Knightsbridge Lane Complex
PRO 1265
Detailed Engineering Evaluation
Quantitative Report
Version 2.0 - Final

Knightsbridge Lane, Aranui

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

Knightsbridge Lane Complex
PRO 1265
Detailed Engineering Evaluation
Quantitative and Strengthening Report
Version 2.0 - Final

Knightsbridge Lane, Aranui

Christchurch City Council

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PRO 1265

Detailed Engineering Evaluation

Quantitative Report - SUMMARY

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Knightsbridge Lane, Aranui

Background

This is a summary of the Quantitative report, and subsequent strengthening, for the buildings that form the Knightsbridge Lane Housing Complex. It is based in general on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, visual inspections on 2 November 2012 and available drawings itemised in Section 5.2.

Building Descriptions

The Knightsbridge Lane Residential Housing Complex consists of single storey multi residential block buildings and is located on Knightsbridge Lane in Aranui. The original buildings were designed in 1976 and consist of 4 Blocks comprising a total of 17 one bedroom residential units. The buildings are solely used as residential housing. Blocks A and B are similar and consist of 5 one bedroom units. Block C consists of 3 one bedroom units and Block D consists of 4 one bedroom units.

Key Damage Observed

Cracking in the plaster lining between the timber framed walls and the concrete masonry walls was observed in all units in Blocks A, B, C and D. Cracking in the plaster lining between the ceiling and the concrete masonry walls was also observed.

Cracking was also observed in all of the units at the corners of windows and door frames.

The site experienced some liquefaction during recent seismic activity. The site is considered to have a low to moderate susceptibility to liquefaction. No damage to the buildings caused by liquefaction induced settlement was observed.

Additional damage specific to each block is listed below.

Block A

A collapsed section of brick masonry veneer was observed at the entrance to Unit 4. Emergency repairs have been carried out to remove the remaining section of brick veneer and to board up the exposed timber wall.

Block B

No additional damage, apart from that noted above, was observed in the block.

Block C

The external brick masonry veneer on the timber framed gable walls at the transverse ends of Block C had collapsed during the seismic activity. Emergency repairs were carried out to board up the exposed timber framed walls with props erected to hold the plywood boards in place.

Water damage to the ceiling in Unit 12 was observed. This is likely to be unrelated to the recent seismic activity.

Block D

Step cracking in the mortar joints along the top of the reinforced concrete masonry wall separating Units 16 and 17 have been repaired.

The doors in Unit 16 have been eased to allow them to close.

Building Capacity Assessment and Strengthening

Following a quantitative assessment Blocks A, B, C and D were assessed to have a seismic capacity in the order of 22% NBS and were deemed to be Earthquake Prone. As a result GHD were engaged by the Christchurch City Council to develop a strengthening solution to achieve a minimum of 67%NBS, and to replace the blockwork veneer gable ends with lightweight cladding.

Strengthening works, involving the installation of Gib bracing elements were commenced on the 31st of May 2013, and completed on all Blocks on the 20th of September. A summary of the strengths pre and post earthquake of each block is outlined in the table below.

Knightsbridge Lane Social Housing Complex	Asset Code	Strength (Pre Repairs)	Strength (Post Repairs)
Block A (Units 1,2,3,4,5)	PRO 1265 B001	22% NBS	73% NBS
Block B (Units 6,7,8,9,10)	PRO 1265 B002	22% NBS	73% NBS
Block C (Units 11,12,13)	PRO 1265 B003	22% NBS	72% NBS
Block D (Units 14,15,16,17)	PRO 1265 B004	22% NBS	72% NBS

Recommendations

As the buildings are no longer deemed to be low strength buildings no further action is required to satisfy the Christchurch City Councils Earthquake Prone buildings policy.

1. Background

GHD has been engaged by Christchurch City Council (CCC) to undertake a detailed engineering evaluation, and strengthening design, for the Knightsbridge Lane Complex in Aranui.

This report is a Quantitative Assessment of the building structure, and is based in general on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011.

A quantitative assessment involves a full site measure of the building which is used to determine the buildings bracing capacity in accordance with manufacturers' guidelines where available. When the manufacturers' guidelines are not available, values for material strengths are taken from Table 11.1 of the NZSEE guidelines for the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes. The demand for the building is determined in accordance with NZS 3604: 2011 and the percentage of New Building Standard (%NBS) is assessed.

At the time of this report, no intrusive site investigation of the building structure had been carried out. The detailed analysis and strengthening design was carried out to achieve a minimum of 67%NBS.

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 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 3.1 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 3.1 NZSEE Risk Classifications Extracted from table 2.2 of the NZSEE 2006 AISPBE

Figure 3.2 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

Figure 3.2 %NBS compared to relative risk of failure

4. Building Descriptions

4.1 General

The Knightsbridge Lane Residential Housing Complex consists of single storey multi residential block buildings and is located on Knightsbridge Lane in Aranui. The original buildings were designed in 1976 and consist of 4 Blocks comprising a total of 17 one bedroom residential units. The buildings are solely used as residential housing. The layout and orientation of the housing blocks are shown below. All blocks have a similar layout and are constructed from similar materials.

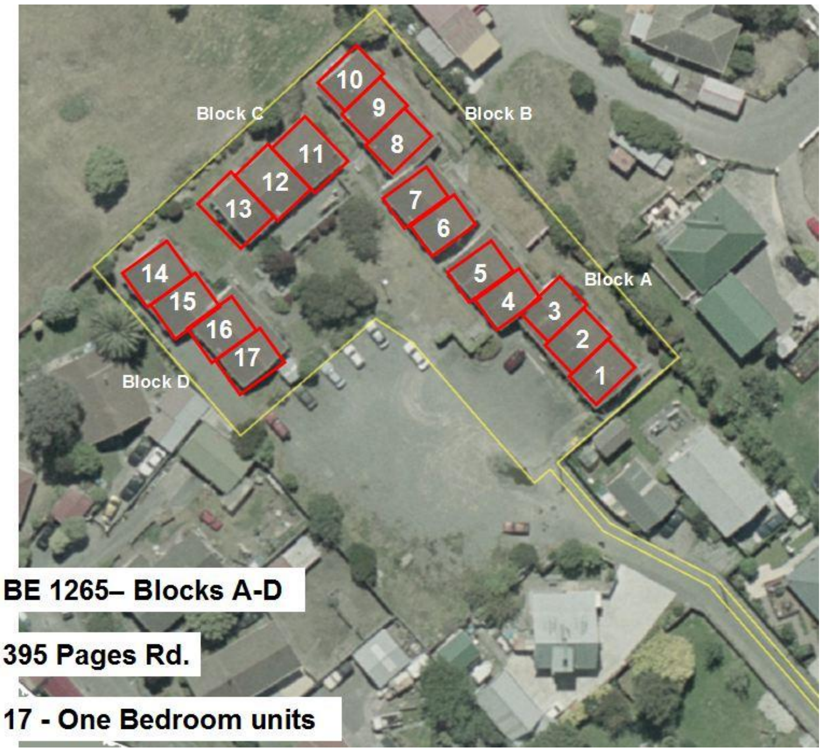


Figure 4.1 Layout of housing blocks

Blocks A and B are similar and consist of 5 one bedroom units. Block C consists of 3 one bedroom units and Block D consists of 4 one bedroom units. Block A and Block B each have dimensions of approximately 29m long, 7.5m wide and 4.4m in height. The overall footprint of these blocks is approximately 214m^2. Block C has dimensions of approximately 17m long, 7.5m wide and 4.4m in height. The overall footprint of Block C is approximately 128m^2. Block D has dimensions of approximately 23m long, 7.5m wide and 4.4m in height. The overall footprint of Block D is approximately 171m^2.

The structure of these buildings consists of timber framed walls lined internally with plasterboard and clad externally with a brick masonry veneer. The timber framed walls have studs at 600mm centres. Adjacent individual residential units are separated by 190mm thick reinforced concrete masonry walls. The concrete masonry walls are reinforced with 12mm diameter vertical bars placed centrally at 600mm centres. A bond beam reinforced with 2 No. 12mm diameter bars runs along the length of the masonry walls at eaves level.

The roof structure consists of timber nail plate roof trusses (shown in Photograph 12) clad with concrete roof tiles. The timber nail plate trusses are spaced at 900mm centres. The ceiling in each residential unit is lined with plasterboard.

The brick masonry cladding on the exterior of the buildings is unreinforced. This is visible in the collapsed gable ends of Block C (see Photographs 5 and 6). There is a 37mm cavity between the timber framed walls and the brick masonry veneer. The brick masonry veneer is restrained with galvanised brick ties.

The foundations of the buildings consist of a concrete slab-on-grade reinforced with 665 Mesh and 500x250mm concrete strip footings beneath the external walls reinforced with 2 No. 12mm diameter bars with 6mm diameter stirrups at 300mm centres. The foundations of the reinforced concrete masonry walls consist of ground beams reinforced with 4 No. 12mm diameter bars with 6mm diameter stirrups at 300mm centres.

Figures 4.2 and 4.3 show the construction details typical to all blocks.

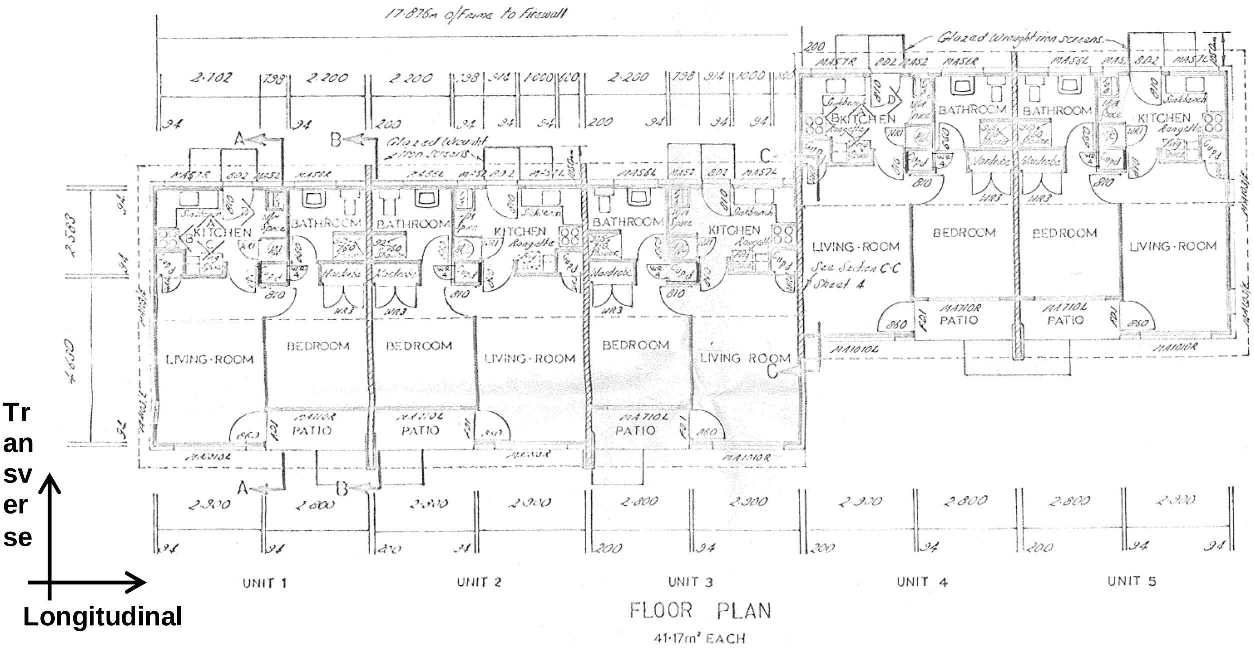


Figure 4.2 Typical Plan of Blocks A & B

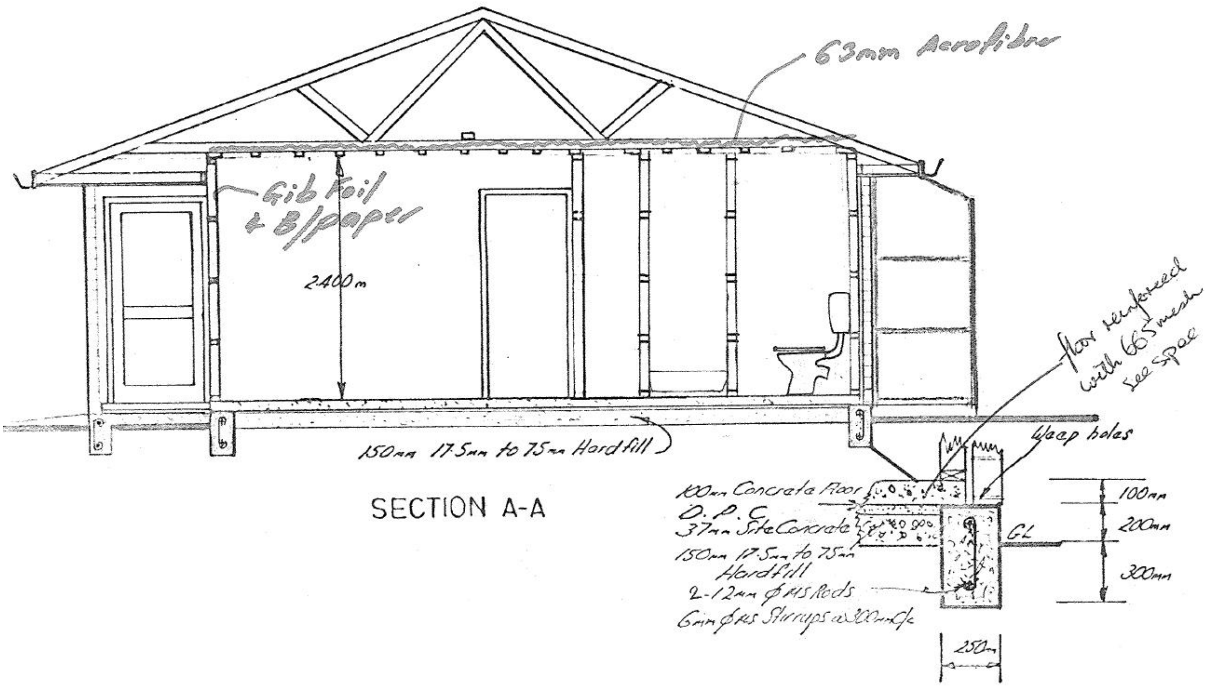


Figure 4.3 Typical Section of a Housing Unit

4.2 Gravity Load Resisting Systems

Gravity loads acting on the buildings are resisted by load bearing timber framed walls. Gravity loads from the concrete roof tiles are transferred via the timber nail plate trusses to the timber framed walls. The gravity loads are transferred through the timber framed walls to the concrete strip footings where they are distributed into the ground. Floor gravity loads are transferred through the reinforced concrete slab to the underlying ground.

4.3 Lateral Load Resisting Systems

The plasterboard lined ceiling to the underside of the timber roof trusses in each residential unit provide a diaphragm to transfer seismic forces in the roof structure to the lateral load resisting walls supporting the diaphragm. The timber framed roof has diagonal timber braces in the plane of the roof which braces the roof structure and allows forces to be transferred to the diaphragm in the ceiling plane.

Lateral seismic loads in the longitudinal direction are resisted by the plasterboard lined timber framed walls which act as in-plane bracing panels. The external walls are likely to have steel diagonal bracing straps or angles present as these are shown on the elevations of the available drawings.

Due to the insufficient lengths of plasterboard lined timber framed walls available to brace the buildings in the longitudinal direction there is effectively only one bracing line through each residential unit. The layout of the bracing elements is therefore asymmetric. The external timber framed walls provide minimal bracing to the structure.

Lateral seismic loads in the transverse direction are resisted by the reinforced concrete masonry walls that separate adjacent residential units. The lateral forces are resisted by the panel action of concrete masonry units. Loads are transferred to the foundations through shear and bending of the concrete masonry walls.

Due to the relatively stiff nature of the concrete masonry walls compared to the timber framed walls in the transverse direction, it is likely that the majority of lateral seismic loads will be taken by the concrete masonry walls. As a result, the contribution of the timber framed walls in the transverse direction to the overall lateral load resisting capacity is negligible.

The 190mm thick concrete masonry partition walls are restrained at eaves level by the plasterboard ceiling diaphragm and along the top edge by the timber roof framing. Out-of-plane loading on these walls is likely to be resisted by the walls spanning vertically between the supporting ground beams and the ceiling diaphragm restraining the walls.

5. Assessment

5.1 Site Inspection

An inspection of the buildings was undertaken on the 2nd of November 2012. Both the interior and exterior of each unit was inspected. Most of the main structural components of the building were internally and externally lined and were unable to be viewed. It should be noted that inspection of the foundations of the structure was limited to the top of the external strips exposed above ground level.

The inspection consisted of observing the building to determine the structural systems and likely behaviours of the building during earthquake. The site was assessed for damage, including observing the ground condition, checking for damage areas where damage would be expected for the structure type observed and noting general damage observed throughout the building in both structural and non-structural elements.

A Hilti PS 200 Ferroscan was used to confirm the position, depth and diameter of the reinforcement in the concrete masonry walls. The scanning equipment confirmed that the reinforcing in the concrete masonry walls is as detailed in the available drawings.

5.2 Available Drawings

The construction drawings of the original structure have been made available.

Key drawings are attached as Appendix B.

5.3 Damage Assessment

5.3.1 Surrounding Buildings

No significant damage to the surrounding buildings was observed during inspections.

5.3.2 General Observations

Cracking in the plaster lining between the timber framed walls and the concrete masonry walls (see Photograph 11) was observed in all units in Block A, B, C and D. Cracking in the plaster lining between the ceiling and the concrete masonry walls was also observed. The cracking to the linings is likely due the difference in stiffness between the concrete masonry walls and timber framed walls causing the walls to deflect differentially during an earthquake.

Cracking was also observed in all of the units at the corners of windows and door frames where stresses are likely to have been concentrated during an earthquake.

Additional damage observed during inspections of each block is listed below.

<u>Block A</u>

A collapsed section of brick masonry veneer was observed at the entrance to Unit 4. Minor repairs have been carried out to remove the remaining section of brick veneer and to board up the exposed timber wall.

Block B

No additional damage, apart from that noted above, was observed in the block.

Block C

The external brick masonry veneer on the timber framed gable walls at the transverse ends of Block C have collapsed during the recent seismic activity as shown in Photographs 5 and 6. Repairs have been carried out to board up the exposed timber framed walls. Props have been erected to hold the plywood boards in place.

Water damage to the ceiling in Unit 12 was observed. This is likely to be unrelated to the recent seismic activity.

Block D

Step cracking in the mortar joints along the top of the reinforced concrete masonry wall separating Units 16 and 17 was observed during inspections. The tenants have indicated that these are pre-existing cracks that have opened further during the recent seismic activity.

The doors in Unit 16 do not close properly suggesting that some settlement of the building's foundations has occurred.

5.3.3 Ground Damage

Evidence of liquefaction was observed in the Knightsbridge Lane Complex car park. This is shown in Photograph 10. No damage to the buildings caused by liquefaction induced settlement was observed.

5.3.4 Level Survey

A level survey of all units within the blocks was undertaken during the inspection of the site on 2 November 2012. The survey was carried out with a zip level, using the entrance to each unit as the datum point. Levels were taken at the corners of each room in the units where accessible.

Units 11, 12 and 13 in Block C have the largest recorded differential settlement of 42mm across the building. Relative settlement of up to 22mm was recorded in Unit 6 of Block B. The remaining Units 7 to 10 in Block B have differential settlement of less than 12mm. Blocks A and D have differential settlement of less than 14mm. These settlements will not affect the seismic performance of the buildings.

6. Geotechnical Consideration

The site is situated in the suburb of Aranui, east of Christchurch City centre. It is relatively flat at approximately 4 m above mean sea level. It is approximately 1.2 km southwest of Avon River, 2.3 km northwest of the Avonhead Heathcote Estuary, and 2.5 km west of the coast (Pegasus Bay).

6.1 Published Information on Ground Conditions

6.1.1 Local Geology

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

- Dominantly sand of fixed and semi-fixed dunes and beaches, Holocene in age, of the Christchurch formation;
- The Riccarton gravels are located approximately 39 m bgl; and
- Groundwater is likely within 1 m of ground level.

6.1.2 Environment Canterbury Records

Information from Environment Canterbury (ECan) indicates that there are seven boreholes located within 200 m of the site. Three of these logs are shown in Table 6.1.

These indicate that the area is underlain by sand.

Table 6.1 ECan Borehole Summary

Bore Name	Log Depth	Groundwater	From Site	Log Summary
M35/2014	81 m	Not recorded	170 m W	0 – 32.3 m Sand 32.3 – 37.7 m Clay 37.7 – 50.2 m Gravel
M35/13323	2.23 m	Not recorded	100 m NE	0 – 2.23 m Sand
M35/16509	1.8 m	Not recorded	180 m SE	0 – 0.3 m Topsoil 0.3 – 1.8 m Sand

It should be noted that 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.

6.1.3 EQC Geotechnical Investigations

The Earthquake Commission has undertaken geotechnical testing in the area of the site which is included in the Tonkin & Taylor Report for Wainoni[2]. Two investigation points were undertaken within 200 m of the site, as summarised below in Table 6.2.

