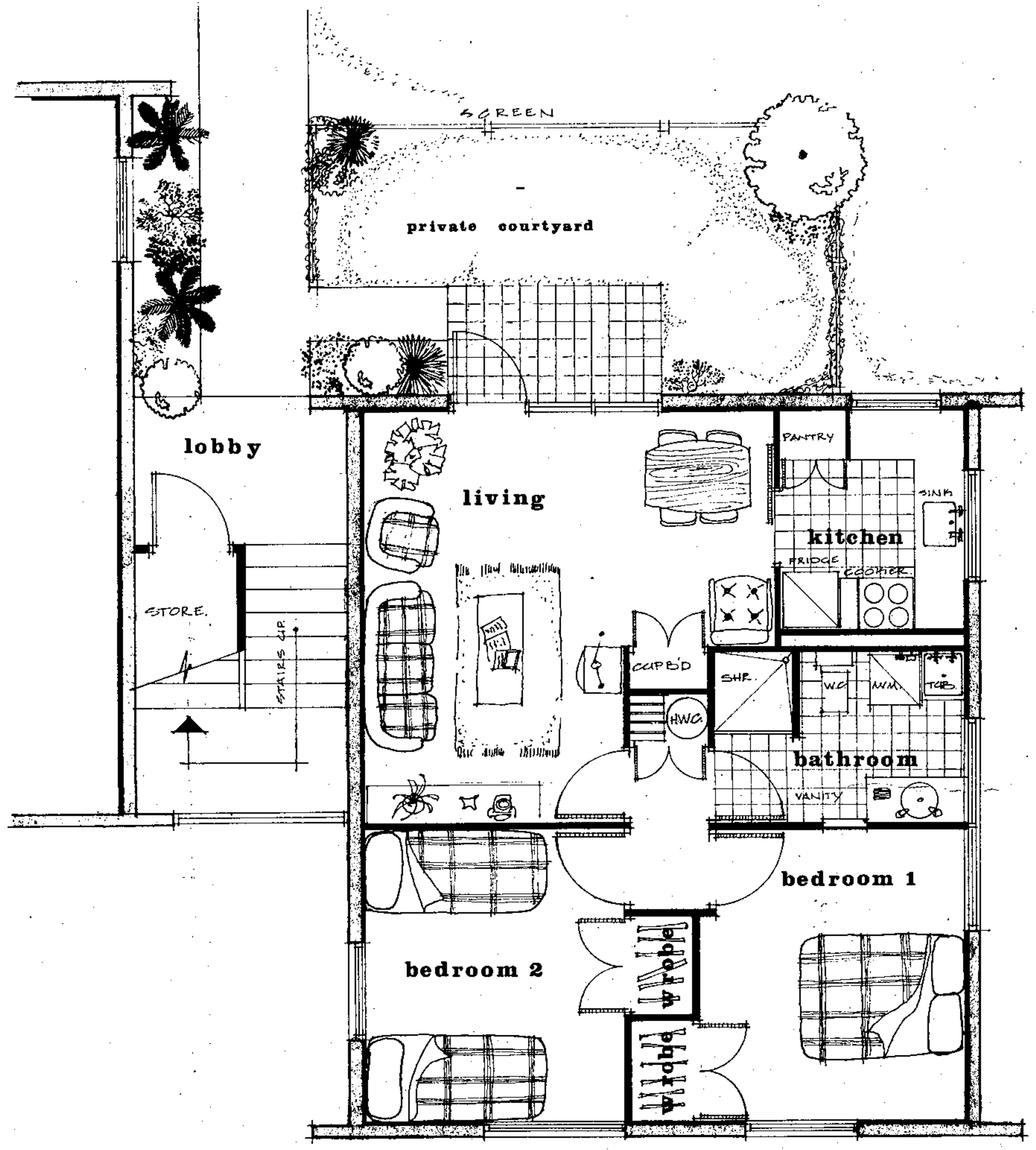
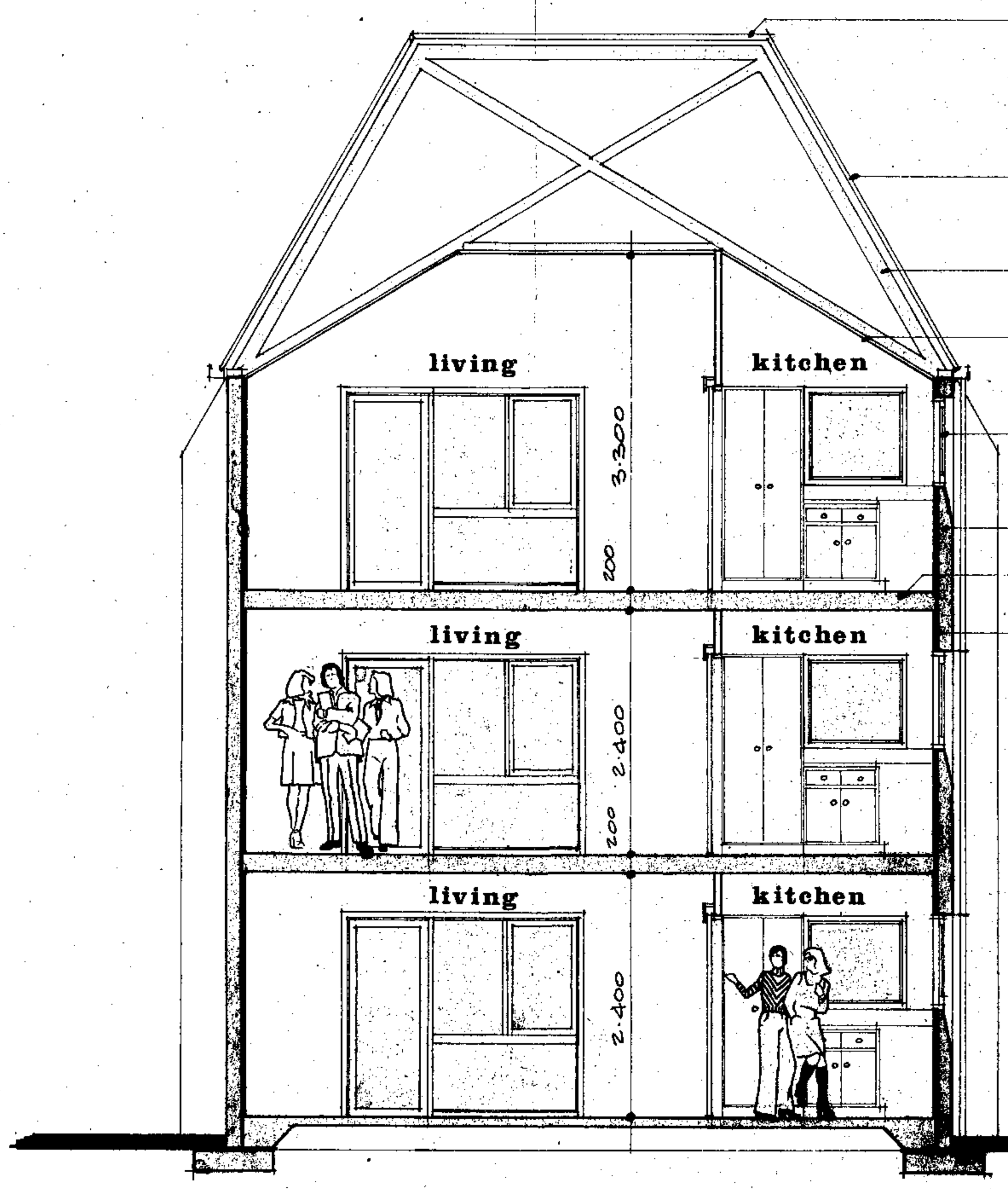
P.O.Box 1766 Christchurch Phone 60323



