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Botanic Gardens Office Store
PRK 1566 BLDG 003 EQ2
Detailed Engineering Evaluation
Quantitative Report
Version FINAL

7 Rolleston Avenue, Christchurch

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Christchurch City Council

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Date
16 September 2013

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Botanic Gardens Office Store

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Detailed Engineering Evaluation

Quantitative Report - SUMMARY

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Background

This is a summary of the Quantitative report for the Botanic Gardens Office Store, and 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 4th April 2012, electromagnetic scans on the 19th September 2012 and available construction drawings.

Brief Description

The Botanical Gardens Office Store is located at 7 Rolleston Avenue, Christchurch Central. The building was constructed in 1969. The building is L shaped. The building utilises two different structural systems; Load bearing masonry walls and steel frames.

The roof is lightweight metal cladding fixed to timber purlins running longitudinally between steel frames in some portions of the building and timber trusses in others. The walls are partially reinforced concrete masonry units. Reinforcement is provided at intersections between walls and around windows and doors. The building has large sliding doors to allow access for vehicles.

Floors throughout the entire building are unreinforced concrete slabs. Foundations are reinforced concrete strip footings under the masonry walls. Foundations under the steel columns have wider base.

Indicative Building Strength

Future plans for the operations facility area will see demolition of a portion of the building and as such that portion was omitted from this report.

From a detailed assessment of all the walls and the structural steel, the building, excluding any structural damage, has been assessed as achieving 24 **%**NBS. The critical elements contributing to this low % NBS are the unfilled and effectively unreinforced concrete masonry walls. Under the New Zealand Society for Earthquake Engineering (NZSEE) guidelines the office store is considered to be an Earthquake Prone building.

Recommendations

The recent seismic activity in Christchurch has caused minor damage to the building, with cracking in concrete masonry walls the only damage noted.

The building has however been assessed as being a potentially Earthquake Prone building as it has achieved less than 34% NBS. As such, GHD Limited recommend that strengthening options be explored in order to increase the % NBS of the building to 67% NBS as recommended by the NZSEE Guidelines.

1. Background

GHD Limited has been engaged by the Christchurch City Council (CCC) to undertake a detailed engineering evaluation of the Botanic Gardens Office Store.

This report is a Quantitative Assessment and is based in general on NZS 1170.5: 2004, NZS 3404: 2009, the New Zealand Society for Earthquake Engineering (NZSEE) guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Resistance (02/2011) and the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes (06/2006).

The quantitative assessment to the building comprises an investigation on in-plane and out-of-plane strength of the unreinforced masonry block walls of the building and an assessment of the capacities of structural steel elements. The investigation is based on the analysis of the seismic loads that the structure is subjected to, the analysis of the distribution of these forces throughout the structure and the analysis of the capacity of existing structural elements to resist the forces applied. The capacity of the existing structural elements is compared to the demand placed on the element to give the percentage of New Building Standard (%NBS) of each of the structural elements.

Electromagnetic scans have been carried out on site to ascertain the extent of the reinforcement in the masonry walls. Reinforcing within the walls was only present where walls intersected at right angles and around window and door openings. No other reinforcement was identified.

Finite element modelling of the building structure has been carried out to ascertain the distribution of seismic loads through the building.

2. Compliance

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

2.1 Canterbury Earthquake Recovery Authority (CERA)

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

Section 38 – Works

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

Section 51 – Requiring Structural Survey

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

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

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

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

- The importance level and occupancy of the building
- The placard status and amount of damage
- The age and structural type of the building
- Consideration of any critical structural weaknesses
- The extent of any earthquake damage

2.2 Building Act

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

Section 112 – Alterations

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

Section 115 – Change of Use

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

2.2.1 Section 121 – Dangerous Buildings

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

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

Section 122 – Earthquake Prone Buildings

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

Section 124 – Powers of Territorial Authorities

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

Section 131 – Earthquake Prone Building Policy

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

2.3 Christchurch City Council Policy

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

The 2010 amendment includes the following:

- A process for identifying, categorising and prioritising Earthquake Prone Buildings, commencing on 1 July 2012;
- A strengthening target level of 67% of a new building for buildings that are Earthquake Prone;
- A timeframe of 15-30 years for Earthquake Prone Buildings to be strengthened; and,
- Repair works for buildings damaged by earthquakes will be required to comply with the above.

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

We anticipate that any building with a capacity of less than 33% NBS (including consideration of critical structural weaknesses) will need to be strengthened to a target of 67% NBS of new building standard as recommended by the Policy.

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

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

2.4 Building Code

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

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

- Hazard Factor increased from 0.22 to 0.3 (36% increase in the basic seismic design load)
- Serviceability Return Period Factor increased from 0.25 to 0.33 (80% increase in the serviceability design loads when combined with the Hazard Factor increase)

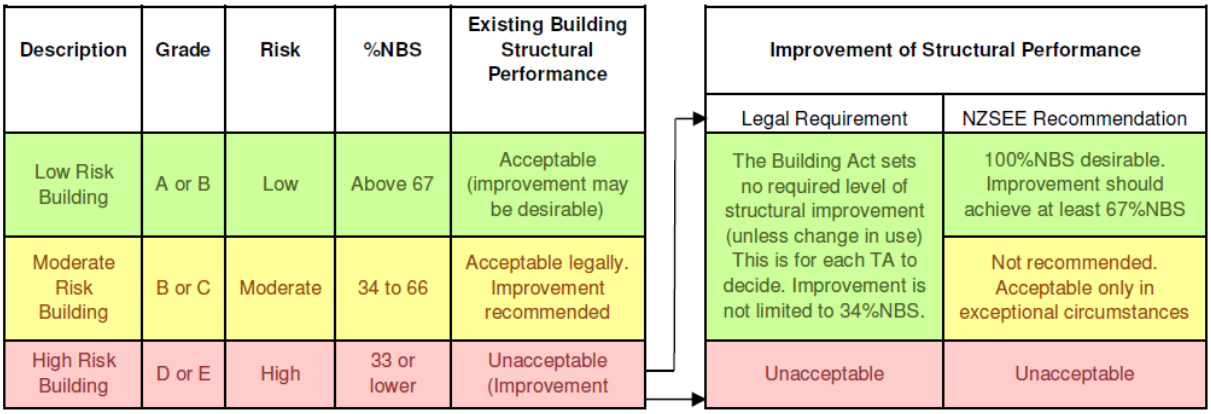
The increase in the above factors has resulted in a reduction in the level of compliance of an existing building relative to a new building despite the capacity of the existing building not changing.

3. Earthquake Resistance Standards

For this assessment, the building's earthquake resistance is compared with the current New Zealand Building Code requirements for a new building constructed on the site. This is expressed as a percentage of new building standard (%NBS). The new building standard load requirements have been determined in accordance with the current earthquake loading standard (NZS 1170.5:2004 Structural design actions - Earthquake actions - New Zealand).

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

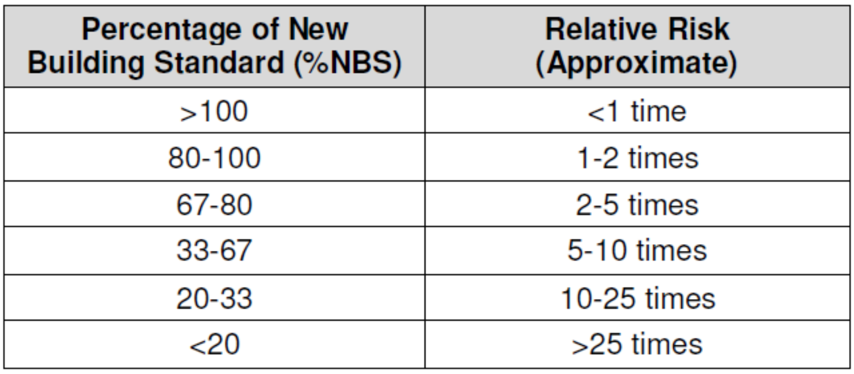
The New Zealand Society for Earthquake Engineering has proposed a way for classifying earthquake risk for existing buildings in terms of %NBS and this is shown in Figure 1 below.



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

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

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



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

Table 1 %NBS compared to relative risk of failure

4. Building Description

4.1 General

The Botanical Gardens Office Store is located at 7 Rolleston Avenue, Christchurch Central. The building was constructed in 1969. The building has various uses which include:

- Garage for vehicles
- Storage for mowers
- Office space
- Tool sheds
- Workshop
- Chemical storage
- General storage

For the purpose of analysis, the building can be split up into two wings (A and B). The western wing of the building that houses the offices, tool shed, chemical store and mower store will be referred to as Wing A. The southern wing that houses the vehicle garages and other storage units will be referred to as Wing B. (See Figure 2)

Future plans for the operations facility area will see demolition of the majority of Wing A.

The building utilises two different structural systems; Load bearing concrete masonry walls and steel portal frames.

In the northern half of Wing B, the main structural system comprises a number of steel frames. The roofs of this portion of the building are mono-pitched, sloping from west to east. The roof is constructed of lightweight metal cladding fixed to timber purlins on the steel portal frames. The eastern side of Wing B has reinforced concrete masonry unit walls between the columns of the steel portal frames. The western side of Wing B has large sliding doors to allow access for vehicles. The mower shed at the eastern end of Wing A is of similar construction.

The roofing system for the southern portion of Wing B comprises lightweight metal roof cladding on timber roof purlins attached to timber roof trusses. The walls are 200mm thick partially filled concrete masonry units. Reinforcement is present in the concrete masonry walls around all openings, at all intersections of perpendicular walls, through the parapets and through all bond beams.

Floors throughout the building are 125mm thick unreinforced concrete slabs. Foundations are reinforced concrete strip footings under the masonry walls. Pad foundations are under the steel posts and columns.

The buildings footprint is approximately L shaped with 2 wings. The western wing is 33.6m in length, 4.85m in width and 4.5m in height. The second wing is approximately 40.5m in length and 6.85m in width.

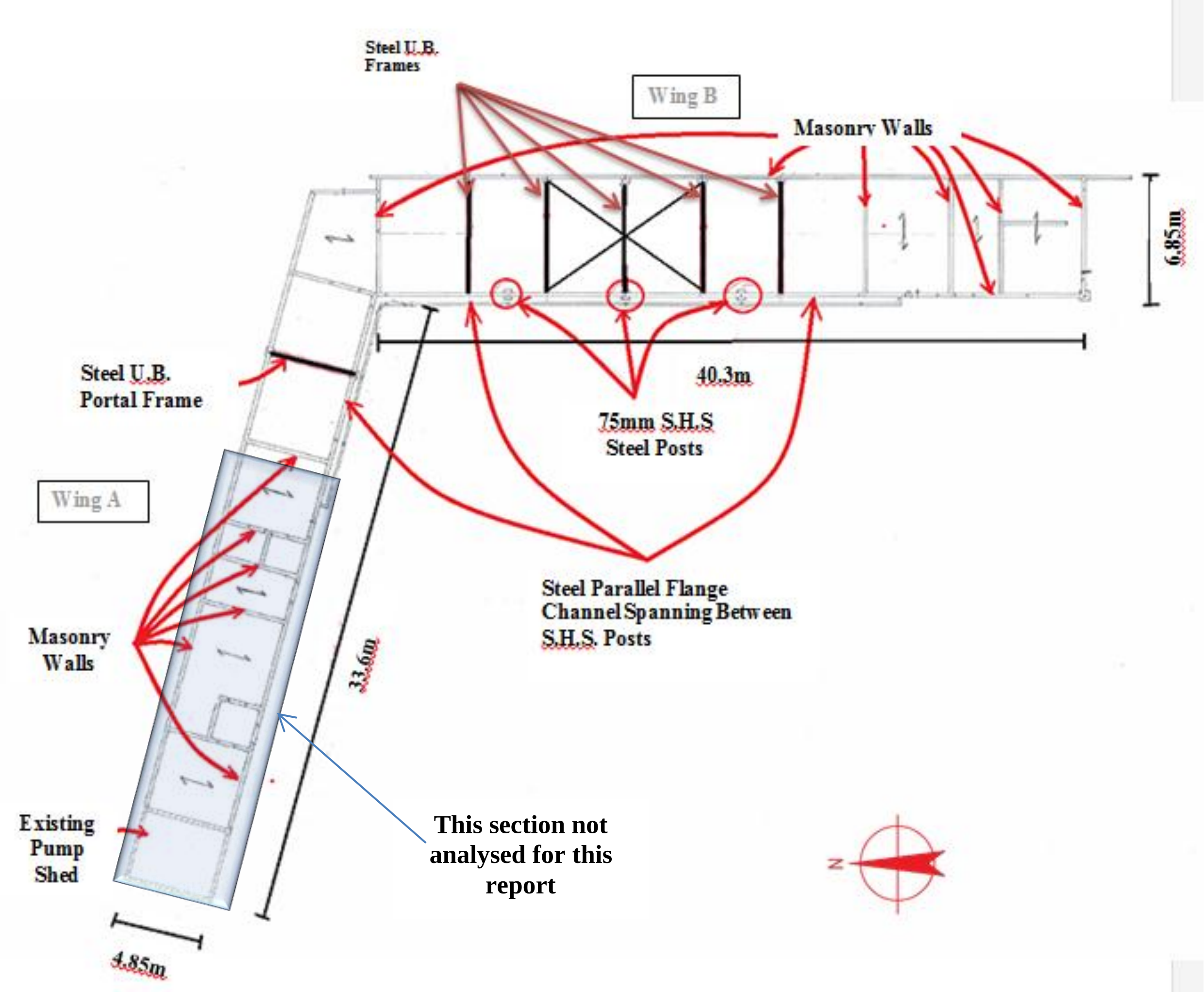


Figure 2 Plan of the building showing key structural elements

4.2 Gravity Load Resisting System

Roof loads are transferred through the lightweight metal cladding to the timber purlins. The timber purlins transfer the gravity loads back to the supporting steel portal frames and timber roof trusses.

Where loads have been transferred to the timber roof trusses the loads are then transferred to the supporting external concrete masonry walls and down to the supporting foundations. In the area of the steel rafters this load is supported by steel columns sitting on concrete pad foundations on the eastern side. At the western side, loads from the steel rafters are transferred to the supporting steel beam and post system and then down to the supporting pad foundations (See Figure 2).

Loads on the internal walls are transferred directly through to the supporting concrete strip foundations.

4.3 Lateral Load Resisting System

In Wing B the primary lateral load resisting structure in the longitudinal direction is the concrete masonry rear wall. It is anticipated that the steel frame system of the western wall will not provide significant resistance to lateral seismic loading. Flat steel strap cross bracing is present in a portion of the roof of the wing (See Figure 2) to transfer the lateral loads back to the rear wall of the building.

In the transverse direction, lateral loads are resisted by the partially filled and partially reinforced masonry walls to the north and south of this wing, and to some degree by the steel portal frames in between. Some diaphragm action is provided by the cross bracing (where present), the existing roof cladding in conjunction with the timber purlins spanning between the steel frames and the concrete masonry walls. The frames and walls transfer the lateral loads to the foundations.

5. Damage Assessment

5.1 Surrounding Buildings

The Office Store is located in the Botanic Gardens work-yard. To the east is a sports hall situated on the Christs College School Campus. To the south of Wing B is the Irrigation Pumphouse. To the west of Wing A are the glass houses and the potting sheds. To the west of Wing B are the Office Library and the Cycle Shelter. Shear cracking was noted to the blockwork in the office library building. Cracking was noted to several of the walls of the glass houses but the majority of these are not believed to be earthquake related.

5.2 Residual Displacements and General Observations

No residual displacements of the structure were noted during the inspection of the building.

No damage was evident to the roof structure.

No cracking was noted to the perimeter strip footing.

Shear cracking was observed to the block walls in several locations.

5.3 Ground Damage

No ground damage was observed during the inspection of the site. Ground remediation works have been carried out slightly to the west of the building. These works included strengthening of the river banks. The river is situated approximately 7m to the north of Wing A and runs parallel to the building. Due to remediation works, any ground damage that may have been present was not identifiable. The presence of a large number of trees and shrubs between the building and the riverbanks man have minimised any significant lateral spreading.

6. Geotechnical Consideration

The site is situated within the Botanic Gardens of Hagley Park, in central Christchurch. It is relatively flat at approximately 8m above mean sea level. The structures are situated between 50m and 100m south of the Avon River, and 9.5km west of the coast (Pegasus Bay) at New Brighton.

6.1 Published Information on Ground Conditions

6.1.1 Local Geology

The geological map of the area[1] indicates that the site is underlain by Holocene alluvial soils of the Yaldhurst Member, sub-group of the Springston Formation, comprising alluvial sand and silt overbank deposits.

Brown and Weeber (1992) indicates the site consists of near surface gravel underlain by sand, silt, clay until approximately 20m bgl where the Riccarton Gravels are located. Groundwater is indicated to be present 1 - 2m bgl.

6.1.2 Environment Canterbury Records

Information from Environment Canterbury (ECan) indicates that three boreholes are located within 200m of the site (see Table 2). Of these, two contained adequate lithographic logs. The site geology described in the logs is stratified gravel, sand, silt and clay. Also present are layers of peat between 20m and 40m bgl.

Groundwater was recorded between 2.7m and 4.3m bgl in the ECan logs.

Table 2 ECan Borehole Summary

Bore Name	Log Depth	Groundwater	Distance & Direction from Site
M35/1936	100.9m	4.3m bgl	50m E of office buildings
M35/10619	104.5m	2.7m bgl	100m E of office buildings

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 not undertaken geotechnical testing in the area of the subject site.

6.1.4 CERA Land Zoning

Canterbury Earthquake Recovery Authority (CERA) has published areas showing the Green Zone Technical Category in relation to the risk of future liquefaction and how these areas are expected to perform in future earthquakes. The site is classified as Technical Category N/A – Urban Non-residential.

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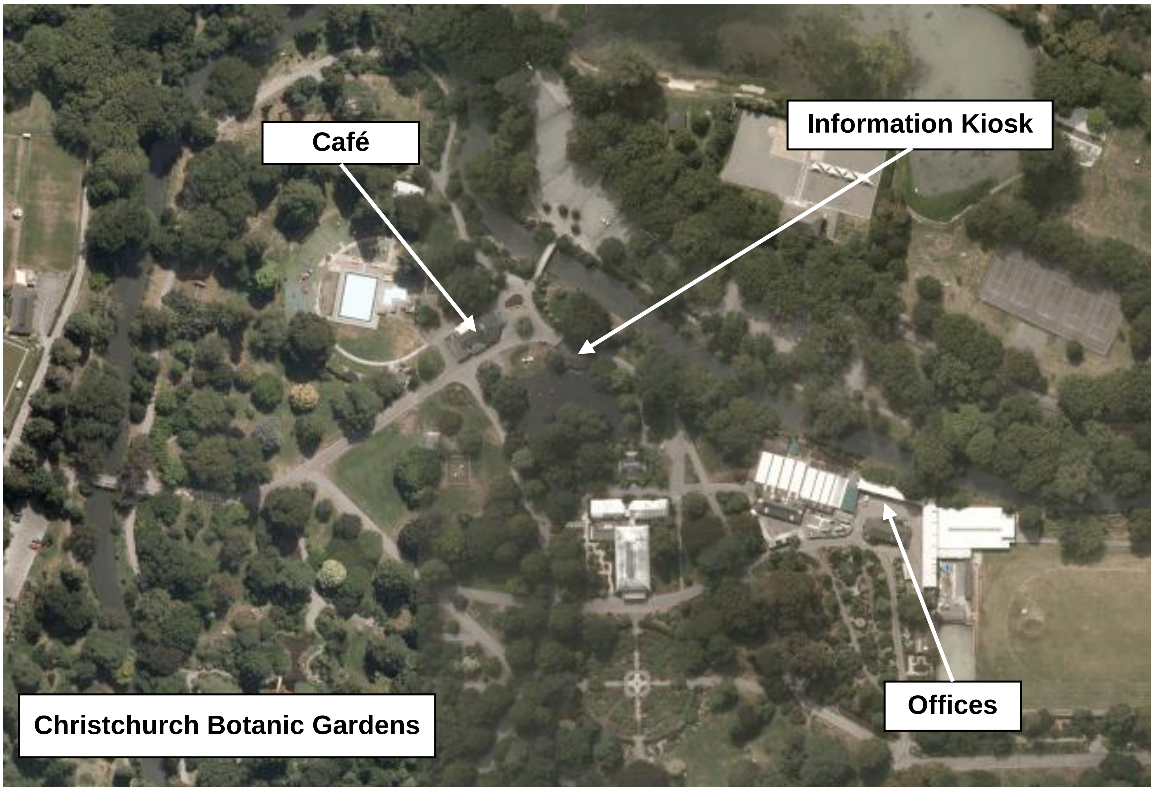
[1] Brown, L. J. and Weeber, J.H. 1992: Geology of the Christchurch Urban Area. Institute of Geological and Nuclear Sciences 1:25,000 Geological Map 1. Lower Hutt. Institute of Geological and Nuclear Sciences Limited.