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

Table 6.2 EQC Geotechnical Investigation Summary Table

Bore Name	Orientation from Site	Depth (m bgl)	Log Summary[3]
CPT-WAI-71	180 m E	0 – 1.2	Pre-drilled
		1.2 – 1.7	CLAY, stiff
		1.7 – 29.9	SAND, medium dense to dense
			(WT at 3.2 m bgl)
CPT-WAI-72	150 m W	0 – 1.2	Pre-drilled
		1.2 – 24.9	SAND, medium dense to dense
			(WT at 1.4 m bgl)

The CPT results indicate the soils are medium dense to dense.

6.1.4 CERA Land Zoning

Canterbury Earthquake Recovery Authority (CERA) has indicated the site is situated within the Green Zone, indicating that repair and rebuild may take place.

Land in the CERA green zone has been divided into three technical categories. These categories describe how the land is expected to perform in future earthquakes.

The site has been categorised as “Technical Category 2”. This means that minor to moderate land damage from liquefaction is possible in future significant earthquakes.

6.1.5 Historical Aerial Photography

Shallow fill is indicated from the CCC Landfill Map[4]. Aerial Photos taken in 1946[5] and 1955[6] show no signs of filling, and instead show a small forest to the north of the property.

6.2 Post-Earthquake Land Observations

6.2.1 Aerial Photography

Aerial photography was taken after each of the major earthquake events. Photos taken following the 4 September 2010 show no signs of liquefaction on the site or in the wider area. Those taken following the 22 February 2011 earthquake show moderate signs of liquefaction in the car park. Signification surface flooding, presumed to be from ejected liquefaction water is evident in the sports field at the rear

[2] Tonkin & Taylor Ltd., 2011: Christchurch Earthquake Recovery, *Geotechnical Factual Report, Wainoni.*

[3] Log Summary for CPT’s interpreted from Soil Behavior Type Robertson 2010

[4] Map of the “Christchurch Landfill Sites”, Christchurch City Council, 29 September 1995

[5] Aerial Photography of, Burwood, Greater Christchurch, taken 30/05/1946, provided by Christchurch City Council

[6] Aerial Photography of Burwood, Greater Christchurch, 2nd Edition, taken 10/05/1955, provided by Christchurch City Council

of the property, as shown in Figure 6.1. Photos from the June 2011 event show reactivation of sand boils in the car park and the sports field resulting in minor liquefaction.

Figure 6.1 Post February 2011 Earthquake Aerial Photography[7]

6.3 Field Observations

During the site investigation the following observations were noted. The brick cladding of the gable ends of units 13 and 11 had suffered damage. Localised minor liquefaction was evident in many of the grassed areas in the gardens of the units.

No significant ground damage due to ground cracking, from neither sand ejection, nor cracking from lateral spread was observed.

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

6.4 Seismicity

6.4.1 Nearby Faults

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

Table 6.3 Summary of Known Active Faults[8,9]

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	130 km	NW	~8.3	~300 years
Greendale Fault (2010)	27 km	W	7.1	~15,000 years
Hope Fault	100 km	N	7.2~7.5	120~200 years
Kelly Fault	110 km	NW	7.2	150 years
Port Hills Fault (2011)	7 km	S	6.3	*Not estimated*

The recent earthquakes since 4 September 2010 have identified the presence of a previously unmapped active fault system underneath the Canterbury Plains; these include the Greendale Fault and Port Hills Fault listed in Table 6.3. Research and published information on this system is in development and the average recurrence interval is yet to be established for the Port Hills Fault.

6.4.2 Ground Shaking

The recent 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 and 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.

Conditional PGA's from the Canterbury Geotechnical Database (CGD)[10] indicate the PGA to be 0.19g during the 4 September 2010 earthquake, 0.49g on 22 February 2011, and 0.30g on 13 June 2011.

6.5 Slope Failure and Rockfall Potential

Given the site's location in Aranui, global slope instability is considered negligible. However, any localised retaining structures or embankments should be further investigated to determine the site-specific slope instability potential.

[8] 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.

[9] GNS Active Faults Database

[10] Canterbury Geotechnical Database (2012): "Conditional PGA for Liquefaction Assessment", Map Layer CGD5110 - 27 Sept 2012, retrieved 31/10/2012 from https://canterburygeotechnicaldatabase.projectorbit.com/

6.6 Field Investigations

The geotechnical field investigation comprised a site walkover, three hand augers (HA01 – HA03) with Scala penetrometer tests, and one cone penetrometer test (CPT01). The CPT was located centrally on the site to give a site wide assessment; additional locations were not possible due to access restrictions and services. The Hand augers were focussed around Block C, (Units 11, 12, 13) where damage was observed and the worst floor level survey was recorded. The investigation layout is shown in Figure 2 and the GPS locations of the tests are tabulated in Table 6.4 below.

Table 6.4 Investigation Locations

Borehole Number	Depth (m bgl)	Northing	Easting
CPT01	22.0	5743580	2486119
HA01	2.4	5743612	2486100
HA02	2.4	5743600	2486087
HA03	2.3	5743590	2486086

The CPT was undertaken by McMillan Drilling Services and all site work was undertaken on 6 November, 2012.

Figure 6.2 Investigation Location Plan

6.7 Ground Conditions Encountered

A summary of the ground conditions encountered in the hand augers are shown in Table 6.5.

Table 6.5 Summary of Hand Auger and DCPs

Depth (m)	Lithology	DCP blows per 100 mm
0 – 0.2	Organic SILT with rootlets, firm	3 – 4
0.2 – 2.4	SAND, loose to medium dense	2 – 12

Detailed engineering bore logs can be found in Appendix D.

A summary of the soil behaviour type determined from the CPT results is shown in Table 6.6.

6.7.1 Groundwater

Whilst groundwater was not recorded in the field investigation, the borelogs indicate water is at 1.4m bgl.

6.8 Liquefaction Assessment

Due to the observed liquefaction and the anticipated presence of loose/soft alluvial soils a comprehensive liquefaction analysis has been undertaken.

6.8.1 Parameters used in Analysis

Assumptions made for the analysis process are as follows:

- Importance Level 2, 50-year design life, giving peak ground accelerations (PGA's) of:
    - → 0.35g for Ultimate Limit State (ULS), and
    - → 0.13g for Serviceability Limit State (SLS);
- Earthquake Magnitude 7.5; and
- Groundwater levels at 1.4m bgl.

Soil unit weights have been approximated using the tip resistance and sleeve friction from the CPT investigation data using formulae from Robertson & Cabal.

The liquefaction analysis process has been conducted using the methodology from Robertson & Wride, and from the NZGS Guidelines. Settlements were estimated using the methodology outlined in Zhang et al (2002).

6.8.2 Results of Liquefaction Analysis

The results of the liquefaction analysis, as outlined in Table 6.6, indicate that several layers are moderately liquefiable.

Please refer to Appendix D for further detail.

Table 6.6 Summary of Liquefaction Susceptibility

Depth (m)	Soil Behaviour Type	Liquefaction Susceptibility [11]
0.0 – 1.4	SANDS	Above the water table
1.4 – 3.2	SANDS	low
3.2 – 3.6	SANDS	Moderate
3.6 – 6.0	SANDS	low
6.0 – 8.8	SANDS	Moderate
8.8 – 10.3	SANDS	low
10.3 – 14.8	SANDS	Low to Moderate
14.8 – 15.6	SANDS	low
15.6 – 20.0	SANDS	Low to Moderate

Settlement estimates for the CPT locations are listed in Table 6.7.

Table 6.7 Estimated Liquefaction Induced Settlements

CPT Number	ULS	SLS	SLS Index Value (top 10 m)
CPT01	88 mm	0 mm	0 mm

The SLS index value reflects the vertical settlement of the shallow soils (<10m) for an SLS event.

Please refer to Appendix D for further details.

6.8.3 "Sufficiently Tested at SLS"

Since the PGA for 22 February (0.49g) exceeds 170% of the magnitude-corrected SLS value (0.30g), the site can be considered "sufficiently tested at SLS". As a result, the ground damage during a future moderate earthquake (SLS) is likely to be similar or less than that observed in the 22 February 2011 earthquake.

6.8.4 Liquefaction Summary

The site is considered to have a low to moderate susceptibility to liquefaction based of the following:

- Observations of minor liquefaction on the site from post-earthquake aerial photography with no clear signs of liquefaction directly outside the structures' footprints;
- Surrounding properties are classified TC2;
- Estimated ULS and SLS settlements are consistent with TC2 classification.

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

- Presence of several liquefiable layers identified in liquefaction assessments;

There was moderate to significant liquefaction observed in the neighbouring field and carpark. The surface flooding in the playing fields could be attributable to over compaction of the indicated historic fill.

The liquefaction analysis indicates discrete narrow layers of moderately liquefaction susceptible layers at 3 m and 6m bgl.

6.8.5 Summary and Recommendations

The subject structure has remained operational throughout the Canterbury earthquake sequence.

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

- A soil class of **D** (in accordance with NZS 1170.5:2004) should be adopted for this site;
- The site has a low to moderate susceptibility to liquefaction.
- The site behaviour is consistent with the TC2 classification which indicates that minor to moderate land damage may occur from future earthquakes.

7. Structural Analysis

7.1 Seismic Parameters

Seismic loading on the structure has been determined using New Zealand Standard 1170.5:2004.

◗ 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
◗ Return Period Factor (R_u)	
(Table 3.5, NZS 1170.5:2004)	1.0 (ULS)
Longitudinal Direction	
◗ Ductility Factor (μ)	3.0
◗ Ductility Scaling Factor (k_μ)	2.14
◗ Performance Factor (S_p)	0.7
Transverse Direction	
◗ Ductility Factor (μ)	1.25
◗ Ductility Scaling Factor (k_μ)	1.14
◗ Performance Factor (S_p)	0.925

An increased Z factor of 0.3 for Christchurch has been used in line with recommendations from the Department of Building and Housing recommendations.

The structural performance factor, S_P, was calculated in accordance with CL 4.4.2 NZS 1170.5.

$$S_P = 1.3 - 0.3\mu$$

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 0.4 was assumed for both direction of the building. The coefficient was then calculated using Equation 5.2(1);

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

Where

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

7.2 Equivalent Static Method

Equivalent Static forces were calculated in accordance with NZS 1170.5:2004. In the transverse direction, the total lateral force acting on the structure has been distributed equally to each of the concrete masonry walls based on the regular layout and similar lengths of the walls in the direction. In the longitudinal direction, the distribution of lateral forces follows the bracing design procedure discussed in Section 5 of NZS 3604:2011. The loading the equivalent static loading in the longitudinal direction was resolved into bracing units (BUs) and compared to the bracing capacity of the timber walls.

A ductility factor of 1.25 has been assumed in the transverse direction based on the age of the building and the lightly reinforced concrete masonry walls resisting lateral load in this direction. The structure is expected to have nominally ductile behavior given the lightly reinforced concrete masonry construction. In the longitudinal direction, a ductility factor of 3.0 has been assumed based on the relatively flexible, lightweight timber framed walls resisting lateral load in this direction.

The elastic site hazard spectrum for horizontal loading:

Longitudinal

$C(T_1)=C_h \cdot Z \cdot R \cdot N(T,D)$

$C_h=3.0$ – Value from Table 3.1 ($T \leq 0.4s$)

Z=0.3 – Hazard factor determined from Table 3.3 (NZS 1170.5:2004)

R=1.0 – Return period factor determined from Table 3.5 (NZS 1170.5:2004)

N (T,D) = 1.0 – Near fault factor from Clause 3.1.6 (NZS 1170.5:2004)

$C(T_1)= 3.0 \cdot 0.3 \cdot 1.0 \cdot 1.0 = 0.9$

The horizontal design action coefficient:

$$C_d(T_1)=\frac{C(T_1) \cdot S_p}{k_\mu}=\frac{0.9 \cdot 0.7}{2.143}=0.294$$

Transverse

$C(T_1)=C_h \cdot Z \cdot R \cdot N(T,D)$

$C_h = 3.0$ – Value from Table 3.1 ($T \leq 0.4s$)

Z = 0.3 – Hazard factor determined from Table 3.3 (NZS 1170.5:2004)

R = 1.0 – Return period factor determined from Table 3.5 (NZS 1170.5:2004)

N (T,D) = 1.0 – Near fault factor from Clause 3.1.6 (NZS 1170.5:2004)

$C(T_1) = 3.0 \cdot 0.3 \cdot 1.0 \cdot 1.0 = 0.9$

The horizontal design action coefficient:

$$C_d(T_1) = \frac{C(T_1) \cdot S_p}{k_\mu} = \frac{0.9 \cdot 0.925}{1.143} = 0.728$$

7.3 Capacity of Structural Elements

7.3.1 Reinforced Masonry Shear Capacity

The shear capacity of the reinforced filled masonry wall was determined using NZS 4230: 2004. As there are no details as to the level of supervision during the construction stage, the Observation Type was classed accordance with Table 3.1. The strength reduction factor, ϕ, for shear and shear friction was taken as 0.75 in accordance with Cl 3.4.7. The overall shear capacity of the wall was calculated from Cl 10.3.2.1, Equation 10-4.

For reinforced concrete masonry;

$$V_m = 0.8 d b_w v_m$$

$$v_m = (C_1 + C_2) v_{bm}$$

$$C_2 = 33 p_w \frac{f_y}{300}$$

$$p_w = A_s / b_w d$$

Where

C_1 = wall proportion factor;

v_m = shear strength of masonry;

b_w = t wall thickness when fully filled;

d = 0.8 x length of wall,

A_s = area of reinforcement.

The shear capacity component from the reinforcing steel, V_S, was calculated using equation below;

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

Where

A_V = area of transverse (horizontal) reinforcing at spacing s;

f_{yt} = characteristic yield strength of the transverse steel;

7.3.2 Reinforced Masonry Out-of-Plane Moment Capacity

The following method was used to calculate the out of plane moment capacity of the reinforced masonry walls.

$$\emptyset M_n = \emptyset \left(\frac{t}{2} - \frac{a}{2}\right)\left(f_{yt} A_s + N_n\right)$$

$$a = {A_s f_{yt}}/{0.85 f'_m b}$$

Where

N_n = the axial load due to the self-weight of the wall

t = thickness of the masonry wall

b = unit width of wall

A_s = area of steel reinforcement

A_m = area of masonry

f'_m = specified compressive strength of masonry from Table 10.1 NZS 4230:2004

f_y = the strength of steel as specified by the NZSEE guidelines

7.3.1 Timber Framed Wall Bracing Capacity

The bracing capacity of the timber framed walls in the longitudinal direction was calculated in accordance with NZS 3604:2011 and the NZSEE guidelines. The demand for the building was calculated in accordance with NZS 1170.5:2004 and resolved into Bracing Units (BUs) for comparison.

There is no reliable information available regarding the bracing capacities of the plasterboard lining to the timber framed walls as the building was constructed in 1976. Assumptions regarding the likely bracing capacity of the plasterboard lined timber walls have been made in accordance with Table 11.1 of the in NZSEE guidelines. A bracing capacity value of 3 kN/m (60 BU/m) and a strength reduction factor of 0.7 have been used in calculations.

Section 11.4 of the NZSEE guidelines suggests that shear panels may utilise their full bracing capacity for aspect ratios (height-to-width) up to 2:1. For aspect ratios greater than 2:1 and up to 3.5:1 a limiting factor may be applied in accordance with the NEHRP Recommended Provisions (BSSC, 2000) as follows;

$$\text{Aspect Ratio Factor} = \frac{2 \times \text{Width}}{\text{Height}}$$

Any sections of wall with an aspect ratio greater than 3.5:1 were not included in the bracing calculations.

The buildings were also checked against the current requirements in NZS 3604:2011 for spacing of bracing lines, minimum bracing line values, diaphragm spans and the bracing capacities of walls supporting diaphragms.

7.3.2 %NBS

The timber framed wall bracing capacity in the longitudinal direction, the in-plane shear capacity, the in-plane bending moment capacity and the out-of-plane bending moment capacity of the concrete masonry walls were compared to their respective demands to determine the overall %NBS for each building.

$$\%NBS = \frac{BU_{\text{provided}}}{BU_{\text{demand}}} \times 100$$

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

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

8. Results

The New Zealand Society for Earthquake Engineering (NZSEE) publication "Assessment & Improvement of Structural Performance of Buildings" (2006, Ref. b) and the relevant New Zealand material standards were used to provide a framework and method for the analysis. Our analysis applied live loads, imposed dead loads and seismic loads to the structure. The elements were then assessed against their respective load capacities.

Our calculations show that the seismic load resisting systems of Blocks A, B, C and D achieve **22% NBS** and are therefore **Earthquake Prone.**

The structural analysis results are discussed in the following sections.

8.1 Blocks A & B

Blocks A and B have identical layouts and construction. As a result, both buildings have the same level of assessed seismic performance. The structural analysis results for both buildings are presented together in Section 8.1.

8.1.1 Timber Framed Walls

The bracing demand was determined by evaluating the seismic weight of the building and multiplying this value by the horizontal design action coefficient for the longitudinal direction. The demand was then resolved into bracing units (BUs) for comparison with bracing capacities of timber framed walls.

$$\mathrm{BU}_{\mathrm{demand}} = 4{,}308 \text{ BUs}$$

A comparison was made with the corresponding demand based on NZS 3604:2011 requirements. The demand calculated from NZS 3604:2011 significantly underestimated the likely seismic weight of the structure due to the presence of the heavy fully filled reinforced concrete masonry walls and brick masonry cladding.

The total bracing capacity of the building in the longitudinal direction was evaluated by determining the lengths of plasterboard lined timber framed walls available that satisfy the aspect ratio limit of 3.5:1 suggested in the NZSEE guidelines. Only a small number of sections of walls in each unit satisfy this requirement. There is a significant lack of walls capable of bracing the structure in the perimeter walls due to large penetrations. As a result, the building effectively has two lines of bracing through each unit. The layout of bracing elements and bracing lines is extremely asymmetric and contributes significantly to the assessed score of 22% NBS.

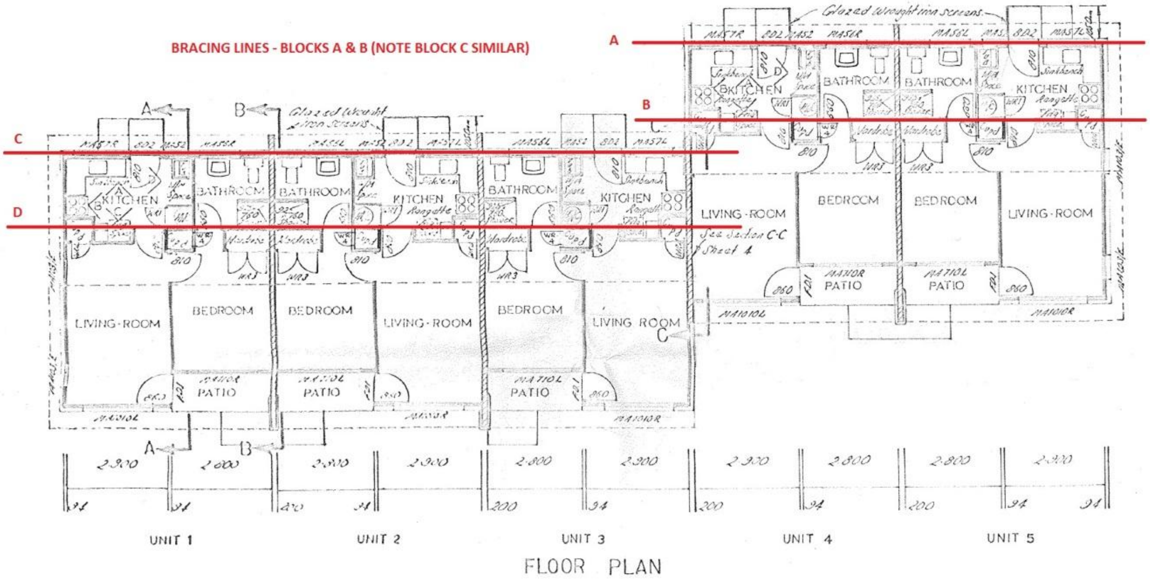


Figure 8.1 Longitudinal bracing lines for Blocks A, B and D

Bracing Line	Bracing Capacity (BU)
A	57
B	318
C	85
D	477
Total Bracing Capacity =	937 BUs

Table 8.1 Block A and B bracing line capacities

$$\%NBS = \frac{937\ BUs}{4{,}308\ BUs} = 22\%\ NBS$$

8.1.2 Reinforced Concrete Masonry Walls

<u>In-Plane Shear</u>

The reinforced concrete masonry walls achieve **100% NBS** under in-plane shear seismic loading.

The reinforced concrete masonry walls are significantly stiffer than the timber framed walls in the transverse direction. As a result, the concrete masonry walls are likely to resist the majority of lateral seismic loads in the transverse direction. The contribution of the timber framed walls to the lateral load resisting capacity has been ignored in the calculations.

The layout of the reinforced concrete masonry walls in the transverse direction is regular. All walls are of a similar length. As a result, it has been assumed that each wall resists an equal portion of the total lateral seismic load. In-plane shear demand for each wall:

$$V^* = 133.4\ kN$$

Shear capacity of 7.5m long reinforced concrete masonry wall:

$$\emptyset V_n = 367.7\ kN$$

$$\%NBS = \frac{367.7\ kN}{133.4\ kN} = 100\%\ NBS$$

In-Plane Moment

The reinforced concrete masonry walls achieve **100% NBS** when considering in-plane bending of the walls.

$$M^* = 320\ kNm$$

In-plane bending moment capacity of 7.5m long reinforced concrete masonry wall:

$$\emptyset M_n = 2{,}182\ kNm$$

$$\%NBS = \frac{2{,}182\ kNm}{320\ kNm} = 100\%\ NBS$$

Out-of-Plane Moment

The reinforced concrete masonry walls achieve **100% NBS** when considering out-of-plane bending of the walls.