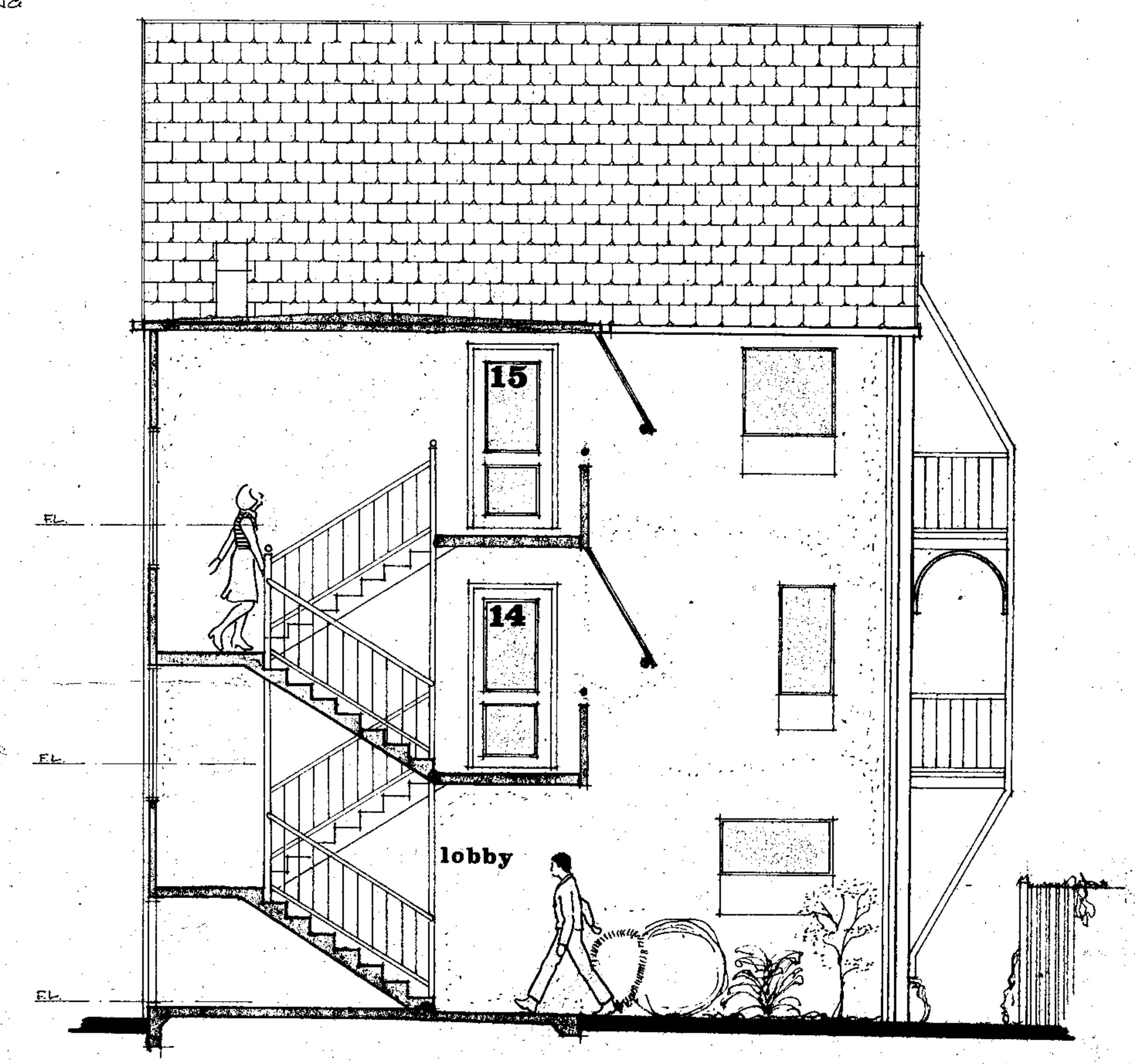

# ELEVATION
HAAST STREET FRONTAGE



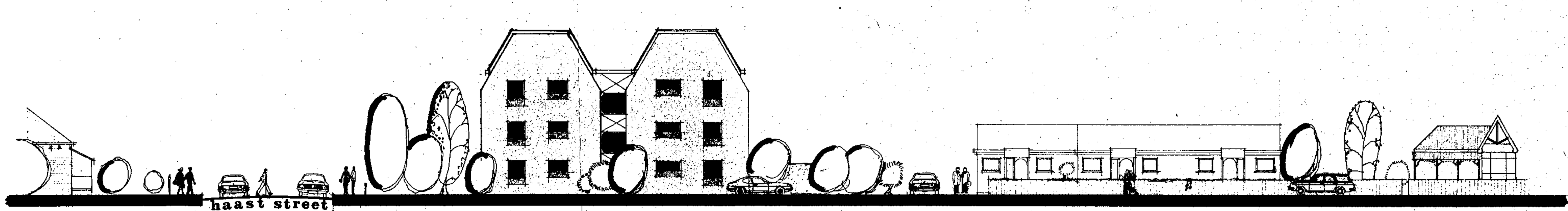

# GROUND FLOOR PLAN




# SECTION




# SECTION




# ELEVATION
STANMORE ROAD FRONTAGE

# STANMORE ROAD HOUSING
# CHRISTCHURCH CITY COUNCIL

| | |
|---|---|
| Date | SEPTEMBER 1982 |
| Scale | 1:50, 1:200 |
| Job No | 571 |

4

**M.L. PAYNTER LTD.**
design construction management
11 dollans lane christchurch ph. 67812

**IAN KRAUSE ASSOCIATES**
REGISTERED ARCHITECTS Urban Design & Environment Consultants
P.O.Box 1766 Christchurch Phone 60323


Stanmore Rd
Stanmore Rd
Stanmore Rd
Block E - Units 1 to 3
Block D - Units 1 to 5
Block F - Units 4 to 5
Block K - Garages 16 to 18
Block C - Units 11 to 14
Block G - Units 6 to 11
Block B - Units 5 to 10
Block A - Units 1 to 4
Block J - Garages 11 to 15
Block H - Units 12 to 14
Block I - Garages 1 to 10 + 5A
Haast St
Haast St
Haast St
Haast St

Appendix C

# CERA Building Evaluation Form

**Detailed Engineering Evaluation Summary Data** V1.11

## Location

| | | | | | |
|---|---|---|---|---|---|
| Building Name: | Haast Courts Block B | | | Reviewer: | David Lee |
| | Unit | No: | Street | CPEng No: | 112052 |
| Building Address: | 5 to 7 | 43 | Haast Street | Company: | GHD |
| Legal Description: | LOT 1 DP 47661 | | | Company project number: | 513059642 |
| | | | | Company phone number: | (03) 3780900 |
| | Degrees | Min | Sec | | |
| GPS south: | 43 | 31 | 39.49 | | 25/10/2012 |
| GPS east: | 172 | 39 | 22.16 | Inspection Date: | 20/07/2012 |
| | | | | Revision: | DRAFT |
| Building Unique Identifier (CCC): | BU 0792-002 EQ2 | | | Is there a full report with this summary? | yes |

## Site

| | | | |
|---|---|---|---|
| Site slope: | flat | Max retaining height (m): | |
| Soil type: | sandy silt | 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): | 10.00 |

## Building

| | | | | |
|---|---|---|---|---|
| No. of storeys above ground: | 2 | single storey = 1 | Ground floor elevation (Absolute) (m): | 11.00 |
| Ground floor split? | no | | Ground floor elevation above ground (m): | 0.15 |
| Storeys below ground | 0 | | | |
| Foundation type: | strip footings | | if Foundation type is other, describe: | |
| Building height (m): | 8.00 | | height from ground to level of uppermost seismic mass (for IEP only) (m): | 7.5 |
| Floor footprint area (approx): | 130 | | | |
| Age of Building (years): | 30 | | 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): | Apartments - 2 Buildings | | | |
| Importance level (to NZS1170.5): | IL2 | | | |

### Gravity Structure

| | | | |
|---|---|---|---|
| Gravity System: | load bearing walls | | |
| Roof: | timber truss | truss depth, purlin type and cladding | |
| Floors: | concrete flat slab | slab thickness (mm) | 200 |
| Beams: | none | overall depth x width (mm x mm) | |
| Columns: | | | |
| Walls: | load bearing concrete | #N/A | |

### Lateral load resisting structure

| | | | | | |
|---|---|---|---|---|---|
| Lateral system along: | concrete shear wall | | **Note: Define along and across in** | note total length of wall at ground (m): | 30 |
| Ductility assumed, μ: | 1.25 | | **detailed report!** | wall thickness (m): | 0.15 |
| Period along: | 0.13 | 0.05 | from parameters in sheet | estimate or calculation? | estimated |
| Total deflection (ULS) (mm): | | | | estimate or calculation? | |
| maximum interstorey deflection (ULS) (mm): | | | | estimate or calculation? | |
| Lateral system across: | concrete shear wall | | | note total length of wall at ground (m): | 33 |
| Ductility assumed, μ: | 1.25 | | | wall thickness (m): | 0.15 |
| Period across: | 0.13 | 0.04 | from parameters in sheet | estimate or calculation? | estimated |
| Total deflection (ULS) (mm): | | | | estimate or calculation? | |
| 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, half height | describe supports | Half landing at top at bottom |
| Wall cladding: | exposed structure | describe | Painted shear walls |
| Roof Cladding: | Membrane | substrate | Plywood |
| Glazing: | aluminium frames | | |
| Ceilings: | sprayed | | Asbestos |
| Services(list): | | | |

**Available documentation**

| | | | |
|---|---|---|---|
| Architectural | partial | original designer name/date | ML Painter Ltd. September 1982 |
| 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: | Good overall | 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): | Insignificant | | |
| Across | Damage ratio: | 0% | | |
| | Describe (summary): | Insignificant | | |
| 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: | none | Describe: | |
| | Building Consent required: | no | Describe: | |
| | Interim occupancy recommendations: | full occupancy | Describe: | |
| Along | Assessed %NBS before: | 45% | If IEP not used, please detail assessment methodology: | Structural modelling/analysis to NZS3101 |
| | Assessed %NBS after: | 45% | | |
| Across | Assessed %NBS before: | 45% | | |
| | Assessed %NBS after: | 45% | | |