6.1.5 Post-Earthquake Land Observations

Aerial photography taken following the 22 February 2011 earthquake shows moderate amounts of liquefaction on the northern side of the Avon River and in Victoria Lake. There is no evidence of liquefaction within the Botanic Gardens themselves.

The Canterbury Geotechnical Database[2] shows several observed ground cracks <10mm within 100m of the café and information kiosk structures and 280m from the office block.

Figure 3 Post February 2011 Earthquake Aerial Photography[3]



6.2 Seismicity

6.2.1 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.

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[2] Canterbury Geotechnical Database (2012) "Observed Ground Crack Locations", Map Layer CGD0400 - 23 July 2012, retrieved 10/10/12 from https://canterburygeotechnicaldatabase.projectorbit.com/

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

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

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	120km	NW	~8.3	~300 years
Greendale Fault (2010)	20km	W	7.1	~15,000 years
Hope Fault	100km	N	7.2~7.5	120~200 years
Kelly Fault	100km	NW	7.2	150 years
Porters Pass Fault	55km	NW	7.0	1100 years
Port Hills Fault (2011)	7km	SE	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 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.

6.2.2 Ground Shaking

This 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.

6.3 Slope Failure and Rockfall Potential

Given the site’s flat elevation and location in Central Christchurch, global slope instability is considered negligible. However, due to the site’s proximity to the Avon River, it may be susceptible to lateral spreading along the river margins. In addition, any localised retaining structures or embankments should be further investigated to determine the site-specific slope instability potential.

6.4 Field Investigations

The geotechnical field investigation comprised a site walkover, two machine boreholes, one located between the café and information kiosk and the other outside the office block. The investigation layout is shown in Figure 2 and the GPS locations of the tests are tabulated in Table 4 below.

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

[5] GNS Active Faults Database

Table 4 Investigation Locations

Borehole Number	Depth	Northing	Easting
BH01	19.5	5741909	2479508
BH02	19.5	5742005	2479326

Machine drilled boreholes were undertaken by McMillan Specialist Drilling from 8th of October.

Figure 4 Investigation Location Plan

6.5 Ground Conditions Encountered

A summary of the ground conditions encountered in BH01 and BH02 are shown in Table 5.

Table 5 Summary of Machine-drilled Boreholes

Depth (m)	Lithology	SPT-N Values
0.0 – 0.8	Gravelly SAND to SAND with some organic material	-
0.8 – 4.5	Sandy fine to coarse GRAVEL with occasional fine sand and silt lenses	9
4.5 – 12.0	Sandy fine to coarse GRAVEL with occasional fine sand and silt lenses	19 to 50
12.0 – 19.5	Stratified layers of silty fine SAND to sandy SILT.	4 to 25
19.5	End of Borehole – Target Depth Achieved	

Detailed engineering borelogs can be found in Appendix E.

Groundwater was encountered at 3.6m and 3.7m in BH01 and BH02 respectively. This correlates with the water level in the Avon River that is within 20m of the boreholes.

6.6 Liquefaction Potential

The site is considered unlikely to liquefy based of the following:

- The surface gravels are unlikely to liquefy because the grainsize is too large;
- The saturated sands present from 10m bgl are considered to have a low susceptibility to liquefaction because their relative density is medium dense to dense;
- Any liquefaction beneath surface gravels would be unlikely to penetrate gravels; and;
- No observations of liquefaction from post-earthquake aerial photography in the immediate vicinity of the sites.

6.7 Recommendations and Summary

The grounds conditions beneath the site comprise sand to 0.8m, underlan by sandy gravel to 10m bgl, underlain by interbedded silt and sand to 19.5m bgl.

The soil class of **D** (in accordance with NZS 1170.5:2004) recommended in Section 8 of the Qualitative DEE is still believed to be appropriate.

The ground performance is considered consistent with the TC1 classification.

The café, information kiosk and office buildings have not suffered any damage as a result of the ground conditions present beneath the site. Therefore no ground treatment is recommended for the buildings.

Should repairs be undertaken to parts of the foundations these foundations should follow foundation requirements in accordance with Ministry of Business, Innovation, and Employment Guidelines for TC1 properties.

Should re-development of the site be undertaken a site specific investigation should be undertaken, but it is likely shallow foundations onto the gravels would be appropriate.

Our investigations confirm the ground conditions in the Geotech Consulting report dated May 2010 and we concur with the foundation recommendations.

7. Assessment Methodology

7.1 Quantitative Assessment

The quantitative assessment of the building comprised of an investigation on the in-plane and out-of-plane strength of the masonry block walls and an assessment of the capacities of the structural steel members. The investigation was based on the analysis of the seismic loads that the structure is subjected to, distribution of these forces throughout the structure and the analysis of the capacity of existing structural elements to resist the forces applied. The capacity of the existing structural elements was compared to the demand placed on the elements to give the %NBS of each of the structural elements.

7.2 Demand

The demand forces for each wall and structural steel element was assessed by applying seismic loading to a finite element model of the building. NZS 1170.5:2004 makes allowance for accidental eccentricity and requires that the earthquake action be applied at an eccentricity of 10% of the building dimension which is perpendicular to the force applied. This results in a torsional action about the centre of resistance of the building, and induces forces in the lateral force resisting structural elements. Cl 5.3.1.2 of NZS 1170.5: 2004 states that for nominally ductile and brittle structures an action set of 100% of the earthquake actions in one direction and 30% in the orthogonal direction must be applied when calculating the demand for any structural member and has such been applied in the analysis.

7.3 Concrete Masonry Assessment

7.3.1 Seismic Weight Coefficient Masonry

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

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

Where

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

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

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

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

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

$$\mathrm{S_P} = 1.33 - 0.3\mu$$

The structural ductility factor, μ, was taken as 1.25 for the out of plane assessment and 2.00 for the in plane analysis of walls as suggested by the NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Resistance (02/2011).

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 0.1 was assumed for 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

$$k_\mu = \frac{(\mu - 1)T_1}{0.7} + 1 \text{ For out of plane assessment}$$

$$k_\mu = 1.2 \text{ For in plane assessment}$$

7.3.2 In-Plane Capacity of Unreinforced Walls

The in-plane capacity of the concrete masonry wall was determined using the NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Resistance (02/2011). The NZSEE guidelines recommend checks for 4 different in-plane response modes.

- Diagonal tension failure mode
- Bed-sliding failure mode
- Toe crushing failure mode
- Rocking failure mode

An analysis of each wall was carried out using the methods set out in Section 8 – In-Plane Wall Response, of the NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Performance (02/2011).

7.3.3 In-plane Wall Shear Capacity of Unreinforced Walls

The in-plane nominal shear capacity of a wall, pier or spandrel was taken as the minimum of the nominal capacity in the diagonal tension failure mode, V_{dt}, the rocking failure mode, V_r, the bed-joint sliding failure mode, V_s, and the toe crushing failure mode, V_{tc}.

$$V_n = \min(V_{dt}, V_s, V_r, V_{tc})$$

7.3.4 Out-of-Plane Capacity of Unreinforced Walls

The % NBS for out-of-plane flexure of the concrete masonry walls was determined using the methods set out in NZSEE guidelines for the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes (06/2006) Section 10.3.

7.4 Structural Steel Assessment

7.4.1 Seismic Weight Coefficient Steel

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

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

Where

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

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

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

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

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

$$\mathrm{S_P} = 1.33 - 0.3\mu$$

Where μ, the structural ductility factor, was taken as 2.00.

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 0.1 was assumed for the building. The coefficient was then calculated using Equation 5.2(1);

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

Where

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

7.4.2 Member Bending Moment Capacity (Section 5.1 of NZS 3404:1997)

A member bent about the section major principle axis shall satisfy:

$$M_x^* \leq \Phi M_{sx} \text{ and}$$

$$M_x^* \leq \Phi M_{bx}$$

A member bent about the section minor principle axis shall satisfy:

$$M_y^* \leq \Phi M_{sy}$$

Where

$M_x^* = the\ design\ bending\ moment\ from\ analysis$

$\Phi = the\ strength\ reduction\ factor\ from\ Table\ 3.3\ of\ NZS\ 3604{:}\ Part\ 1\ 1997$

$M_{sx} = the\ nominal\ section\ capacity\ in\ bending\ about\ the\ x\ axis, as\ specified\ in\ Clause\ 5.2$

$of\ NZS\ 3404{:}\ Part\ 1\ 1997$

$M_{sy} = the\ nominal\ section\ capacity\ in\ bending\ about\ the\ y\ axis, as\ specified\ in\ Clause\ 5.2$

$of\ NZS\ 3404{:}\ Part\ 1\ 1997$

$M_{bx} = the\ nominal\ member\ capacity\ in\ bending, as\ specified\ in\ Clause\ 5.3\ or\ 5.6\ of$

$NZS\ 3404{:}\ Part\ 1\ 1997$

For hollow sections, the nominal member capacity in bending, M_{bx}, always equals the nominal section capacity in bending, M_{sx}, according to clause 5.6.1.4 of NZS 3404: Part 2 1997.

7.4.3 Member Shear Capacity (Section 5.9 of NZS 3404:1997)

A member web subjected to shear force, V^*, shall satisfy:

$$V^* \leq \Phi V_v$$

Where

$V^* = the\ design\ shear\ force\ from\ analysis$

$\Phi = the\ strength\ reduction\ factor\ from\ Table\ 3.3\ of\ NZS\ 3404{:}\ Part\ 1\ 1997$

$V_v = the\ nominal\ section\ capacity\ of\ the\ web, as\ determined\ in\ Clause\ 5.11.2\ of$

$NZS\ 3404{:}\ Part\ 1\ 1997$

7.4.4 Member Shear and Bending Moment Interaction (Section 5.12 of NZS 3404:1997)

A member subjected to bending moment, M_x^*, and shear force, V^*, shall have its nominal web shear capacity, V_{vm}, calculated using the equations set out in Clause 5.12.2 of NZS 3404: Part 1 1997. The web design shear capacity in the presence of bending moment shall satisfy

$$V^* \leq \Phi V_{vm}$$

Where

$V^* = the\ design\ shear\ force\ from\ analysis$

$\Phi = the\ strength\ reduction\ factor\ from\ Table\ 3.3\ of\ NZS\ 3404{:}\ Part\ 1\ 1997$

$V_{vm} = the\ nominal\ section\ capacity\ of\ the\ web, modefied\ for\ the\ presence\ of\ bending\ as$

$determined\ in\ Clause\ 5.12.2\ of\ NZS\ 3404{:}\ Part\ 1\ 1997$

7.4.5 Member Axial Capacity (Section 6 of NZS 3404:1997)

A concentrically loaded member subject to a design axial compressive force, N^*, shall comply with both:

$$N^* \leq \Phi N_s$$

$$N^* \leq \Phi N_c$$

Where

$N^* = the\ design\ axial\ force\ from\ analysis$

$\Phi = the\ strength\ reduction\ factor\ from\ Table\ 3.3\ of\ NZS\ 3404{:}\ Part\ 1\ 1997$

$N_s = the\ nominal\ section\ capacity\ ,as\ determined\ in\ Clause\ 6.2.1.1\ of\ NZS\ 3404{:}\ Part\ 1\ 1997$

$N_c = the\ nominal\ member\ capacity\ ,as\ determined\ in\ Clause\ 6.3\ of\ NZS\ 3404{:}\ Part\ 1\ 1997$

7.4.6 Member Combined Axial and Bending Moment Capacity (Section 8 of NZS 3404:1997)

A member subject to uniaxial bending and axial actions need not be checked for combined actions when the axial force is not significant as defined by Cl 8.1.4 of NZS 3404:1997. The design axial force shall be considered significant unless it complies with:

$$N^* \leq 0.05\Phi N_s\ if\ the\ member\ is\ subject\ to\ uniaxial\ bending\ and\ is\ an\ I\ or\ channel\ section$$

$$N^* \leq 0.05\Phi N_c\ if\ the\ member\ is\ subject\ to\ uniaxial\ bending\ and\ is\ any\ other\ cross\ section$$

Where axial force is considered significant, the following general design provision should be satisfied:

$$M_x^* \leq \Phi M_{rx}$$

Where

$M_x^* = the\ design\ bending\ moment\ from\ analysis$

$\Phi = the\ strength\ reduction\ factor\ from\ Table\ 3.3\ of\ NZS\ 3404{:}\ Part\ 1\ 1997$

$M_{rx} = the\ nominal\ section\ moment\ capacity, reduced\ by\ axial\ force\ as\ specified\ in\ Clause$

$8.3.2.1\ of\ NZS\ 3404{:}\ Part\ 1\ 1997$

7.4.7 Connection Fillet Weld Capacity (Section 9.7.3 of NZS 3404:1997)

A fillet weld subject to a design force per unit length of weld shal satisfy: (Cl 9.7.3.10.1 of NZS 3404:1997)

$$v_w^* \leq \Phi v_w$$

Where

$v_w^* = the\ design\ shear\ action\ from\ analysis$

$\Phi = the\ strength\ reduction\ factor\ from\ Table\ 3.3\ of\ NZS\ 3404{:}\ Part\ 1\ 1997$

$v_w = the\ nominal\ \ weld\ shear\ capacity, as\ specified\ in\ Clause\ 9.7.3.10.3\ of\ NZS\ 3404{:}\ 1997$

The nominal capacity of a fillet weld per unit length, v_w, shall be calculated as follows (Cl 9.7.3.10.3 of NZS 3404:1997);

$$v_w = 0.6 f_{uw} t_t k_r$$

Where

$f_{uw} = the\ nominal\ tensile\ strength\ of\ weld\ metal$

$t_t = design\ throat\ thickness$

$k_r = reduction\ factor\ given\ in\ table\ 9.7.3.10(2) of\ NZS\ 3404{:}\ Part\ 1\ 1997 to\ account$

$for\ the\ length\ of\ a\ weld\ lap\ connection$

8. Initial Capacity Assessment

8.1 Seismic Loading Investigation

A 3D structure analysis using Robot Structure Analysis Professional engineering software was undertaken to model the building structure for 100% NBS loads. Loads were applied in both the along and across directions of the building. The loads from the analysis were then checked against the structural steel member capacities derived from NZS 3404: 1997 and the masonry wall capacities derived from the NZSEE Guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Performance (02/2011) and the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes (06/2006).

8.2 Building Analysis and Results

For the purposes of analysing the demand, each wall between the steel frames was considered separately (See Figure 5). Forces acting on the bottom edge of each of the walls were extracted for comparison with their individual capacities. Parapets above roof level were analysed as being separate to the supporting wall.

The structural steel frame was modelled in tandem with the masonry wall panels and as shown below in Figure 4. Following the finite element analysis of the structure, the design actions acting on each steel member were extracted and analysed. The members were analysed as groups with the same cross section and only the member of each group subjected to the largest design actions were considered.

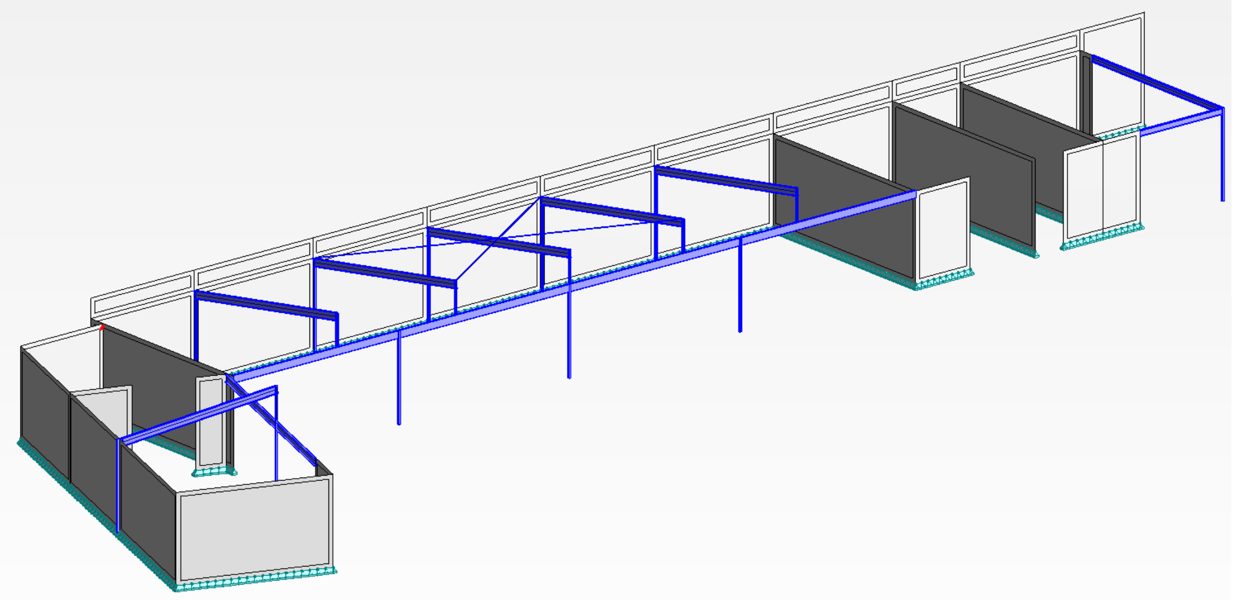


Figure 5 Structural walls and steel members modelled

8.3 Office Store Wall Analysis Results

The position of each wall is indicated in Figure 6 and each wall is named accordingly.

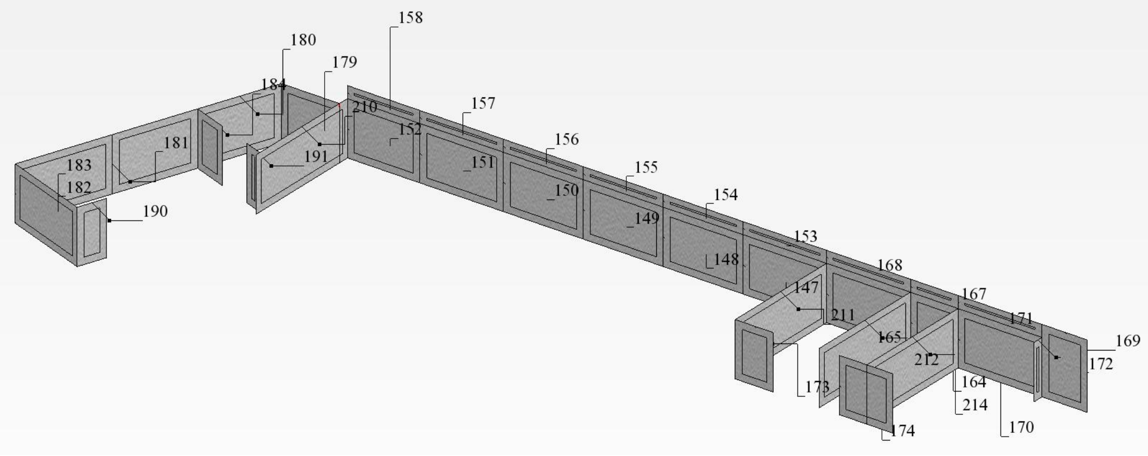


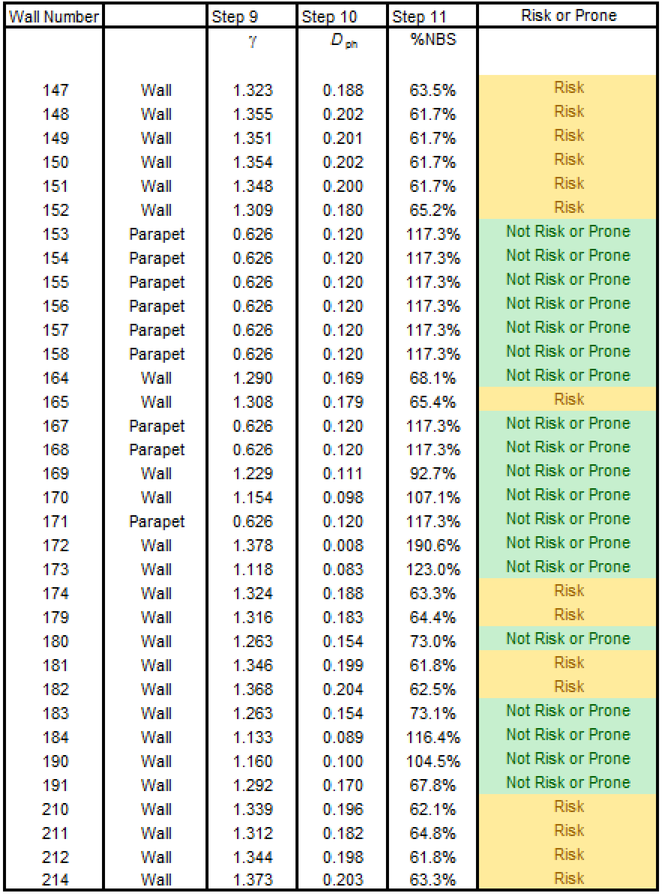
Figure 6 Wall numbers and locations of Office Store

The results of the in-plane analysis and subsequent earthquake designation under the NZSEE guidelines are listed below in Table 6 .