The 190mm thick reinforced concrete masonry walls are restrained out-of-plane at eaves level by the ceiling diaphragm at a height of 2.4m. The top of the walls are restrained by the braced timber roof framing. The walls were assumed to have pinned connections at the top and bottom of the wall. Out-of-plane bending moment demands and capacities were evaluated per metre width of wall.

$$M^* = 2.4\ kNm/m$$

Out-of-plane bending moment capacity of 7.5m long reinforced concrete masonry wall:

$$\emptyset M_n = 7.0\ kNm/m$$

$$\%NBS = \frac{7.0\ kNm/m}{2.4\ kNm/m} = 100\%\ NBS$$

8.2 Block C

8.2.1 Timber Framed Walls

The bracing demand was determined by evaluating the seismic weight of the building and multiplying this value by the horizontal design action coefficient for the longitudinal direction. The demand was then resolved into bracing units (BUs) for comparison with bracing capacities of timber framed walls.

$$\mathrm{BU}_{\mathrm{demand}} = 2{,}610\ \mathrm{BUs}$$

A comparison was made with the corresponding demand based on NZS 3604:2011 requirements. The demand calculated from NZS 3604:2011 significantly underestimated the likely seismic weight of the structure due to the presence of the heavy fully filled reinforced concrete masonry walls and brick masonry cladding.

The total bracing capacity of the building in the longitudinal direction was evaluated by determining the lengths of plasterboard lined timber framed walls available that satisfy the aspect ratio limit of 3.5:1 suggested in the NZSEE guidelines. Only a small number of sections of walls in each unit satisfy this requirement. There is a significant lack of walls capable of bracing the structure in the perimeter walls due to large penetrations. As a result, the building effectively has two lines of bracing through each unit. The layout of bracing elements and bracing lines is extremely asymmetric and contributes significantly to the assessed score of 22% NBS.

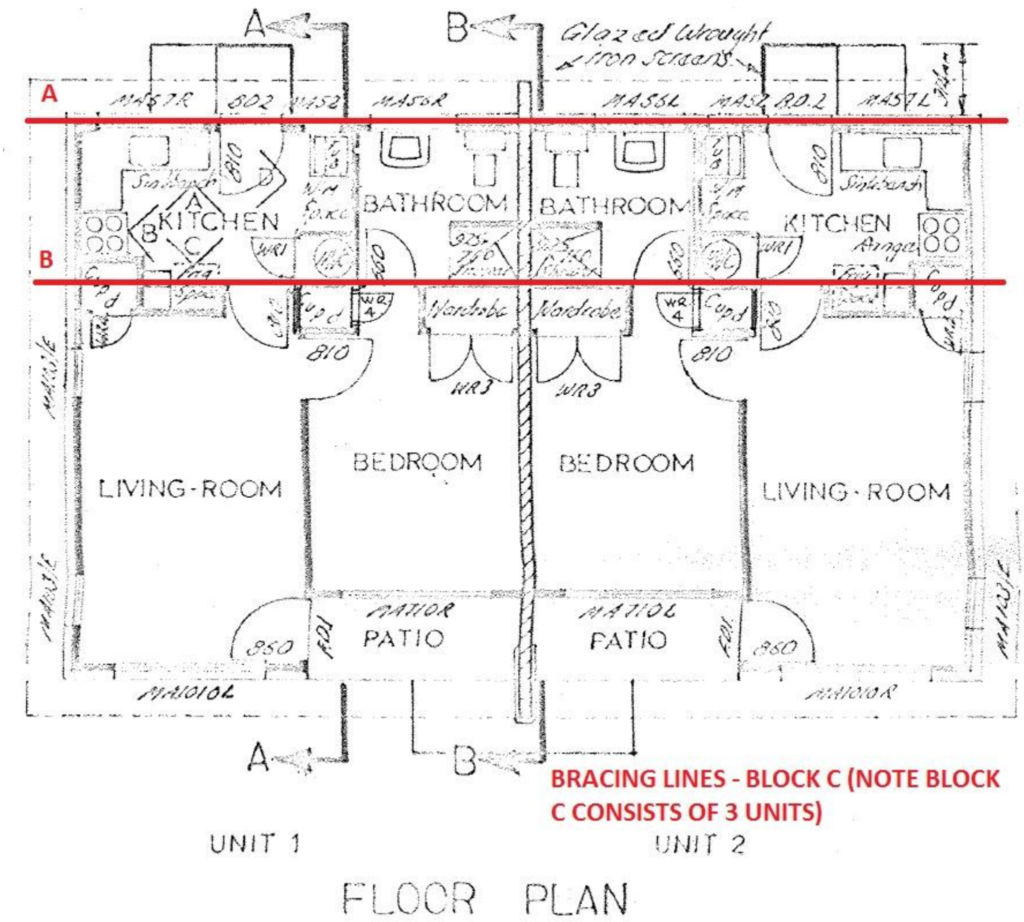


Figure 8.2 Longitudinal bracing lines for Block C

Bracing Line	Bracing Capacity (BU)
A	85
B	477
Total Bracing Capacity =	562 BUs

Table 8.2 Block C bracing line capacities

$$\%NBS = \frac{562\ BUs}{2{,}610\ BUs} = 22\%\ NBS$$

8.2.2 Reinforced Concrete Masonry Walls

<u>In-Plane Shear</u>

The reinforced concrete masonry walls achieve **100% NBS** under in-plane shear seismic loading.

The reinforced concrete masonry walls are significantly stiffer than the timber framed walls in the transverse direction. As a result, the concrete masonry walls are likely to resist the majority of lateral seismic loads in the transverse direction. The contribution of the timber framed walls to the lateral load resisting capacity has been ignored in the calculations.

The layout of the reinforced concrete masonry walls in the transverse direction is regular. All walls are of a similar length. As a result, it has been assumed that each wall resists an equal portion of the total lateral seismic load. In-plane shear demand for each wall:

$$V^* = 161.6\ kN$$

Shear capacity of 7.5m long reinforced concrete masonry wall:

$$\emptyset V_n = 367.7\ kN$$

$$\%NBS = \frac{367.7\ kN}{161.6\ kN} = 100\%\ NBS$$

<u>In-Plane Moment</u>

The reinforced concrete masonry walls achieve **100% NBS** when considering in-plane bending of the walls.

$$M^* = 388\ kNm$$

In-plane bending moment capacity of 7.5m long reinforced concrete masonry wall:

$$\emptyset M_n = 2{,}182\ kNm$$

$$\%NBS = \frac{2{,}182\ kNm}{388\ kNm} = 100\%\ NBS$$

Out-of-Plane Moment

The reinforced concrete masonry walls achieve **100% NBS** when considering out-of-plane bending of the walls.

The 190mm thick reinforced concrete masonry walls are restrained out-of-plane at eaves level by the ceiling diaphragm at a height of 2.4m. The top of the walls are restrained by the braced timber roof framing. The walls were assumed to have pinned connections at the top and bottom of the wall. Out-of-plane bending moment demands and capacities were evaluated per metre width of wall.

$$M^* = 2.4\ kNm/m$$

Out-of-plane bending moment capacity of 7.5m long reinforced concrete masonry wall:

$$\emptyset M_n = 7.0\ kNm/m$$

$$\%NBS = \frac{7.0\ kNm/m}{2.4\ kNm/m} = 100\%\ NBS$$

8.3 Block D

8.3.1 Timber Framed Walls

The bracing demand was determined by evaluating the seismic weight of the building and multiplying this value by the horizontal design action coefficient for the longitudinal direction. The demand was then resolved into bracing units (BUs) for comparison with bracing capacities of timber framed walls.

$$\mathrm{BU}_{\mathrm{demand}} = 3{,}460\ \mathrm{BUs}$$

A comparison was made with the corresponding demand based on NZS 3604:2011 requirements. The demand calculated from NZS 3604:2011 significantly underestimated the likely seismic weight of the structure due to the presence of the heavy fully filled reinforced concrete masonry walls and brick masonry cladding.

The total bracing capacity of the building in the longitudinal direction was evaluated by determining the lengths of plasterboard lined timber framed walls available that satisfy the aspect ratio limit of 3.5:1 suggested in the NZSEE guidelines. Only a small number of sections of walls in each unit satisfy this requirement. There is a significant lack of walls capable of bracing the structure in the perimeter walls due to large penetrations. As a result, the building effectively has two lines of bracing through each unit. The layout of bracing elements and bracing lines is extremely asymmetric and contributes significantly to the assessed score of 22% NBS.

Bracing Line	Bracing Capacity (BU)
A	57
B	318
C	57
D	318
Total Bracing Capacity =	750 BUs

Table 8.3 Block D bracing line capacities

$$\%NBS = \frac{750\ BUs}{3{,}460\ BUs} = 22\%\ NBS$$

8.3.2 Reinforced Concrete Masonry Walls

<u>In-Plane Shear</u>

The reinforced concrete masonry walls achieve **100% NBS** under in-plane shear seismic loading.

The reinforced concrete masonry walls are significantly stiffer than the timber framed walls in the transverse direction. As a result, the concrete masonry walls are likely to resist the majority of lateral seismic loads in the transverse direction. The contribution of the timber framed walls to the lateral load resisting capacity has been ignored in the calculations.

The layout of the reinforced concrete masonry walls in the transverse direction is regular. All walls are of a similar length. As a result, it has been assumed that each wall resists an equal portion of the total lateral seismic load. In-plane shear demand for each wall:

$$V^* = 142.8\ kN$$

Shear capacity of 7.5m long reinforced concrete masonry wall:

$$\emptyset V_n = 367.7\ kN$$

$$\%NBS = \frac{367.7\ kN}{142.8\ kN} = 100\%\ NBS$$

<u>In-Plane Moment</u>

The reinforced concrete masonry walls achieve **100% NBS** when considering in-plane bending of the walls.

$$M^* = 343\ kNm$$

In-plane bending moment capacity of 7.5m long reinforced concrete masonry wall:

$$\emptyset M_n = 2{,}182\ kNm$$

$$\%NBS = \frac{2{,}182\ kNm}{343\ kNm} = 100\%\ NBS$$

Out-of-Plane Moment

The reinforced concrete masonry walls achieve **100% NBS** when considering out-of-plane bending of the walls.

The 190mm thick reinforced concrete masonry walls are restrained out-of-plane at eaves level by the ceiling diaphragm at a height of 2.4m. The top of the walls are restrained by the braced timber roof framing. The walls were assumed to have pinned connections at the top and bottom of the wall. Out-of-plane bending moment demands and capacities were evaluated per metre width of wall.

$$M^* = 2.4\ kNm/m$$

Out-of-plane bending moment capacity of 7.5m long reinforced concrete masonry wall:

$$\emptyset M_n = 7.0\ kNm/m$$

$$\%NBS = \frac{7.0\ kNm/m}{2.4\ kNm/m} = 100\%\ NBS$$

8.4 Summary

Element	Seismic Action	Block A %NBS	Block B %NBS	Block C %NBS	Block D %NBS
Transverse Direction					
Concrete Masonry Walls	In-Plane Shear	100	100	100	100
	In-Plane Bending	100	100	100	100
	Out-of-Plane Bending	100	100	100	100
Longitudinal Direction					
Timber Framed Walls	In-Plane Shear	22	22	22	22

Table 8.4 Summary of %NBS scores

8.5 Discussion of Results

The results obtained from the analysis are generally consistent with those expected for a building of this age and construction type, founded on Class D soils.

The Knightsbridge Lane Complex was designed in 1976 and was likely designed in accordance with the previous loading standard, NZS 1900:1965, superseded that year. The design loads used are likely to have been less than those required by the current loading standard.

The buildings perform well in the transverse direction with the concrete masonry walls achieving 100% NBS. However, the lack of suitable lengths of both internal and perimeter plasterboard lined timber framed walls combined with a poor distribution of bracing lines leads to an assessed score of 22% NBS for all of the buildings in the longitudinal direction.

The layout of the timber framed walls is extremely asymmetric and also fails to satisfy current NZS 3604:2011 requirements for minimum bracing line values and minimum bracing line values for walls supporting a diaphragm. Based on the age of the building and the above issues regarding the timber framed walls in the longitudinal direction of the buildings, it is reasonable to expect the buildings to be Earthquake Prone.

8.6 Strengthening

Following the quantitative assessment of the buildings at Knightsbridge Lane GHD were engaged by the Christchurch City Council to develop a strengthening solution to achieve a minimum of 67%NBS, and to replace the blockwork veneer gable ends with lightweight cladding (Refer Appendix E for details).

Strengthening works involved the installation of Gib bracing in the along direction. The resultant strength for each of the buildings is as detailed in Table 5 below

Element	**Seismic Action**	**Block A %NBS**	**Block B %NBS**	**Block C %NBS**	**Block D %NBS**
Transverse Direction					
Concrete Masonry Walls	In-Plane Shear	100	100	100	100
	In-Plane Bending	100	100	100	100
	Out-of-Plane Bending	100	100	100	100
Longitudinal Direction					
Timber Framed Walls	In-Plane Shear	73	73	72	72

Table 5 Strengthened building indicative strength

9. Conclusions and Recommendations

Following a quantitative assessment Blocks A, B, C and D were assessed to have a seismic capacity in the order of 22% NBS and were deemed to be buildings with low strength. As a result GHD were engaged by the Christchurch City Council to develop a strengthening solution to achieve a minimum of 67%NBS, and to replace the blockwork veneer gable ends with lightweight cladding.

Strengthening works, involving the installation of Gib bracing elements were commenced on the 31st of May 2013, and completed on all Blocks on the 20th of September. A summary of the strengths pre and post earthquake of each block is outlined in the table below.

Knightsbridge Lane Social Housing Complex	Asset Code	Strength (Pre Repairs)	Strength (Post Repairs)
Block A (Units 1,2,3,4,5)	PRO 1265 B001	22% NBS	73% NBS
Block B (Units 6,7,8,9,10)	PRO 1265 B002	22% NBS	73% NBS
Block C (Units 11,12,13)	PRO 1265 B003	22% NBS	72% NBS
Block D (Units 14,15,16,17)	PRO 1265 B004	22% NBS	72% NBS

10. Limitations

10.1 General

This report has been prepared subject to the following limitations:

- Available drawings itemised in Section 5.2 were used in the assessment.
- The foundations of the building were unable to be inspected beyond those exposed above ground level externally.
- No material testing has 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 report.

10.2 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 View of Block A from Knightsbridge Lane

Photograph 2 View of rear of Block A

Photograph 3 View of Block B from Knightsbridge Lane

Photograph 4 View of rear of Block B

Photograph 5 View of collapsed brick gable veneer at north-eastern end of Block C

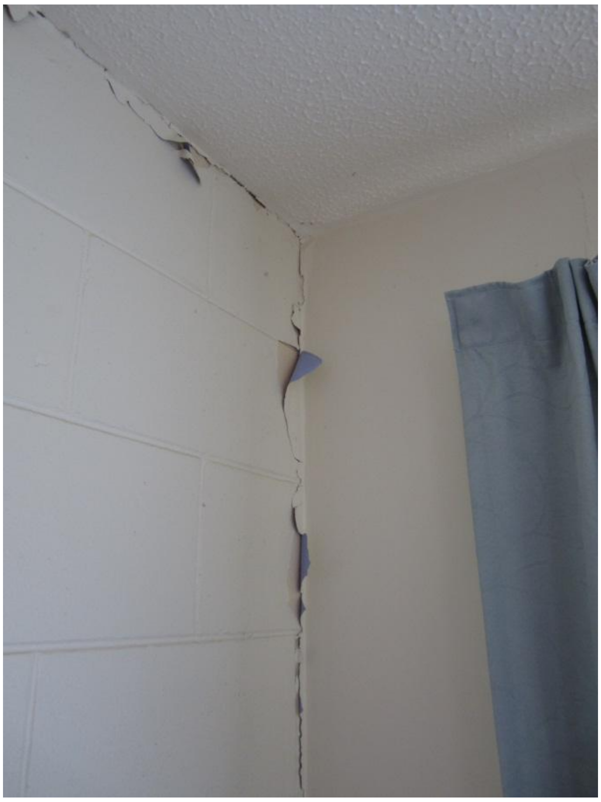
Photograph 6 View of collapsed brick gable veneer at south-western end of Block C

Photograph 7 **View of front of Block C**

Photograph 8 **View of rear of Block D**

Photograph 9 View of front of Block D

Photograph 10 Evidence of liquefaction occurring in the car park

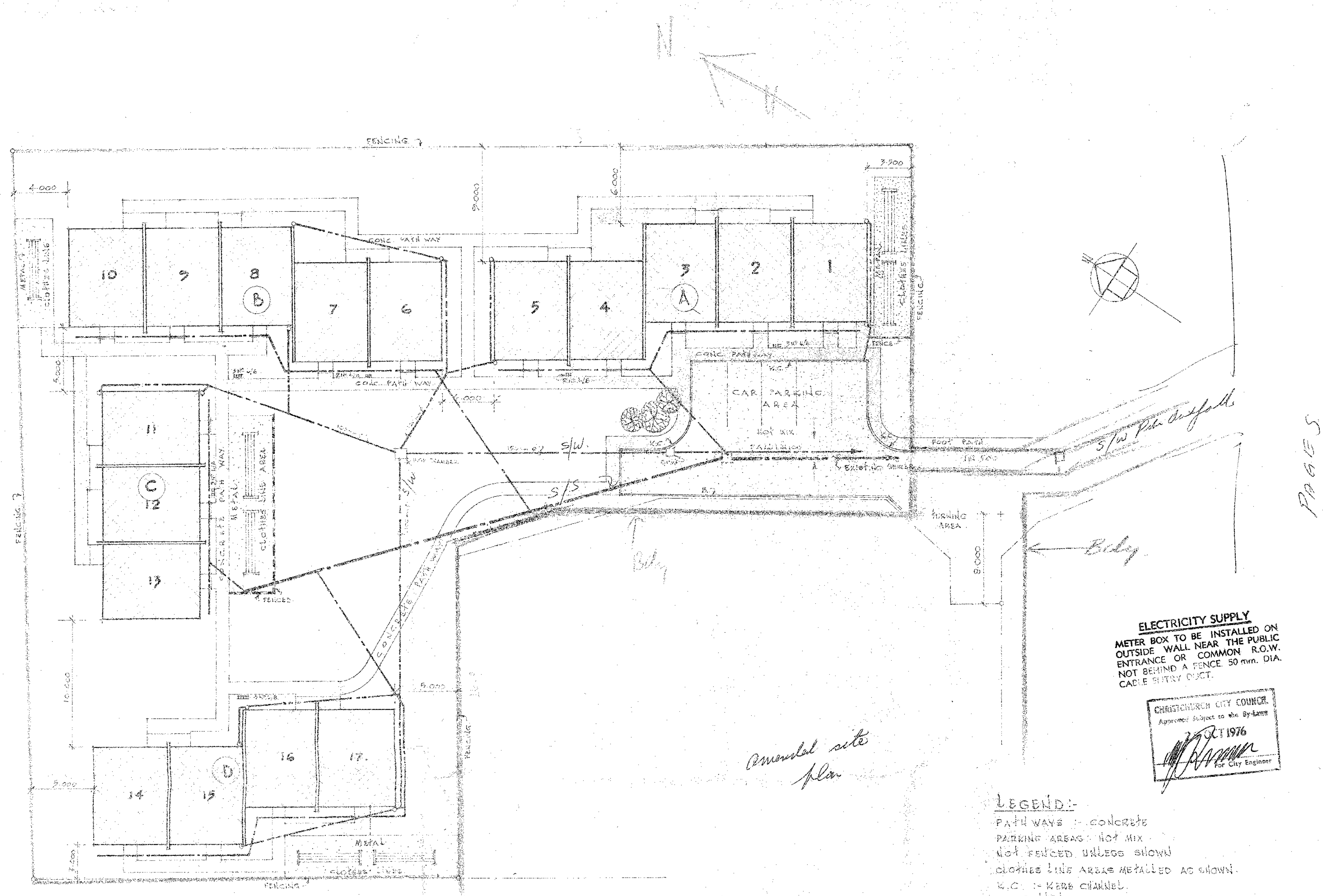


Photograph 11 Typical damage observed in residential units

Photograph 12 Timber nail plate roof trusses and concrete tile cladding

Appendix B

Existing Drawings



ELECTRICITY SUPPLY
METER BOX TO BE INSTALLED ON OUTSIDE WALL NEAR THE PUBLIC ENTRANCE OR COMMON R.O.W. NOT BEHIND A FENCE. 50 mm. DIA. CABLE ENTRY DUCT.