**Detailed Engineering Evaluation Summary Data** V1.11

## Location

| | | | |
|---|---|---|---|
| Building Name: | Haast Courts Block B | Reviewer: | David Lee |
| | Unit / No: / Street | CPEng No: | 112052 |
| Building Address: | 8 to 10 / 43 / Haast Street | Company: | GHD |
| Legal Description: | LOT 1 DP 47661 | Company project number: | 513059642 |
| | | Company phone number: | (03) 3780900 |
| | Degrees / Min / Sec | | |
| GPS south: | 43 / 31 / 39.49 | | 25/10/2012 |
| GPS east: | 172 / 39 / 22.16 | Inspection Date: | 20/07/2012 |
| | | Revision: | DRAFT |
| Building Unique Identifier (CCC): | BU 0792-002 EQ2 | Is there a full report with this summary? | yes |

## Site

| | | | |
|---|---|---|---|
| Site slope: | flat | Max retaining height (m): | |
| Soil type: | sandy silt | 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): | 10.00 |

## Building

| | | | | |
|---|---|---|---|---|
| No. of storeys above ground: | 2 | single storey = 1 | Ground floor elevation (Absolute) (m): | 11.00 |
| Ground floor split? | no | | Ground floor elevation above ground (m): | 0.15 |
| Storeys below ground | 0 | | | |
| Foundation type: | strip footings | | if Foundation type is other, describe: | |
| Building height (m): | 8.00 | | height from ground to level of uppermost seismic mass (for IEP only) (m): | 7.5 |
| Floor footprint area (approx): | 130 | | | |
| Age of Building (years): | 30 | | 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): | Apartments - 2 Buildings | | | |
| Importance level (to NZS1170.5): | IL2 | | | |

Gravity Structure

| | | | |
|---|---|---|---|
| Gravity System: | load bearing walls | | |
| Roof: | timber truss | truss depth, purlin type and cladding | |
| Floors: | concrete flat slab | slab thickness (mm) | 200 |
| Beams: | none | overall depth x width (mm x mm) | |
| Columns: | | | |
| Walls: | load bearing concrete | #N/A | |

Lateral load resisting structure

| | | | | |
|---|---|---|---|---|
| Lateral system along: | concrete shear wall | **Note: Define along and across in detailed report!** | note total length of wall at ground (m): | 30 |
| Ductility assumed, μ: | 1.25 | | wall thickness (m): | 0.15 |
| Period along: | 0.13 | 0.05 from parameters in sheet | estimate or calculation? | estimated |
| Total deflection (ULS) (mm): | | | estimate or calculation? | |
| maximum interstorey deflection (ULS) (mm): | | | estimate or calculation? | |
| Lateral system across: | concrete shear wall | | note total length of wall at ground (m): | 33 |
| Ductility assumed, μ: | 1.25 | | wall thickness (m): | 0.15 |
| Period across: | 0.13 | 0.04 from parameters in sheet | estimate or calculation? | estimated |
| Total deflection (ULS) (mm): | | | estimate or calculation? | |
| maximum interstorey deflection (ULS) (mm): | | | estimate or calculation? | |

Separations:

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

<u>Non-structural elements</u>

| | | | |
|---|---|---|---|
| Stairs: | precast, half height | describe supports | Half landing at top at bottom |
| Wall cladding: | exposed structure | describe | Painted shear walls |
| Roof Cladding: | Membrane | substrate | Plywood |
| Glazing: | aluminium frames | | |
| Ceilings: | sprayed | | Asbestos |
| Services(list): | | | |

**Available documentation**

| | | | |
|---|---|---|---|
| Architectural | partial | original designer name/date | ML Painter Ltd. September 1982 |
| 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**

<u>Site:</u>
(refer DEE Table 4-2)

| | | | |
|---|---|---|---|
| Site performance: | Good overall | 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): | |

<u>Building:</u>

| | | | | |
|---|---|---|---|---|
| | Current Placard Status: | green | | |
| Along | Damage ratio: | 0% | Describe how damage ratio arrived at: | |
| | Describe (summary): | Insignificant | | |
| Across | Damage ratio: | 0% | | |
| | Describe (summary): | Insignificant | | |
| 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: | none | Describe: | |
| | Building Consent required: | no | Describe: | |
| | Interim occupancy recommendations: | full occupancy | Describe: | |
| Along | Assessed %NBS before: | 50% | If IEP not used, please detail assessment methodology: | Structural modelling/analysis to NZS3101 |
| | Assessed %NBS after: | 50% | | |
| Across | Assessed %NBS before: | 50% | | |
| | Assessed %NBS after: | 50% | | |

Appendix D

# Geotechnical Appendix

# 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 ($u_C$).

| | |
|---|---|
| red | sand (and gravel) |
| yellow | silt-sand |
| green | silt |
| cyan | clay-silt |
| blue | clay |
| black | 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).

| | |
|---|---|
| red | high susceptibility |
| orange | medium susceptibility |
| green | 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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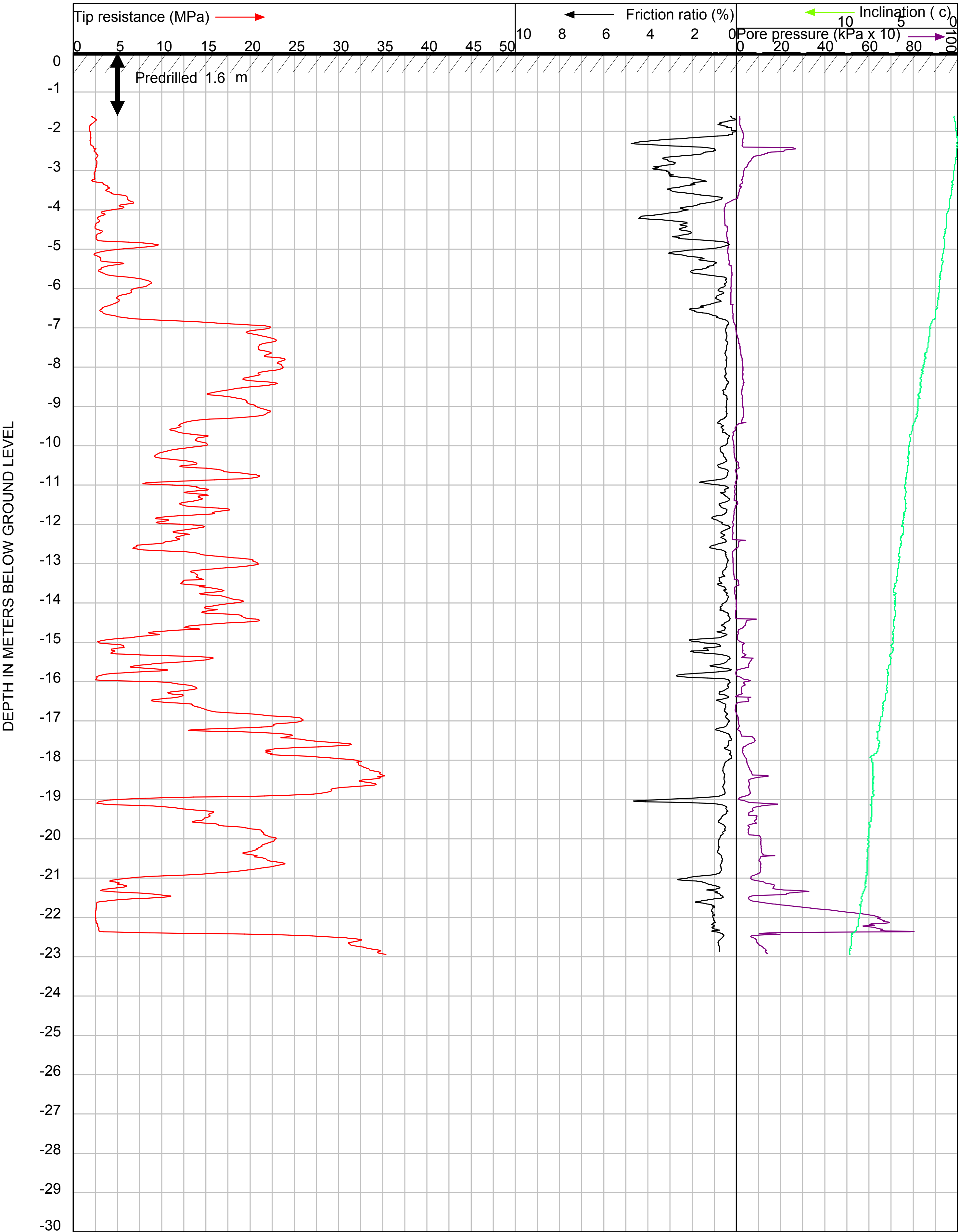
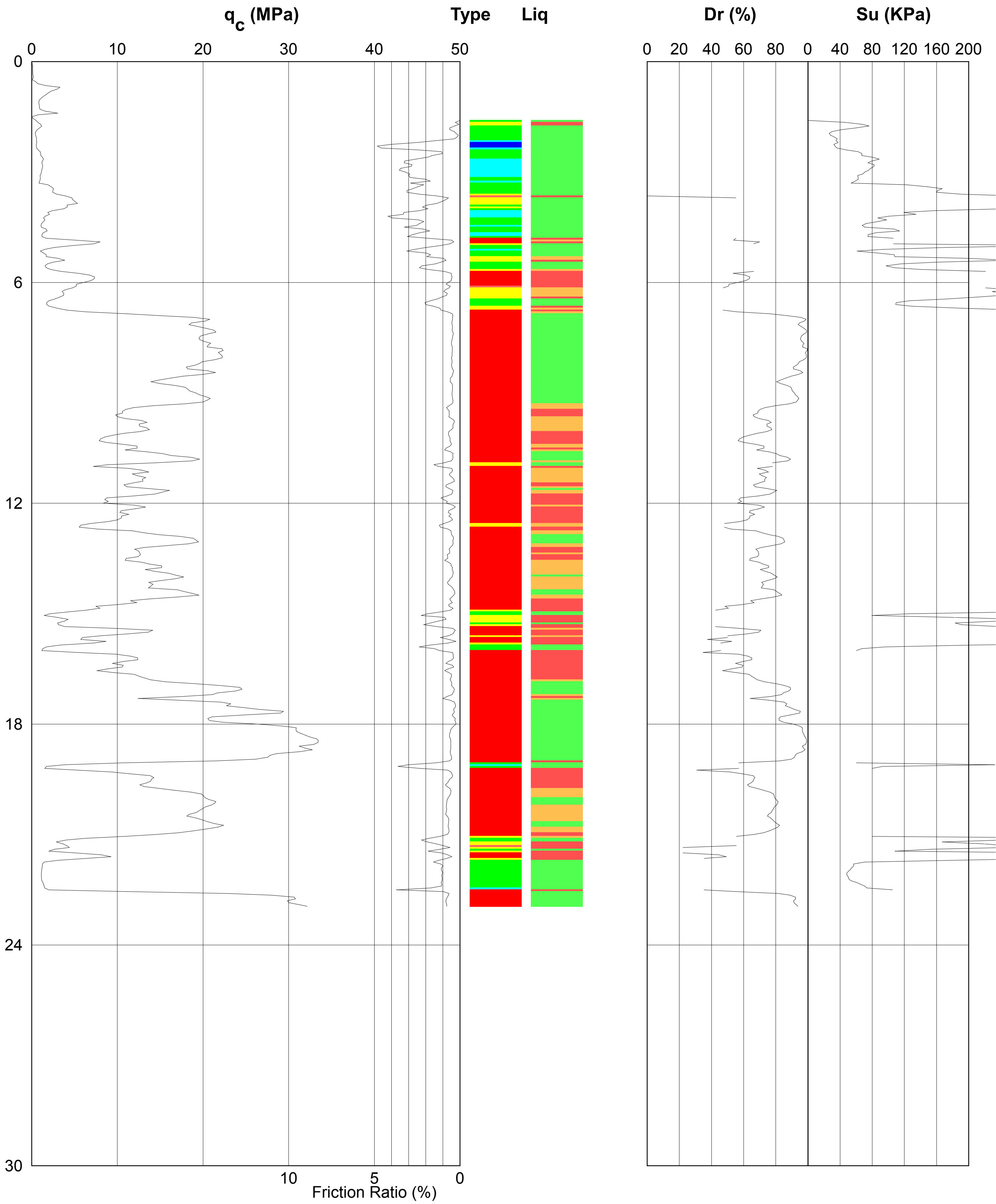