Wall Number	Governing Mode	Critical shear capacity	Demand	Reduced Capacity	% NBS	Risk or Prone
		V_n kN	kN	ϕV_n kN		
147	Diagonal Tension	28.1	32.7	21.1	64.60%	Risk
148	Diagonal Tension	24.1	26.8	18.1	67.53%	Not Risk or Prone
149	Diagonal Tension	24.5	25.9	18.3	70.82%	Not Risk or Prone
150	Diagonal Tension	24.2	24.9	18.2	72.86%	Not Risk or Prone
151	Diagonal Tension	25.8	24.7	19.4	78.47%	Not Risk or Prone
152	Diagonal Tension	25.6	18.2	19.2	105.96%	Not Risk or Prone
153	Rocking Failure	19.5	7.4	14.7	199.47%	Not Risk or Prone
154	Rocking Failure	17.9	4.4	13.4	308.27%	Not Risk or Prone
155	Rocking Failure	17.8	3.5	13.3	377.39%	Not Risk or Prone
156	Rocking Failure	17.7	3.5	13.3	384.34%	Not Risk or Prone
157	Rocking Failure	19.3	3.8	14.5	376.64%	Not Risk or Prone
158	Rocking Failure	15.3	6.1	11.5	187.47%	Not Risk or Prone
164	Rocking Failure	13.9	23.4	10.4	44.61%	Risk
165	Diagonal Tension	29.8	37.5	22.3	59.61%	Risk
167	Rocking Failure	6.4	0.6	4.8	839.61%	Not Risk or Prone
168	Rocking Failure	19.5	3.9	14.6	379.37%	Not Risk or Prone
169	Rocking Failure	24.8	13.1	18.6	141.90%	Not Risk or Prone
170	Diagonal Tension	47.4	39.8	35.5	89.37%	Not Risk or Prone
171	Rocking Failure	19.8	12.7	14.9	117.35%	Not Risk or Prone
172	Toe Crushing	11.3	32.3	8.5	26.25%	Prone
173	Rocking Failure	18.3	38.6	13.7	35.67%	Risk
174	Rocking Failure	14.8	31.9	11.1	34.96%	Risk
179	Rocking Failure	18.2	57.1	13.7	23.95%	Prone
180	Diagonal Tension	32.7	31.3	24.6	78.41%	Not Risk or Prone
181	Diagonal Tension	25.5	22.0	19.1	86.92%	Not Risk or Prone
182	Diagonal Tension	26.8	26.4	20.1	76.18%	Not Risk or Prone
183	Diagonal Tension	36.9	45.8	27.7	60.42%	Risk
184	Rocking Failure	15.1	35.2	11.4	32.23%	Prone
190	Rocking Failure	8.5	22.3	6.4	28.53%	Prone
191	Rocking Failure	1.9	5.5	1.5	26.22%	Prone
210	Diagonal Tension	39.1	53.9	29.3	54.38%	Risk
211	Diagonal Tension	42.8	70.9	32.1	45.23%	Risk
212	Diagonal Tension	38.4	49.8	28.8	57.80%	Risk
214	Diagonal Tension	34.6	61.9	25.9	41.89%	Risk

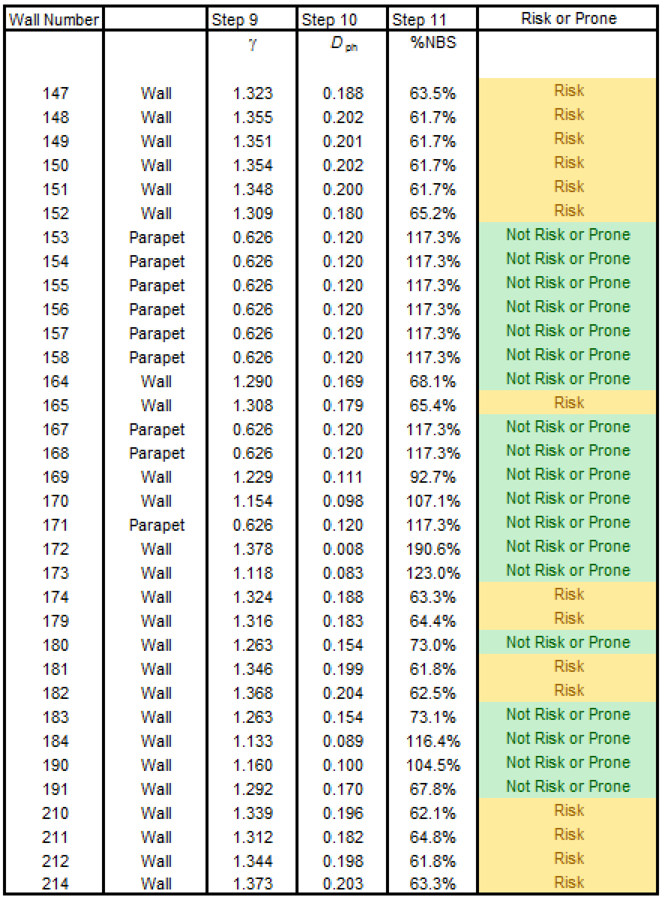
Table 6 In-plane analysis results

The results of the out-of-plane displacement response capability analysis and subsequent designation under the NZSEE guidelines are listed in



Wall Number		Step 9	Step 10	Step 11	Risk or Prone
		γ	D_{ph}	%NBS	
147	Wall	1.323	0.188	63.5%	Risk
148	Wall	1.355	0.202	61.7%	Risk
149	Wall	1.351	0.201	61.7%	Risk
150	Wall	1.354	0.202	61.7%	Risk
151	Wall	1.348	0.200	61.7%	Risk
152	Wall	1.309	0.180	65.2%	Risk
153	Parapet	0.626	0.120	117.3%	Not Risk or Prone
154	Parapet	0.626	0.120	117.3%	Not Risk or Prone
155	Parapet	0.626	0.120	117.3%	Not Risk or Prone
156	Parapet	0.626	0.120	117.3%	Not Risk or Prone
157	Parapet	0.626	0.120	117.3%	Not Risk or Prone
158	Parapet	0.626	0.120	117.3%	Not Risk or Prone
164	Wall	1.290	0.169	68.1%	Not Risk or Prone
165	Wall	1.308	0.179	65.4%	Risk
167	Parapet	0.626	0.120	117.3%	Not Risk or Prone
168	Parapet	0.626	0.120	117.3%	Not Risk or Prone
169	Wall	1.229	0.111	92.7%	Not Risk or Prone
170	Wall	1.154	0.098	107.1%	Not Risk or Prone
171	Parapet	0.626	0.120	117.3%	Not Risk or Prone
172	Wall	1.378	0.008	190.6%	Not Risk or Prone
173	Wall	1.118	0.083	123.0%	Not Risk or Prone
174	Wall	1.324	0.188	63.3%	Risk
179	Wall	1.316	0.183	64.4%	Risk
180	Wall	1.263	0.154	73.0%	Not Risk or Prone
181	Wall	1.346	0.199	61.8%	Risk
182	Wall	1.368	0.204	62.5%	Risk
183	Wall	1.263	0.154	73.1%	Not Risk or Prone
184	Wall	1.133	0.089	116.4%	Not Risk or Prone
190	Wall	1.160	0.100	104.5%	Not Risk or Prone
191	Wall	1.292	0.170	67.8%	Not Risk or Prone
210	Wall	1.339	0.196	62.1%	Risk
211	Wall	1.312	0.182	64.8%	Risk
212	Wall	1.344	0.198	61.8%	Risk
214	Wall	1.373	0.203	63.3%	Risk

Table 77.



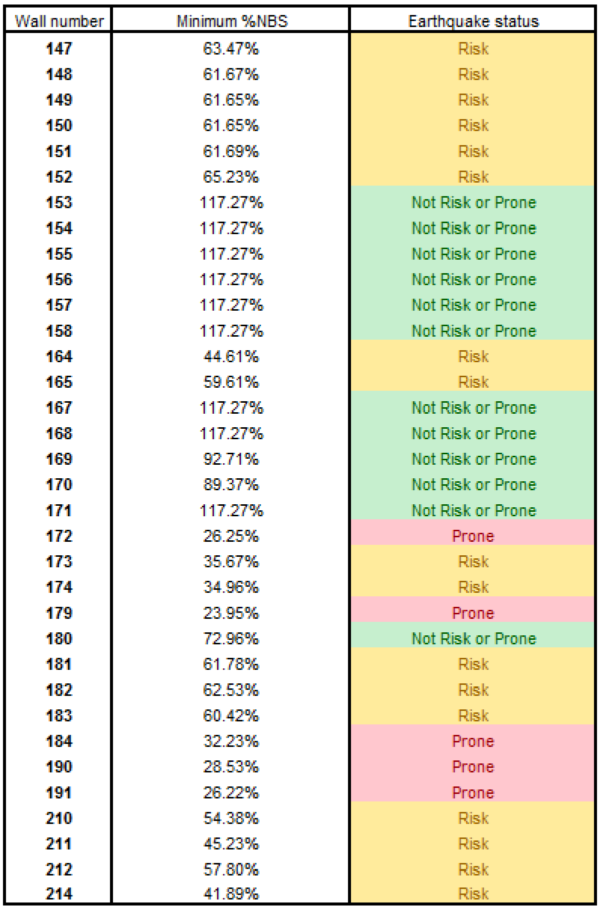
Wall Number		Step 9	Step 10	Step 11	Risk or Prone
		γ	D_{ph}	%NBS	
147	Wall	1.323	0.188	63.5%	Risk
148	Wall	1.355	0.202	61.7%	Risk
149	Wall	1.351	0.201	61.7%	Risk
150	Wall	1.354	0.202	61.7%	Risk
151	Wall	1.348	0.200	61.7%	Risk
152	Wall	1.309	0.180	65.2%	Risk
153	Parapet	0.626	0.120	117.3%	Not Risk or Prone
154	Parapet	0.626	0.120	117.3%	Not Risk or Prone
155	Parapet	0.626	0.120	117.3%	Not Risk or Prone
156	Parapet	0.626	0.120	117.3%	Not Risk or Prone
157	Parapet	0.626	0.120	117.3%	Not Risk or Prone
158	Parapet	0.626	0.120	117.3%	Not Risk or Prone
164	Wall	1.290	0.169	68.1%	Not Risk or Prone
165	Wall	1.308	0.179	65.4%	Risk
167	Parapet	0.626	0.120	117.3%	Not Risk or Prone
168	Parapet	0.626	0.120	117.3%	Not Risk or Prone
169	Wall	1.229	0.111	92.7%	Not Risk or Prone
170	Wall	1.154	0.098	107.1%	Not Risk or Prone
171	Parapet	0.626	0.120	117.3%	Not Risk or Prone
172	Wall	1.378	0.008	190.6%	Not Risk or Prone
173	Wall	1.118	0.083	123.0%	Not Risk or Prone
174	Wall	1.324	0.188	63.3%	Risk
179	Wall	1.316	0.183	64.4%	Risk
180	Wall	1.263	0.154	73.0%	Not Risk or Prone
181	Wall	1.346	0.199	61.8%	Risk
182	Wall	1.368	0.204	62.5%	Risk
183	Wall	1.263	0.154	73.1%	Not Risk or Prone
184	Wall	1.133	0.089	116.4%	Not Risk or Prone
190	Wall	1.160	0.100	104.5%	Not Risk or Prone
191	Wall	1.292	0.170	67.8%	Not Risk or Prone
210	Wall	1.339	0.196	62.1%	Risk
211	Wall	1.312	0.182	64.8%	Risk
212	Wall	1.344	0.198	61.8%	Risk
214	Wall	1.373	0.203	63.3%	Risk

Table 7 The Office Store out-of-plane analysis results

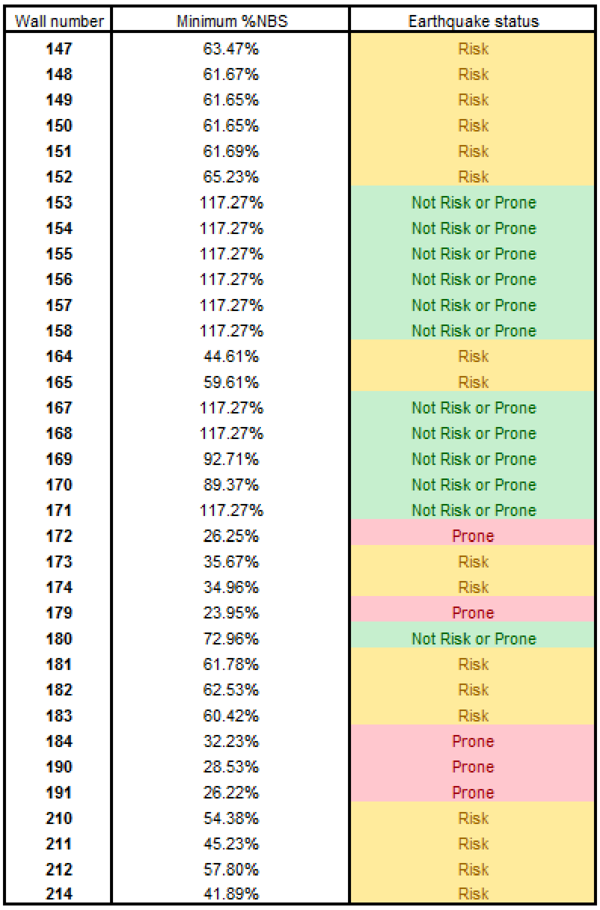
8.4 Office Store Walls Percentage of New Building Standard (%NBS)

A comparison between the in-plane and out-of plane % NBS of each wall was carried out and the critical % NBS for each wall is listed below in

Table 88.



Wall number	Minimum %NBS	Earthquake status
147	63.47%	Risk
148	61.67%	Risk
149	61.65%	Risk
150	61.65%	Risk
151	61.69%	Risk
152	65.23%	Risk
153	117.27%	Not Risk or Prone
154	117.27%	Not Risk or Prone
155	117.27%	Not Risk or Prone
156	117.27%	Not Risk or Prone
157	117.27%	Not Risk or Prone
158	117.27%	Not Risk or Prone
164	44.61%	Risk
165	59.61%	Risk
167	117.27%	Not Risk or Prone
168	117.27%	Not Risk or Prone
169	92.71%	Not Risk or Prone
170	89.37%	Not Risk or Prone
171	117.27%	Not Risk or Prone
172	26.25%	Prone
173	35.67%	Risk
174	34.96%	Risk
179	23.95%	Prone
180	72.96%	Not Risk or Prone
181	61.78%	Risk
182	62.53%	Risk
183	60.42%	Risk
184	32.23%	Prone
190	28.53%	Prone
191	26.22%	Prone
210	54.38%	Risk
211	45.23%	Risk
212	57.80%	Risk
214	41.89%	Risk



Wall number	Minimum %NBS	Earthquake status
147	63.47%	Risk
148	61.67%	Risk
149	61.65%	Risk
150	61.65%	Risk
151	61.69%	Risk
152	65.23%	Risk
153	117.27%	Not Risk or Prone
154	117.27%	Not Risk or Prone
155	117.27%	Not Risk or Prone
156	117.27%	Not Risk or Prone
157	117.27%	Not Risk or Prone
158	117.27%	Not Risk or Prone
164	44.61%	Risk
165	59.61%	Risk
167	117.27%	Not Risk or Prone
168	117.27%	Not Risk or Prone
169	92.71%	Not Risk or Prone
170	89.37%	Not Risk or Prone
171	117.27%	Not Risk or Prone
172	26.25%	Prone
173	35.67%	Risk
174	34.96%	Risk
179	23.95%	Prone
180	72.96%	Not Risk or Prone
181	61.78%	Risk
182	62.53%	Risk
183	60.42%	Risk
184	32.23%	Prone
190	28.53%	Prone
191	26.22%	Prone
210	54.38%	Risk
211	45.23%	Risk
212	57.80%	Risk
214	41.89%	Risk

Table 8 %NBS results

8.5 Office Store Steel Member Analysis Results

The position of each structural steel member is indicated in Figure 7 below and each wall is named accordingly.

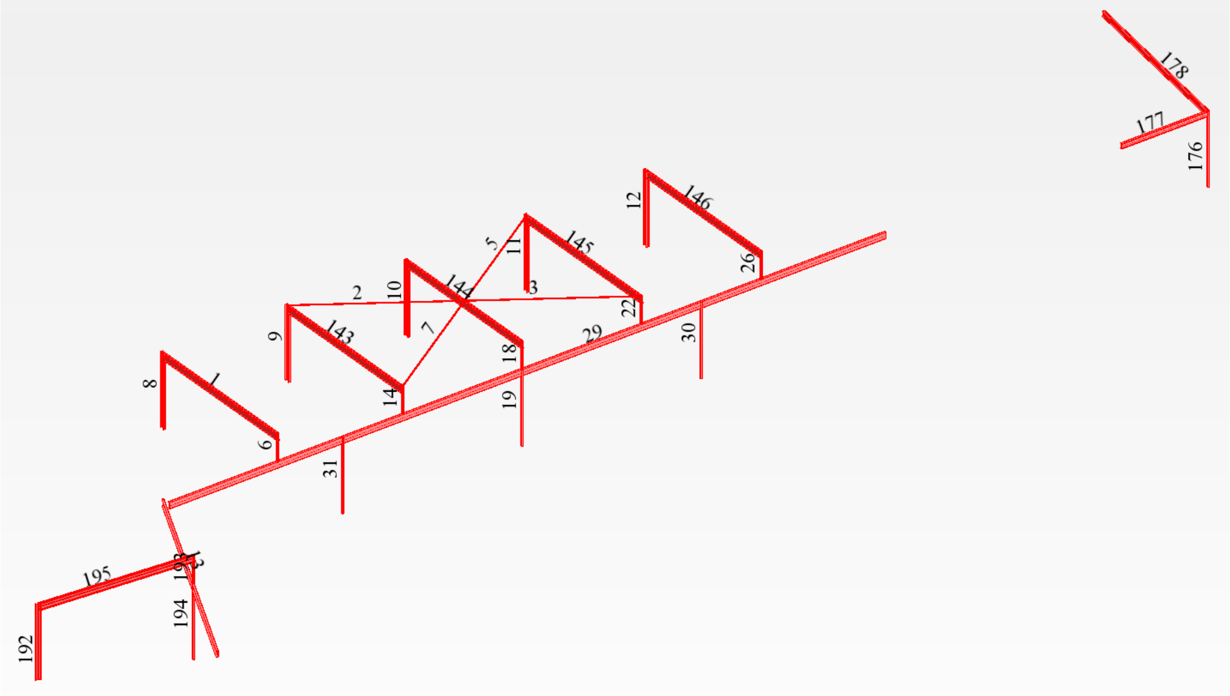


Figure 7 Locations of structural steel members and corresponding member names

The critical members for each steel section group and the % NBS for those members is shown below in Table 9.

Critical Member Number	Member Group	Section Dimensions	% NBS %
1	Code group : 1 Rafters	UB 254x102x22	126
7	Code group : 2 Cross Bracing	Cross Bracing	156
22	Code group : 3 Short BSJs	BSJ 76x50x6.67	175
9	Code group : 4 BSJ columns	UB 178x102x19	136
19	Code group : 5 SHS columns	SHS 75x75x4	294
29	Code group : 6 PFC Beam	PFC 254x76x28.39	667
177	Code group : 7 PFC Beam	RSC 203x76	1111
178	Code group : 8 I Beam	UB 203x102x23	3333

Table 9 Critical steel members, section types and % NBS

8.6 Discussion of Results

The Office Store building was designed in 1969 and likely designed for the loading standard current at the time: NZS 1900:1965. The design loads used in this code are less than those required by the current loading standard. In addition, the detailing requirements for ductile seismic behaviour that are present in the current codes are unlikely to have been considered in the design of this building. As a result, it would be expected that the building would not achieve 100% NBS. The increase in the hazard factor for Christchurch to 0.3 would be expected to further reduce the %NBS score.

Following a detailed assessment, the building has been assessed as achieving 24**%** New Building Standard (NBS). Under the New Zealand Society for Earthquake Engineering (NZSEE) guidelines the building is considered potentially an Earthquake Prone building as it achieves less than 34% NBS.