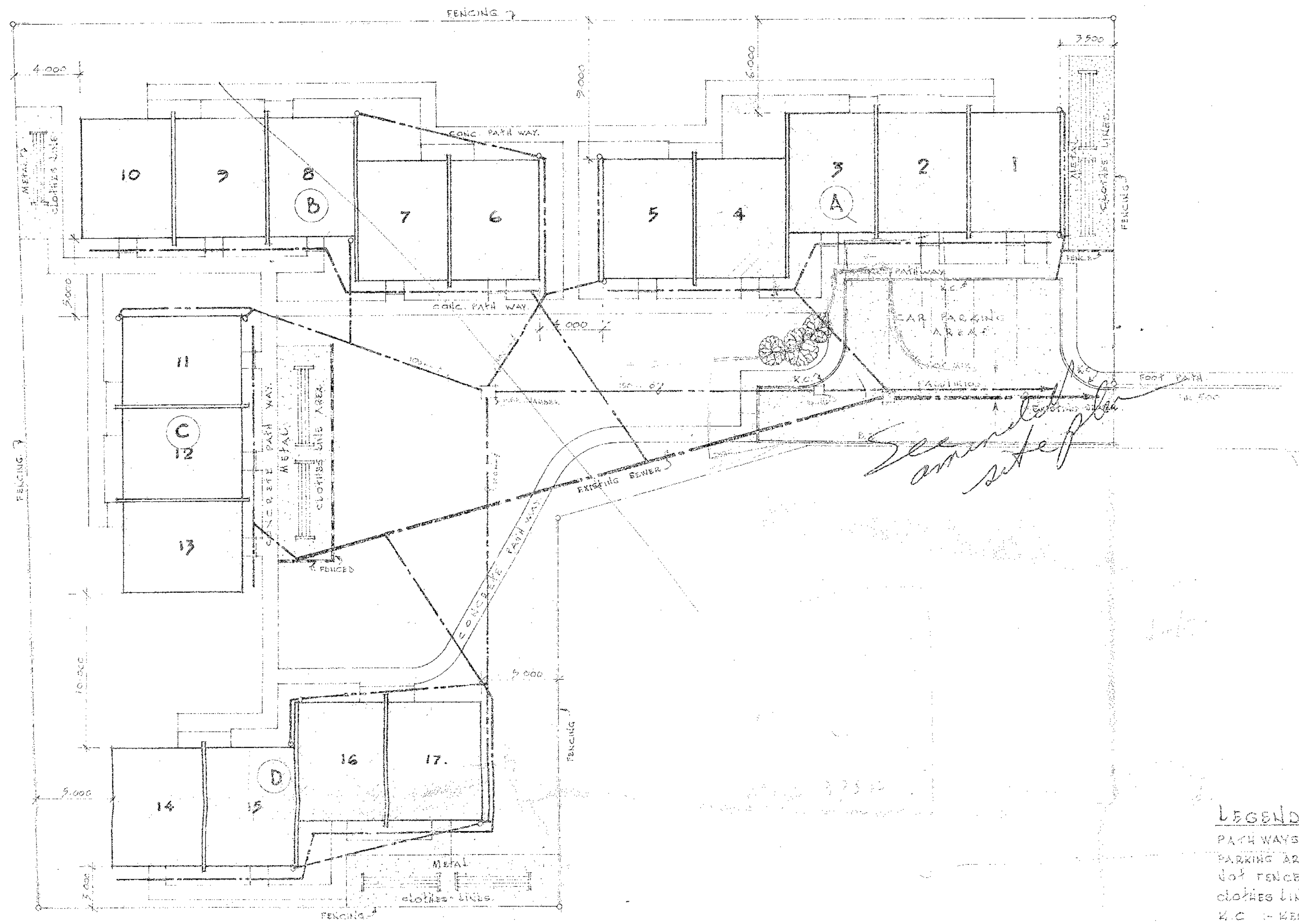
CHRISTCHURCH CITY COUNCIL
Approved Subject to the By-Laws
OCT 1976
For City Engineer

LEGEND:-
PATH WAYS :- CONCRETE
PARKING AREAS :- HOT MIX
NOT FENCED UNLESS SHOWN
CLOTHES LINE AREAS METALLED AS SHOWN.
K.C. :- KERB CHANNEL.
B :- BATTENED.
SEWER
STORM WATER

(MARK III UNITS.)

MB PROPOSED MERRITT BEAZLEY COTTAGES FOR THE ELDERLY - PAGES ROAD - CHRISTCHURCH. SCALE 1:200.

DHS 3866



LEGEND:-
PATHWAYS :- CONCRETE
PARKING AREAS : HOT MIX
NOT FENCED UNLESS SHOWN
CLOTHES LINE AREAS METALLED AS SHOWN
K.C :- KERB CHANNEL
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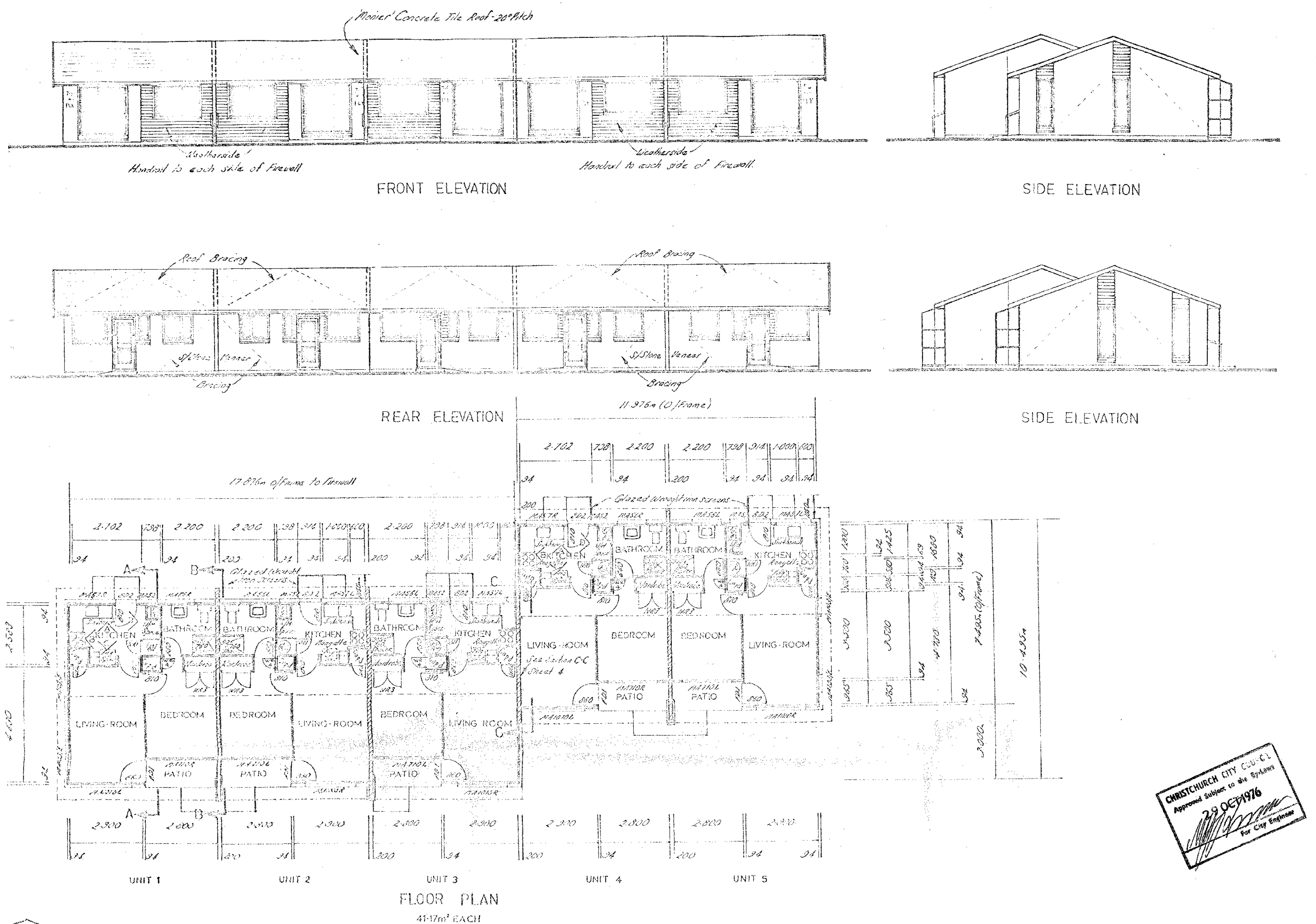
STORM WATER

(MARK III UNITS)

MB PROPOSED MERRITT BEAZLEY COTTAGES FOR THE ELDERLY - PAGES ROAD - CHRISTCHURCH. SCALE 1:200



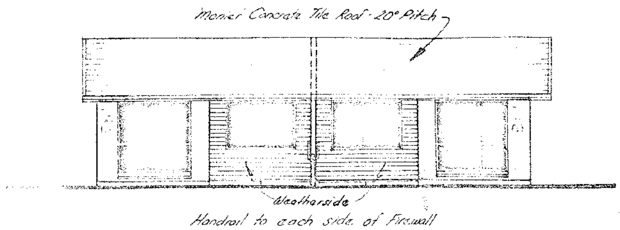
TYPICAL MARK 3 UNITS

MB

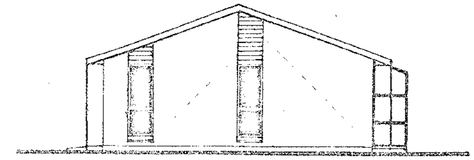
PROPOSED MERRITT-BEAZLEY COTTAGES FOR THE ELDERLY — PAGES ROAD — CHRISTCHURCH

SCALE 1:100

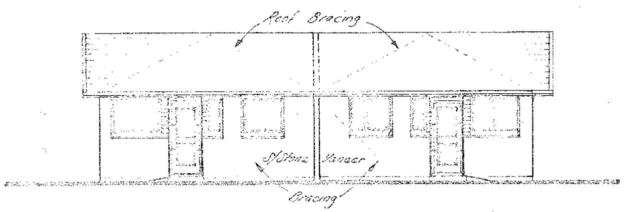
3



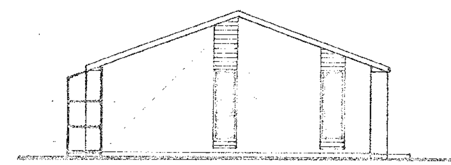
FRONT ELEVATION



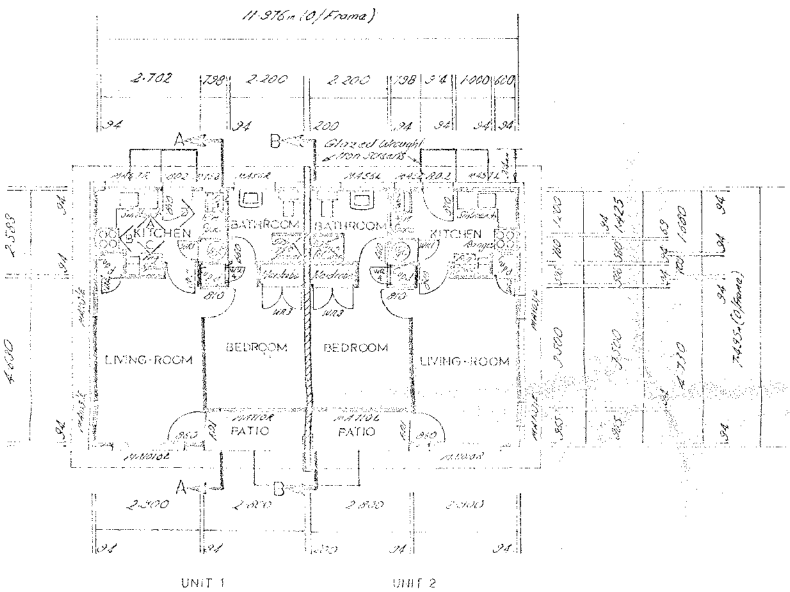
SIDE ELEVATION



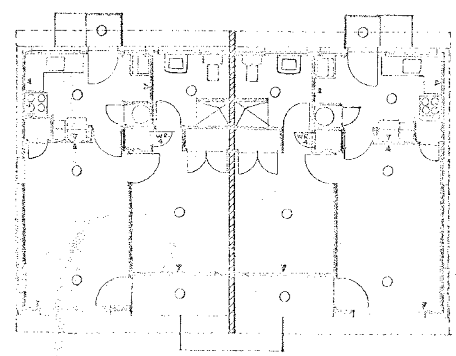
REAR ELEVATION



SIDE ELEVATION



FLOOR PLAN



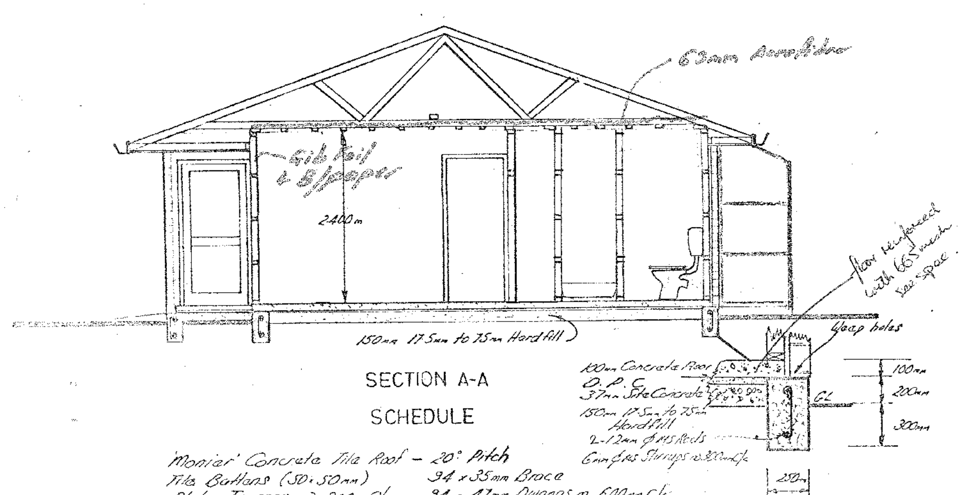
ELECTRICAL PLAN

CHRISTCHURCH CITY COUNCIL
Approved Subject to the By-Laws
OCT 1976
For City Engineer

MARK 3

PROPOSED MERRITT BEAZLEY COTTAGES FOR THE ELDERLY — [illegible] — CHRISTCHURCH SCALE 1:100

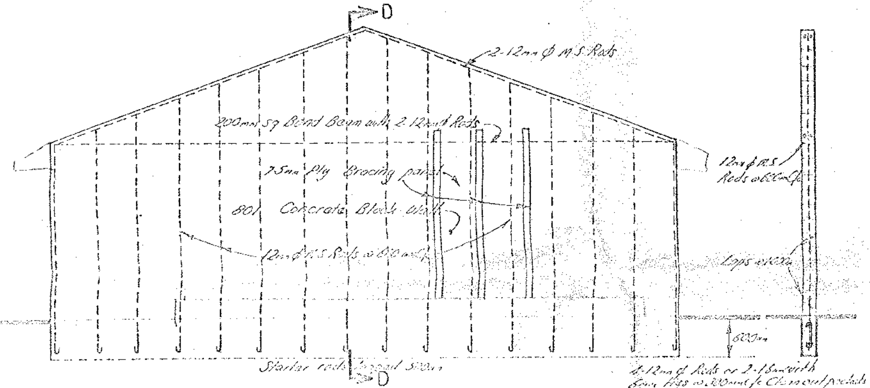
3A



SECTION A-A

SCHEDULE

'Monier' Concrete Tile Roof – 20° Pitch
Tile Battens (50 x 50mm)
Plate Trusses @ 900mm C/c
Pryda Plates (By M Alpines)
75 x 37mm Ceiling Battens @ 400mm C/c
144 x 37mm Top plate.
94 x 47mm Top Plate
PVC Spouting
150 x 35mm Fascia
4.75 Polite 300 x 600mm Soffit
Veneer tied
37mm Cavity
94 x 47mm Studs @ 600mm C/c
94 x 35mm Brace
94 x 47mm Dwangs @ 600mm C/c
24G Galv brick ties
400mm Vertical
500mm Horizontal
94 x 47mm Bottom plate
100mm Concrete Floor
Mesh № 665
D.P.C.
37mm Site Concrete
2-12mm ø M.S. Rods.
6mm ø M.S. Stirrups @ 300mm C/c

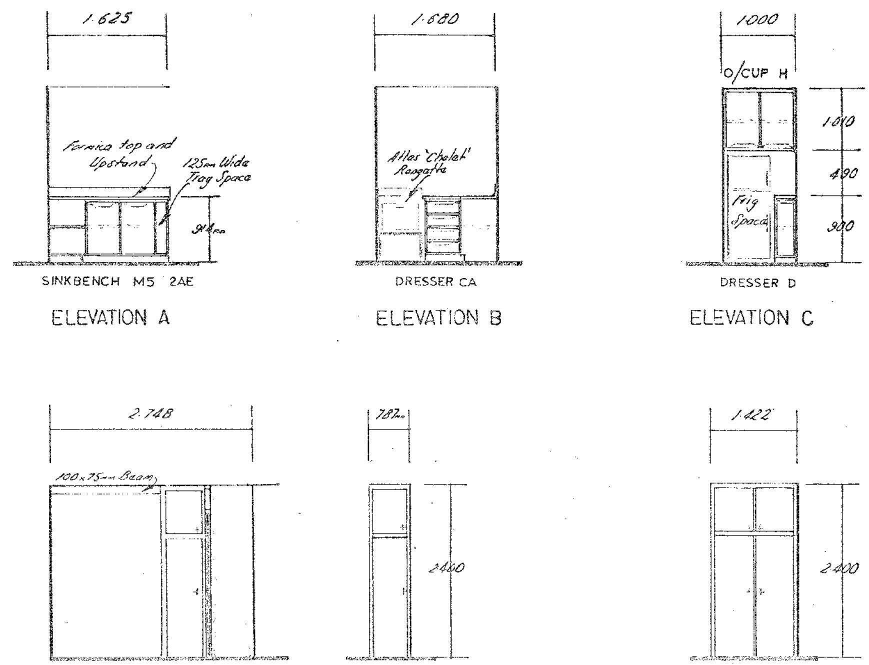


SECTION B-B THRU FIREWALL

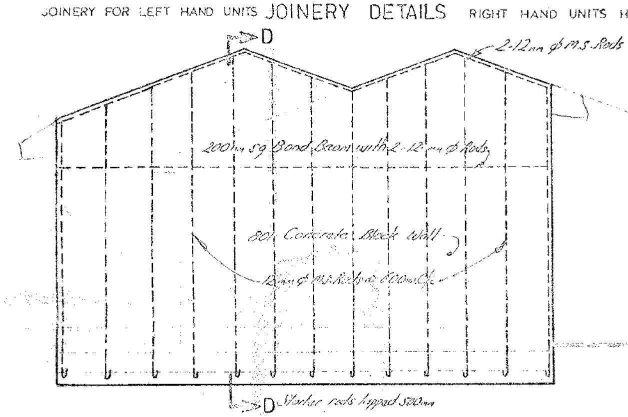
SECTION D-D

WINDOW LINTEL SIZES

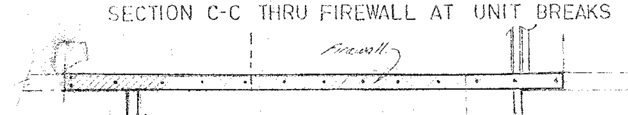
150 x 100mm 0 Up To 2.130
200 x 100mm 2.130 Up To 3.040
250 x 100mm 3.070 Up To 3.500
400 x 100mm 3.520 Up To 4.260



JOINERY FOR LEFT HAND UNITS JOINERY DETAILS RIGHT HAND UNITS HANDED



SECTION C-C THRU FIREWALL AT UNIT BREAKS



PLAN

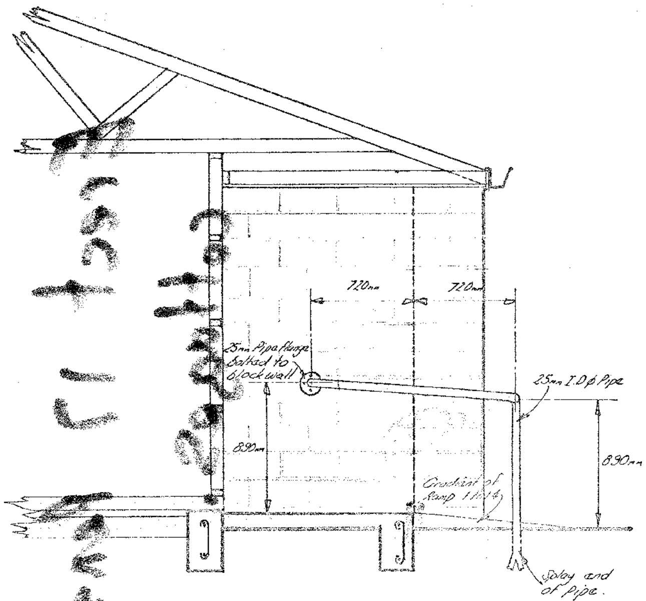
CHRISTCHURCH CITY COUNCIL Approved

MARK 3

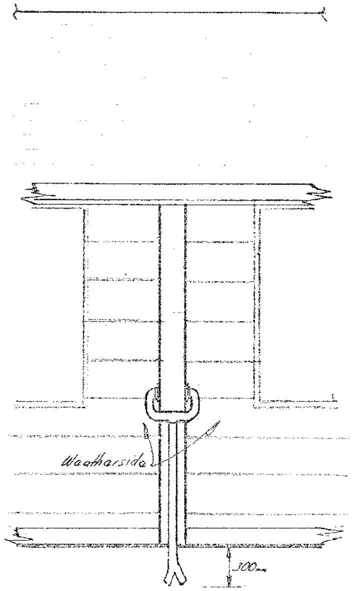
AMMENDED 16-2-76

MB

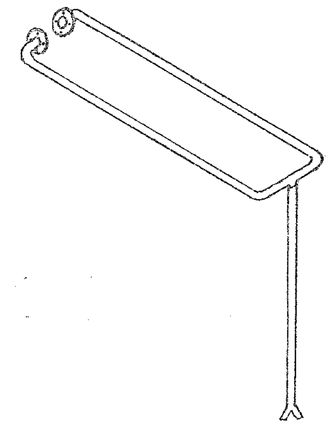
PROPOSED MERRITT - BEAZLEY COTTAGES FOR THE ELDERLY — CHRISTCHURCH | SCALE 1:50 | 4