CLIENT : GHD NZ Ltd
LOCATION : 149 Stanmore Road, Christchurch
DATE : 26-6-2012
OPERATOR : S.Cardona
REMARK 1 : CPTu001
REMARK 2 : Effective Refusal

JOB # : 10677
TEST # : 1
CONE TYPE/SERIAL #: I-CFXYP20-10/ 071017T

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: 10677

CPT No: CPTu001

Project: GHD NZ Ltd

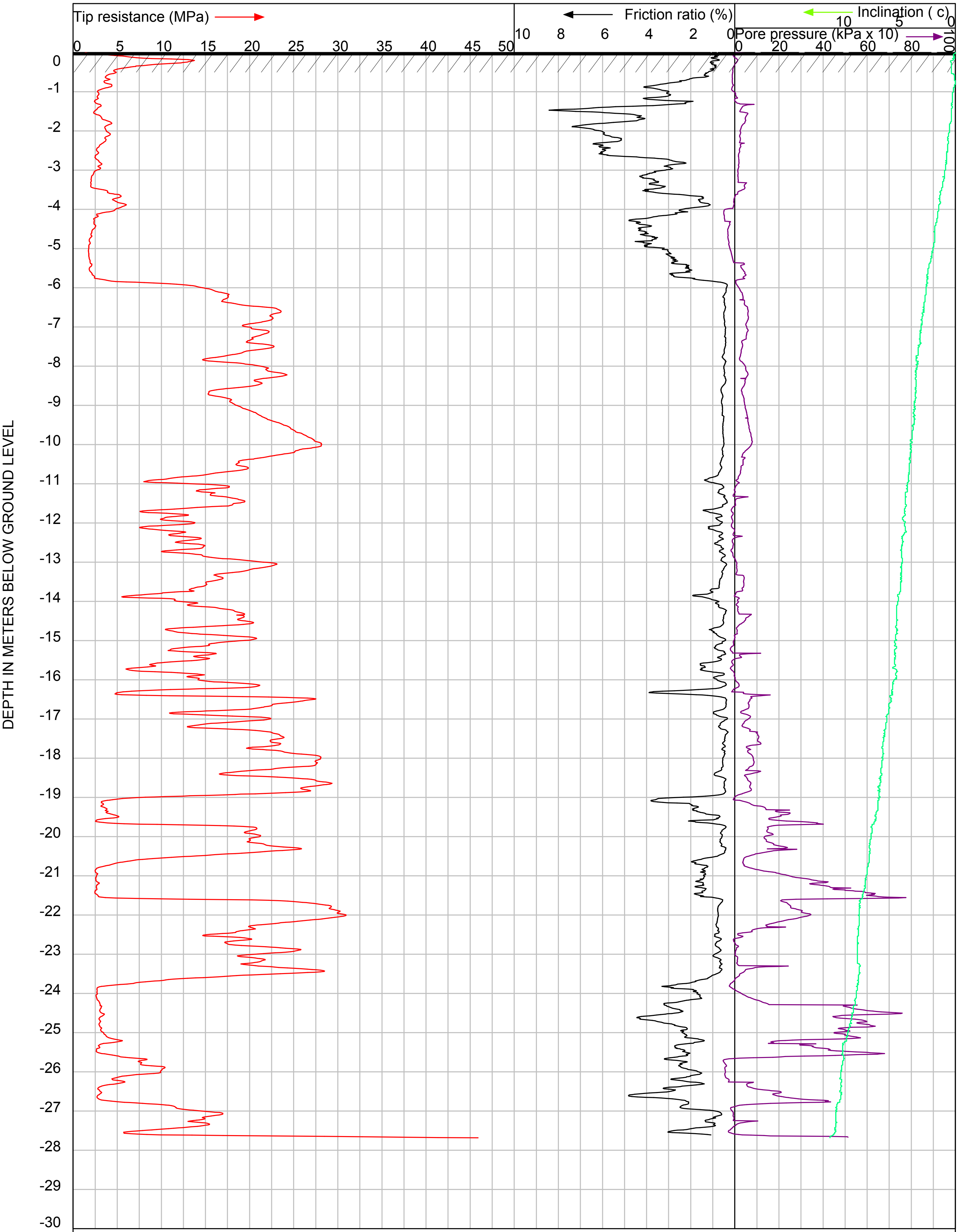
Location: 149 Stanmore Road, Christchurch