The low in-plane shear capacity of a number of the walls was found to be the governing capacity and has resulted in a % NBS of less than 34% for the overall building.

9. Recommendations

The recent seismic activity in Christchurch has caused minor damage to the building, with cracking in concrete masonry walls the only damage noted.

The building has however been assessed as being a potentially Earthquake Prone building as it has achieved less than 34% NBS. As such, GHD Limited recommend that strengthening options be explored in order to increase the % NBS of the building to 67% NBS as recommended by the NZSEE Guidelines.

10. Limitations

10.1 General

This report has been prepared subject to the following limitations:

- Drawings of the building were available.
- No intrusive structural investigations have been undertaken.
- No level or verticality surveys have been undertaken.
- No material testing has been undertaken.
- No calculations, other than those detailed in Section 7 have been carried out on the structure.

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 Limited 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.

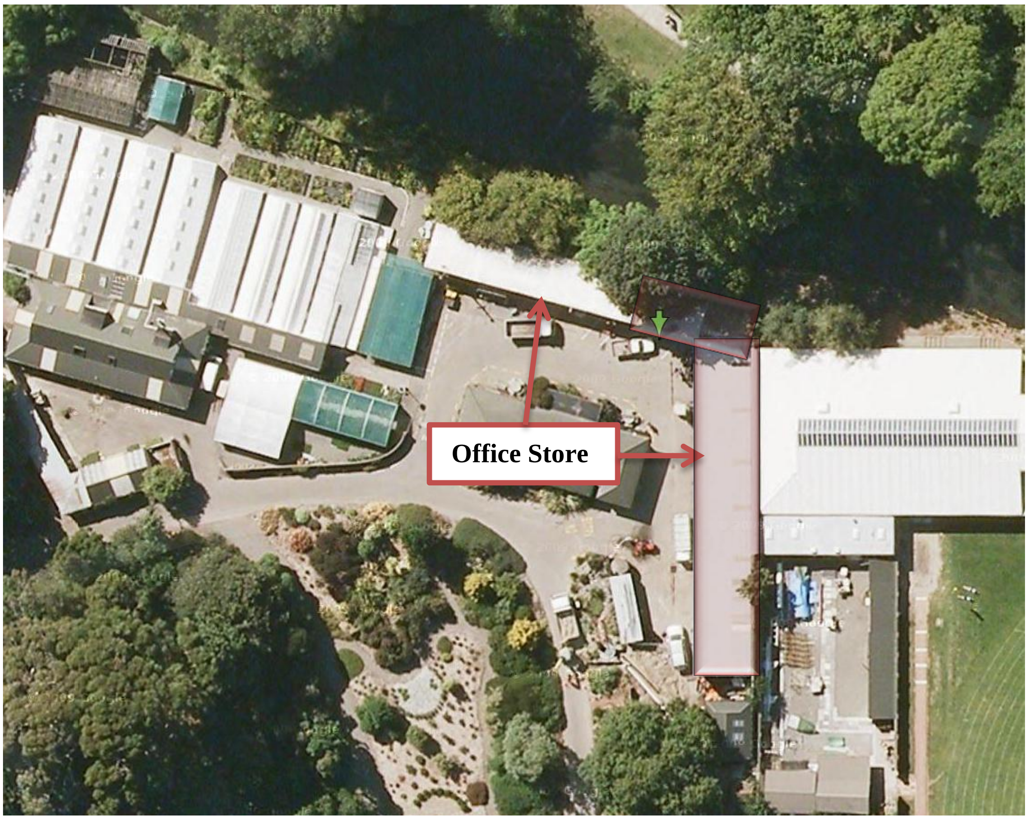
It should be noted that because of the inherent uncertainties in subsurface evaluations, changed or unanticipated subsurface conditions may occur that could affect total project cost and/or execution. GHD does not accept responsibility for the consequences of significant variances in the conditions and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

Photograph 2 Southern wall of Wing A and western wall of Wing B.

Photograph 3 Western wall of the original pre-1969 pump house attached to the Office Store.

Photograph 4 Connection between pre-1969 pump house and office store building. Horizontal cracking along mortar line is visible.

Photograph 5 Cracking to masonry wall of the pre-1969 pump house.

Photograph 6 Cracking to masonry walls at connection between Wings A and B.

Photograph 7 Western side of Wing B showing the large openings for the garage doors.

Photograph 8 Cracking to masonry wall at the northern end of Wing B.

Photograph 9 Timber framed roof supports at the northern end of Wing B.

Photograph 10 Roofing system of the Garage portion of Wing B showing timber roof purlins on steel portal frames. Flat steel cross bracing straps are clearly visible.

Photograph 11 A typical connection between the steel frame columns and concrete masonry unit walls.

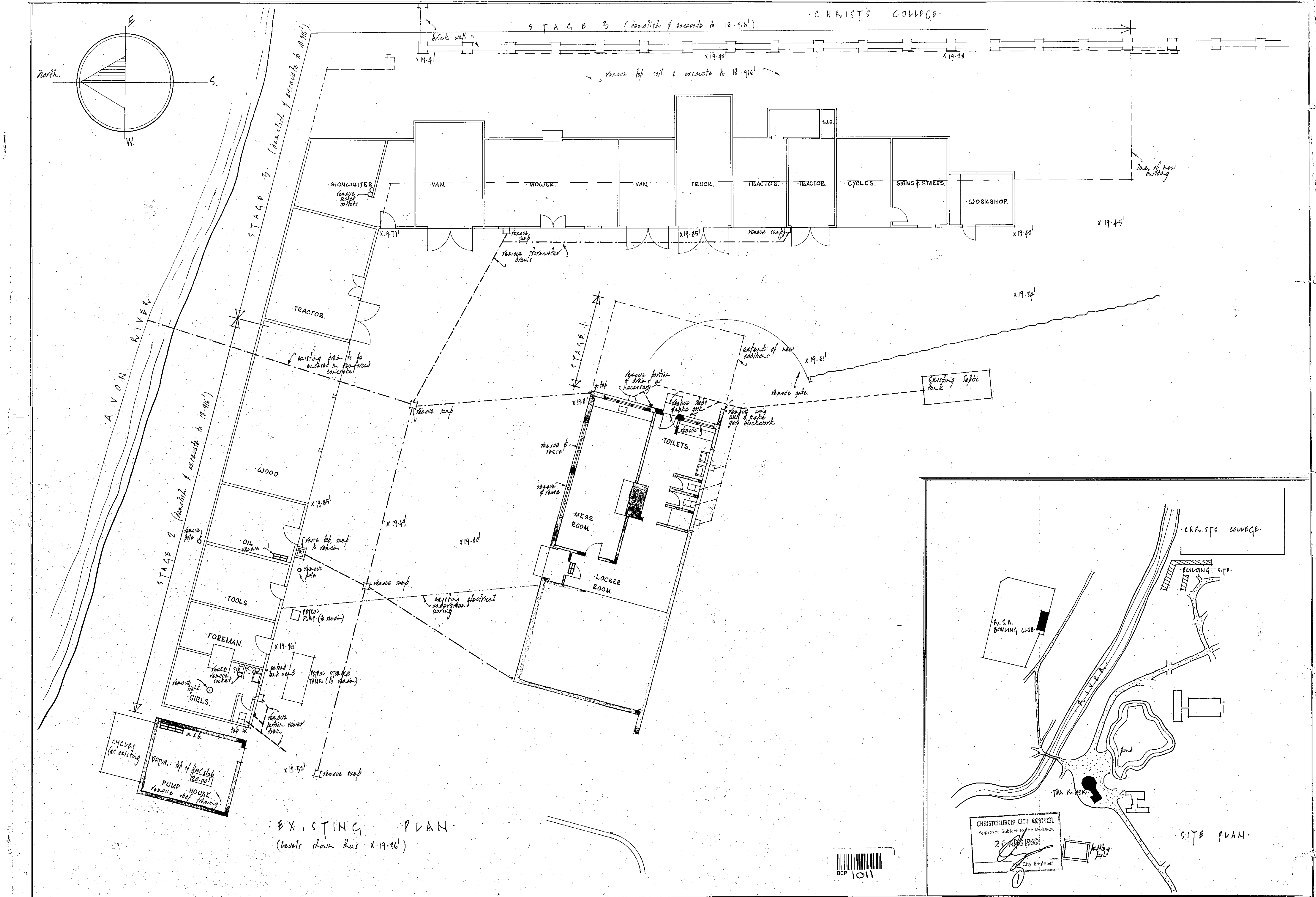
Photograph 12 Typical connections at the western end of the portal frames showing steel beams welded to steel columns.

Photograph 13 Typical timber framed roof supports above concrete masonry partition walls.

Photograph 14 Electromagentic scan of wall revealed one single steel bar at locations of intersecting perpendicular walls.

Appendix B

Existing Drawings

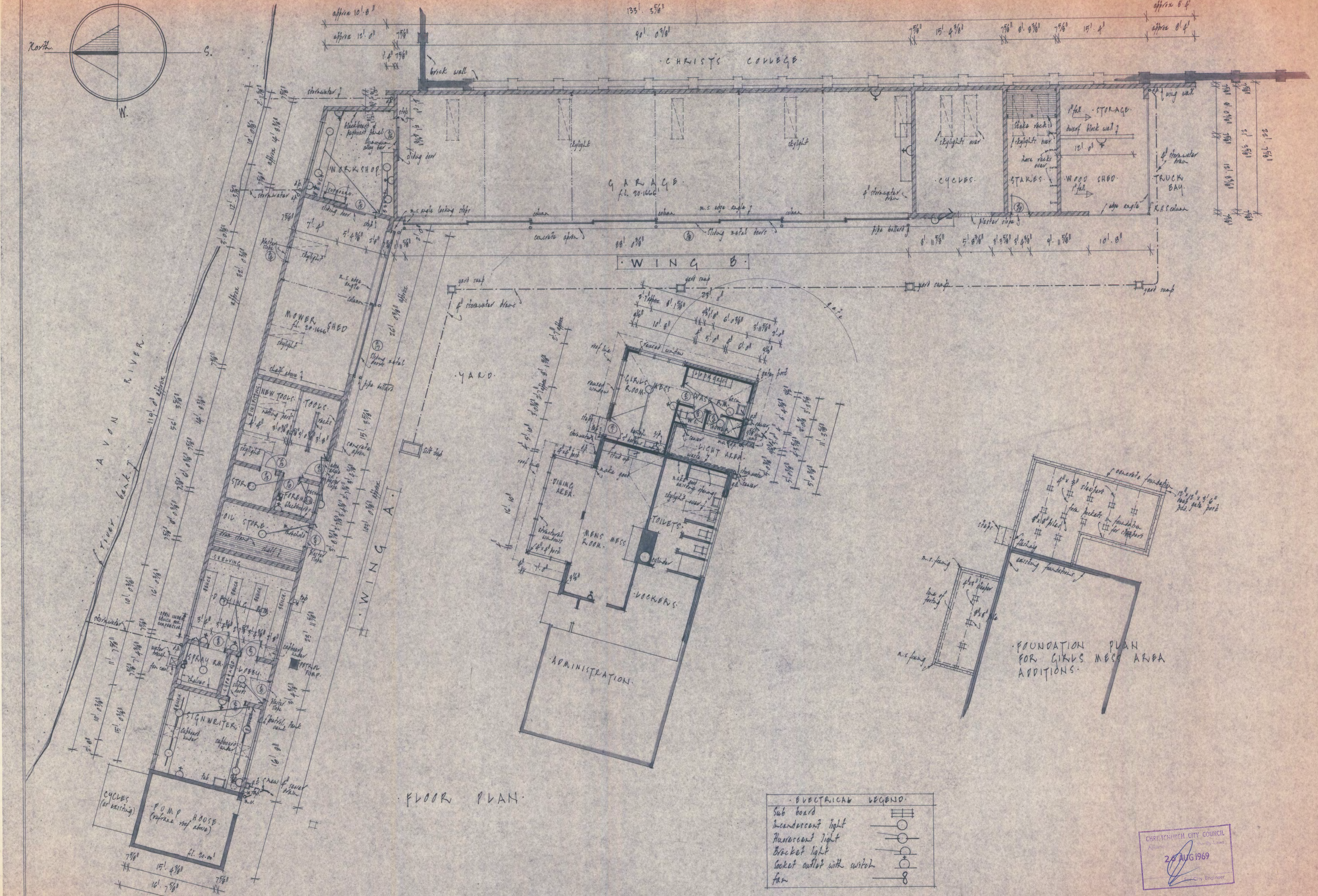


·EXISTING PLAN·
(levels shown thus x 19·36')

·SITE PLAN·

CHRISTCHURCH CITY COUNCIL
Approved Subject to the By-Laws
26 AUG 1969
For City Engineer
①

GERALD K. AUSTIN A.N.Z.I.A. Registered Architect 87 Victoria St. Christchurch Phone 40-213	HARDIE & ANDERSON Consulting Engineers Christchurch	IMPLEMENT SHEDS & STAFF FACILITIES · BOTANIC GARDENS for the CHRISTCHURCH CITY COUNCIL·	FILE No. 43/2 7.69	Existing plans showing drainage, levels, electrical fittings & demolition etc.	SCALES. Eight feet to One inch	SHEET No. 1 of 8	SET No. 12



CHRISTCHURCH CITY COUNCIL
Approved ... the By-Laws
2? AUG 1969
For City Engineer

GERALD K. AUSTIN A.N.Z.I.A.
Registered Architect
87 Victoria St. Christchurch. Phone 40-213

HARDIE & ANDERSON
Consulting Engineers
Christchurch

IMPLEMENT SHEDS & STAFF FACILITIES · BOTANIC GARDENS
for the CHRISTCHURCH CITY COUNCIL

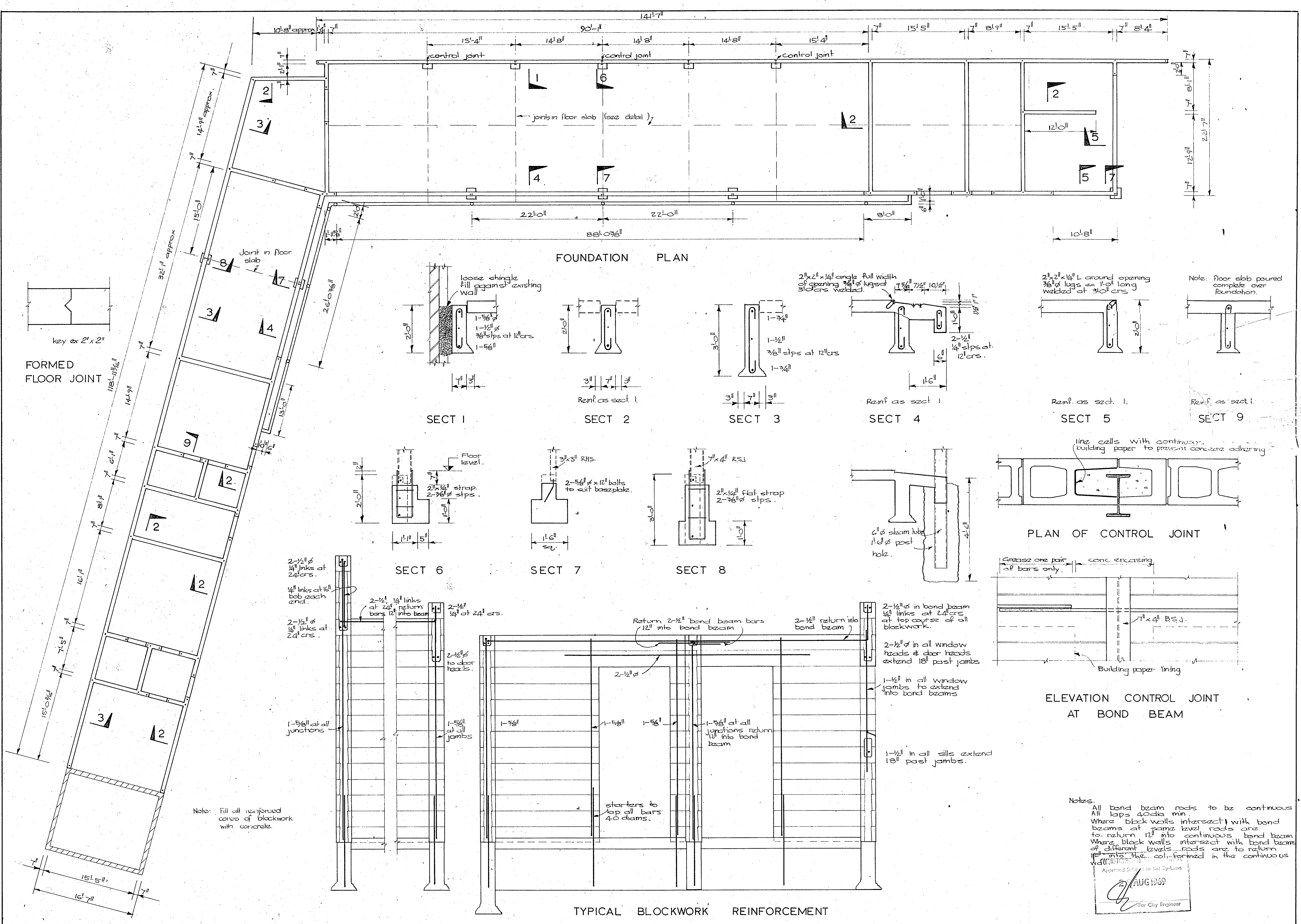
FILE NO
43/2
7.68

Floor & drainage plan
Foundation plan Mess additions
Electrical Plan

SCALES.
Eight feet to One inch

SHEET NO.
2 of 8

SET NO.
12



FOUNDATION PLAN

FORMED FLOOR JOINT

SECT 1

SECT 2

SECT 3

SECT 4

SECT 5

SECT 9

SECT 6

SECT 7

SECT 8

PLAN OF CONTROL JOINT

ELEVATION CONTROL JOINT
AT BOND BEAM

TYPICAL BLOCKWORK REINFORCEMENT

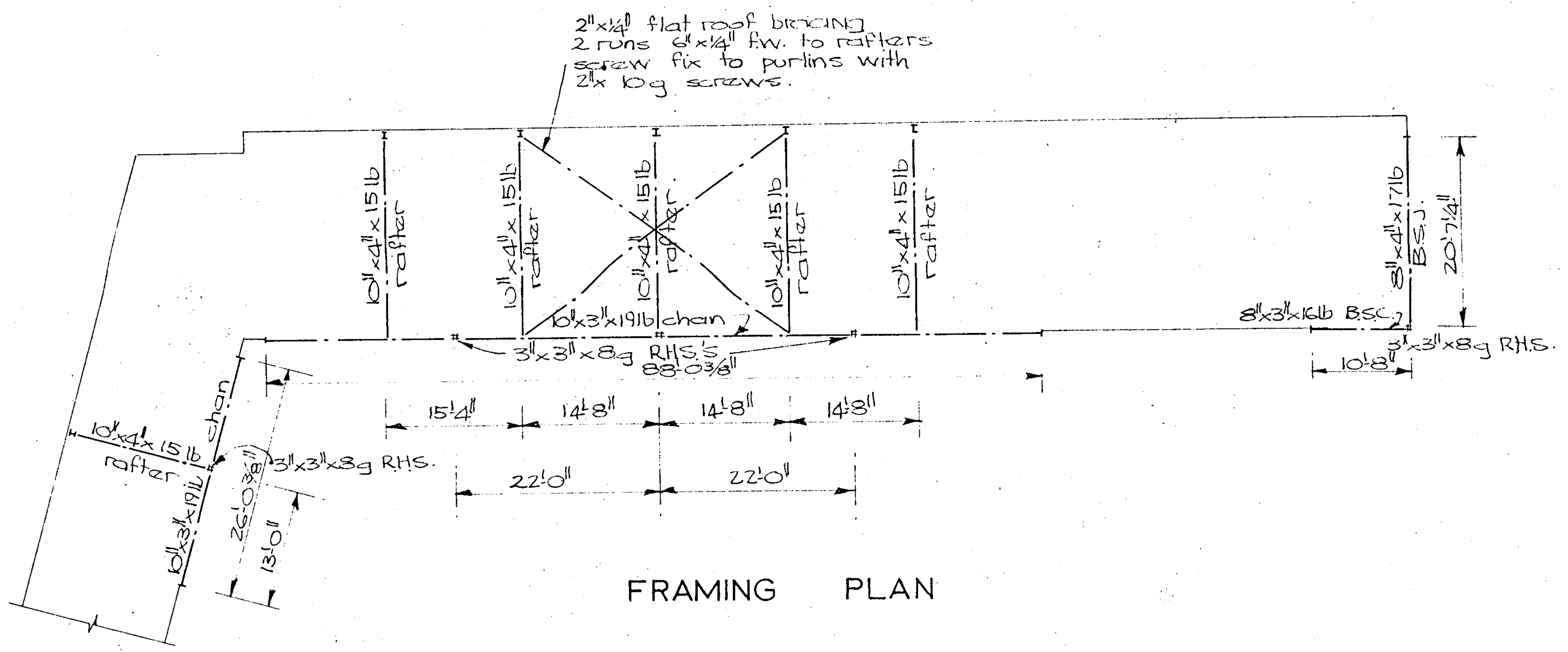
Note: Fill all reinforced cores of blockwork with concrete.