SIDE VIEW

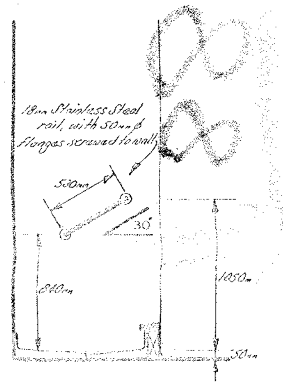


END VIEW

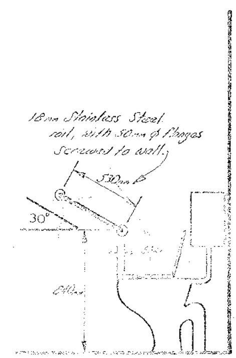


ISOMETRIC VIEW

HANDRAIL DETAILS



SHOWER HANDRAIL



W.C. HANDRAIL

FINAL PLAN

MARK 2

MB | PROPOSED MERRITT-BEAZLEY COTTAGES FOR THE ELDERLY — CHRISTCHURCH | SCALE 1:25 | 5

Appendix C

CERA Forms

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Knightsbridge Lane Complex Block A			Reviewer:	Stephen Lee
	Unit	No:	Street	CPEng No:	1006840
Building Address:			Knightsbridge Lane	Company:	GHD
Legal Description:				Company project number:	513090276
				Company phone number:	04 472 0799
	Degrees	Min	Sec		
GPS south:				Date of submission:	
GPS east:				Inspection Date:	2/11/2012
				Revision:	
Building Unique Identifier (CCC):	BE 1265 EQ2			Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:		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):	0.10
Storeys below ground	0			
Foundation type:	strip footings		if Foundation type is other, describe:	
Building height (m):	3.80		height from ground to level of uppermost seismic mass (for IEP only) (m):	3.8
Floor footprint area (approx):	214			
Age of Building (years):	36		Date of design:	1965-1976
Strengthening present?	no		If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):	multi-unit residential		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 truss	truss depth, purlin type and cladding	
Floors:	other (note)	describe sytem	Slab on grade
Beams:	timber	type	
Columns:	brick masonry	typical dimensions (mm x mm)	
Walls:	fully filled concrete masonry	#N/A	190

Lateral load resisting structure

Lateral system along:	lightweight timber framed walls	**Note: Define along and across in detailed report!**		
Ductility assumed, μ:	3.00		note typical wall length (m)	
Period along:	0.40	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	fully filled CMU			
Ductility assumed, μ:	1.25		note total length of wall at ground (m):	
Period across:	0.40	##### enter height above at H31	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:			
Wall cladding:	brick or tile	describe (note cavity if exists)	37mm Cavity
Roof Cladding:	Heavy tiles	describe	Concrete Tiles
Glazing:	aluminium frames		
Ceilings:	fibrous plaster, fixed		
Services(list):			

Available documentation

Architectural	partial	original designer name/date	
Structural	partial	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	Describe damage:	
Settlement:	none observed	notes (if applicable):	
Differential settlement:	0-1:350	notes (if applicable):	Minor foundation settlement.
Liquefaction:	0-2 m³/100m²	notes (if applicable):	Liquefaction in car park.
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):	Minor damage. Less than 5%		
Across	Damage ratio:	0%		
	Describe (summary):	Minor damage. Less than 5%		
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	yes	Describe:	Damaged linings.

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

Recommendations

	Level of repair/strengthening required:			Describe:	
	Building Consent required:			Describe:	
	Interim occupancy recommendations:			Describe:	
Along	Assessed %NBS before e'quakes:	73%	##### %NBS from IEP below	If IEP not used, please detail assessment methodology:	Quantitative analysis and strengthening
	Assessed %NBS after e'quakes:	73%			
Across	Assessed %NBS before e'quakes:	100%	##### %NBS from IEP below		
	Assessed %NBS after e'quakes:	100%			

IEP **Use of this method is not mandatory - more detailed analysis may give a different answer, which would take precedence. Do not fill in fields if not using IEP.**

Period of design of building (from above):	1965-1976	h_n from above:	3.8m
Seismic Zone, if designed between 1965 and 1992:		not required for this age of building	
		not required for this age of building	

	along	across
Period (from above):	0.4	0.4
(%NBS)nom from Fig 3.3:		

Note:1 for specifically design public buildings, to the code of the day: pre-1965 = 1.25; 1965-1976, Zone A =1.33; 1965-1976, Zone B = 1.2; all else 1.0

Note 2: for RC buildings designed between 1976-1984, use 1.2

Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0)

	along	across
Final (%NBS)$_{nom}$:	**0%**	**0%**

2.2 Near Fault Scaling Factor

Near Fault scaling factor, from NZS1170.5, cl 3.1.6:

	along	across
Near Fault scaling factor (1/N(T,D), **Factor A**:	#DIV/0!	#DIV/0!

2.3 Hazard Scaling Factor

Hazard factor Z for site from AS1170.5, Table 3.3:	
Z_{1992}, from NZS4203:1992	
Hazard scaling factor, **Factor B:**	#DIV/0!

2.4 Return Period Scaling Factor

Building Importance level (from above):	2
Return Period Scaling factor from Table 3.1, **Factor C**:	

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.2)		
Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:		
Ductiity Scaling Factor, **Factor D**:	0.00	0.00

2.6 Structural Performance Scaling Factor:

	along	across
Sp:		
Structural Performance Scaling Factor **Factor E**:	#DIV/0!	#DIV/0!

2.7 Baseline %NBS, (NBS%)$_b$ = (%NBS)$_{nom}$ x A x B x C x D x E

	along	across
%NBS$_b$:	**#DIV/0!**	**#DIV/0!**

Global Critical Structural Weaknesses: (refer to NZSEE IEP Table 3.4)

3.1. Plan Irregularity, factor A:	insignificant	1
3.2. Vertical irregularity, Factor B:	insignificant	1
3.3. Short columns, Factor C:	insignificant	1

3.4. Pounding potential

Pounding effect D1, from Table to right	
Height Difference effect D2, from Table to right	
Therefore, Factor D:	0

Table for selection of D1	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Alignment of floors within 20% of H	**0.7**	**0.8**	**1**
Alignment of floors not within 20% of H	**0.4**	**0.7**	**0.8**

Table for Selection of D2	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Height difference > 4 storeys	0.4	0.7	1
Height difference 2 to 4 storeys	0.7	0.9	1
Height difference < 2 storeys	1	1	1

3.5. Site Characteristics	insignificant	1

3.6. Other factors, Factor F

	Along	Across
For ≤ 3 storeys, max value =2.5, otherwise max valule =1.5, no minimum		
Rationale for choice of F factor, if not 1		

Detail Critical Structural Weaknesses: (refer to DEE Procedure section 6)

List any: Refer also section 6.3.1 of DEE for discussion of F factor modification for other critical structural weaknesses

3.7. Overall Performance Achievement ratio (PAR)	**0.00**	**0.00**

4.3 PAR x (%NBS)b:	PAR x Baselline %NBS:	**#DIV/0!**	**#DIV/0!**
4.4 Percentage New Building Standard (%NBS), (before)			**#DIV/0!**

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Knightsbridge Lane Complex Block B			Reviewer:	Stephen Lee
	Unit	No:	Street	CPEng No:	1006840
Building Address:			Knightsbridge Lane	Company:	GHD
Legal Description:				Company project number:	513090276
				Company phone number:	04 472 0799
	Degrees	Min	Sec		
GPS south:				Date of submission:	
GPS east:				Inspection Date:	2/11/2012
				Revision:	
Building Unique Identifier (CCC):	BE 1265 EQ2			Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:		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):	0.10
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Floor footprint area (approx):	214			
Age of Building (years):	36		Date of design:	1965-1976
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			And what load level (%g)?	
Use (ground floor):	multi-unit residential		Brief strengthening description:	
Use (upper floors):				
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Gravity System:	load bearing walls		
Roof:	timber truss	truss depth, purlin type and cladding	
Floors:	other (note)	describe sytem	Slab on grade
Beams:	timber	type	
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Lateral system along:	lightweight timber framed walls	**Note: Define along and across in detailed report!**		
Ductility assumed, μ:	3.00		note typical wall length (m)	
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Period across:	0.40	##### enter height above at H31	estimate or calculation?	estimated
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Separations:

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Stairs:			
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Damage

Site:
(refer DEE Table 4-2)

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Settlement:	none observed	notes (if applicable):	
Differential settlement:	0-1:350	notes (if applicable):	Minor foundation settlement.
Liquefaction:	0-2 m³/100m²	notes (if applicable):	Liquefaction in car park.
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:

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	Describe (summary):	Minor damage. Less than 5%		
Across	Damage ratio:	0%		
	Describe (summary):	Minor damage. Less than 5%		
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	yes	Describe:	Damaged linings.

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

Recommendations

	Level of repair/strengthening required:			Describe:	
	Building Consent required:			Describe:	
	Interim occupancy recommendations:			Describe:	
Along	Assessed %NBS before e'quakes:	73%	##### %NBS from IEP below	If IEP not used, please detail assessment methodology:	Quantitative analysis and strengthening
	Assessed %NBS after e'quakes:	73%			
Across	Assessed %NBS before e'quakes:	100%	##### %NBS from IEP below		
	Assessed %NBS after e'quakes:	100%			

IEP **Use of this method is not mandatory - more detailed analysis may give a different answer, which would take precedence. Do not fill in fields if not using IEP.**

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Seismic Zone, if designed between 1965 and 1992:		not required for this age of building	
		not required for this age of building	

	along	across
Period (from above):	0.4	0.4
(%NBS)nom from Fig 3.3:		

Note:1 for specifically design public buildings, to the code of the day: pre-1965 = 1.25; 1965-1976, Zone A =1.33; 1965-1976, Zone B = 1.2; all else 1.0

Note 2: for RC buildings designed between 1976-1984, use 1.2

Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0)

	along	across
Final (%NBS)$_{nom}$:	**0%**	**0%**

2.2 Near Fault Scaling Factor

Near Fault scaling factor, from NZS1170.5, cl 3.1.6:

	along	across
Near Fault scaling factor (1/N(T,D), **Factor A**:	#DIV/0!	#DIV/0!

2.3 Hazard Scaling Factor

Hazard factor Z for site from AS1170.5, Table 3.3:	
Z_{1992}, from NZS4203:1992	
Hazard scaling factor, **Factor B:**	#DIV/0!

2.4 Return Period Scaling Factor

Building Importance level (from above):	2
Return Period Scaling factor from Table 3.1, **Factor C**:	

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.2)		
Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:		
Ductiity Scaling Factor, **Factor D**:	0.00	0.00

2.6 Structural Performance Scaling Factor:

	along	across
Sp:		
Structural Performance Scaling Factor **Factor E**:	#DIV/0!	#DIV/0!

2.7 Baseline %NBS, (NBS%)$_b$ = (%NBS)$_{nom}$ x A x B x C x D x E

	along	across
%NBS$_b$:	**#DIV/0!**	**#DIV/0!**

Global Critical Structural Weaknesses: (refer to NZSEE IEP Table 3.4)

3.1. Plan Irregularity, factor A:	insignificant	1
3.2. Vertical irregularity, Factor B:	insignificant	1
3.3. Short columns, Factor C:	insignificant	1
3.4. Pounding potential Pounding effect D1, from Table to right		
Height Difference effect D2, from Table to right		
Therefore, Factor D:		0
3.5. Site Characteristics	insignificant	1

Table for selection of D1	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Alignment of floors within 20% of H	**0.7**	**0.8**	**1**
Alignment of floors not within 20% of H	**0.4**	**0.7**	**0.8**

Table for Selection of D2	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Height difference > 4 storeys	0.4	0.7	1
Height difference 2 to 4 storeys	0.7	0.9	1
Height difference < 2 storeys	1	1	1

3.6. Other factors, Factor F

	Along	Across
For ≤ 3 storeys, max value =2.5, otherwise max valule =1.5, no minimum		
Rationale for choice of F factor, if not 1		

Detail Critical Structural Weaknesses: (refer to DEE Procedure section 6)

List any: Refer also section 6.3.1 of DEE for discussion of F factor modification for other critical structural weaknesses

3.7. Overall Performance Achievement ratio (PAR)	**0.00**	**0.00**

4.3 PAR x (%NBS)b:	PAR x Baselline %NBS:	**#DIV/0!**	**#DIV/0!**
4.4 Percentage New Building Standard (%NBS), (before)			**#DIV/0!**

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Knightsbridge Lane Complex Block C			Reviewer:	Stephen Lee
	Unit	No:	Street	CPEng No:	1006840
Building Address:			Knightsbridge Lane	Company:	GHD
Legal Description:				Company project number:	513090276
				Company phone number:	04 472 0799
	Degrees	Min	Sec		
GPS south:				Date of submission:	
GPS east:				Inspection Date:	2/11/2012
				Revision:	
Building Unique Identifier (CCC):	BE 1265 EQ2			Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:		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):	0.10
Storeys below ground	0			
Foundation type:	strip footings		if Foundation type is other, describe:	
Building height (m):	3.80		height from ground to level of uppermost seismic mass (for IEP only) (m):	3.8
Floor footprint area (approx):	128			
Age of Building (years):	36		Date of design:	1965-1976
Strengthening present?	no		If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):	multi-unit residential		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 truss	truss depth, purlin type and cladding	
Floors:	other (note)	describe sytem	Slab on grade
Beams:	timber	type	
Columns:	brick masonry	typical dimensions (mm x mm)	
Walls:	fully filled concrete masonry	#N/A	190

Lateral load resisting structure

Lateral system along:	lightweight timber framed walls	**Note: Define along and across in detailed report!**		
Ductility assumed, μ:	3.00		note typical wall length (m)	
Period along:	0.40	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	fully filled CMU			
Ductility assumed, μ:	1.25		note total length of wall at ground (m):	
Period across:	0.40	##### enter height above at H31	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:			
Wall cladding:	brick or tile	describe (note cavity if exists)	37mm Cavity
Roof Cladding:	Heavy tiles	describe	Concrete Tiles
Glazing:	aluminium frames		
Ceilings:	fibrous plaster, fixed		
Services(list):			

Available documentation

Architectural	partial	original designer name/date	
Structural	partial	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	Describe damage:	
Settlement:	none observed	notes (if applicable):	
Differential settlement:	0-1:350	notes (if applicable):	Minor foundation settlement.
Liquefaction:	0-2 m³/100m²	notes (if applicable):	Liquefaction in car park.
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):	Minor damage. Less than 5%		
Across	Damage ratio:	0%		
	Describe (summary):	Minor damage. Less than 5%		
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	yes	Describe:	Collapsed gable veneers.

$$Damage\ \_\ Ratio = \frac{(\% NBS\ (before) - \% NBS\ (after))}{\% NBS\ (before)}$$

Recommendations

	Level of repair/strengthening required:			Describe:	
	Building Consent required:			Describe:	
	Interim occupancy recommendations:			Describe:	
Along	Assessed %NBS before e'quakes:	72%	##### %NBS from IEP below	If IEP not used, please detail assessment methodology:	Quantitative analysis and strengthening
	Assessed %NBS after e'quakes:	72%			
Across	Assessed %NBS before e'quakes:	100%	##### %NBS from IEP below		
	Assessed %NBS after e'quakes:	100%			

IEP **Use of this method is not mandatory - more detailed analysis may give a different answer, which would take precedence. Do not fill in fields if not using IEP.**

Period of design of building (from above):	1965-1976	h_n from above:	3.8m
Seismic Zone, if designed between 1965 and 1992:		not required for this age of building	
		not required for this age of building	

	along	across
Period (from above):	0.4	0.4
(%NBS)nom from Fig 3.3:		

Note:1 for specifically design public buildings, to the code of the day: pre-1965 = 1.25; 1965-1976, Zone A =1.33; 1965-1976, Zone B = 1.2; all else 1.0
Note 2: for RC buildings designed between 1976-1984, use 1.2
Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0)

	along	across
Final $(\%NBS)_{nom}$:	**0%**	**0%**

2.2 Near Fault Scaling Factor

Near Fault scaling factor, from NZS1170.5, cl 3.1.6:

	along	across
Near Fault scaling factor (1/N(T,D), **Factor A**:	#DIV/0!	#DIV/0!

2.3 Hazard Scaling Factor

Hazard factor Z for site from AS1170.5, Table 3.3:
Z_{1992}, from NZS4203:1992
Hazard scaling factor, **Factor B:** #DIV/0!

2.4 Return Period Scaling Factor

Building Importance level (from above): 2
Return Period Scaling factor from Table 3.1, **Factor C**:

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.2)		
Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:		
Ductiity Scaling Factor, **Factor D**:	0.00	0.00

2.6 Structural Performance Scaling Factor:

Sp:		
Structural Performance Scaling Factor **Factor E**:	#DIV/0!	#DIV/0!

2.7 Baseline %NBS, $(NBS\%)_b = (\%NBS)_{nom} \times A \times B \times C \times D \times E$

$\%NBS_b$:	**#DIV/0!**	**#DIV/0!**

Global Critical Structural Weaknesses: (refer to NZSEE IEP Table 3.4)

3.1. Plan Irregularity, factor A:	insignificant	1
3.2. Vertical irregularity, Factor B:	insignificant	1
3.3. Short columns, Factor C:	insignificant	1
3.4. Pounding potential	Pounding effect D1, from Table to right	
	Height Difference effect D2, from Table to right	
	Therefore, Factor D:	0
3.5. Site Characteristics	insignificant	1

Table for selection of D1	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Alignment of floors within 20% of H	**0.7**	**0.8**	**1**
Alignment of floors not within 20% of H	**0.4**	**0.7**	**0.8**

Table for Selection of D2	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Height difference > 4 storeys	0.4	0.7	1
Height difference 2 to 4 storeys	0.7	0.9	1
Height difference < 2 storeys	1	1	1

3.6. Other factors, Factor F

	Along	Across
For ≤ 3 storeys, max value =2.5, otherwise max valule =1.5, no minimum		
Rationale for choice of F factor, if not 1		

Detail Critical Structural Weaknesses: (refer to DEE Procedure section 6)
List any: Refer also section 6.3.1 of DEE for discussion of F factor modification for other critical structural weaknesses

3.7. Overall Performance Achievement ratio (PAR)	**0.00**	**0.00**

4.3 PAR x (%NBS)b:	PAR x Baselline %NBS:	**#DIV/0!**	**#DIV/0!**
4.4 Percentage New Building Standard (%NBS), (before)			**#DIV/0!**

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Knightsbridge Lane Complex Block D			Reviewer:	Stephen Lee
	Unit	No:	Street	CPEng No:	1006840
Building Address:			Knightsbridge Lane	Company:	GHD
Legal Description:				Company project number:	513090276
				Company phone number:	04 472 0799
	Degrees	Min	Sec		
GPS south:				Date of submission:	
GPS east:				Inspection Date:	2/11/2012
				Revision:	
Building Unique Identifier (CCC):	BE 1265 EQ2			Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:		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):	0.10
Storeys below ground	0			
Foundation type:	strip footings		if Foundation type is other, describe:	
Building height (m):	3.80		height from ground to level of uppermost seismic mass (for IEP only) (m):	3.8
Floor footprint area (approx):	171			
Age of Building (years):	36		Date of design:	1965-1976
Strengthening present?	no		If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):	multi-unit residential		Brief strengthening description:	
Use (upper floors):				
Use notes (if required):				
Importance level (to NZS1170.5):	IL2			

<u>Gravity Structure</u>

Gravity System:	load bearing walls		
Roof:	timber truss	truss depth, purlin type and cladding	
Floors:	other (note)	describe sytem	Slab on grade
Beams:	timber	type	
Columns:	brick masonry	typical dimensions (mm x mm)	
Walls:	fully filled concrete masonry	#N/A	190

<u>Lateral load resisting structure</u>

Lateral system along:	lightweight timber framed walls		**Note: Define along and across in detailed report!**		
Ductility assumed, μ:	3.00			note typical wall length (m)	
Period along:	0.40	0.00		estimate or calculation?	estimated
Total deflection (ULS) (mm):				estimate or calculation?	
maximum interstorey deflection (ULS) (mm):				estimate or calculation?	
Lateral system across:	fully filled CMU				
Ductility assumed, μ:	1.25			note total length of wall at ground (m):	
Period across:	0.40	#####	enter height above at H31	estimate or calculation?	estimated
Total deflection (ULS) (mm):				estimate or calculation?	
maximum interstorey deflection (ULS) (mm):				estimate or calculation?	

<u>Separations:</u>

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)	37mm Cavity
Roof Cladding:	Heavy tiles	describe	Concrete Tiles
Glazing:	aluminium frames		
Ceilings:	fibrous plaster, fixed		
Services(list):			

Available documentation

Architectural	partial	original designer name/date	
Structural	partial	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	Describe damage:	
Settlement:	none observed	notes (if applicable):	
Differential settlement:	0-1:350	notes (if applicable):	Minor foundation settlement.
Liquefaction:	0-2 m³/100m²	notes (if applicable):	Liquefaction in car park.
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):	Minor damage. Less than 5%		
Across	Damage ratio:	0%		
	Describe (summary):	Minor damage. Less than 5%		
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	yes	Describe:	Damage to linings.