Date: 26-6-2012

Operator: S.Cardona

Remark: Effective Refusal

McMILLAN DRILLING SERVICES

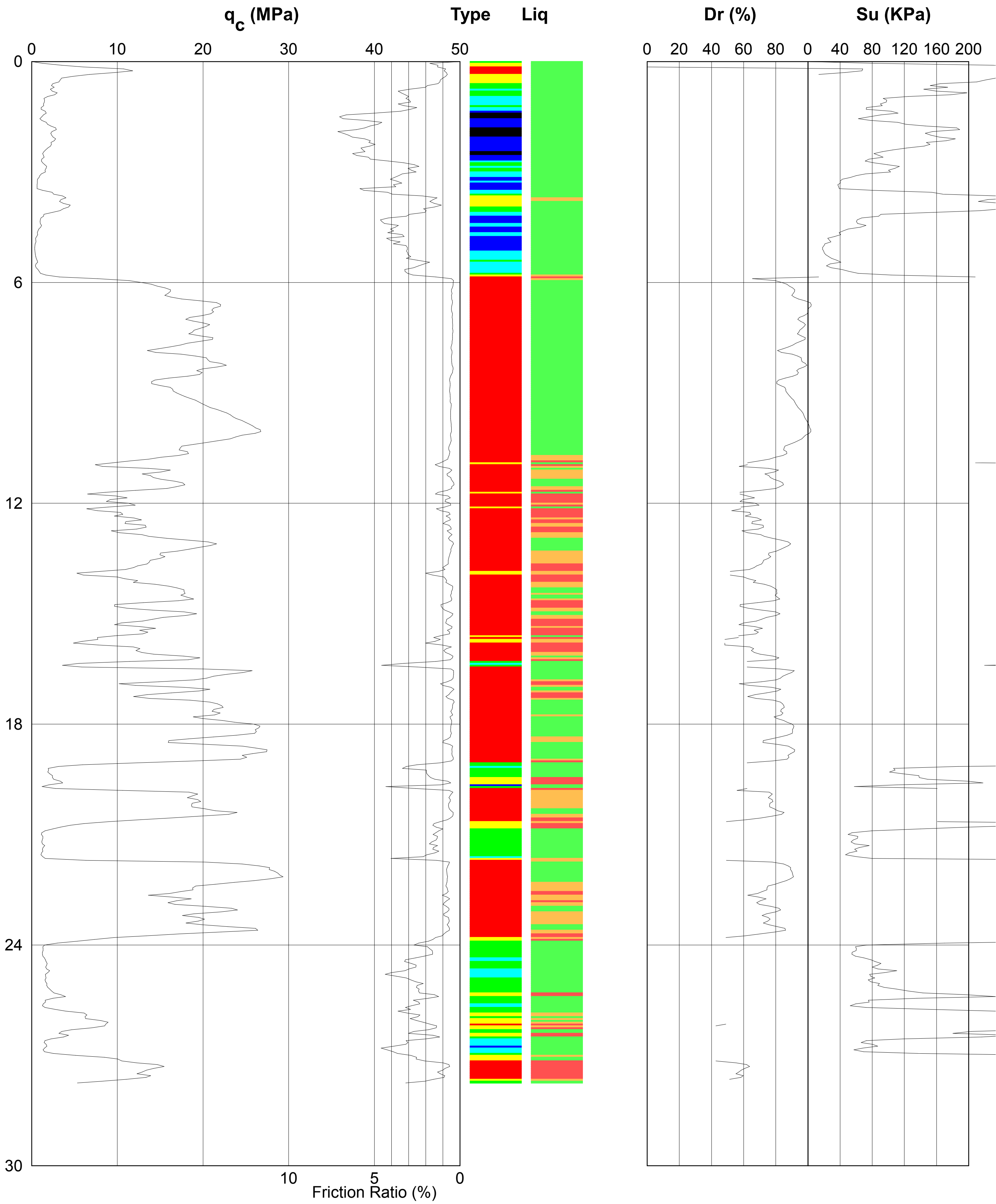



CLIENT : GHD NZ Ltd
LOCATION : 149 Stanmore Road, Christchurch
DATE : 26-6-2012
OPERATOR : S.Cardona
REMARK 1 : CPTu002
REMARK 2 : Effective Refusal

JOB # : 10677
TEST # : 2
CONE TYPE/SERIAL #: I-CFXYP20-10/ 071017T

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: | 10677 | Date: | 26-6-2012 |
| CPT No: | CPTu002 | Operator: | S.Cardona |
| Project: | GHD NZ Ltd | Remark: | Effective Refusal |
| Location: | 149 Stanmore Road, Christchurch | | |

McMILLAN DRILLING SERVICES

# CIVIL CONSTRUCTION OVERVIEW

- 5 x Piling Rigs (20 to 80 tonne);
- 4 x Tieback/Micro-Piling Rigs (0.5 to 20 tonne);
- Sheet Piling & Injection Grouting;
- Dewatering;
- 26 x Drilling Rigs Company wide.

## A NEW ZEALAND FIRST METHOD – INTRODUCED TO THE MARKET BY MCMILLAN'S:

**Provisionally Patented Vibration Free Stone Column Method:**


- Can be used next to sensitive buildings;
- No mess (dry);
- Cost effective (minimal setup times);
- Further savings possible for building construction – i.e. ground beams, deep rafts, pile starters, boxing to piles;
- No corrosion issues, all natural materials;
- Reliance on individual piles, and the risk of differential settlement is reduced.

**Fully Instrumented Continuous Flight Auger / Displacement Auger Piling:**

- Cost effective;
- Sizes 350mm to 900mm and 19m depth;
- Fast (150m of 600mm diameter reinforced concrete pile can be installed per day);
- Lateral load capacity of RC piles exceed some other piling methods;
- Quiet & vibration free;
- Fully reinforced concrete piles, with no corrosion issues.

## McMILLAN'S ALSO OFFER THE FOLLOWING SERVICES:

- Screw Piles;
- Conventional Bored Concrete Piles;
- Mini & Micro Piles;
- Retaining Walls;
- Sheet Piling;
- Anchors & Tiebacks.

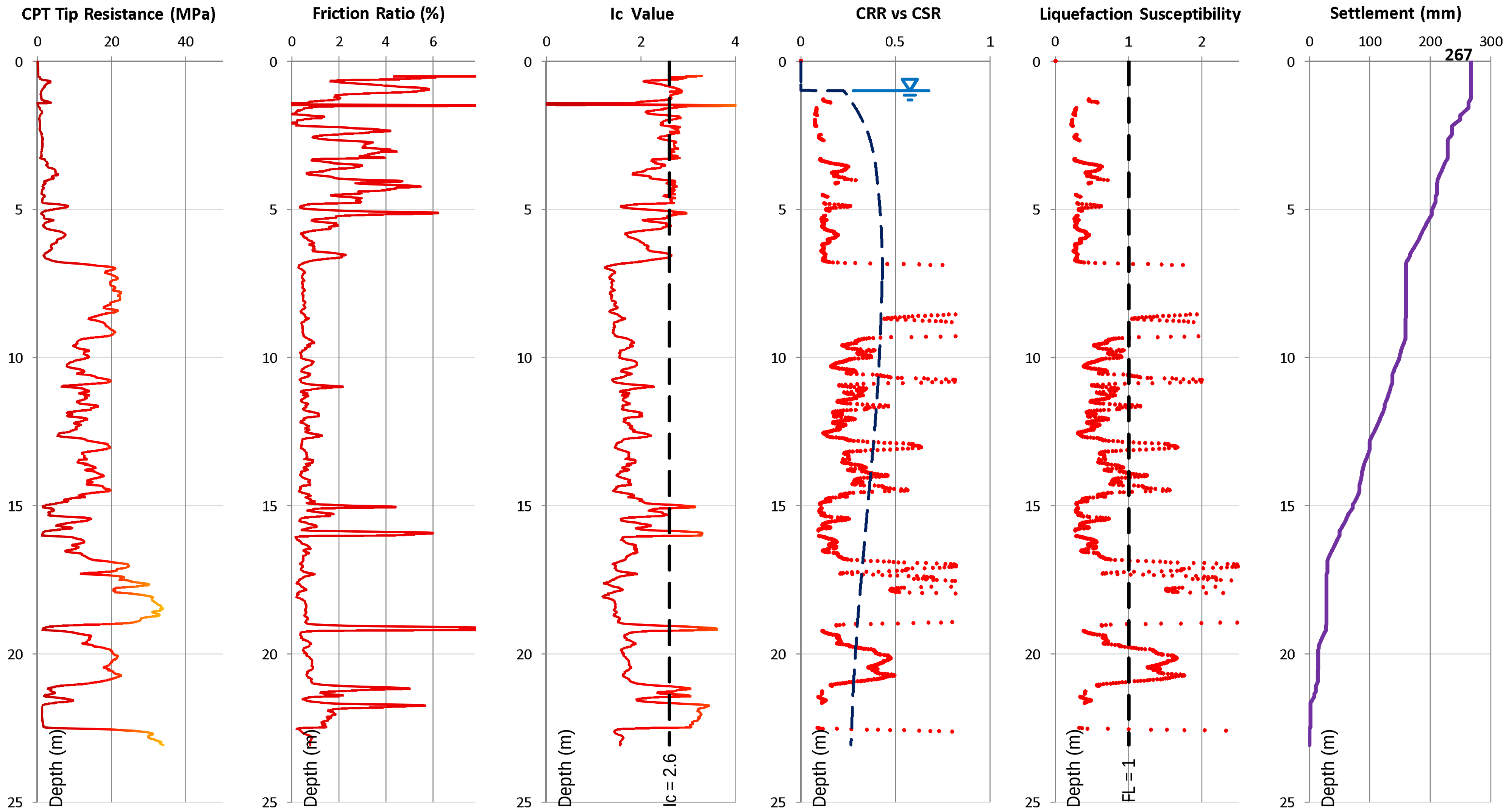
Please contact us to find out more information or visit our website www.drilling.co.nz

McMILLAN
DRILLING SERVICES

# SOIL LIQUEFACTION SUSCEPTIBILITY ASSESSMENT

| | | | |
|---|---|---|---|
| LOCATION : | **CPT 1** | SHEET : | **1** |
| PROJECT : | **Haast Courts - 149 Stanmore Road** | CALCULATED : | **HNN** |
| JOB NO : | **51 30596 41** | CHECKED BY : | |
| TEST DATE : | **28 Jun 2012** | DATE : | **4 Jul 2012** |

| | | | | | | |
|---|---|---|---|---|---|---|
| PGA ($a_{max}$): | **0.35 g** | Groundwater Level (m bgl): | **1.0** | Bore depth (m): | **23.07** | **Total Estimated Settlement (mm)** |
| EQ Magnitude: | **7.5** | Atmospheric Pressure (kPa): | **101** | Test data step (m): | **0.01** | **267** |