Notes.
All bond beam rods to be continuous
All laps 40dia min.
Where block walls intersect with bond beams at same level rods are to return 12" into continuous bond beam
Where block walls intersect with bond beams of different levels rods are to return 12" into the col. formed in the continuous wall.

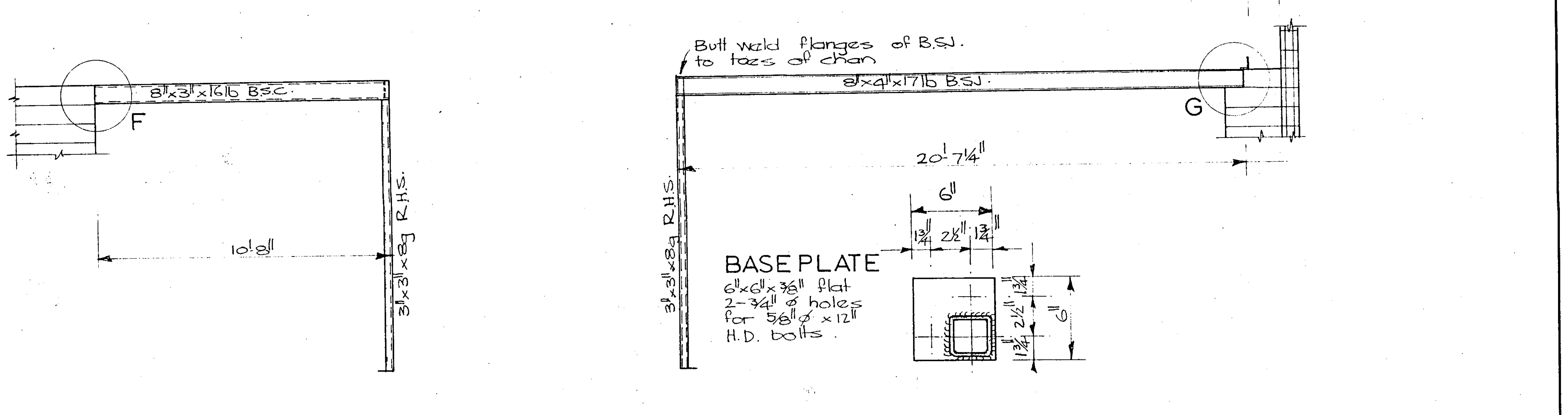
CHRISTCHURCH CITY COUNCIL
Approved Subject to the By-Laws
2? AUG 1969
For City Engineer

IMPLEMENT SHEDS & STAFF FACILITIES BOTANIC GARDENS FOR CHRISTCHURCH CITY COUNCIL	FOUNDATIONS & BLOCKWORK	Gerald K Austin ANZIA Registered Architect. 87 Victoria Street Christchurch Ph 40-213.	Hardie & Anderson Consulting Engineers 234 Armagh street Christchurch Ph 60969.	Scales ⅛". ½". 1½" To 1'-0"	Date July 1969	Ref Nº 527	Dwg Nº S1

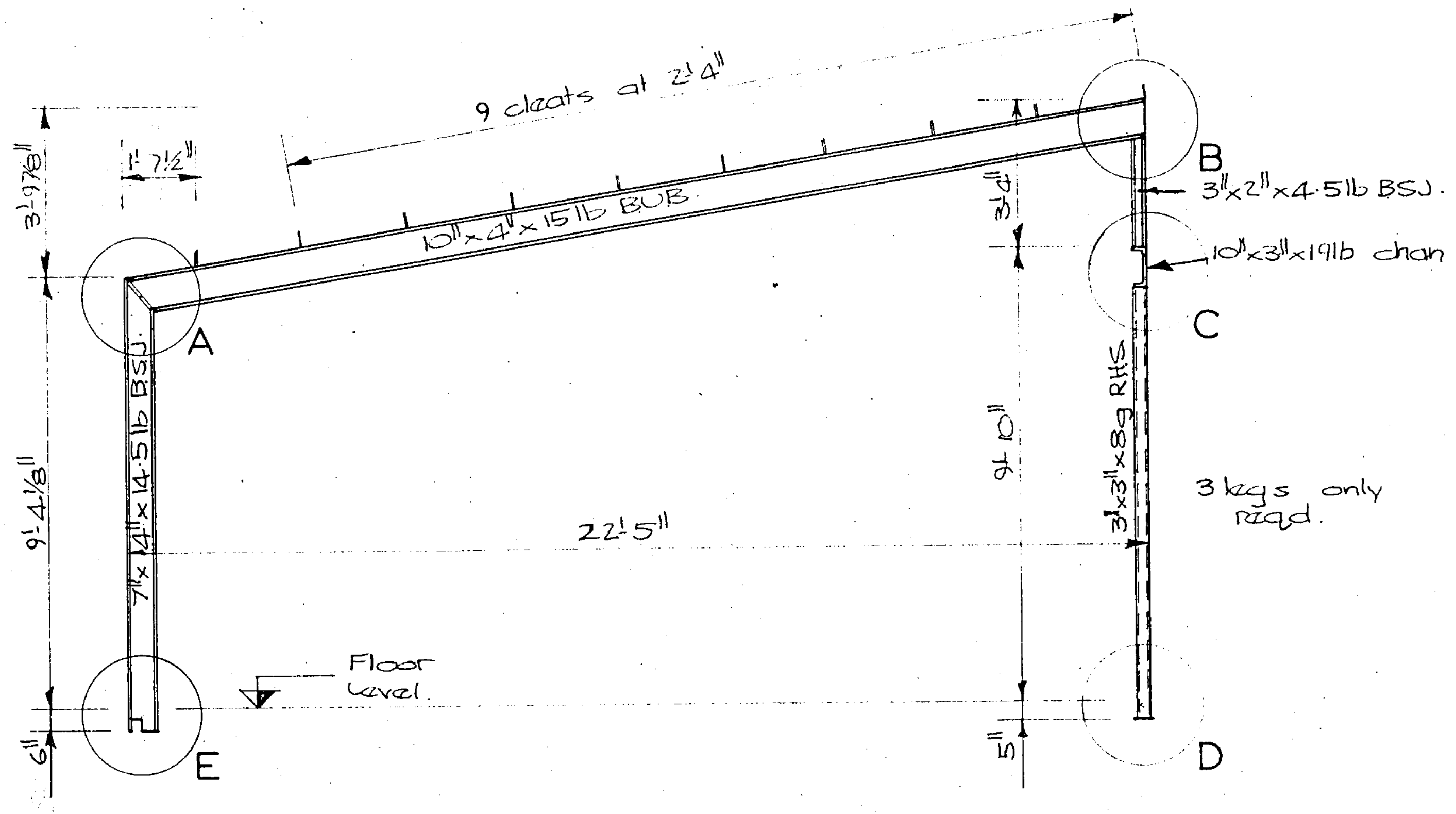
12



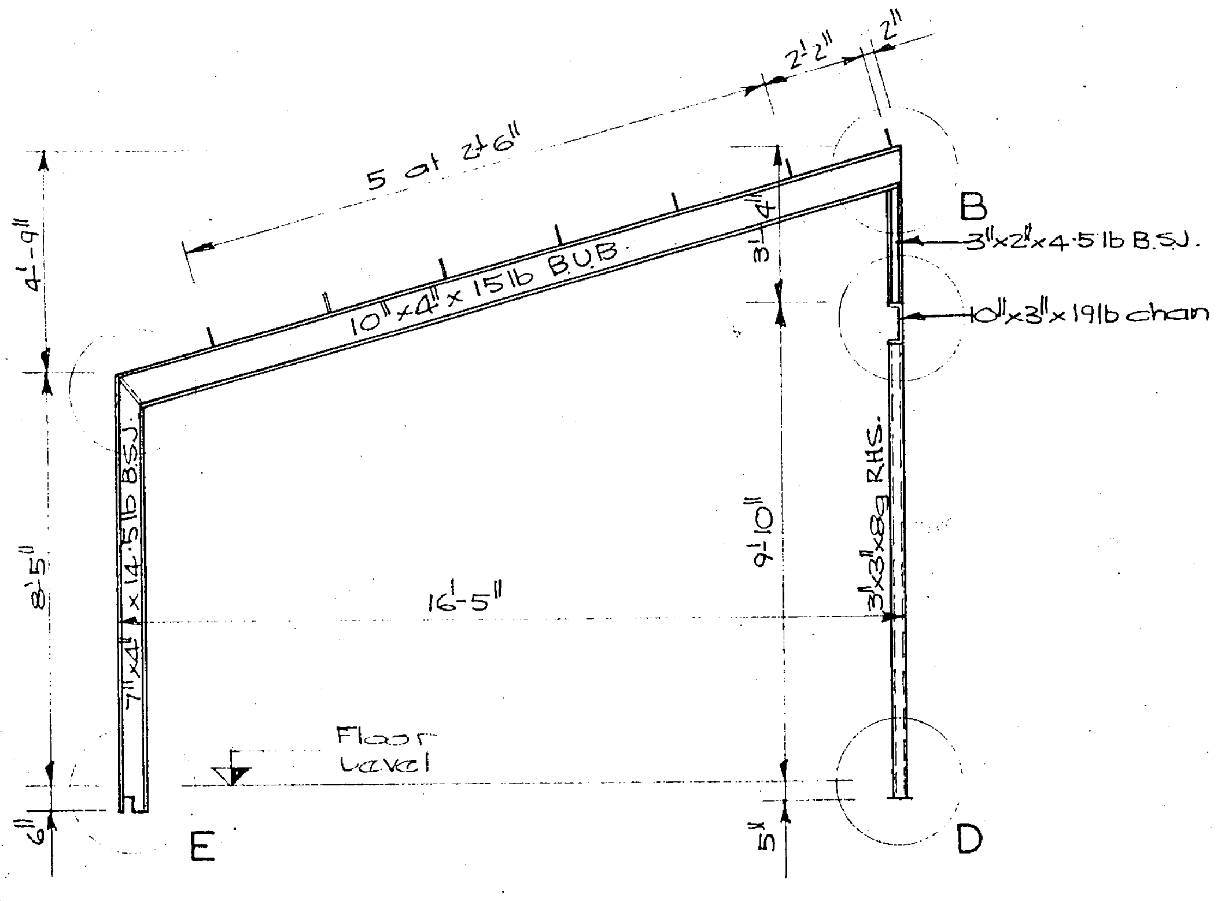
FRAMING PLAN



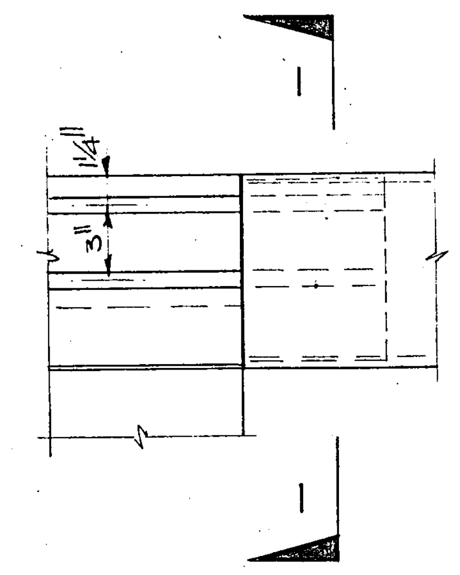
FRAMING TO WOOD SHED



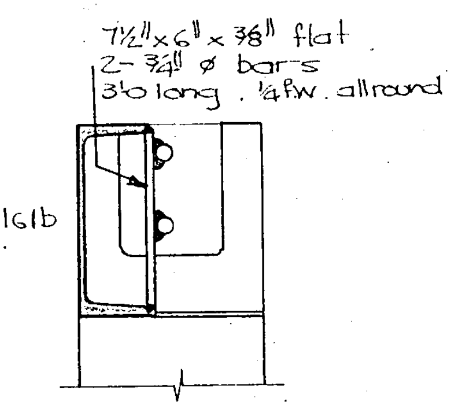
PORTAL



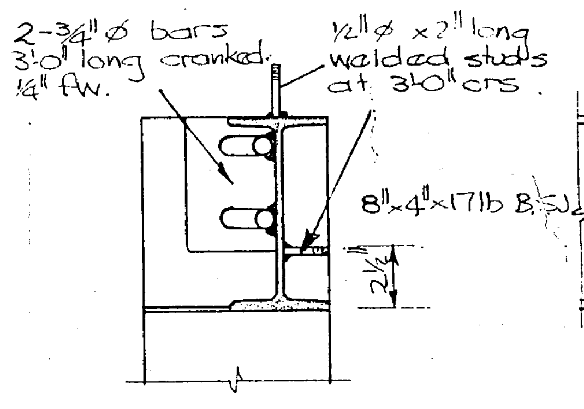
PORTAL



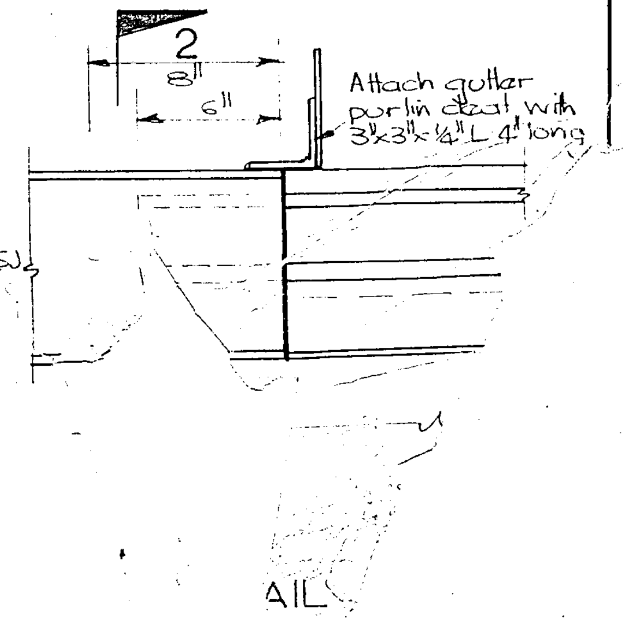
DETAIL F



SECT 1-1



SECT 2-2



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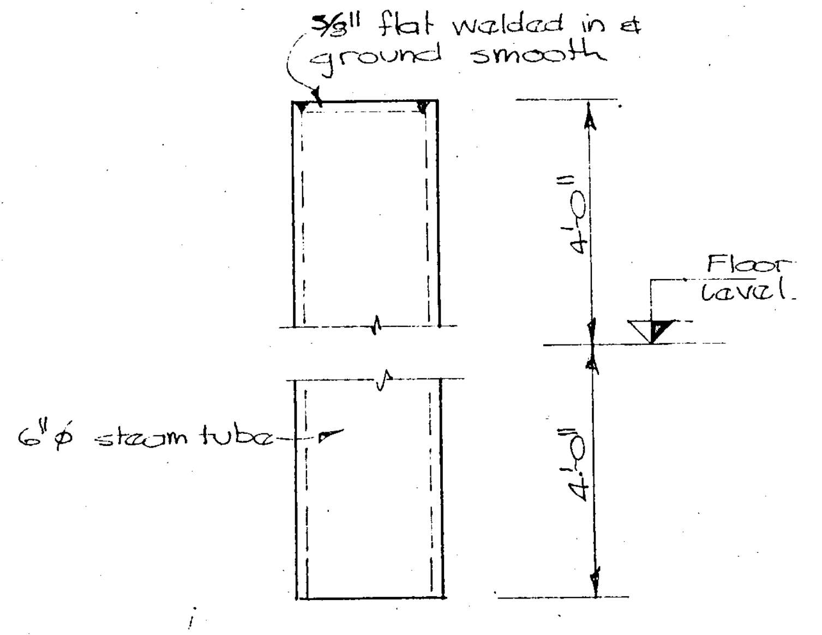
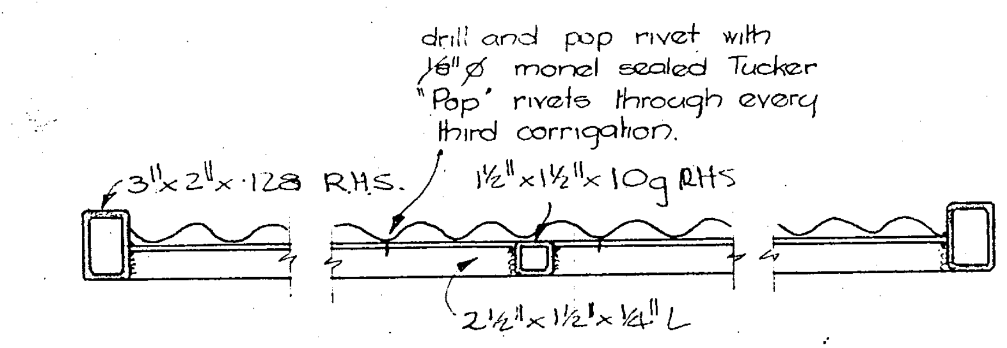
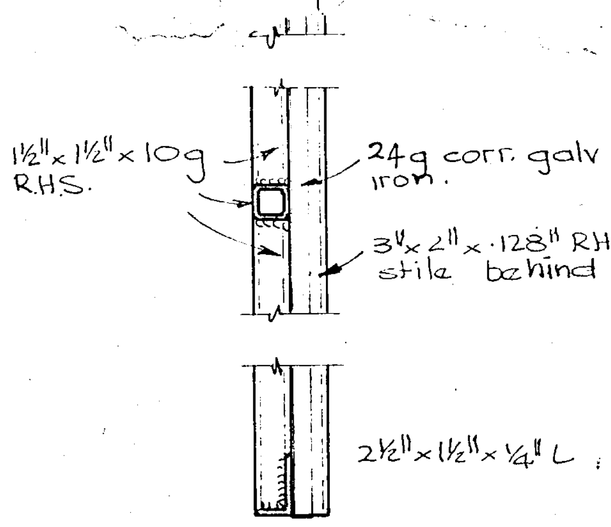


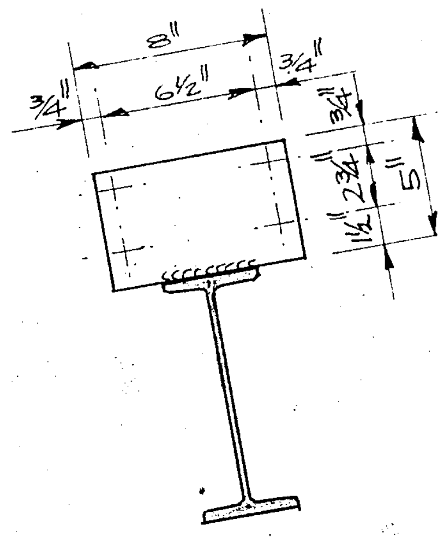
BOLLARDS



SECT 4-4

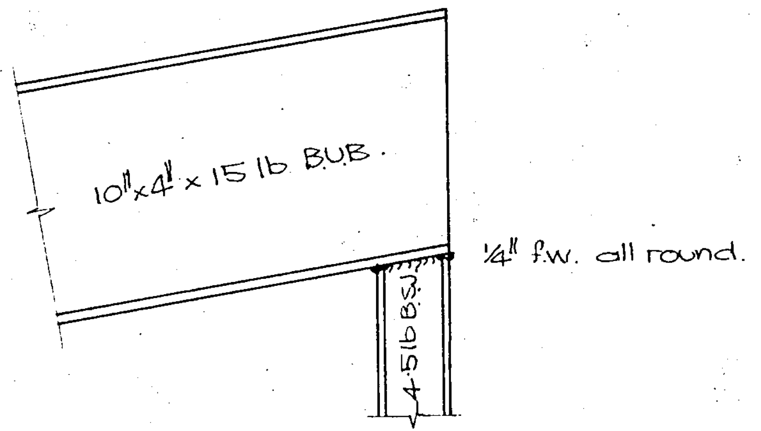


SECT 5-5

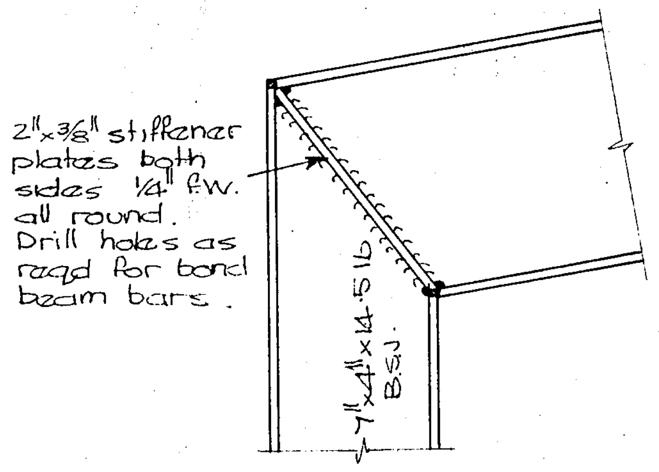


PURLIN CLEAT

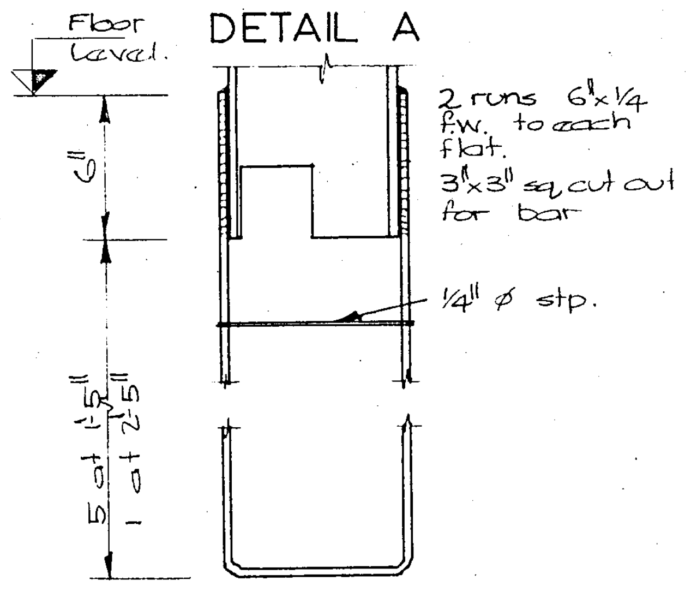
8"× 5"×¼" flat
4-9/16"ø holes
for ½"ø bolts
¼" f.w. to BUB.