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

Recommendations

	Level of repair/strengthening required:			Describe:	
	Building Consent required:			Describe:	
	Interim occupancy recommendations:			Describe:	
Along	Assessed %NBS before e'quakes:	72%	##### %NBS from IEP below	If IEP not used, please detail assessment methodology:	Quantitative analysis and strengthening
	Assessed %NBS after e'quakes:	72%			
Across	Assessed %NBS before e'quakes:	100%	##### %NBS from IEP below		
	Assessed %NBS after e'quakes:	100%			

IEP **Use of this method is not mandatory - more detailed analysis may give a different answer, which would take precedence. Do not fill in fields if not using IEP.**

Period of design of building (from above):	1965-1976	h_n from above:	3.8m
Seismic Zone, if designed between 1965 and 1992:		not required for this age of building	
		not required for this age of building	

	along	across
Period (from above):	0.4	0.4
(%NBS)nom from Fig 3.3:		

Note:1 for specifically design public buildings, to the code of the day: pre-1965 = 1.25; 1965-1976, Zone A =1.33; 1965-1976, Zone B = 1.2; all else 1.0

Note 2: for RC buildings designed between 1976-1984, use 1.2

Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0)

	along	across
Final $(\%NBS)_{nom}$:	**0%**	**0%**

2.2 Near Fault Scaling Factor

Near Fault scaling factor, from NZS1170.5, cl 3.1.6:

	along	across
Near Fault scaling factor (1/N(T,D), **Factor A**:	#DIV/0!	#DIV/0!

2.3 Hazard Scaling Factor

Hazard factor Z for site from AS1170.5, Table 3.3:

Z_{1992}, from NZS4203:1992

Hazard scaling factor, **Factor B:** #DIV/0!

2.4 Return Period Scaling Factor

Building Importance level (from above): 2

Return Period Scaling factor from Table 3.1, **Factor C**:

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.2)		
Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:		
Ductiity Scaling Factor, **Factor D**:	0.00	0.00

2.6 Structural Performance Scaling Factor:

Sp:		
Structural Performance Scaling Factor **Factor E**:	#DIV/0!	#DIV/0!

2.7 Baseline %NBS, $(NBS\%)_b = (\%NBS)_{nom} \times A \times B \times C \times D \times E$

$\%NBS_b$:	**#DIV/0!**	**#DIV/0!**

Global Critical Structural Weaknesses: (refer to NZSEE IEP Table 3.4)

3.1. Plan Irregularity, factor A: insignificant 1

3.2. Vertical irregularity, Factor B: insignificant 1

3.3. Short columns, Factor C: insignificant 1

3.4. Pounding potential

Pounding effect D1, from Table to right

Height Difference effect D2, from Table to right

Therefore, Factor D: 0

3.5. Site Characteristics insignificant 1

Table for selection of D1	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Alignment of floors within 20% of H	**0.7**	**0.8**	**1**
Alignment of floors not within 20% of H	**0.4**	**0.7**	**0.8**

Table for Selection of D2	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Height difference > 4 storeys	0.4	0.7	1
Height difference 2 to 4 storeys	0.7	0.9	1
Height difference < 2 storeys	1	1	1

3.6. Other factors, Factor F

	Along	Across
For ≤ 3 storeys, max value =2.5, otherwise max valule =1.5, no minimum		
Rationale for choice of F factor, if not 1		

Detail Critical Structural Weaknesses: (refer to DEE Procedure section 6)

List any: Refer also section 6.3.1 of DEE for discussion of F factor modification for other critical structural weaknesses

3.7. Overall Performance Achievement ratio (PAR)	**0.00**	**0.00**

4.3 PAR x (%NBS)b:	PAR x Baselline %NBS:	**#DIV/0!**	**#DIV/0!**
4.4 Percentage New Building Standard (%NBS), (before)			**#DIV/0!**

Appendix D

Geotechnical Investigation

Appendix *A*

HAND AUGER LOG

GHD Limited
PO Box 13468
Christchurch 8141

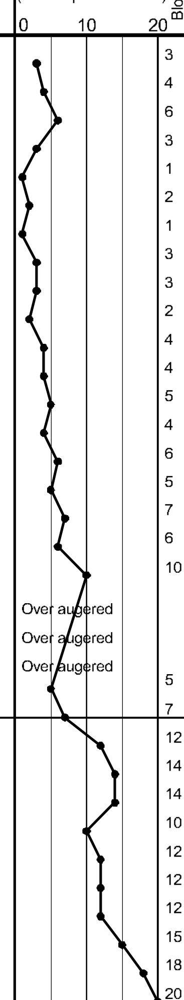
Site Identification: **HA01**

Sheet 1 of

Project: Knightsbridge Lane	**Coordinates:** E 2486 100, N 5743 612		**Datum:** NZMG
Client: Christchruch City Council	**Surface RL (m):**		**Total Depth:** 2.4m
Site: Knigthsbridge Lane	**Commenced:** 06-Nov-12	**Contractor:**	
Job No.: 513090276	**Completed:** 06-Nov-12		

Equipment:	**Shear Vane:**	**Logged:**	DW/DF
Hole Diameter (mm):		**Processed:**	DW
		Checked:	

Depth (m)	Water	Depth (m)	Geological Unit	Graphic Log	Classification	SOIL DESCRIPTION: (Soil Code), Soil Name [minor MAJOR], colour, structure [zoning, defects, cementing], plasticity or grain size, secondary components, structure. (Geological Formation)	Moisture Condition	Consistency/ Relative Density	Sample Type & Depth	Sample No.	Sample/ Test Records & Comments	Test Results (blows per 100mm) Blowcounts 0 10 20	Depth Scale (m)
		0.25			OL	TOPSOIL; organic SILT with rootlets, brown. Stiff; moist.	M	St					
		0.35			ML	SILT; brown. Stiff; low plasticity.	M	St					
		0.65			SP	Fine to medium SAND; brown. Loose; poorly graded.	M	L					
		0.75			SP	Fine to medium organic SAND; blackish brown. Loose; poorly graded.	M	L					
1		1.50			SP	Fine to medium SAND; brownish grey. Loose to mediuom dense; poorly graded.	M	L					1
2		2.40			SM	Silty fine to medium SAND; grey. Medium dense; poorly graded.	M	MD					2
3						Termination Depth = 2.4m (Collapse)							3
4													4
5													5



HAND AUGER LOG

GHD Limited
PO Box 13468
Christchurch 8141

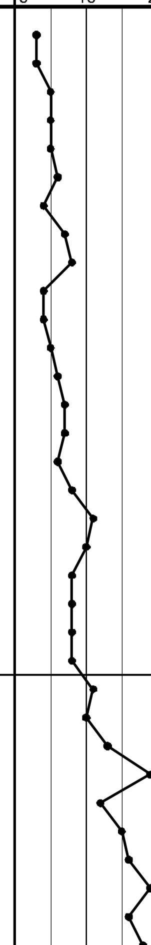
Site Identification: **HA02**
Sheet 1 of

Project: Knightsbridge Lane	**Coordinates:** E 2486 087, N 5743 600		**Datum:** NZMG
Client: Christchruch City Council	**Surface RL (m):**		**Total Depth:** 2.4m
Site: Knigthsbridge Lane	**Commenced:** 06-Nov-12	**Contractor:**	
Job No.: 513090276	**Completed:** 06-Nov-12		

Equipment:		**Logged:**	DW/DF
	Shear Vane:	**Processed:**	DW
Hole Diameter (mm):		**Checked:**	

Depth (m)	Water	Depth (m)	Geological Unit	Graphic Log	Classification	SOIL DESCRIPTION: (Soil Code), Soil Name [minor MAJOR], colour, structure [zoning, defects, cementing], plasticity or grain size, secondary components, structure. (Geological Formation)	Moisture Condition	Consistency/ Relative Density	Sample Type & Depth	Sample No.	Sample/ Test Records & Comments
		0.20			OL	TOPSOIL; organic SILT with rootlets, brown. Firm; moist.	M	St			
					SP	Fine to medium SAND; brown. Loose to medium dense; poorly graded.	M	L			
		2.35				@ 1.4m becomes grey	M	MD			
						Termination Depth = 2.35m (Collapse)					

Test Results (blows per 100mm) 0 10 20 Blowcounts



Blowcounts: 3, 3, 5, 5, 5, 6, 4, 7, 8, 4, 4, 5, 6, 7, 7, 6, 8, 11, 10, 8, 8, 8, 8, 11, 10, 13, 19, 12, 15, 16, 19, 16, 18

Depth Scale (m): 1, 2, 3, 4, 5

HAND AUGER LOG

GHD Limited

PO Box 13468
Christchurch 8141

Site Identification: **HA03**

Sheet 1 of

Project: Knightsbridge Lane	**Coordinates: E** 2486 086, **N** 5743 590		**Datum:** NZMG
Client: Christchruch City Council	**Surface RL (m):**		**Total Depth:** 2.3m
Site: Knigthsbridge Lane	**Commenced:** 06-Nov-12	**Contractor:**	
Job No.: 513090276	**Completed:** 06-Nov-12		

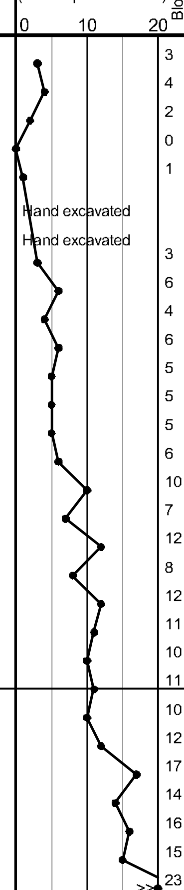
Equipment:

Shear Vane:

Hole Diameter (mm):

Logged:	DW/DF
Processed:	DW
Checked:	

Depth (m)	Water	Depth (m)	Geological Unit	Graphic Log	Classification	SOIL DESCRIPTION: (Soil Code), Soil Name [minor MAJOR], colour, structure [zoning, defects, cementing], plasticity or grain size, secondary components, structure. (Geological Formation)	Moisture Condition	Consistency/ Relative Density	Sample Type & Depth	Sample No.	Sample/ Test Records & Comments	Test Results (blows per 100mm) 0 10 20 Blowcounts	Depth Scale (m)
		0.20			OL	TOPSOIL; organic SILT with rootlets, brown. Stiff; moist.	M	St					
					SP	Fine to medium SAND; brown. Loose; poorly graded.	M	L					
1						@0.9 becomes medium dense	M	MD					1
						@ 1.4m becomes grey	M	MD					
2		2.35											2
						Termination Depth = 2.3m (Collapse)							
3													3
4													4
5													5



Appendix B

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.

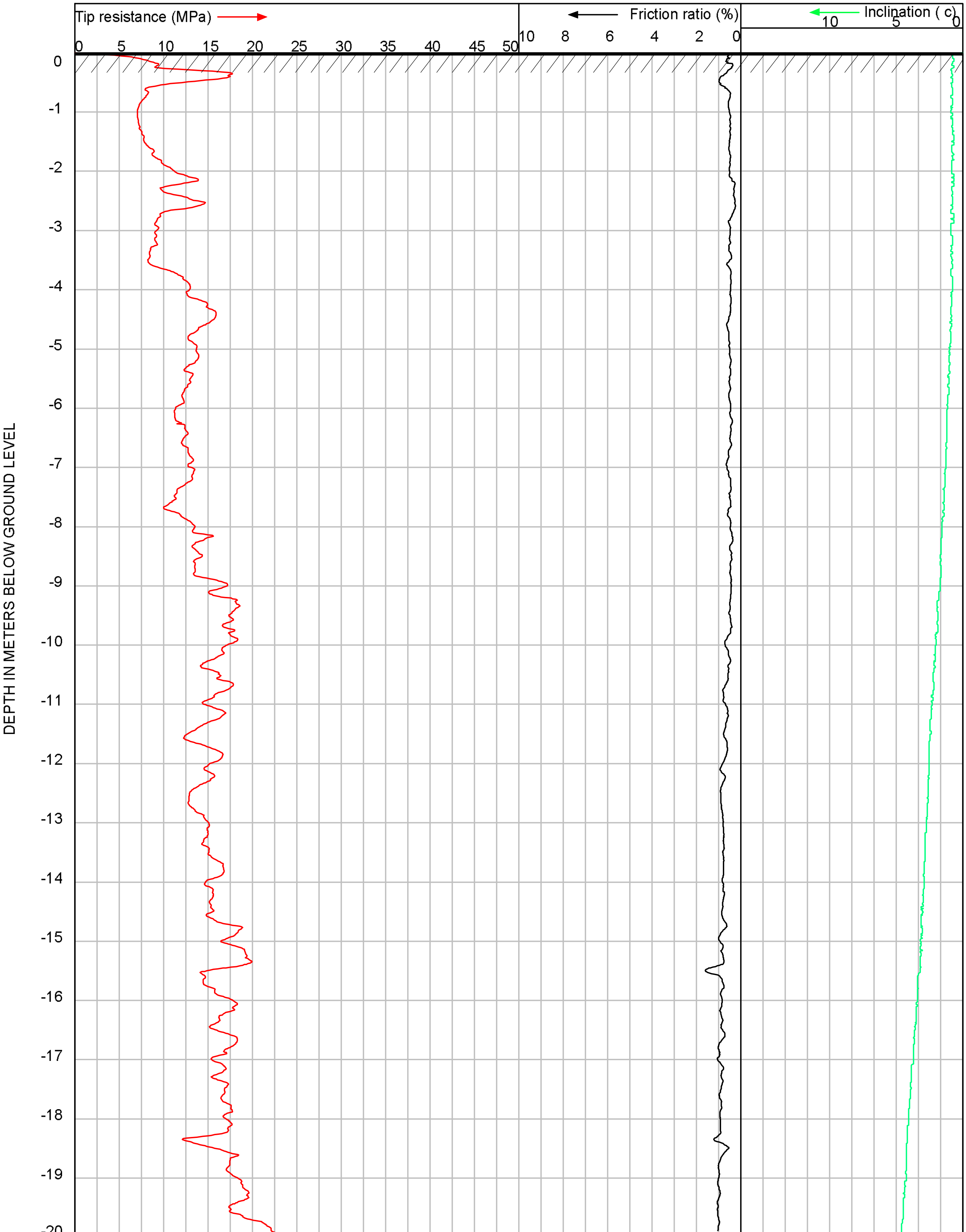
Relative Density (D_R)

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 $S_U = (q_C - \sigma_{VO})/15$.

McMILLAN
DRILLING SERVICES



CLIENT : GHD NZ Ltd
LOCATION : 395 Pages Road, Christchurch
DATE : 6-11-2012
OPERATOR : S.Cardona
REMARK 1 : CPT001
REMARK 2 : Target depth

JOB # : 11260
TEST # : 1
CONE TYPE/SERIAL #: I-CFXY-10 / 080238

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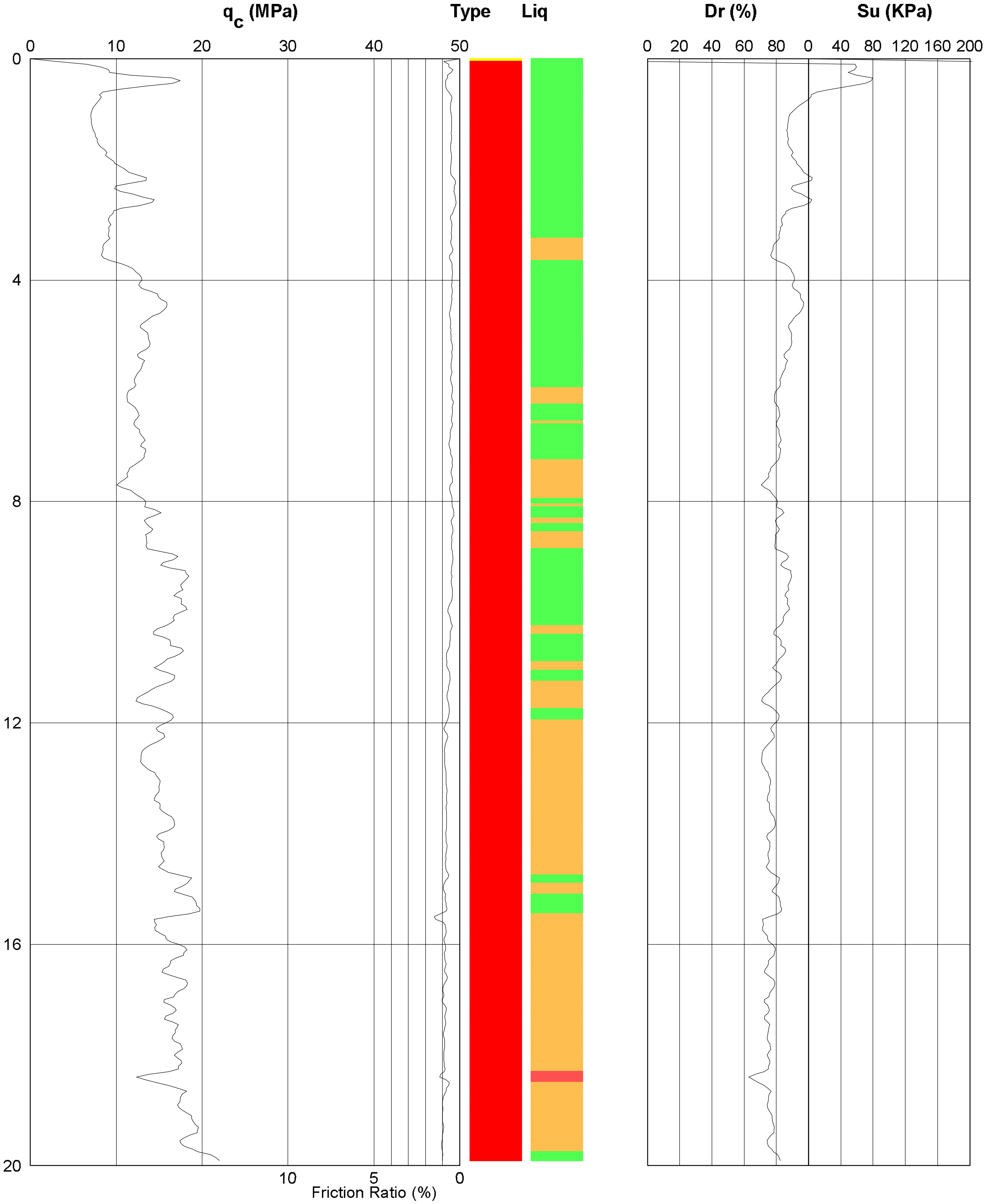
120 High St Southbridge CANTERBURY NZ
Ph +64 3 324 2571 Fax +64 3 324 2431
www.drilling.co.nz

PIEZOCONE PENETROMETER TEST (CPTU) INTERPRETIVE REPORT



Job No:	11260	Date:	6-11-2012
CPT No:	CPT001	Operator:	S.Cardona
Project:	GHD NZ Ltd	Remark:	Target depth
Location:	395 Pages Road, Christchurch		

McMILLAN DRILLING SERVICES

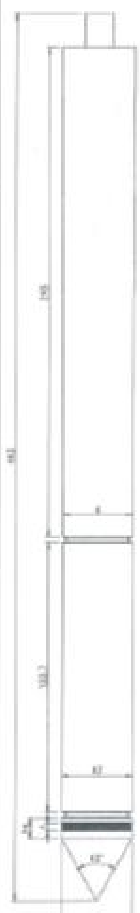
CPT CALIBRATION AND TECHNICAL NOTES

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

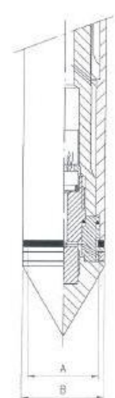
- ELCI-10CFXY measuring cone resistance, sleeve friction and inclination (standard cone);
- ELCI-CFXYP20-10 measuring cone resistance, sleeve friction, inclination and pore pressure (piezo cone).

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:	$d \le d_2 < d_1 + 0,35$	
Tip: (production dimension)	$d = 35,7\ ^{+0,2}_{0}$	
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



A=0,25x3,14x30,9x30,9=750 MM^2
B=0,25x3,14x35,7x35,7=1000 MM^2

$\alpha = A/B \qquad \beta = 1 - A/B$

$\alpha = 750/1000 = 0,75$
$\beta = 1 - 0,75 = 0,25$

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:	55346.001
Client:	Mc Millan
Cone-type:	ELCI 10 CFXY
Cone-number:	080238

To test / To check item		**Required value**	**Checked value**
Isolation-resistance		>0.5 G-Ohm	O.K. Gohm
Straightness		S=<0,2 mm	O.K. mm
Zero-Value Tip		Good	-2,74 MPa
Zero-Value Local Friction		Good	-0,043 MPa
Zero-Value Pore Pressure	/	Good	n.v.t. kPa
Zero-Value Inclination X Zero-Value Inclination Y		-2°< X <+2° -2° < Y <+2°	-0,1 ° 0,0 °
Measurements Tip resistance OK?		Yes	0-50 MPa
Influence of Tip on Local Friction? (Tip: 100 kN; Mantle free?)		No influence	O.K.
Measurements Local Friction OK?		Yes	0-0,667 MPa
Measurements Pore Pressure OK?	/	Yes	n.v.t. kPa
Measurements Inclination OK?		Yes	-24-0-24
Cone recognition on disconnecting and connecting Icone again?		Yes	O.K.
Software version 1.7 installed? Check at opening screen		Yes	O.K.
Thresholds for rapid exit set to maximum		Yes	O.K.