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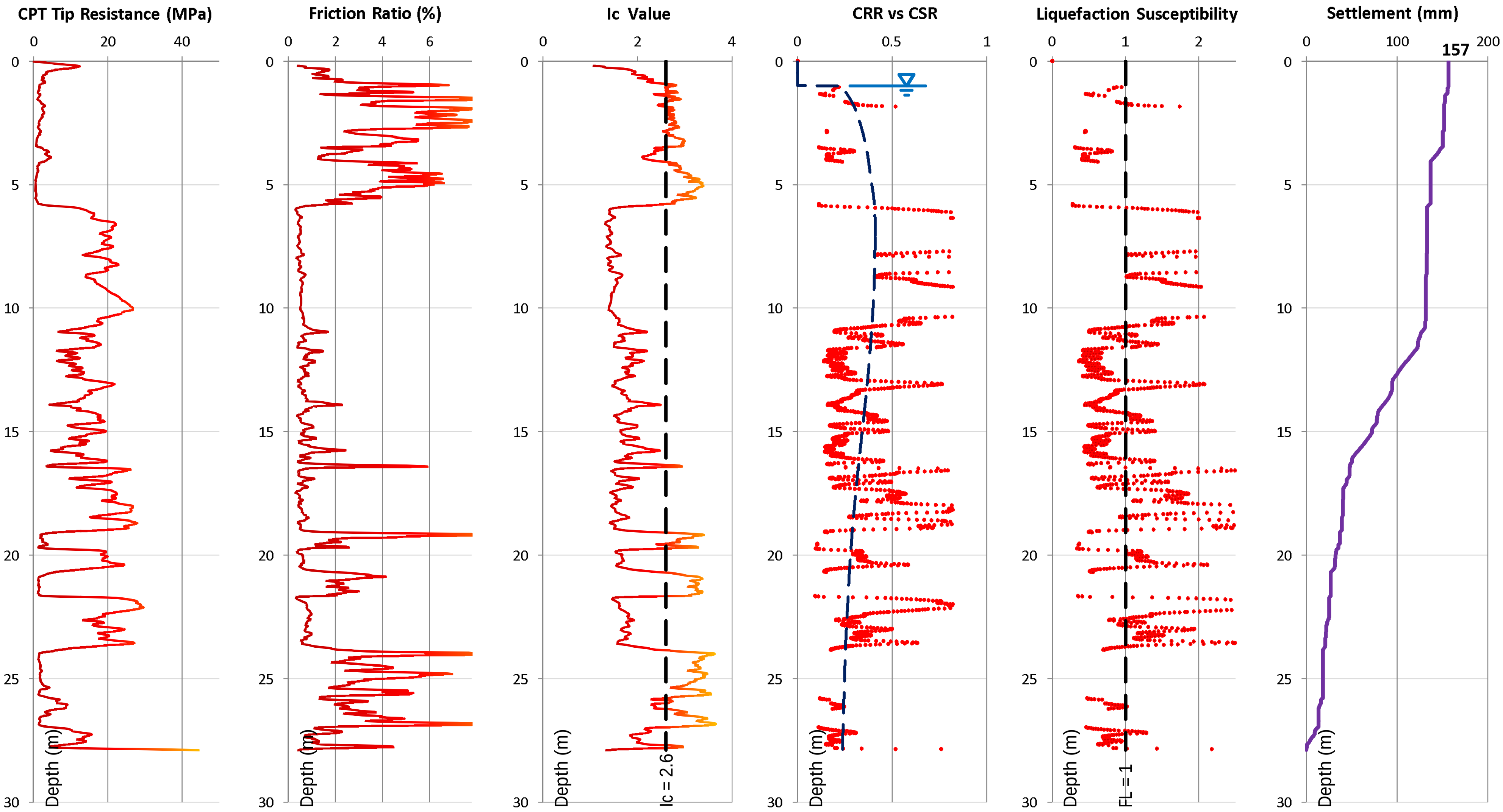
# SOIL LIQUEFACTION SUSCEPTIBILITY ASSESSMENT

| | | | |
|---|---|---|---|
| LOCATION : | CPT 2 | SHEET : | 2 |
| PROJECT : | Haast Courts - 149 Stanmore Road | CALCULATED : | HNN |
| JOB NO : | 51 30596 41 | CHECKED BY : | |
| TEST DATE : | 28 Jun 2012 | DATE : | 4 Jul 2012 |

| | | | | | |
|---|---|---|---|---|---|
| PGA ($a_{max}$): | 0.35 g | Groundwater Level (m bgl): | 1.0 | Bore depth (m): | 27.89 |
| EQ Magnitude: | 7.5 | Atmospheric Pressure (kPa): | 101 | Test data step (m): | 0.01 |

**Total Estimated Settlement (mm)**
**157**

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Appendix E

# Basis of Design

# Basis of Design

## General

The basic assumptions, design codes and references, practice advisory, material strengths and properties, and loading data used in the analysis and design are presented below.

## Codes, Standards and Design manual

### New Zealand Standard

- NZS 1170.0:2002 Structural Design Actions Part 0: General Principles
- NZS 1170.1:2002 Structural Design Actions Part 1: Permanent, Imposed and Other Actions
- NZS 1170.5:2004 Structural Design Actions Part 5: Earthquake Actions – New Zealand and the NZBC Clause B1 Structure
- NZS 3101:2006 Concrete Structures Standard
- New Zealand Society of Earthquake Engineering Guidelines for Assessment and Improvement of the Structural Performance of Buildings in Earthquakes

## Materials

The material strengths and properties used in the analysis of the existing structures are as follows:

- Seel ($f_y$): 300 MPa (assumed)
- Concrete (f'c) 25 MPa (assumed)

## Assessment Load Criteria

### Basic Assessment Information: (Block B, 2 units each 3 levels)

Properties of the structure that will be used in the structural assessment are:

| | |
|---|---|
| Height of each building: | 7.6 m |
| Dimensions of each building: | 8m x 8m (see floor plan – Drawing) |
| Site Location: | 43 Haast Street, Linwood, Christchurch, New Zealand |
| Importance level: | 2 (Residential) |

### Dead Loads

Dead load to be considered as specified in New Zealand Code (NZS 1170.1:2002)

The weights of various materials being considered in the assessment are as follows:

| | |
|---|---|
| Ceiling Fibrous Plasterboard Lining (10mm thick) | 0.09 $kN/m^2$ |
| Timber | 4.6 $kN/m^3$ |
| Concrete | 24 $kN/m^3$ |

**Live Loads**

Live loads to be considered as indicated in New Zealand Code (NZS 1170.1:2002)

| | |
|---|---|
| Roof Live Load | 0.25 $kN/m^2$ or 1.80/A + 0.12 |

**Snow Load**

Snow Load is not considered in the analysis.

**Wind Load**

Wind loading is not considered in the analysis.

**Seismic Load**

Earthquake loads shall be calculated using New Zealand Code.

| | |
|---|---|
| Site Classification | D |
| Seismic Zone factor (Z) | |
| *(Table 3.3, NZS 1170.5:2004 and NZBC Clause B1 Structure)* | 0.30 (Christchurch) |
| Annual Probability of Exceedance | |
| (*Table 3.3, NZS 1170.0:2002)* | 1/500 (ULS) Importance Level 2 |
| Annual Probability of Exceedance | |
| *(Table 3.3, NZS 1170.0:2002)* | 1/25 (SLS) |
| Return Period Factor (Ru) | |
| (*Table 3.5, NZS 1170.5:2004)* | 1.0 (ULS) |
| Return Period Factor (Rs) | |
| *(Table 3.5, NZS 1170.5:2004 and NZBC Clause B1 Structure)* | 0.33 (SLS) |
| Ductility Factor ($\mu$) | 1.25 |
| Performance Factor (Sp) | 0.925 |
| Gravitational Constant (g) | 9.81 $m/sec^2$ |
| Liquefaction Potential | high to severe |

**Seismic Mass**

| | |
|---|---|
| Total Building Mass | 128778 kg |
| | 1263 kN |

| | |
|---|---|
| Base Shear N-S | 1186 kN |
| Base Shear E-W | 1033 kN |
| Period *(Equivalent Static Method)* | 0.13 seconds |
| $k\mu$ | 1.046 |
| Structural Performance Factor, $S_p$ | 0.925 |
| Elastic Site Spectrum for Horizontal Loading, C(T) | 0.9 |
| Horiontal Design Action Coefficient, $C_d(T_1)$ | 0.795 |

**Site Description**

The site is located within Linwood, a flat suburb in east-central Christchurch.