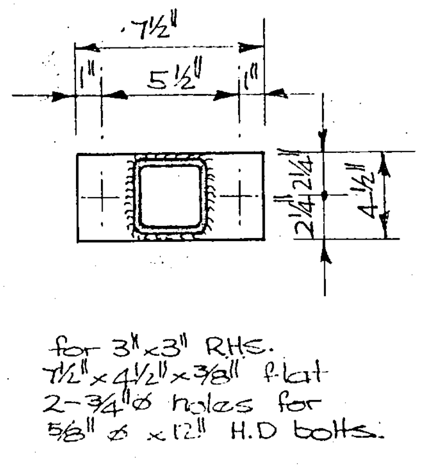
DETAIL B



DETAIL A

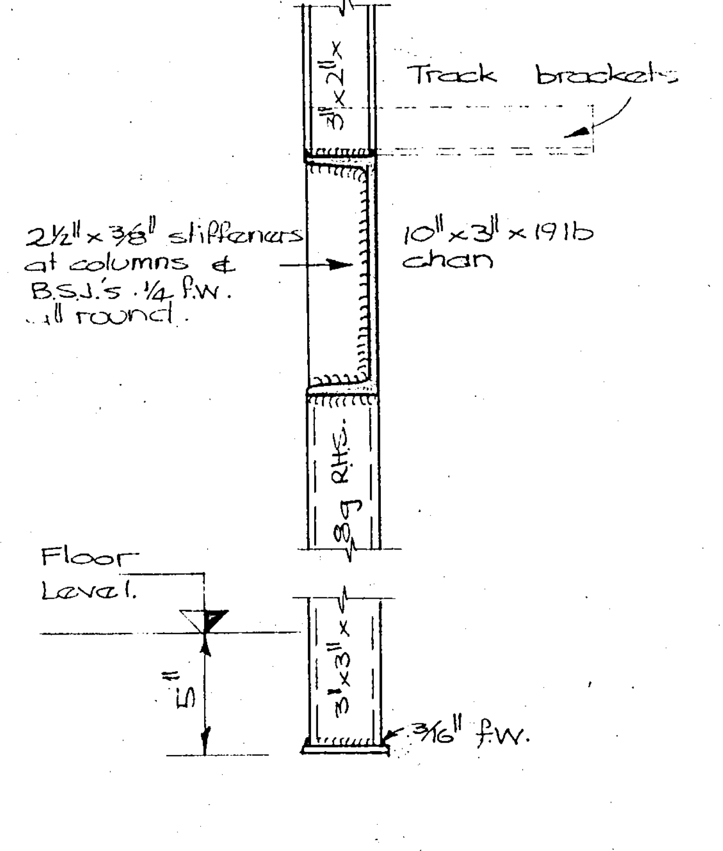


DETAIL E

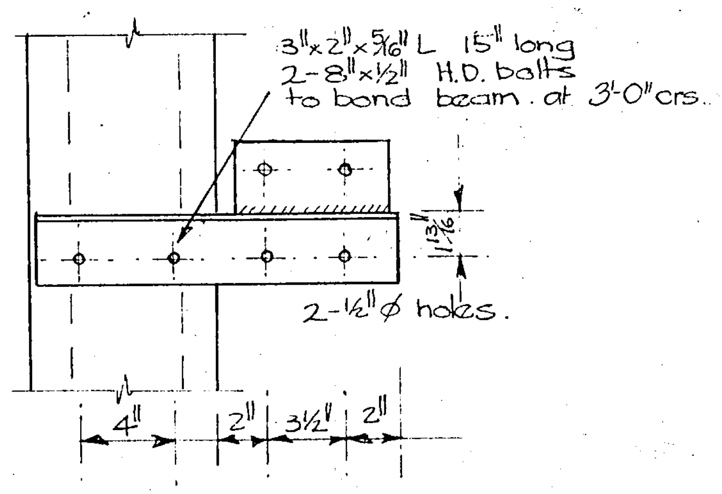


for 3"×3" RHS.
7½"×4½"×⅜" flat
2-¾"ø holes for
⅝" ø ×12" H.D bolts.

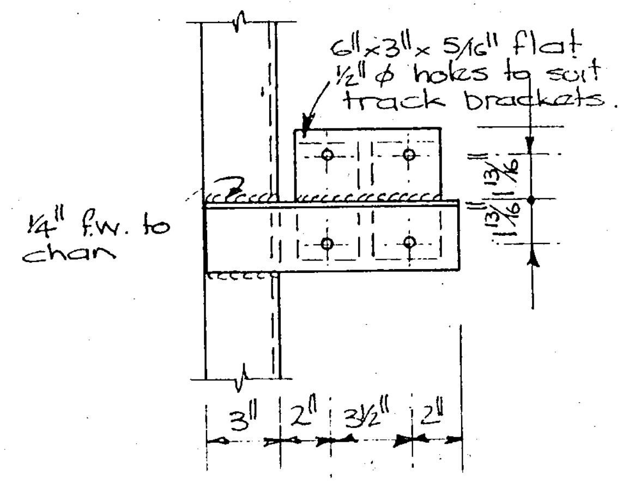
BASE PLATE



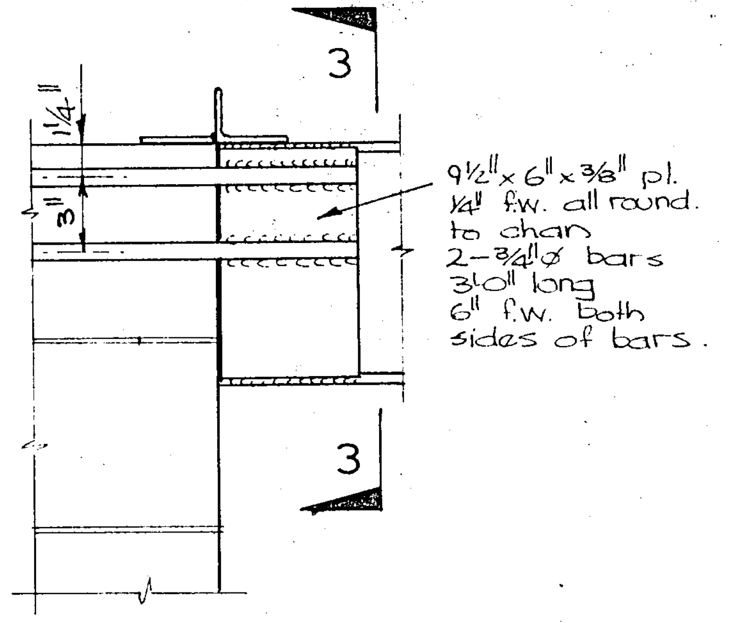
DETAIL D



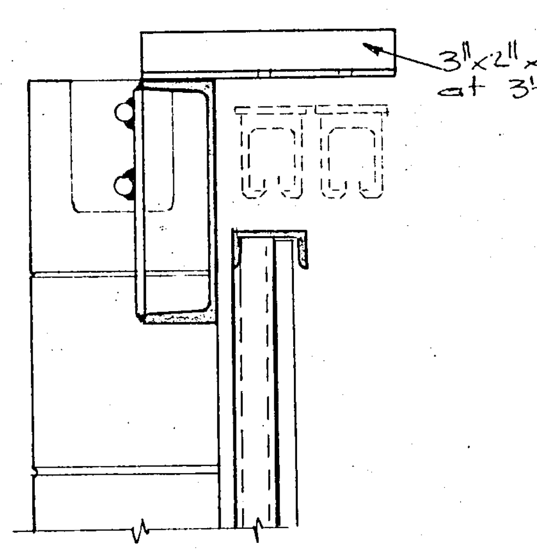
FASCIA BRACKETS (Type A)



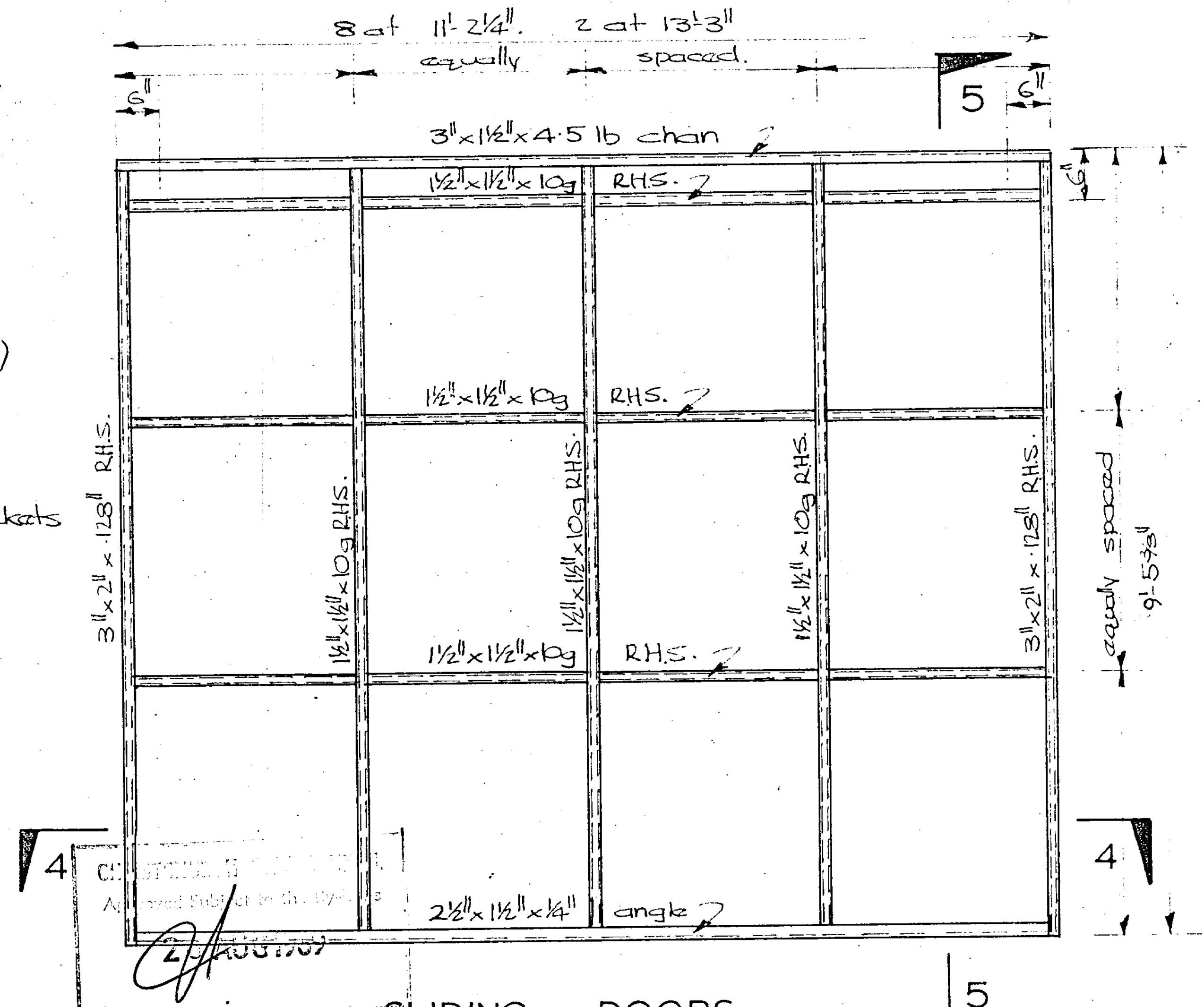
FASCIA BRACKETS (Type B)



DETAIL AT END OF LINTELS



SECT 3-3



Approved Subject to the By-laws

For City Engineer

SLIDING DOORS

IMPLEMENT SHEDS & STAFF FACILITIES BOTANIC GARDENS
FOR CHRISTCHURCH CITY COUNCIL

STEELWORK

Gerald. K. Austin. A.N.Z.I.A.
Registered Architect
87 Victoria Street.
Christchurch Ph 40·213

Hardie & Anderson
Consulting Engineers
234 Armagh Street
Christchurch Ph 60969.

Scales	Date	Ref No	Dwg No
1/16", ¼", 1½" To 1'-0"	July 1969.	527	S2

Appendix C

Masonry Wall Calculation Methodology

a. Quantitative Assessment

The quantitative assessment to the building comprised an investigation of in-plane and out-of-plane strength of the masonry block walls. The investigation was based on the analysis of the seismic loads that the structure is subjected to, the analysis of the distribution of these forces throughout the structure and the analysis of the capacity of existing structural elements to resist the forces applied. The capacity of the existing structural elements was compared to the demand placed on the element to give the %NBS of each of the structural elements.

b. Demand

The in-plane shear demand of each wall was assessed by completing a torsion analysis to the building. NZS 1170.5:2004 makes allowance for accidental eccentricity and requires that the earthquake action be applied at an eccentricity of 10% of the building dimension which is perpendicular to the force applied. This results in a torsional action about the centre of resistance of the building, and induces forces in the lateral force resisting (in-plane) walls in addition to the direct shear. As each wall was made of the same material and with the same properties, the direct shear and the force induced in each wall are proportional to the length squared. Cl 5.3.1.2 of NZS 1170.5: 2004 states that for nominally ductile and brittle structures an action set of 100% of the earthquake actions in one direction and 30% in the orthogonal direction must be applied when calculating the demand for any structural member and has such been applied in the analysis.

c. Seismic Coefficient

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

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

Where

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

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

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

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

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

$$\mathrm{S_P} = 1.33 - 0.3\mu$$

Where μ, the structural ductility factor, was taken as 1.00.

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 0.1 was assumed for the building. The coefficient was then calculated using Equation 5.2(1);

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

Where

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

d. In-Plane Capacity of the Unreinforced Walls

The in-plane capacity of the unreinforced concrete masonry wall was determined using the NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Resistance (06/2006). The NZSEE guidelines recommend checks for 4 different in-plane response modes.

- Diagonal tension failure mode
- Bed-sliding failure mode
- Toe crushing failure mode
- Rocking failure mode

An analysis of each wall was carried out using the methods set out in Section 8 – In-Plane Wall Response, of the NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Performance (06/2006).

e. In-plane Wall Properties of the Unreinforced Walls

Properties of in-plane loaded URM walls, piers or spandrels for use in the calculation of nominal in-plane shear capacity were as follows:

- **Unit Weight of Masonry**

2.10 kN/m^2 was adapted for the unit weight of 20-series concrete hollow block masonry with standard aggregate (see Table A2 from NZS 1170.1:2002).

- **Weight of Wall**

The weight of the wall, W_w, was calculated in accordance with the equation.

$$W_w = 1.82 \times l_w \times h$$

Where: l_w is the total wall length and h is the wall height.

- **Normal Force at Base of Wall**

The normal force acting on the cross section of the base of the wall, N_b, was calculated in accordance with the equation.

$$\mathrm{N_b = W_w + N_t}$$

Where: Values for weight of the wall, W_w, and axial load above the wall, N_t.

- **Diagonal Tension Strength**

The diagonal tension strength of masonry, f_{dt}, was calculated in accordance with the equation below for walls, piers and spandrels.

$$\mathrm{f_{dt}} = \frac{1}{2}\left(\mathrm{c} + \frac{\mathrm{N_t}}{\mathrm{A_w}} 0.8\mu_\mathrm{f}\right)$$

Where: Values for cohesion, c, and coefficient of friction, μ_f, were given in Section 2.5.5 of NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Performance. The factor of 0.8 is to account for vertical accelerations and other dynamic effects.

- **Distance to Centre of Inertia of Wall**

Distance to the centre of inertia of the wall from the compression toe, a_i, was calculated in accordance with the equation for walls with no flanges:

- **Average Compressive Stress**

Average compressive stress acting on the wall, σ_{ave}, was calculated in accordance with the equation

$$\frac{\quad}{\quad}$$

Where: Value for width of the block shell, b_w which was equivalent to 0.45 of the block width. This reduced value of b_w was calculated by multiplying the actual width by a modification factor based on the difference between the unit density of the block compared to the unit density of concrete.

$$\frac{\quad}{\quad}$$

f. In-plane Wall Shear Capacity of the Unreinforced Walls

The in-plane nominal shear capacity of a wall, pier or spandrel was taken as the minimum of the nominal capacity in the diagonal tension failure mode, V_{dt}, the rocking failure mode, V_r, the bed-joint sliding failure mode, V_s, and the toe crushing failure mode, V_{tc}.

$$\min(\mathrm{V} \qquad\qquad)$$

Nominal capacity of each failure mode was derived as following:

- **Capacity in Diagonal Tension Failure Mode, V_{dt}**

Nominal shear capacity corresponding to diagonal tension failure, V_{dt}, was calculated in accordance with the equation below for walls where no perpendicular flanges are present

$$\cdot\sqrt{\left(1 \quad \frac{\quad}{\quad}\right)}$$

Where: ζ was a factor to correct for nonlinear stress distribution (See Table 1010)

	ζ
Slender walls, where $h/l_w > 2$	1.5
Stout walls, where $h/l_w < 0.5$	1.0
Linear interpolation may be used for values of h/l_w	

Table 10 Shear stress factor for inclusion in diagonal tension failure mode equation

- **Capacity in Rocking Failure Mode, V_r**

Nominal shear capacity corresponding to the rocking failure mode, V_r, was calculated in accordance with the equation;

$$\frac{\quad}{\quad}\cdot\left[a \quad \frac{\quad}{\quad}\right]$$

Where: l_{er} was the effective length of the wall in rocking, taken as 0.1 x l_w.

- **Capacity in Bed-joint Sliding Failure Mode, V_s**

Bed-joint sliding failure was not an expected behaviour of URM walls subjected to seismic loading. The bed-joint sliding capacity of an in-plane loaded wall needed only be assessed when conditions suited the initiation of bed-joint sliding, specifically, when either or both the brick compressive strength and mortar compressive strength fell in the bounds of “soft”.

Ultimate shear capacity corresponding to bed-joint sliding failure, V_s, was calculated in accordance with the equation

$$V_s = l_w.b_w.c + 0.8.\mu_f.N_t$$

Where: Values for cohesion, c, and coefficient of friction, μ_f, were given in Section 2.5.5 of NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Performance. The factor of 0.8 is to account for vertical accelerations and other dynamic effects.

- **Capacity in Toe Crushing Failure Mode, V_{tc}**

Nominal shear capacity corresponding to toe crushing failure, V_{tc}, was calculated in accordance with the below equation for walls where perpendicular flanges were present:

$$V_{tc} = \frac{N_b}{h}.\left[\frac{1}{2}.l_w - \frac{1}{3}.l_{etc}\right]$$

Where the effective length of wall was calculated as:

$$l_{etc} = \frac{2.N_b}{1.3.f'_m.b_w}$$

g. Out-of-Plane Capacity of the Unreinforced Walls

The % NBS for out-of-plane flexure of the concrete masonry walls was determined using the methods set out in NZSEE guidelines for the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes Section 10.3. The following steps were those required to assess the displacement response capability and the displacement demand, from which the adequacy of the walls can be determined.