Remarks:

/

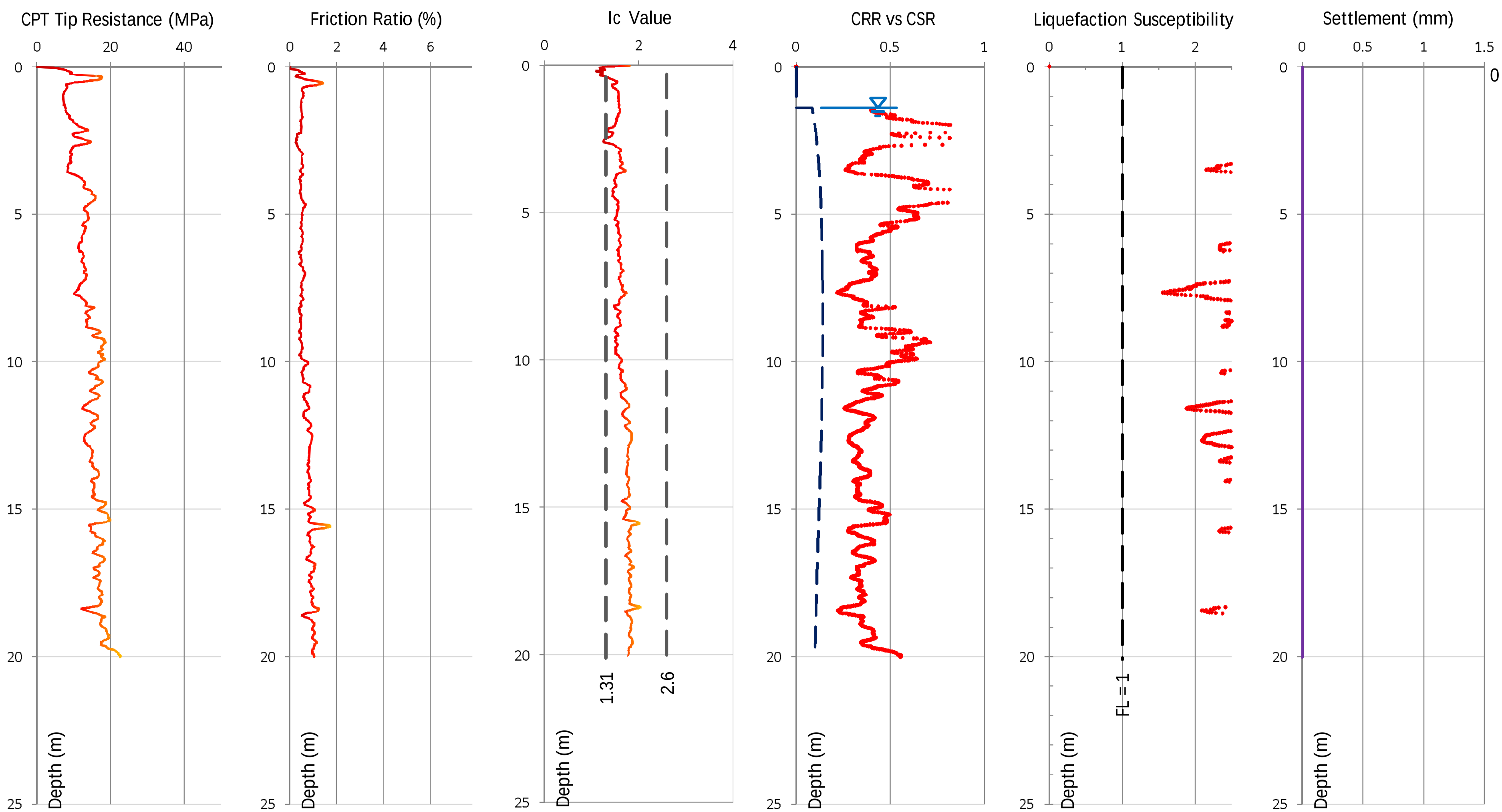
Calibrated by: C.J. Ouwejan	Date: 22-11-'11	Sign.:
Final check: J.E. Tenhage	Date: 22-11-'11	Sign.:

Appendix C

SOIL LIQUEFACTION SUSCEPTIBILITY ASSESSMENT

LOCATION :	CPT01	SHEET :	2 of 2
PROJECT :	Knightsbridge Lane	CALCULATED :	RV
JOB NO :	51 30902 76	CHECKED BY :	
TEST DATE :	6 Nov 2012	DATE :	27 Nov 2012

PGA (a_{max}):	0.13 g	Groundwater Level (m bgl):	1.4	Bore depth (m):	20	**Total Estimated Settlement (mm)**
EQ Magnitude:	7.5	Atmospheric Pressure (kPa):	101	Test data step (m):	0.01	**0**



N:\NZ\Wellington\Projects\51\30902\76 Knightsbridge Lane\Investigation\CPT\CPT01 SLS Liquefaction Analysis

SOIL LIQUEFACTION SUSCEPTIBILITY ASSESSMENT

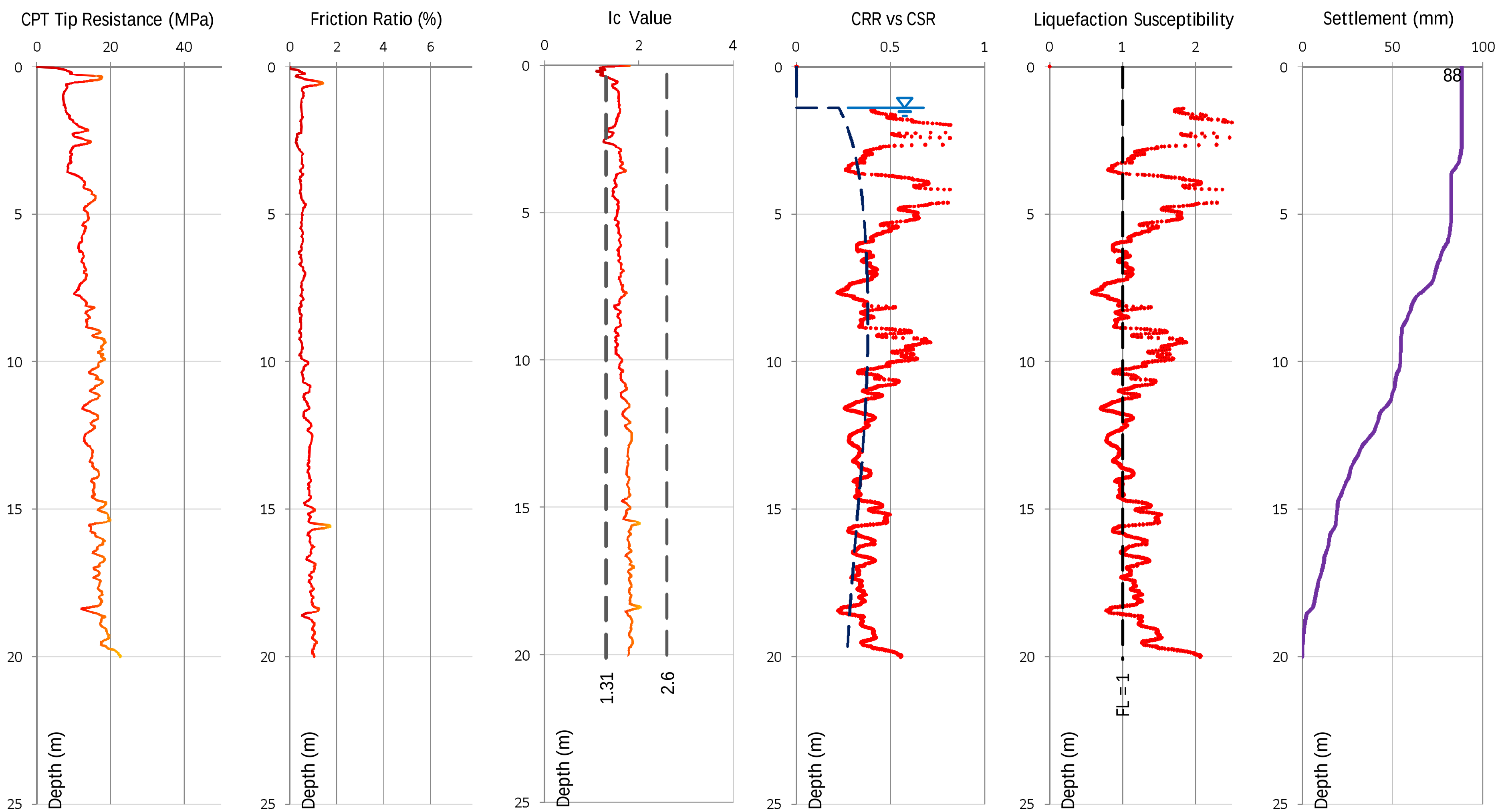
LOCATION :	CPT01	SHEET :	1 of 2
PROJECT :	Knightsbridge Lane	CALCULATED :	RV
JOB NO :	51 30902 76	CHECKED BY :	
TEST DATE :	6 Nov 2012	DATE :	27 Nov 2012

PGA (a_{max}): **0.35 g**
EQ Magnitude: **7.5**

Groundwater Level (m bgl): **1.4**
Atmospheric Pressure (kPa): **101**

Bore depth (m): **20**
Test data step (m): **0.01**

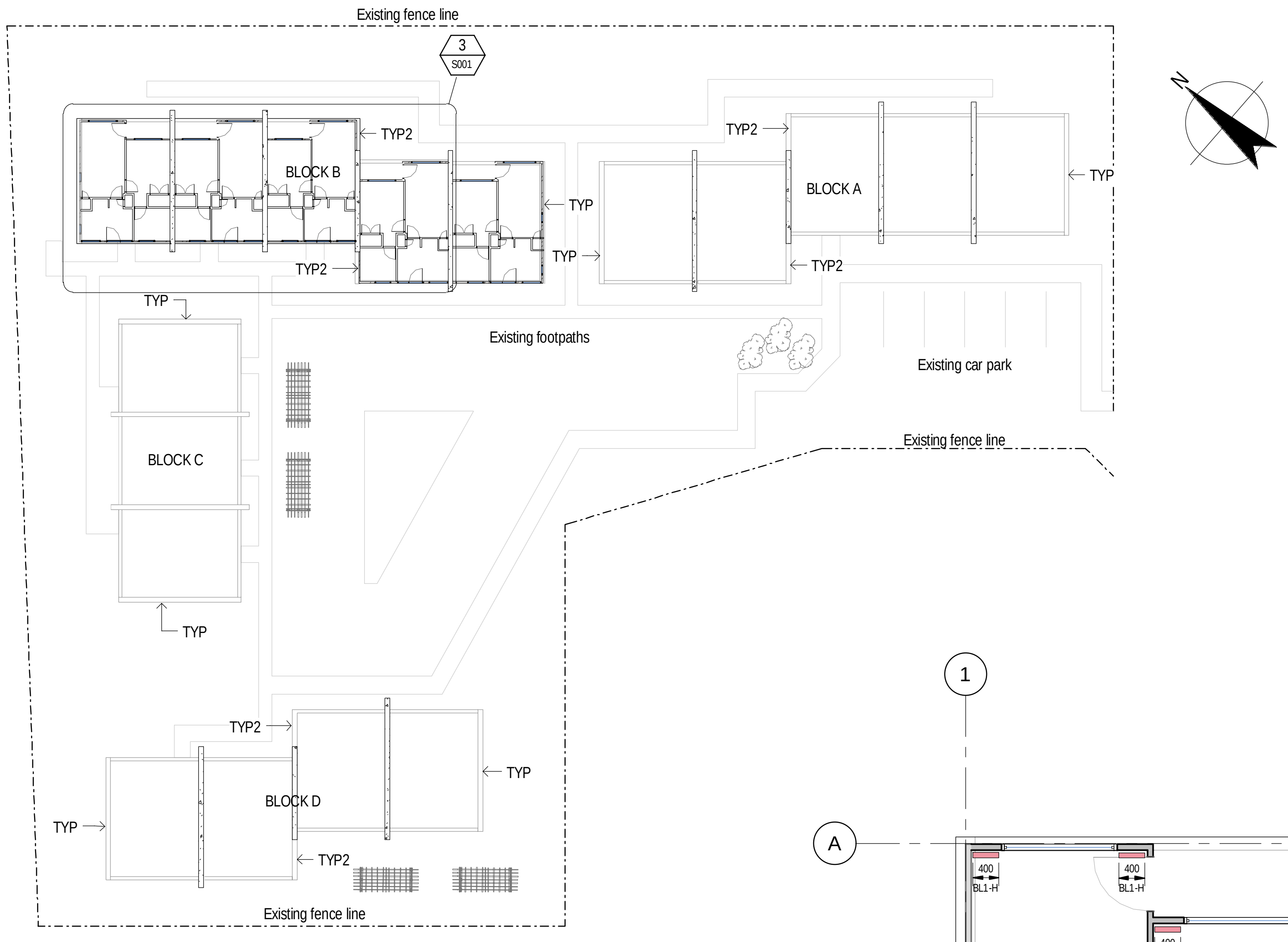
Total Estimated Settlement (mm)
88



N:\NZ\Wellington\Projects\51\30902\76 Knightsbridge Lane\Investigation\CPT\CPT01 ULS Liquefaction Analysis

Appendix E

Repair and Strengthening Drawings

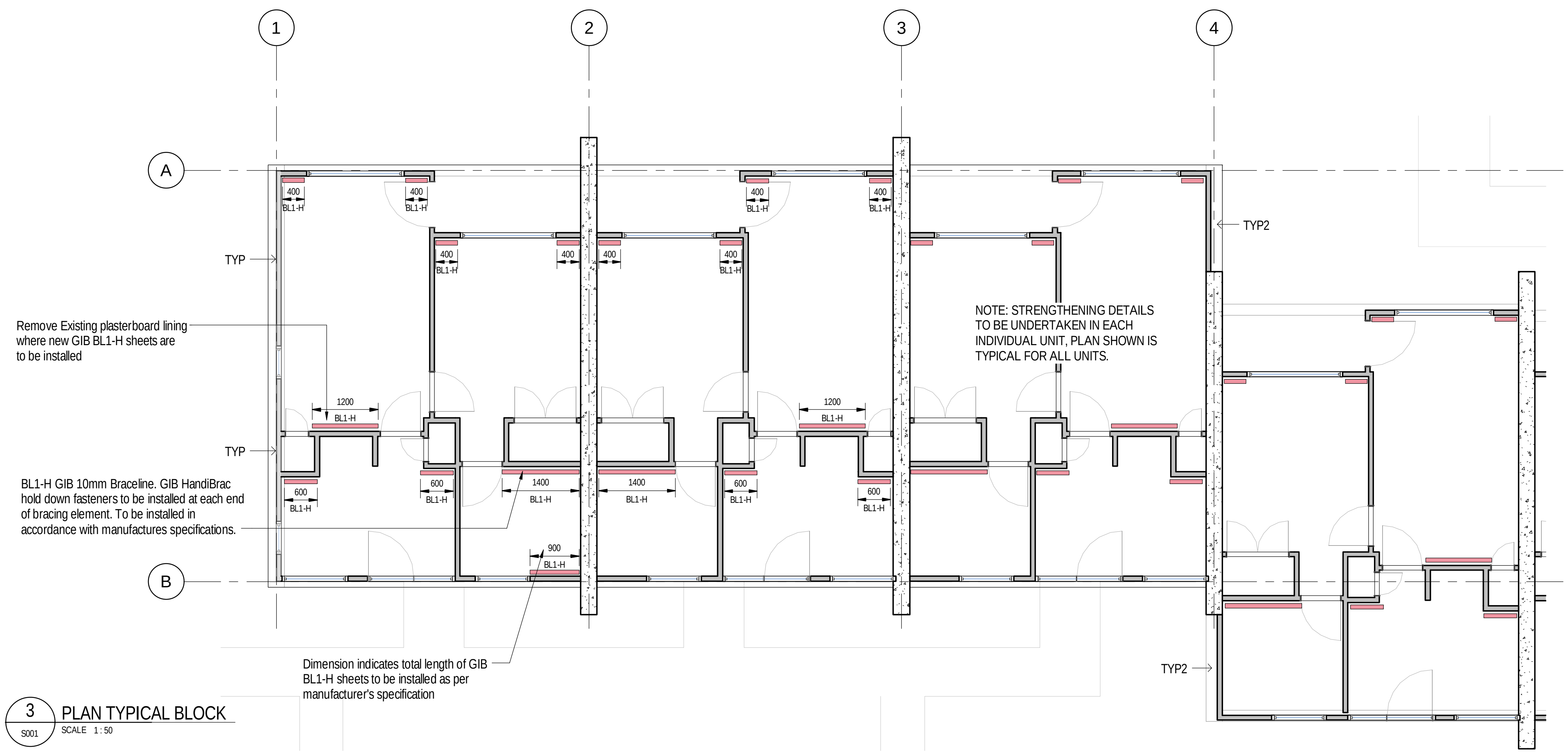


1 OVERALL SITE PLAN

SCALE 1 : 200

GENERAL NOTES

1. BL1-H 10mm GIB Braceline
2. Bracing to be applied to one side only as shown in plan view
3. GIB HandiBrac hold down fasteners to be installed at each end of the bracing element as per the manufacturer's specification, included on S102
4. All dimension are in millimeters unless noted otherwise
5. Re-Lining of walls required for all units
6. (TYP) notation indicates alterations to cladding as per drawing and details on S002
7. (TYP2) notation indicates alterations to cladding as per drawing and details on S002 - detail 8



3 PLAN TYPICAL BLOCK

S001

SCALE 1 : 50

CONSTRUCTION ISSUE

No	Revision	Drawn	Job Manager	Project Director	Date
B	REVISED BRACING PLAN	BL	SL	NW	31.05.13
A	FOR BUILDING CONSENT	SW	SL	NW	01/03/13

Note: * indicates signatures on original issue of drawing or last revision of drawing

GHD

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15 Willeston Street, Wellington New Zealand
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DO NOT SCALE

Drawn	S. WICKHAM	Designer	A. BAYLIS
Drafting Check	D. ANGUS	Design Check	S. LEE
Approved (Project Director)			
Date	01/03/13		
Scale	As indicated	This Drawing must not be used for Construction unless signed as Approved	

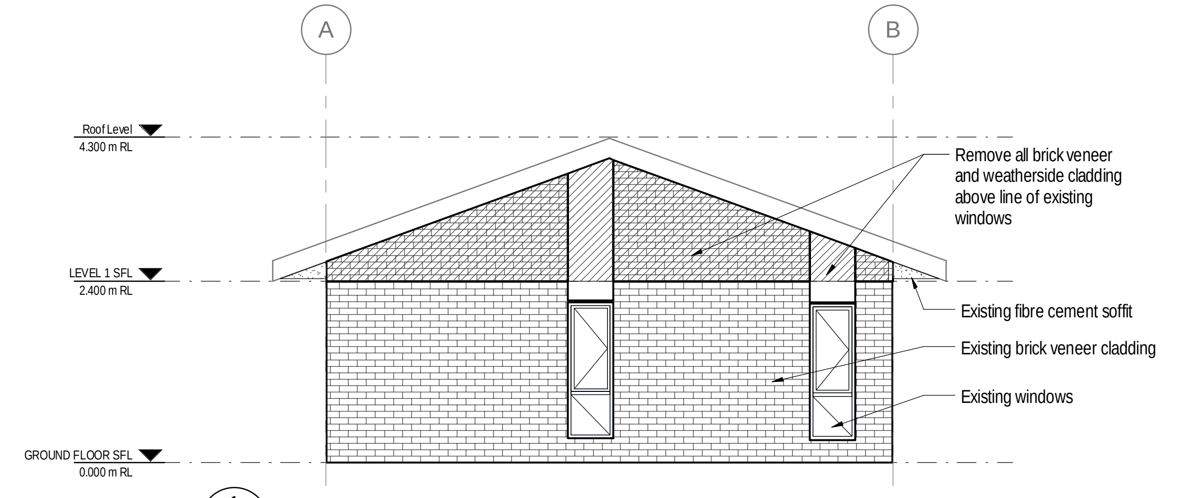
Client **CHRISTCHURCH CITY COUNCIL**

Project **KNIGHTSBRIDGE LANE COMPLEX**

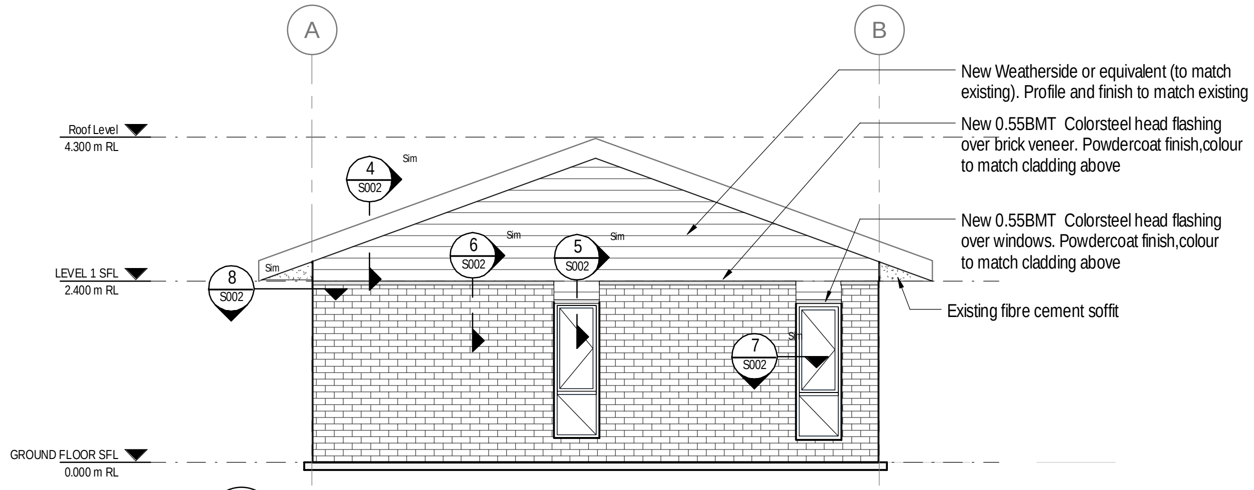
Title **STRENGTHENING GROUND FLOOR PLAN**