Appendix F

# Robot Structural Analysis Panel Locations

Note: All views are from the exterior of the building

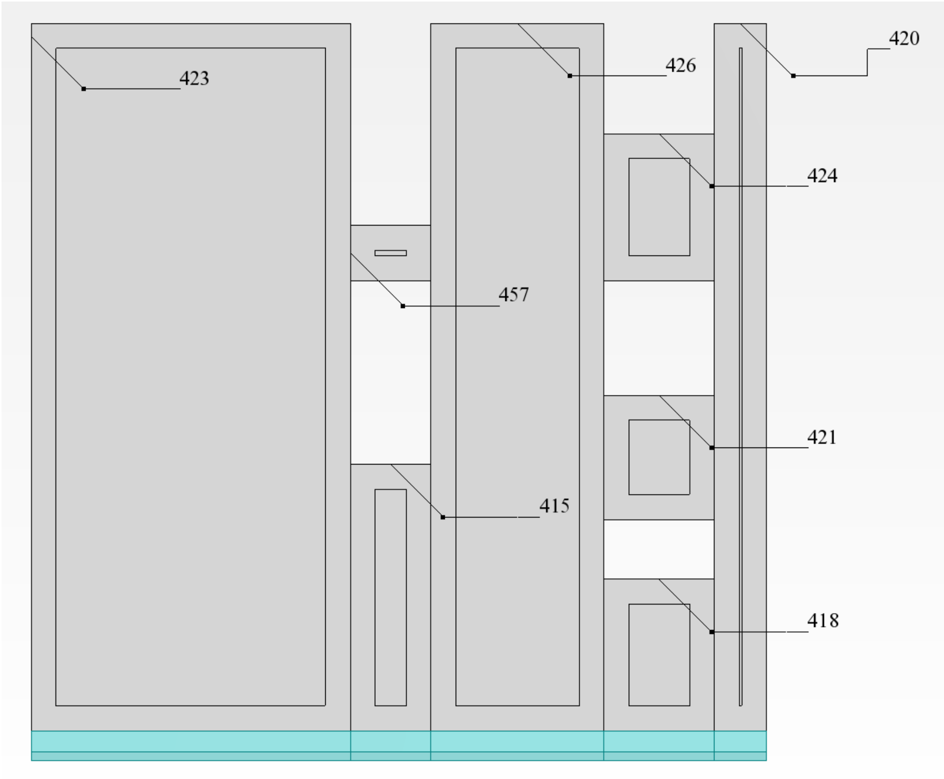


**Figure F.1 Units 5, 6 and 7 Northern Wall**

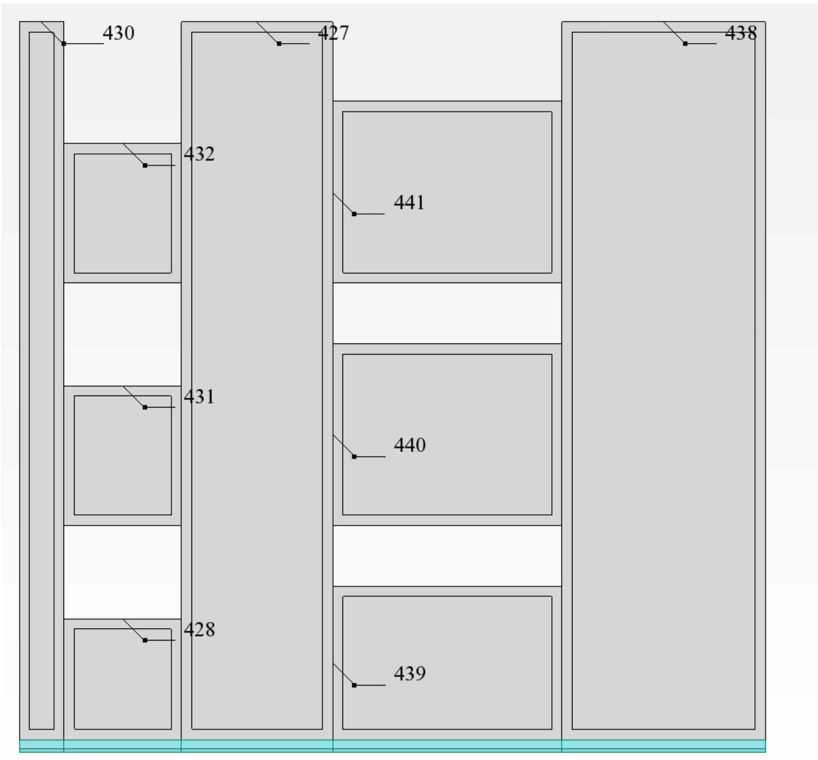


**Figure F.2 Units 5, 6 and 7 Southern Wall**

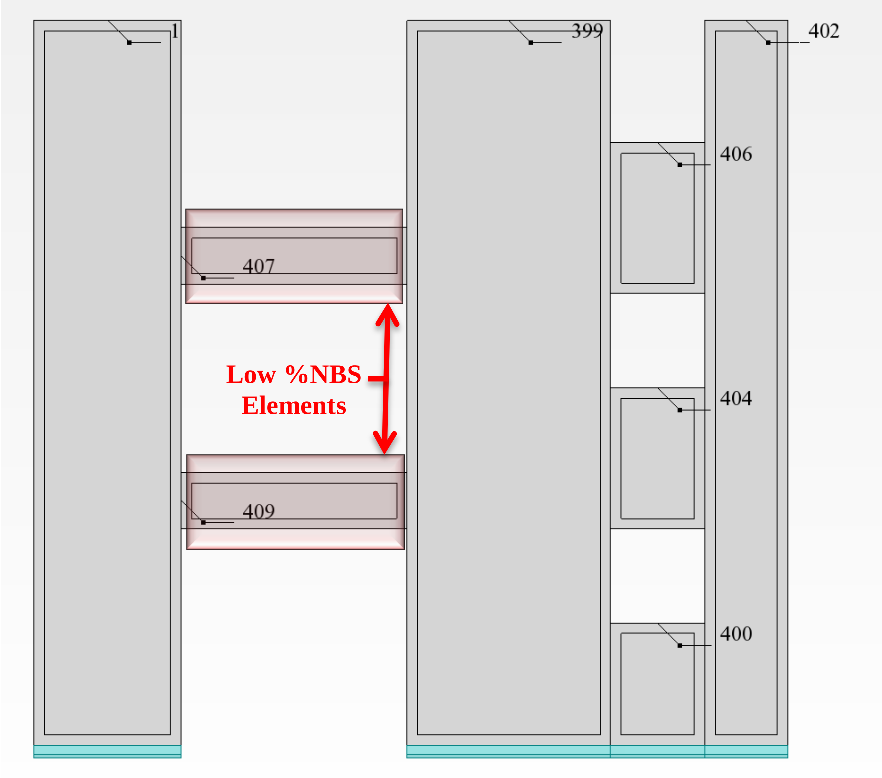


**Figure F.3 Units 5, 6 and 7 Western Wall**

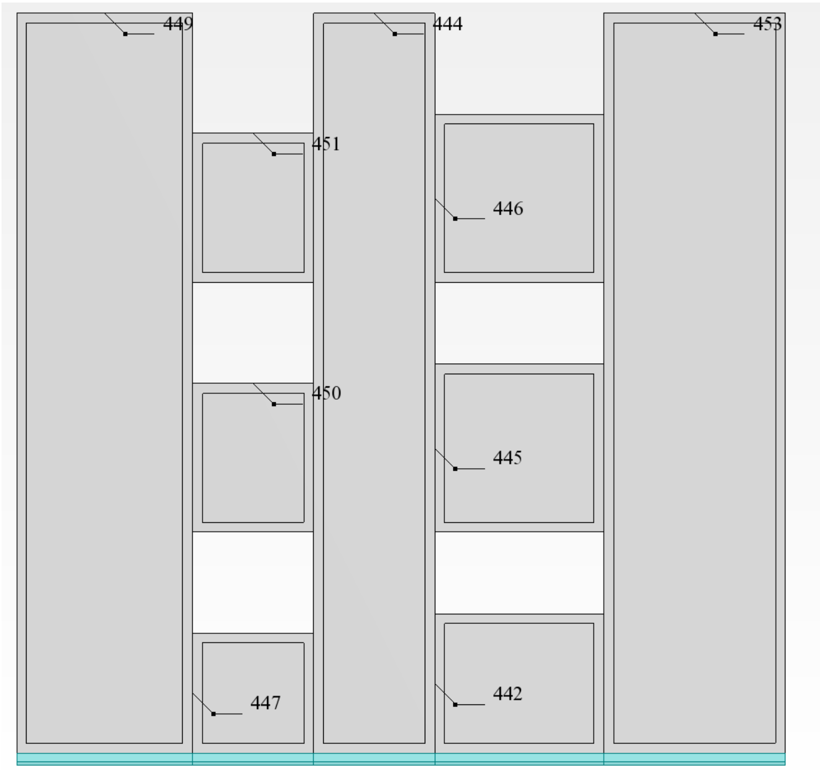


**Figure F.4 Units 5, 6 and 7 Eastern Wall**

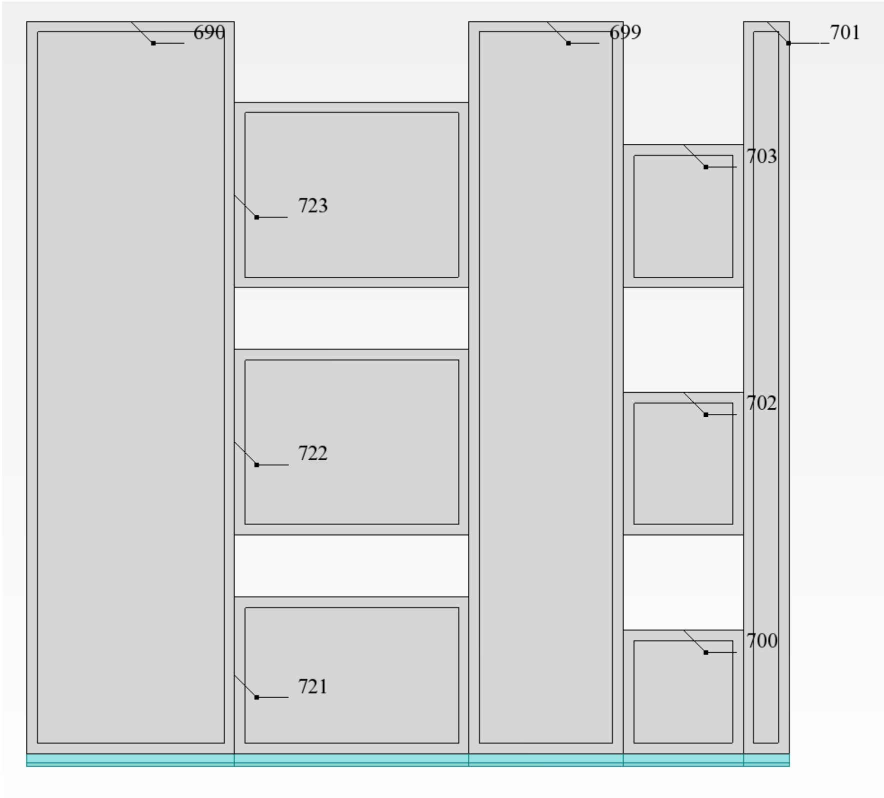


**Figure F.5 Units 8, 9 and 10 Northern Wall**

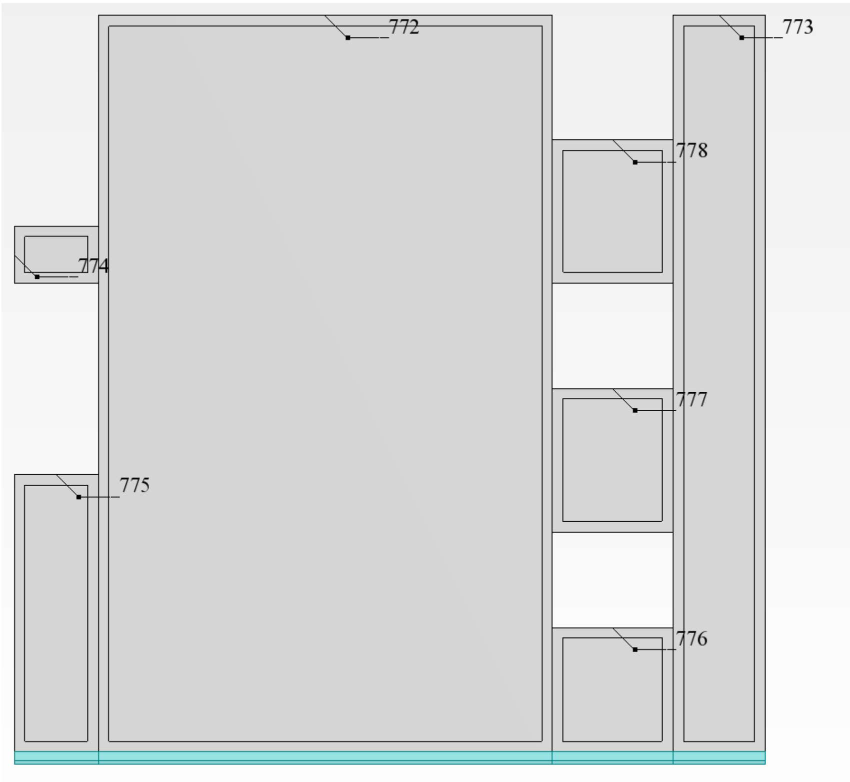