The wall panel was assumed to form hinge lines at the points where effective horizontal restraint was assumed to be applied. The centre of compression on each of these hinge lines was assumed to form a pivot point. The height between these pivot points was the effective panel height h. At mid-height between these pivots, a third pivot point is assumed to form.

Step 1

The wall panel was divided into two parts, a top part bounded by the upper pivot and the mid-height between the top and bottom pivots, and a bottom part bounded by the mid-height pivot and the bottom pivot.

Step 2

The weight of the wall parts, W_b of the bottom part and W_t of the top part, and the weight acting at the top of the storey, P were calculated.

Step 3

From the nominal thickness of the wall, t_{nom}, the effective thickness, t was calculated as follows:

$$t = t_{nom}\left(0.975 - 0.025\frac{P}{W}\right)$$

Step 4

The eccentricity values e_p, e_b, e_t and e_o were calculated. Usually, the eccentricities e_b and e_p will each vary between 0 and *t/2* (where *t* is the effective thickness of the wall). Exceptionally they may be negative.

Where,

e_p = eccentricity of the P measured from the centroid of W_t

e_t = eccentricity of the mid-height pivot measure from the centroid of W_t

e_b = eccentricity of the pivot at the bottom of the panel measured from the centroid of W_b

e_o = eccentricity of the mid-height pivot measured from the centroid of W_b

Step 5

The mid-height deflection, Δ_i was calculated, which would cause instability under static conditions. The following formula was used to calculate this deflection.

$$\Delta_i = \frac{b\mathrm{h}}{2a}$$

Where

$$b = W_b e_b + W_t(e_0 + e_b + e_t) + P\left(e_0 + e_b + e_t + e_p\right) - \Psi(W_b y_b + W_t y_t)$$

And

$$a = W_b y_b + W_t\left(\frac{h}{2} + y_t\right) + Ph$$

And

$$\Psi = Initial\ slope\ of\ wall$$

Step 6

The maximum usable deflection, Δ_m was calculated as 0.6 Δ_i.

Step 7

The period of the wall, T_p, was four times the duration for the wall to return from a displaced position measured by Δ_m to the vertical. The period was calculated from the following equation:

$$T_p = 6.27\sqrt{\frac{J}{a}}$$

Where J was the rotational inertia of the masses associated with W_b, W_t and P and any ancillary masses, and was given by the following equation.

$$J = J_{bo} + J_{to} + \frac{1}{g}\left\{W_b[{e_b}^2 + {y_b}^2] + W_t[(e_0 + e_b + e_t)^2 + {y_t}^2] + P\left[\left(e_0 + e_b + e_t + e_p\right)^2\right]\right\} + J_{ancillary}$$

Where;

$$J_{bo} = J_{to} = \frac{\left\{\left(\frac{W}{h}\right)[h^2 + 16t^2] + 4Pt^2\right\}}{g}$$

Where y_t was the distant from the top of the wall to the centroid of the top wall and y_b was the distant from the bottom of the wall to the centroid of the bottom wall.

Step 8

The seismic coefficient ($C_p(T_p)$) for an elastically responding part ($\mu_p = 1$) with this period (T_p), was calculated as follows:

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

Where

C(0) = the site hazard coefficient for T = 0 determined from NZS 1170.5 Section 3.1, using the values for the modal response spectrum method and numerical integration time history methods

C_{Hi} = the floor height coefficient for level I, from NZS 1170.5 Section 8.3.

$C_i(T_p)$ = the part spectral shape factor at level I, from NZS 1170.5 Section 8.4

Step 9

The participation factor, γ for the rocking system was taken as:

$$\gamma = \frac{(W_b y_b + W_t y_t)h}{2Jg}$$

Step 10

From $C_p(T_p)$, T_p, R_p *and* γ, the displacement response, D_{ph} was obtained from;

$$D_{ph} = \gamma \left(\frac{T_p}{2\pi}\right)^2 \times C_p(T_p) \times R_p \times g$$

Where R_p was from NZS 1170.5 Table 8.1

Appendix D

CERA Building Evaluation Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name: Botanic Gardens - Office Store

Building Address: Unit: No: 7 Street: Riccarton Avenue

Legal Description:

GPS south: Degrees: Min: Sec:

GPS east: Degrees: Min: Sec:

Building Unique Identifier (CCC): PRK 1566 BLDG 003 EQ2

Reviewer: Derek Chinn

CPEng No: 177243

Company: GHD

Company project number: 513059687

Company phone number: 03 3780900

Date of submission: 6/11/2012

Inspection Date: 19/09/2012

Revision:

Is there a full report with this summary? yes

Site

Site slope: flat

Soil type: mixed

Site Class (to NZS1170.5): D

Proximity to waterway (m, if <100m): 7

Proximity to clifftop (m, if < 100m):

Proximity to cliff base (m,if <100m):

Max retaining height (m): 0

Soil Profile (if available):

If Ground improvement on site, describe:

Approx site elevation (m): 10.00

Building

No. of storeys above ground: 1 single storey = 1

Ground floor split? no

Storeys below ground 0

Foundation type: strip footings

Building height (m): 4.50

Floor footprint area (approx): 450

Age of Building (years): 43

Ground floor elevation (Absolute) (m):

Ground floor elevation above ground (m): 0.15

if Foundation type is other, describe:

height from ground to level of uppermost seismic mass (for IEP only) (m): 4.5

Date of design: 1965-1976

Strengthening present? no

If so, when (year)?

And what load level (%g)?

Brief strengthening description:

Use (ground floor): public

Use (upper floors):

Use notes (if required): Mixed Usage

Importance level (to NZS1170.5): IL2

Gravity Structure

Gravity System: load bearing walls

Roof: timber framed

rafter type, purlin type and cladding: Timber purlins running the length of each wing attached to timber roof trusses.

Floors: concrete flat slab

slab thickness (mm): 125

Beams: steel non-composite

beam and connector type: Parallel Flange Channels. Welded Connections

Columns: structural steel

typical dimensions (mm x mm): 75 x 75

Walls: partially filled concrete masonry

thickness (mm): 200

Lateral load resisting structure

Lateral system along: partially filled CMU

Ductility assumed, μ: 1.50

Period along: 0.40 ##### enter height above at H31

Total deflection (ULS) (mm):

maximum interstorey deflection (ULS) (mm):

Note: Define along and across in detailed report!

note total length of wall at ground (m):

wall thickness (m): 0.2

estimate or calculation? estimated

estimate or calculation?

estimate or calculation?

Lateral system across: partially filled CMU

Ductility assumed, μ: 1.50

Period across: 0.40 ##### enter height above at H31

Total deflection (ULS) (mm):

maximum interstorey deflection (ULS) (mm):

note total length of wall at ground (m):

wall thickness (m): 0.2

estimate or calculation? estimated

estimate or calculation?

estimate or calculation?

Separations:

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

Non-structural elements

Stairs:
Wall cladding:
Roof Cladding: Metal
Glazing: aluminium frames
Ceilings:
Services(list):

describe

Available documentation

Architectural	partial	original designer name/date	Gerald K Austin, 1969
Structural	partial	original designer name/date	Hardie & Anderson Consulting Engineers, 1969
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:	none observed	notes (if applicable):	
Liquefaction:	none apparent	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	none apparent	notes (if applicable):	

Building:

Current Placard Status: green

Along
Damage ratio: 100%
Describe how damage ratio arrived at:
Describe (summary): Minor Damage

Across
Damage ratio:
Describe (summary): Minor Damage

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

Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	no	Describe:	

Recommendations

Level of repair/strengthening required: Describe:
Building Consent required: Describe:
Interim occupancy recommendations: Describe:

Along
Assessed %NBS before: 24%
Assessed %NBS after:

If IEP not used, please detail assessment methodology: NZS3404:1997 + NZSEE Guidelines

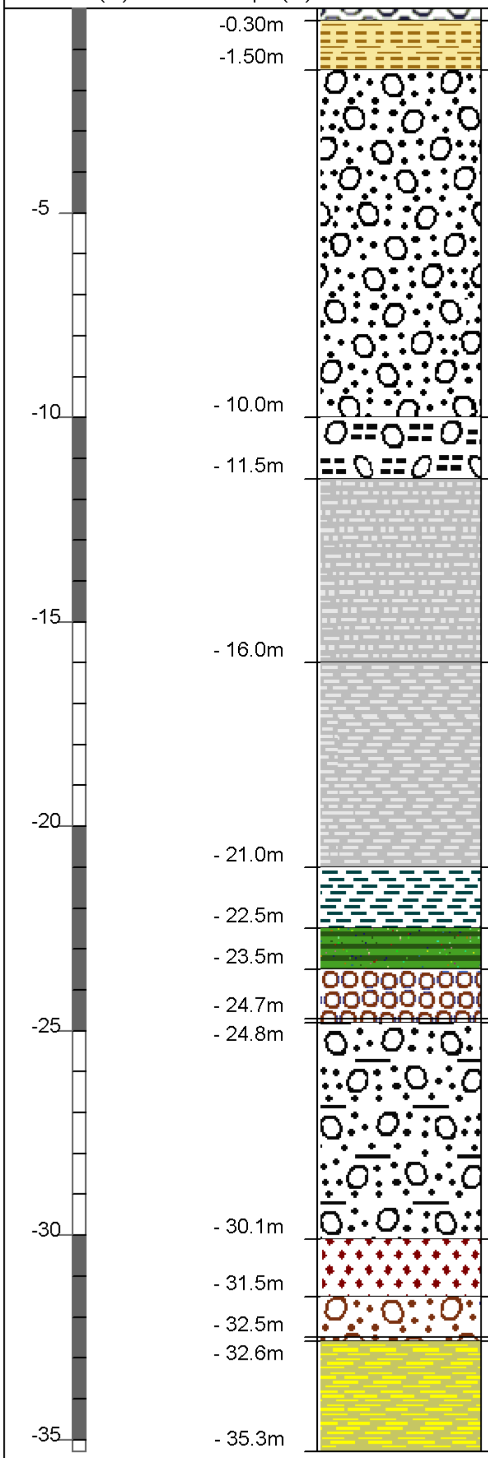
Across
Assessed %NBS before: 24%
Assessed %NBS after:

Appendix E

Geotechnical Appendix

Borelog for well M35/10619 page 1 of 3

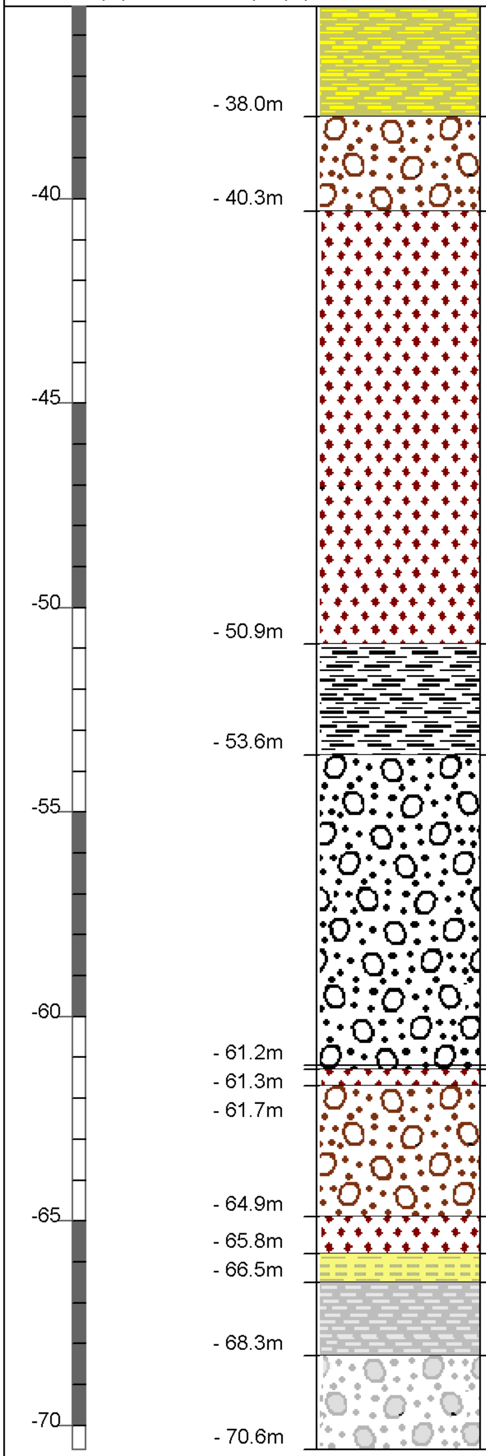
Gridref: M35:7952-4188 Accuracy : 3 (1=high, 5=low)
Ground Level Altitude : 7.5 +MSD
Driller : Clemence Drilling Contractors
Drill Method : Rotary/Percussion
Drill Depth : -105.9m Drill Date : 6/10/2006



Depth(m)	Full Drillers Description	Formation Code
-0.30m	topsoil	sp
-1.50m	silty brown clay	sp
- 10.0m	gravel and sand	sp
- 11.5m	grey pug and gravel	sp
- 16.0m	puggy grey sand	ch?
- 21.0m	soft silty grey pug	ch?
- 22.5m	hard blue/green pug	ch?
- 23.5m	soft puggy peat	ch
- 24.7m	tight brown gravel	ri
- 24.8m	yellow clay seam	
- 30.1m	tight sandy brown gravel (traces of clay)	ri
- 31.5m	brown sand	ri
- 32.5m	loose brown sandy gravel	ri
- 32.6m	yellow clay seam	
- 35.3m	hard sticky yellow clay	ri

Borelog for well M35/10619 page 2 of 3

Gridref: M35:7952-4188 Accuracy : 3 (1=high, 5=low)
Ground Level Altitude : 7.5 +MSD
Driller : Clemence Drilling Contractors
Drill Method : Rotary/Percussion
Drill Depth : -105.9m Drill Date : 6/10/2006

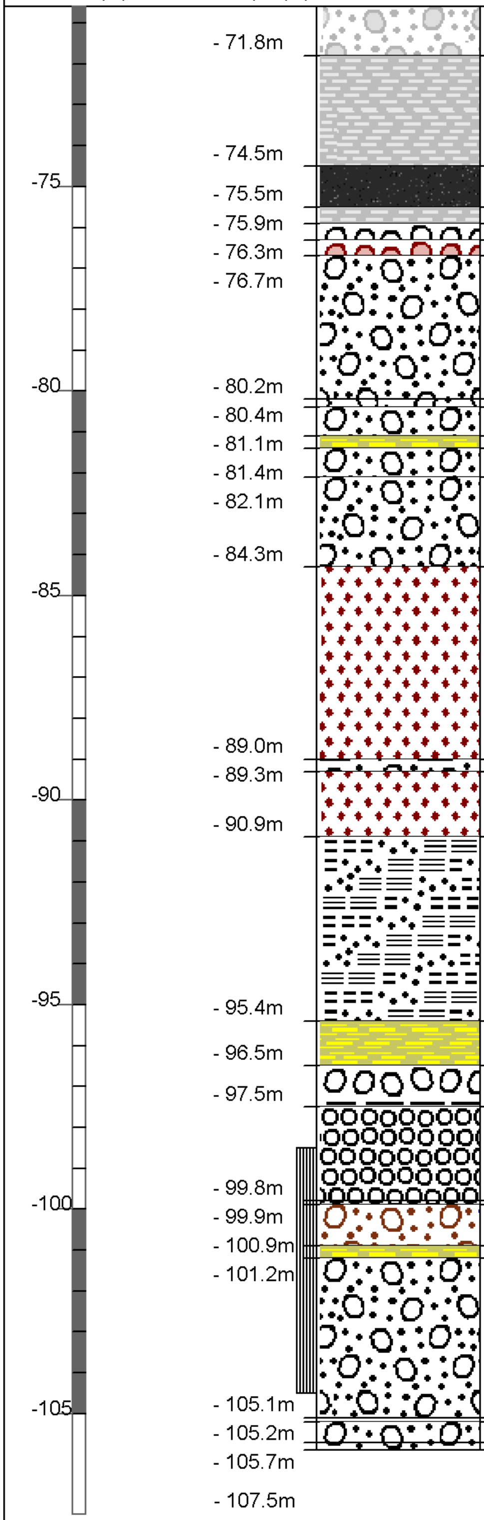


Depth(m)	Full Drillers Description	Formation Code
- 38.0m	hard sticky yellow clay	ri
- 40.3m	small brown gravel - progressively sandier	br
- 50.9m	brown sand	br
- 53.6m	hard sticky yellow/orange clay	br
- 61.2m	sandy grey/brown gravel	li-1
- 61.3m	clay seam	li-2
- 61.7m	brown sand (rusty water)	li-2
- 64.9m	tight brown stained gravel - sandy	li-2
- 65.8m	brown sand (traces gravel)	li-2
- 66.5m	hard silty yellow clay	li-2
- 68.3m	silty grey pug (traces peat)	li-2
- 70.6m	loose grey sandy gravel	li-2

Borelog for well M35/10619 page 3 of 3

Gridref: M35:7952-4188 Accuracy : 3 (1=high, 5=low)
Ground Level Altitude : 7.5 +MSD
Driller : Clemence Drilling Contractors
Drill Method : Rotary/Percussion
Drill Depth : -105.9m Drill Date : 6/10/2006

Environment Canterbury Regional Council

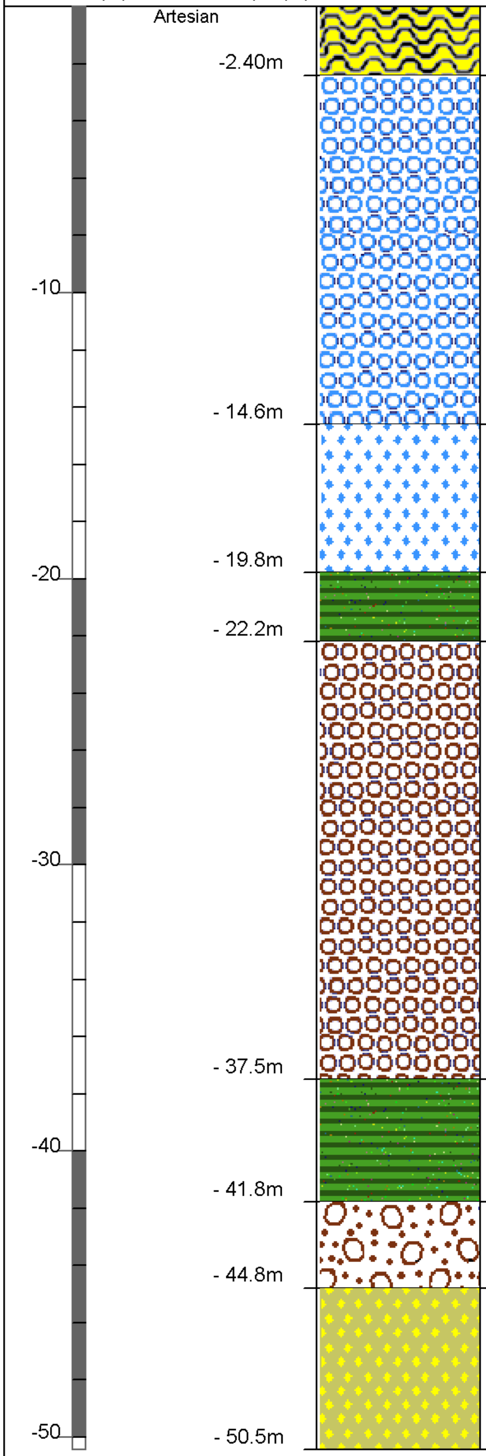


Depth(m)	Full Drillers Description	Formation Code
- 71.8m	loose grey sandy gravel	li-2
- 74.5m	soft sticky grey pug (traces peat)	li-2
- 75.5m	peat (some timber)	li-2
- 75.9m	hard sticky grey pug	li-2
- 76.3m	grey/blue clay bound gravel	li-3
- 76.7m	brown clay bound gravel	li-3
- 80.2m	loose very sandy heavily stained gravel	li-3
- 80.4m	hard sticky yellow clay	li-3
- 81.1m	tight sandy stained gravel	li-3
- 81.4m	hard yellow clay	li-3
- 82.1m	tight lightly stained sandy gravel	li-3
- 84.3m	tight lightly stained very sandy gravel	li-3
- 89.0m	brown sand (traces gravel)	he
- 89.3m	small sandy brown gravel (traces clay)	he
- 90.9m	brown sand (traces gravel)	he
- 95.4m	hard silty/sandy yellow/brown clay	he
- 96.5m	hard sticky yellow clay	he
- 97.5m	claybound gravel	bu
- 99.8m	loose grey/brown gravel	bu
- 99.9m	yellow clay seam	bu
- 100.9m	loose sandy brown gravel	bu
- 101.2m	hard yellow clay	bu
- 105.1m	very loose sandy grey/brown gravel	bu
- 105.2m	yellow clay seam	bu
- 105.7m	large loose stained sandy gravel (some heavily stained)	bu?
- 107.5m		