Original Size **A1** Drawing No: **51/31526/01-S001** Rev: **B**

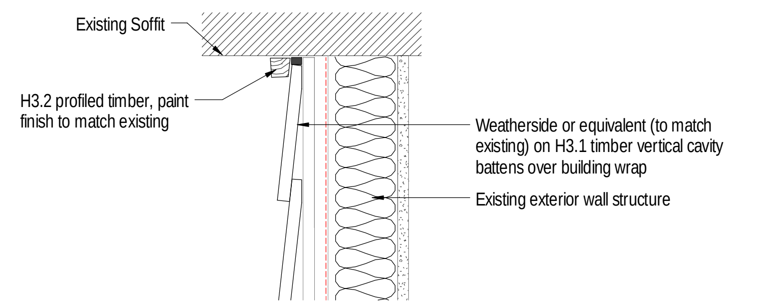
Plot Date: 31/05/2013 11:43:52 a.m. Cad File No: N:\NZ\Wellington\Projects\51\30902\76 Knightsbridge Lane\Drawings\Knightsbridge Courts Housing 28.02.13.rvt



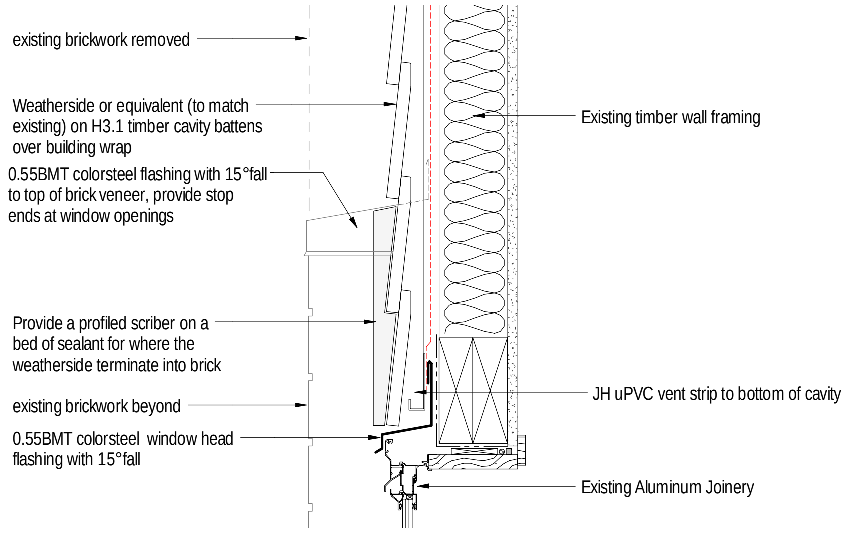
1 EXISTING WEST ELEVATION - TYP
SCALE 1 : 50



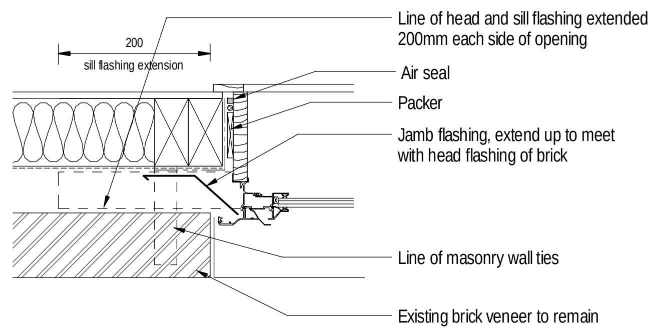
2 PROPOSED WEST ELEVATION - TYP
SCALE 1 : 50



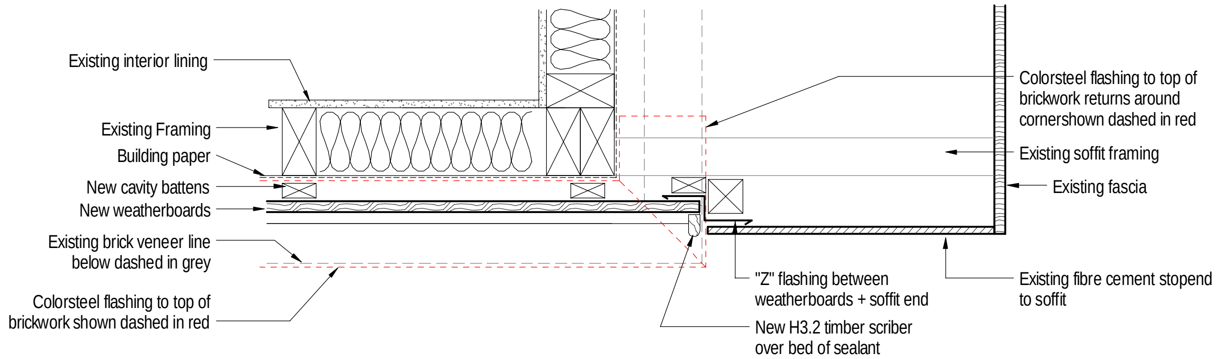
4 DETAIL SECTION
S002 Weatherside Soffit Detail 1 : 5



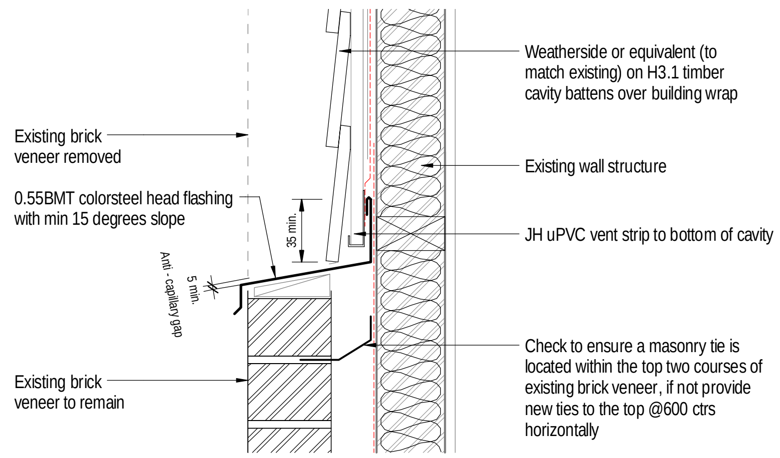
5 DETAIL SECTION
S002 Weatherside Over Window Detail 1 : 5



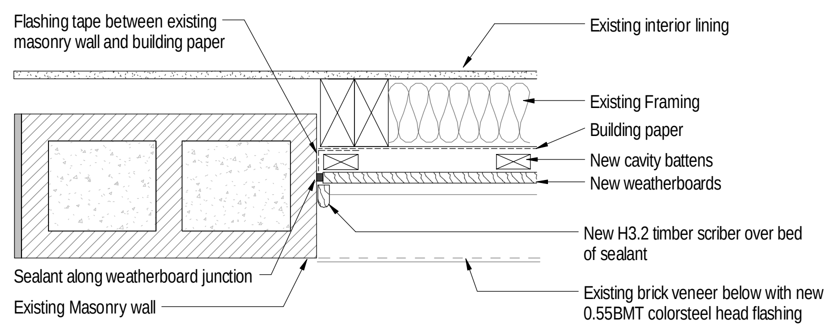
7 DETAIL SECTION
S002 Window Jamb 1 : 5



3 DETAIL SECTION
Weatherboard Junction with Existing Soffit 1 : 5



6 DETAIL SECTION
S002 Weatherside Over Brick Detail 1 : 5



8 DETAIL SECTION
S002 Weatherboard / Masonry Wall Junction 1 : 5

CONSTRUCTION ISSUE

No	Revision Note: * indicates signatures on original issue of drawing or last revision of drawing	Drawn	Job Manager	Project Director	Date
C	DETAILS REVISED	BL	SL	NW	30.05.13
B	FOR BUILDING CONSENT - ALTERATIONS CLOUDED	BL	SL	NW	24.04.13
A	FOR BUILDING CONSENT	SW	SL	NW	01/03/13

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DO NOT SCALE

GHD Limited

Drawn	S. WICKHAM	Designer	A. BAYLIS
Drafting Check	D. ANGUS	Design Check	S. LEE
Approved (Project Director)			
Date	01/03/13		
Scale	As indicated	This Drawing must not be used for Construction unless signed as Approved	

Client **CHRISTCHURCH CITY COUNCIL**
Project **KNIGHTSBRIDGE LANE COMPLEX**
Title **STRENGTHENING**
ELEVATIONS AND DETAILS SHEET 1

Original Size **A1** **Drawing No: 51/31526/01-S002** **Rev: C**

Plot Date: 31/05/2013 11:43:54 a.m.
Cad File No: N:\NZ\Wellington\Projects\51\30902\76 Knightsbridge Lane\Drawings\Knightsbridge Courts Housing 28.02.13.rvt

Bottom Plate Fixing

JUNE 2011

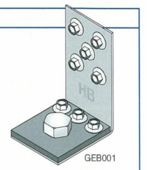
Bottom plate fixings for GIB® Bracing Elements			
Brace type	**Concrete slabs**		**Timber floors**
	External wall	**Internal wall**	**External and Internal walls**
GS1-N	As per NZS 3604:2011. No specific additional fastening required	As per NZS 3604:2011. Alternatively use 75 x 3.8mm shot-fired fasteners with 16mm washers, 150mm and 300mm from each end of the bracing element and at 600mm thereafter.	Pairs of 100 x 3.75mm flat head hand driven nails or 3 / 90 x 3.15mm power driven nails at 600mm centres in accordance with NZS 3604:2011
GS2-N	Not applicable		
GSP-H BL1-H BLP-H	Intermediate fastenings to comply with NZS 3604:2011. In addition: GIB Handibrac® fixings or metal wrap-around strap fixings and bolt as illustrated on pages 19 and 20.		Pairs of 100 x 3.75mm flat head hand driven nails or 3 / 90 x 3.15mm power driven nails at 600mm centres in accordance with NZS 3604:2011. In addition: GIB Handibrac® fixings or metal wrap-around strap fixings and bolt as illustrated below.
BLG-H	Not applicable	As for GSP-N, BL1-H, BLP-H on concrete slab above	

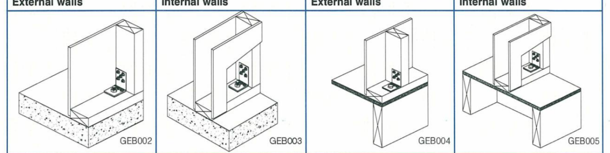
Panel Hold-down Details

GIB HandiBrac® – RECOMMENDED METHOD

Developed in conjunction with MiTek™ NZ, the GIB HandiBrac® has been designed and tested for use as a hold-down in GIB® BL and GSP bracing elements.

- The GIB HandiBrac® registered design provides for quick and easy installation
- The GIB HandiBrac® provides a flush surface for the wall linings because it is fitted inside the framing. There is no need to check in the framing as recommended with conventional straps
- The GIB HandiBrac® is suitable for both new and retrofit construction
- The design also allows for installation and inspection at any stage prior to fitting internal linings



Concrete Floor		Timber Floor	
External walls	**Internal walls**	**External walls**	**Internal walls**
			
Position GIB HandiBrac® as close as practicable to the internal edge of the bottom plate	Position GIB HandiBrac® at the stud / plate junction	Position GIB HandiBrac® in the centre of the perimeter joist or bearer	Position GIB HandiBrac® in the centre of floor joist or full depth solid block

Hold-down fastener requirements	
A mechanical fastening with a minimum characteristic uplift capacity of 15kN.	12x150mm galvanised coach screw

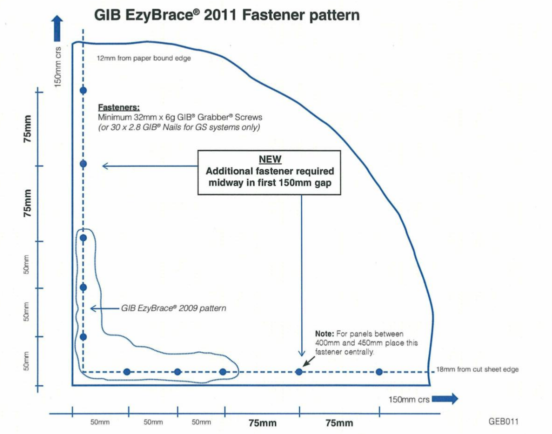
Refer to gib.co.nz/cad for CAD details.

Construction Details

JUNE 2011

Revised Fastener Pattern for *all four corners* of GIB EzyBrace® Elements

As GIB Braceline® screws are no longer required for BL bracing elements, two additional fasteners must be installed in ***all four corners*** of GIB EzyBrace® GS and BL elements, as shown.
Fasteners must be placed no closer than 12mm from the paper bound sheet edge and no closer than 18mm from sheet ends or cut edges.



Refer to gib.co.nz/cad for CAD details.

PERMITTED GIB® PLASTERBOARD SUBSTITUTIONS IN GIB EZYBRACE® SYSTEMS

GIB Ezybrace® Systems have been designed and tested using only the products specified. Occasionally additional properties may be required to be provided by a different GIB® Plasterboard product. The following table provides acceptable substitution options.

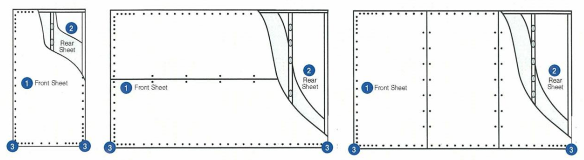
Specified	Permitted alternative GIB® Plasterboard products								
	GIB® Standard	GIB Ultraline®	GIB Braceline/ Noiseline®	GIB Aqualine®	GIB Toughline®	GIB Fyreline® 10mm	13mm	16mm	19mm
GIB® Standard		OK	OK	OK	OK	OK	NOTE 2		
GIB Braceline®	X	X		NOTE 1	OK	X	NOTES 1 and 2		

NOTE 1 The element must be 900mm or longer. Use 32mm x 6g GIB® Grabber® drywall screws at ***100mm*** centres to the perimeter of the bracing element. The bracing corner fastening pattern, as illustrated above, applies to all four corners of the element. Panel hold-down fixings are required.

NOTE 2 The fastener type and length must be as required for the relevant FRR system but the fixing pattern must be as shown above.

Construction Details

JUNE 2011



System	Lining one side 1		Lining opposite side 2		Panel Hold-Down Fixings 3	Fastener spacing
	Lining	**Fasteners**	**Lining**	**Fasteners**		
GS1-N	Any 10mm or 13mm GIB® Plasterboard	30mm GIB® nails, or minimum 32mm x 6g GIB® Grabber® high thread screws	Not required	Not required	Not required	*GIB® Plasterboard* Corner fastening pattern as illustrated above Fasteners at 150mm to bracing element perimeter, and: • at 300mm centres to intermediate sheet joints for vertical fixing, or • at stud / sheet junction for horizontally fixed elements, and • GIBFix adhesive daubs at 300mm crs to intermediate framing *Plywood* Fasteners at 150mm around the perimeter of every sheet and at 300mm centres to intermediate studs. Place fasteners no closer than 7mm from sheet edges. Plasterboard corner fastener pattern does not apply to plywood.
GS2-N			Any 10mm or 13mm GIB® Plasterboard	30mm GIB® nails, or minimum 32mm x 6g GIB® Grabber® high thread screws		
GSP-H			Minimum 7mm Ecoply manufactured to AS/NZS 2269	50mm x 2.8mm Flat head galvanised or stainless steel nails	Yes, see Pages 19 and 20	
BL1-H	10mm or 13mm GIB Braceline®	minimum 32mm x 6g GIB® Grabber® high thread screws	Not required	Not required		
BLG-H			Any 10mm or 13mm GIB® Plasterboard	30mm GIB® nails, or minimum 32mm x 6g GIB® Grabber® high thread screws		
BLP-H		GIB Braceline® Nails may be used for 10mm GIB Braceline® ONLY	Minimum 7mm Ecoply manufactured to AS/NZS 2269	50mm x 2.8mm flat head galvanised or stainless steel nails		

GIB EzyBrace® System Specification – BL1-H

JUNE 2011

Specification Code	Minimum Length (m)	Lining requirement	Other requirements
BL1-H	0.4	10mm or 13mm GIB Braceline® to one side only	Hold downs

WALL FRAMING
Wall framing to comply with;
- NZBC B1 - Structure; AS1 Clause 3 Timber (NZS 3604:2011)
- NZBC B2 - Durability AS1 Clause 3.2 Timber (NZS 3602)

Framing dimensions and height as determined by NZS 3604 stud and top plate tables for load bearing and non-bearing walls. The use of kiln dried stress graded timber is recommended.

BOTTOM PLATE FIXING

Timber Floor
Use panel hold downs at each end of the bracing element. The GIB HandiBrac® is recommended. See details in GIB Ezybrace® Systems 2011 or GIB® Site Guide.
Pairs of hand driven 100 x 3.75mm nails at 600mm centres; or
Three power driven 90 x 3.15 nails at 600mm centres.

Concrete floor
Use panel hold downs at each end of the bracing element. The GIB HandiBrac® is recommended. See details in GIB Ezybrace® Systems 2011 or GIB® Site Guide. Within the length of the bracing element bottom plates are to be fixed in accordance with the requirements of NZS 3604.

WALL LINING
One layer 10mm or 13mm GIB® Braceline.
Sheets can be fixed vertically or horizontally.
Sheet joints shall be touch fitted.
Use full length sheets where possible.

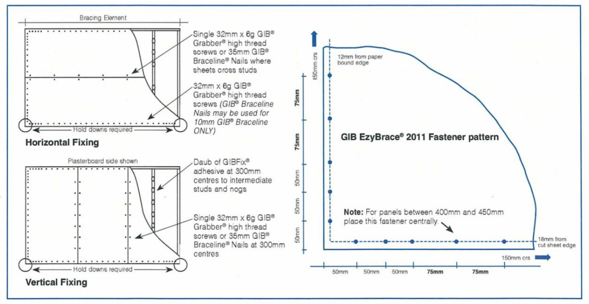
PERMITTED SUBSTITUTION
For permitted GIB® Plasterboard substitutions refer to Page 21 in GIB Ezybrace® Systems 2011.

FASTENING THE LINING

Fasteners
32mm x 6g GIB® Grabber® high thread screws.
(GIB Braceline® Nails may be used with 10mm GIB Braceline® only.)

Fastener centres
50,100,150, 225, 300mm from each corner and 150mm thereafter around the perimeter of the bracing element.
For vertically fixed sheets place fasteners at 300mm centres to the sheet joint.
For horizontally fixed sheets place single fasteners to the sheet edge where it crosses the stud.
Use daubs of GIB Fix® adhesive at 300mm centres to intermediate studs.
Place fasteners no closer than 12mm from paper bound sheet edges and 18mm from any sheet end or cut edge.

JOINTING
All fastener heads stopped and all sheet joints paper tape reinforced and stopped in accordance with the GIB® Site Guide.



In order for GIB® systems to perform as tested, all components must be installed exactly as prescribed. Substituting components produces an entirely different system and may seriously compromise performance. Follow the specifications. This Specification sheet is issued in conjunction with the publication GIB EzyBrace® Systems 2011 and has been appraised in accordance with the BRANZ Appraisal No. 294 (2011).

BRANZ Appraised
Appraisal No.294 [2011]

CONSTRUCTION ISSUE

No	Revision	Drawn	Job Manager	Project Director	Date
A	FOR BUILDING CONSENT	SW	SL	NW	01/03/13

Note: * indicates signatures on original issue of drawing or last revision of drawing

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DO NOT SCALE

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Approved (Project Director)			
Date	01/03/13		
Scale		This Drawing must not be used for Construction unless signed as Approved	

Client **CHRISTCHURCH CITY COUNCIL**
Project **KNIGHTSBRIDGE LANE COMPLEX**
Title **STRENGTHENING STRUCTURAL NOTES**

Original Size **A1** **Drawing No: 51/31526/01-S003** **Rev: A**

Plot Date: 31/05/2013 11:44:01 a.m.
Cad File No: N:\NZ\Wellington\Projects\51\30902\76 Knightsbridge Lane\Drawings\Knightsbridge Courts Housing 28.02.13.rvt

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Document Status

Rev No.	Author		Reviewer	Approved for Issue		
	Author	Signature	Name	Name	Signature	Date
Final (V2.0)	Hamish Mackinven		Peter O'Brien	Nick Waddington		October 2013