**Figure F.6 Units 8, 9 and 10 Southern Wall**

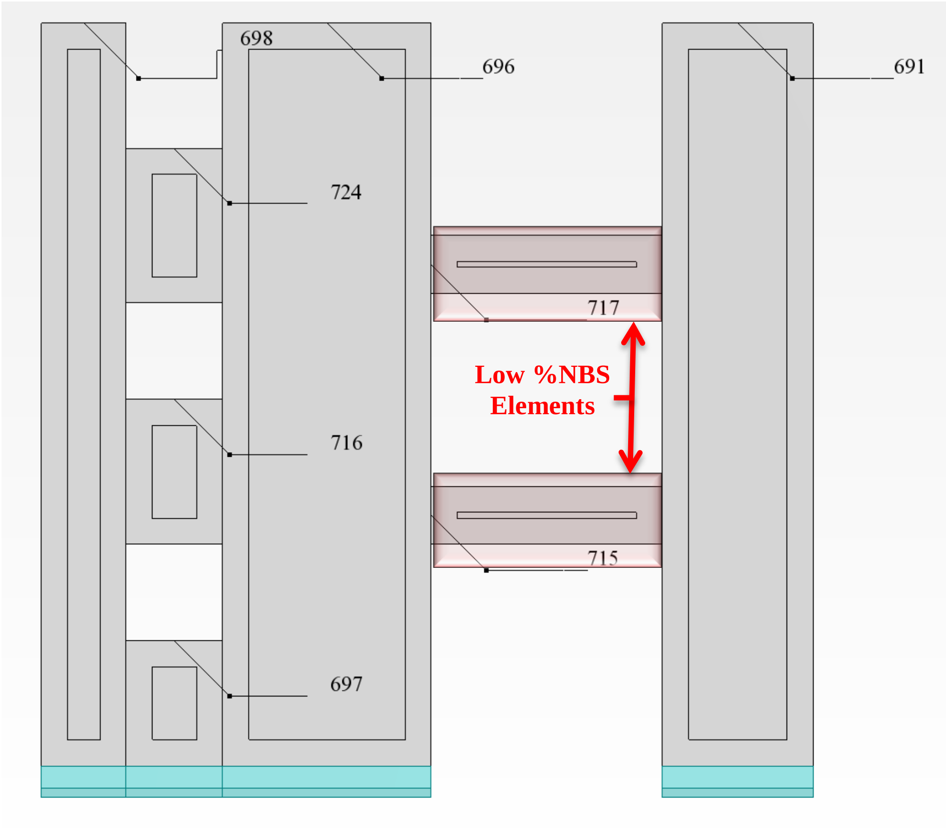


**Figure F.7 Units 8, 9 and 10 Western Wall**

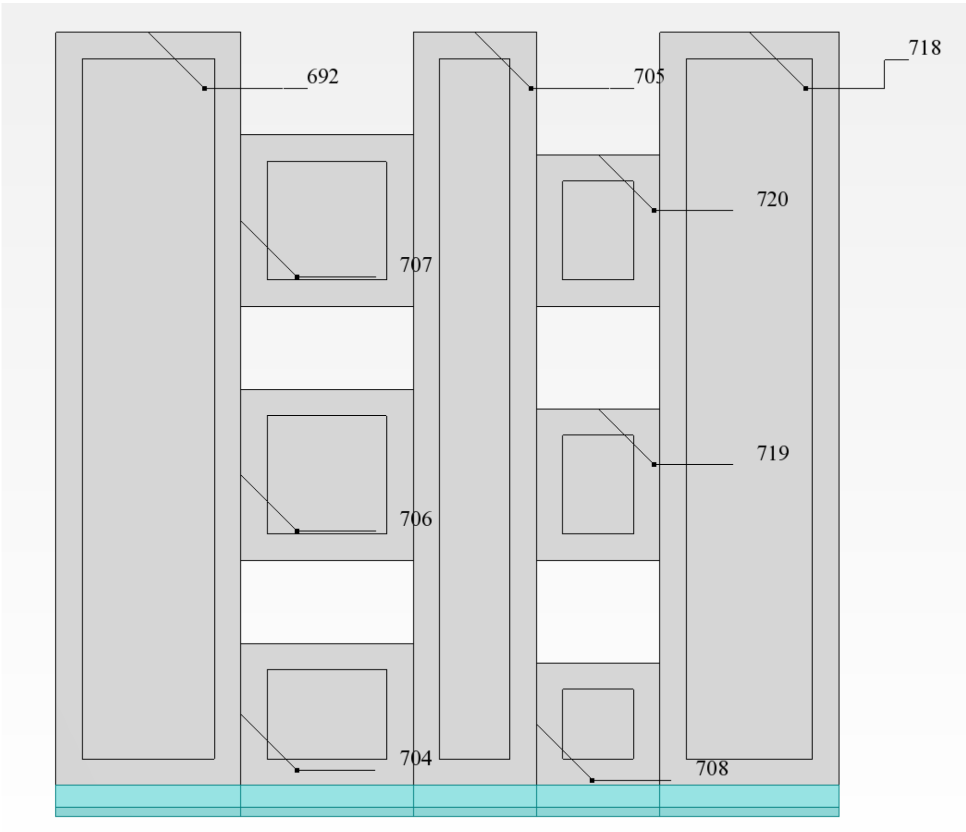


**Figure F.8 Units 8, 9 and 10 Eastern Wall**

**GHD**

GHD Building
226 Antigua Street, Christchurch 8013
T: 64 3 378 0900   F: 64 3 377 8575   E: chcmail@ghd.com



**Document Status**

| Rev No. | Author | Reviewer | | Approved for Issue | | |
|---|---|---|---|---|---|---|
| | | Name | Signature | Name | Signature | Date |
| Final | Peter O'Brien | David Lee | DLee | Nick Waddington | | 31/10/12 |
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