Borelog for well M35/1936 page 1 of 2

Environment Canterbury Regional Council

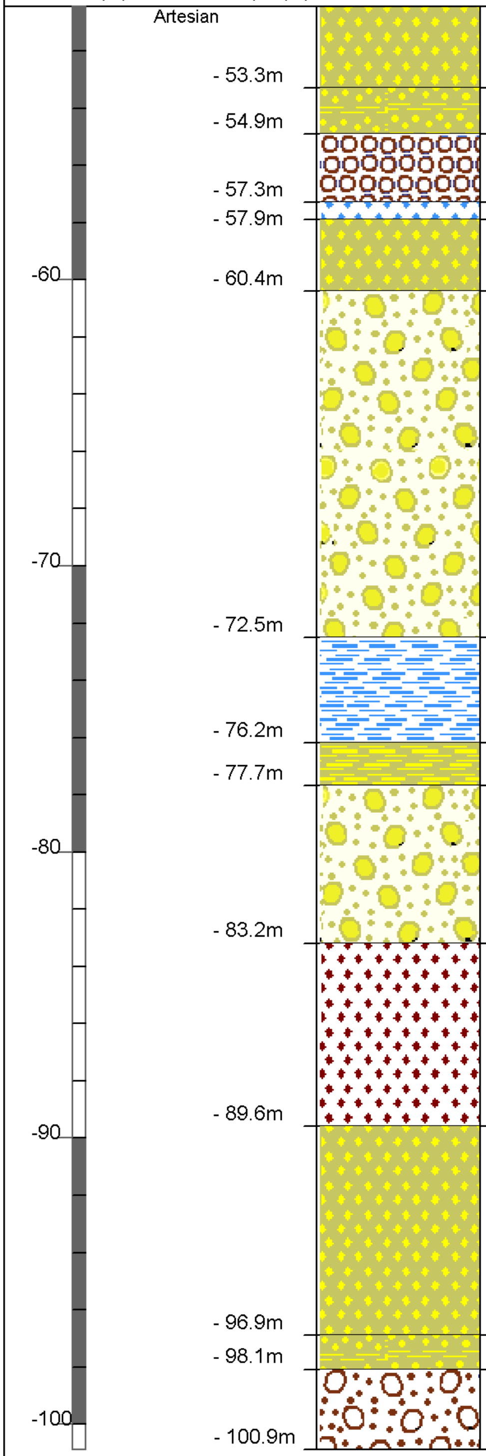
Gridref: M35:79554-41858 Accuracy : 2 (1=high, 5=low)
Ground Level Altitude : 7.6 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Hydraulic/Percussion
Drill Depth : -100.9m Drill Date : 2/07/1898



Water Level	Depth(m)	Full Drillers Description	Formation Code
Artesian	-2.40m	Surface soil & Yellow sand	sp?
	- 14.6m	Hard Blue shingle	sp?
	- 19.8m	Soft Blue sands	ch
	- 22.2m	Clay and peat	ch
	- 37.5m	Brown shingle 1st strata water .9m below surface	ri
	- 41.8m	Clay & peat	br
	- 44.8m	Hard Brown sand & gravel	br
	- 50.5m	Soft Yellow sand	br

Borelog for well M35/1936 page 2 of 2

Gridref: M35:79554-41858 Accuracy : 2 (1=high, 5=low)
Ground Level Altitude : 7.6 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Hydraulic/Percussion
Drill Depth : -100.9m Drill Date : 2/07/1898



Water Level	Depth(m)	Full Drillers Description	Formation Code
Artesian	- 53.3m	Soft Yellow sand	br
	- 54.9m	Soft Yellow clay & sand mixed	br
	- 57.3m	Hard Brown shingle	li-1
	- 57.9m	Soft Blue sand	li-2
	- 60.4m	Soft Yellow sand	li-2
	- 72.5m	Hard Yellow sand & gravel water rise +0.91m flow 6 gpm	li-2
	- 76.2m	Soft Blue clay	li-2
	- 77.7m	Soft Yellow clay	li-2
	- 83.2m	Hard Yellow sand & gravel. Water rise +2.7m & flow 30gpm 0.61m high	li-3
	- 89.6m	Soft Brown sand	he
	- 96.9m	Soft Yellow sand	he
	- 98.1m	Soft Yellow sand with clay	he
	- 100.9m	Brown gravel & sand. Water rise +4.3m flow 45gpm 1.1m high	bu

BOREHOLE LOG

Site Identification: **BH01**

Sheet 1 of 2

Project: Christchurch Botanic Gardens
Client: Christchurch City Council
Site: Botanic Gardens
Job No.: 513059691

Coordinates: E 2479 508, **N** 5741 909
Datum: NZMG
Surface RL (m): +8.0m
Total Depth: 19.5m
Commenced: 08-Oct-12
Contractor: McMillan Specialist Driling
Completed: 10-Oct-12
Driller: McMillan

Equipment: Truck 9700D
Inclination: -90
Shear Vane:
Comments:
Bore Diameter (mm):

Logged: DW
Processed: DW
Checked:

Depth (m)/ [Elev.]	Drilling Method	Core Run / Recovery (%)	Support / Casing (m)	Water	Geological Fm	Classification	Graphic Log	SOIL DESCRIPTION: (Soil Code), Soil Name [minor MAJOR], colour, structure [zoning, defects, cementing], plasticity or grain size, secondary components, structure. (Geological Formation) / ROCK DESCRIPTION: Weathering, colour, fabric, ROCK NAME (Formation Name)	Moisture Condition	Consistency/ Relative Density	Weathering	Estimated Rock Strength EW VW W MS S VS ES	RQD (%)	Defect Spacing (mm) 20 60 200 600 2000	TESTS & SAMPLES / ROCK MASS DEFECTS: Depth, Type, Inclinations, Roughness, Texture, Aperture, Coating
	Dual Tube	54	None		Springston Formation	SP		Gravelly medium to coarse SAND; brown. Dry; well graded; gravel, fine to coarse, angular to subrounded, greywacke.	D						
0.3 [+7.8]						SP		Organic fine SAND with some silt; dark brown. Moist; poorly graded.	M						
0.5 [+7.5]						SP		Fine SAND; brown. Moist; poorly graded.	M						
0.8 [+7.2]								CORELOSS							
1.5 [+6.5]		67				GP		Sandy fine to coarse GRAVEL; brown. Loose; moist; well graded, angular to subrounded, greywacke; sand, fine to coarse.	M	L					SPT 8,8, 4,2, 2,1, [9]
2.5 [+5.5]								CORELOSS *inferred* Sandy GRAVEL							
3.0 [+5.0]		53				GP		Sandy fine to coarse GRAVEL; brown. Loose; moist; poorly graded, angular to subrounded, greywacke; sand, fine to coarse.	M	L					SPT 2,1, 2,1, 3,3, [9]
3.8 [+4.2]								CORELOSS							
4.5 [+3.5]		67				SP		Medium to coarse SAND; grey. Dense; wet; poorly graded.	W	D					SPT 6,6, 6,8, 9,9, [32]
4.8 [+3.2]						GP		Sandy fine to coarse GRAVEL; grey. Dense; wet; poorly graded, angular to subrounded, greywacke; sand, medium to coarse.	W	D					
5.5 [+2.5]								CORELOSS *inferred* Sandy GRAVEL							
6.0 [+2.0]		40				GP		Sandy fine to coarse GRAVEL; grey. Dense; wet; poorly graded, angular to subrounded, greywacke; sand, medium to coarse.	W	D					SPT 6,7, 10,9, 10,12, [41]
6.1 [+1.9]						SP		Medium SAND; grey. Medium dense; wet; poorly graded.	W	MD					
6.2 [+1.8]						GP		Sandy fine to coarse GRAVELwith trace silt; grey. Dense; wet; poorly graded, angular to subrounded, greywacke; sand, medium to coarse.	W	D					
6.6 [+1.4]								CORELOSS *inferred* Sandy GRAVEL							
7.5 [+0.5]		73				SP		Gravelly medium to coarse SAND; grey. Dense; wet; gravel, fine to medium, angular to subrounded, greywacke	W	D					SPT 11,12, 15,16, 16,17, [50]
7.7 [+0.3]						GP		Sandy fine to coarse GRAVELwith trace silt; grey. Medium dense to dense; wet; poorly graded, angular to subrounded, greywacke; sand, medium to coarse.	W	MD					
8.6 [-0.6]								CORELOSS *inferred* Sandy GRAVEL							
9.0 [-1.0]		100				GP		Sandy medium to coarse GRAVELwith trace silt; grey. Medium dense; wet; poorly graded, angular to subrounded, greywacke; sand, fine to coarse.	W	MD					SPT 10,7, 5,5, 5,5, [20]

BOREHOLE LOG

GHD Limited

PO Box 13468
Christchurch 8141

Site Identification: **BH01**

Sheet 2 of 2

Project: Christchurch Botanic Gardens
Client: Christchurch City Council
Site: Botanic Gardens
Job No.: 513059691

Coordinates: **E** 2479 508, **N** 5741 909
Surface RL (m): +8.0m
Commenced: 08-Oct-12
Completed: 10-Oct-12

Datum: NZMG
Total Depth: 19.5m
Contractor: McMillan Specialist Driling
Driller: McMillan

Equipment: Truck 9700D
Shear Vane:
Bore Diameter (mm):

Inclination: -90
Comments:

Logged:	DW
Processed:	DW
Checked:	

Depth (m)/ [Elev.]	Drilling Method	Core Run / Recovery (%)	Support / Casing (m)	Water	Geological Fm	Classification	Graphic Log	SOIL DESCRIPTION: (Soil Code), Soil Name [minor MAJOR], colour, structure [zoning, defects, cementing], plasticity or grain size, secondary components, structure. (Geological Formation) / ROCK DESCRIPTION: Weathering, colour, fabric, ROCK NAME (Formation Name)	Moisture Condition	Consistency/ Relative Density	Weathering	Estimated Rock Strength EW VW W MS S VS ES	RQD (%)	Defect Spacing (mm) 20 60 200 600 2000	TESTS & SAMPLES / ROCK MASS DEFECTS: Depth, Type, Inclinations, Roughness, Texture, Aperture, Coating
	Dual Tube	100 67	None		Springston Formation	GP		Sandy medium to coarse GRAVELwith trace silt; grey. Medium dense; wet; poorly graded, angular to subrounded, greywacke; sand, fine to coarse.	W	MD					SPT 3,4, 5,7, 6,5, [23]
11.5 [-3.5]								CORELOSS *inferred* Sandy GRAVEL							
12.0 [-4.0]		68				GP		Sandy medium to coarse GRAVELwith trace silt; grey. Medium dense; wet; poorly graded, angular to subrounded, greywacke; sand, fine to coarse.	W	MD					SPT 3,4, 4,5, 5,5, [19]
12.9 [-4.9]						ML		SILT with some organic material; grey. Stiff; wet; low plasticity.		St					
13.0 [-5.0]								CORELOSS							
13.5 [-5.5]		100				SP		Fine SAND with some silt; grey. Medium dense; wet; poorly graded.	W	MD					SPT 3,4, 6,6, 6,7, [25]
15.0 [-7.0]		100				ML		SILT; grey. Stiff; wet; low plasticity.		S					SPT 2,1, 1,1, 1,1, [4]
15.8 [-7.8]						SP		Fine SAND with some silt; grey. Loose; saturated; well graded. @15.87m shell fragments	S	L					
16.5 [-8.5]		100				ML		SILT; grey. Stiff; wet; low plasticity.	S	S					SPT 1,1, 1,2, 1,2, [6]
17.4 [-9.4]						SW		Fine to medium SAND with some silt; grey. Medium dense; saturated; well graded.	S	'MD'					
18.0 [-10.0]		100				SP		Fine SAND with some silt; grey. Medium dense; saturated; poorly graded.	S	MD					SPT 1,2, 2,6, 7,8, [23]
19.5 [-11.5]								Termination Depth = 19.5m, Target Depth							SPT 1,2, 3,5, 7,7, [22]

GHD Limited

PO Box 13468
Christchurch 8141

BOREHOLE LOG

Site Identification: **BH01**

Sheet 3 of 2

Project: Christchurch Botanic Gardens
Client: Christchurch City Council
Site: Botanic Gardens
Job No.: 513059691

Coordinates: **E** 2479 508, **N** 5741 909
Surface RL (m): +8.0m
Commenced: 08-Oct-12
Completed: 10-Oct-12

Datum: NZMG
Total Depth: 19.5m
Contractor: McMillan Specialist Driling
Driller: McMillan

Equipment: Truck 9700D
Shear Vane:
Bore Diameter (mm):

Inclination: -90
Comments:

Logged:	DW
Processed:	DW
Checked:	

Depth (m)/ [Elev.]	Drilling Method	Core Run / Recovery (%)	Support / Casing (m)	Water	Geological Fm	Classification	Graphic Log	SOIL DESCRIPTION: (Soil Code), Soil Name [minor MAJOR], colour, structure [zoning, defects, cementing], plasticity or grain size, secondary components, structure. (Geological Formation) / ROCK DESCRIPTION: Weathering, colour, fabric, ROCK NAME (Formation Name)	Moisture Condition	Consistency/ Relative Density	Weathering	Estimated Rock Strength EW VW W MS S VS ES	RQD (%)	Defect Spacing (mm) 20 60 200 600 2000	TESTS & SAMPLES / ROCK MASS DEFECTS: Depth, Type, Inclinations, Roughness, Texture, Aperture, Coating
21															21
22															22
23															23
24															24
25															25
26															26
27															27
28															28
29															29
30															30

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PO Box 13468
Christchurch 8141

BOREHOLE LOG

Site Identification: **BH02**

Sheet 1 of 2

Project: Christchurch Botanic Gardens
Client: Christchurch City Council
Site: Botanic Gardens
Job No.: 513059691

Coordinates: **E** 2479 326, **N** 5742 005
Surface RL (m): +8.0m
Commenced: 10-Oct-12
Completed: 11-Oct-12

Datum: NZMG
Total Depth: 19.5m
Contractor: McMillan Specialist Driling
Driller: McMillan

Equipment: Truck 9700D
Shear Vane:
Bore Diameter (mm):

Inclination: -90
Comments:

Logged:	JS
Processed:	JS
Checked:	

Drilling Method: Dual Tube
Support / Casing (m): None
Geological Fm: Springston Formation

Depth (m)/ [Elev.]	Classification	SOIL DESCRIPTION: (Soil Code), Soil Name [minor MAJOR], colour, structure [zoning, defects, cementing], plasticity or grain size, secondary components, structure. (Geological Formation) / ROCK DESCRIPTION: Weathering, colour, fabric, ROCK NAME (Formation Name)	Moisture Condition	Consistency/ Relative Density
0.0	GP	Sandy fine GRAVEL; brown. Dry; poorly graded, angular greywacke gravel; sand, fine to coarse.	D	
0.3 [+7.7]	ML	SILT with trace gravel; brown. Very stiff; dry; low plasticity; gravel, fine, angular greywacke.	D	VSt
0.6 [+7.4]	ML	SILT with some sand and trace gravel; brown. Stiff; moist; low plasticity; sand, fine; gravel, fine, subrounded greywacke.	M	'St'
0.7 [+7.3]	GP	Sandy fine to medium GRAVEL; brown. Medium dense; dry; poorly graded; subrounded greywacke; sand, fine to medium.	D	MD
2.0 [+6.0]	GP	Sandy fine to medium GRAVEL; grey. Medium dense; wet; poorly graded; subrounded greywacke; sand, fine to coarse.	W	MD
3.7 [+4.3]	GP	Sandy fine to medium GRAVEL; grey. Medium dense; saturated; poorly graded; subrounded greywacke; sand, fine to coarse.	S	'MD'
4.1 [+3.9]	SP	Fine SAND; grey. Medium dense; moist; uniformly graded.	M	'MD'
4.3 [+3.7]	GC	Sandy fine to medium GRAVEL; grey. Dense; moist; poorly graded; subrounded greywacke; sand, fine to medium.	M	'D'
4.5 [+3.5]	SP	Fine to medium SAND; grey. Dense; moist; poorly graded.	M	D
5.0 [+3.0]	GP	Sandy fine to coarse GRAVEL; grey. Dense; wet; well graded; subrounded greywacke; sand fine to medium.	W	D
7.5 [+0.5]		Core loss		
8.0 [+0.0]	GP	Gravelly fine to medium SAND; grey. Medium dense; moist; poorly graded; gravel, fine to medium, subrounded greywacke.	M	MD
8.5 [-0.5]	SP	Fine to medium SAND; grey. Medium dense; moist; poorly graded.	M	'MD'
8.7 [-0.7]	GP	Sandy fine to medium GRAVEL; grey. Medium dense; wet; poorly graded; subrounded greywacke; sand, fine to medium.	W	'MD'
8.8 [-0.8]	SP	Coarse SAND with wood; grey. Medium dense; wet; uniformly graded.	W	'MD'
9.0 [-1.0]	SW	Fine to coarse SAND; grey. Medium dense; saturated; well graded.	S	MD
9.4 [-1.4]	GP		S	'MD'
9.7 [-1.7]				
9.9				

Core Run / Recovery (%)
73
100
87
87
87
53
80

Weathering: EW VW W MS S VS ES — Estimated Rock Strength — RQD (%) — Defect Spacing (mm): 20 60 200 600 2000

TESTS & SAMPLES / ROCK MASS DEFECTS: Depth, Type, Inclinations, Roughness, Texture, Aperture, Coating

SPT	5,5, 4,3, 4,5, [16]
SPT	6,7, 6,7, 8,7, [28]
SPT	6,7, 8,8, 8,9, [33]
SPT	5,6, 6,7, 9,11, [33]
SPT	3,2, 4,7, 6,4, [21]
SPT	3,5, 6,3, 3,5, [17]

BOREHOLE LOG

GHD Limited

PO Box 13468
Christchurch 8141

Site Identification: **BH02**

Sheet 2 of 2

Project: Christchurch Botanic Gardens
Client: Christchurch City Council
Site: Botanic Gardens
Job No.: 513059691

Coordinates: **E** 2479 326, **N** 5742 005
Surface RL (m): +8.0m
Commenced: 10-Oct-12
Completed: 11-Oct-12

Datum: NZMG
Total Depth: 19.5m
Contractor: McMillan Specialist Driling
Driller: McMillan

Equipment: Truck 9700D
Shear Vane:
Bore Diameter (mm):

Inclination: -90
Comments:

Logged: JS
Processed: JS
Checked:

Depth (m)/ [Elev.]	Classification	SOIL DESCRIPTION: (Soil Code), Soil Name [minor MAJOR], colour, structure [zoning, defects, cementing], plasticity or grain size, secondary components, structure. (Geological Formation) / ROCK DESCRIPTION: Weathering, colour, fabric, ROCK NAME (Formation Name)	Moisture Condition	Consistency/ Relative Density
[-1.9]	GP	Sandy fine to medium GRAVEL; grey. Very dense; saturated; poorly graded; subrounded greywacke; sand, fine to medium.	S	'MD'
10.0 [-2.0]	SP	Wooden Log (~200mm thick).	W	'MD'
10.2 [-2.2]				
10.5 [-2.5]	SP	Sandy fine to coarse GRAVEL; grey. Medium dense; saturated; well graded; subrounded greywacke; sand, fine to coarse. Fine to medium SAND; grey. Medium dense; wet; poorly graded. Core loss	M	VD
11.5 [-3.5]	ML	Fine to medium SAND with rare wood fragment; grey. Very dense; moist; poorly graded.	M	'S'
11.6 [-3.6]	SP	SILT; grey. Soft; moist; low plasticity. Fine SAND; grey. Medium dense; wet; uniformly graded.	W	MD
12.4 [-4.4]	SP	Silty fine SAND; grey. Medium dense; wet; poorly graded.	W	MD
13.0 [-5.0]		Core loss		
13.5 [-5.5]	ML	Sandy SILT; grey. Firm; wet; low plasticity.	W	F
14.0 [-6.0]	SP	Silty fine SAND with some shell fragments; grey. Medium dense; wet; poorly graded.	W	'MD'
14.5 [-6.5]	ML	Sandy SILT; grey. Firm; wet; low plasticity.	W	'F'
15.0 [-7.0]	SP	Silty fine SAND; grey. Medium dense; wet; poorly graded.	W	MD
15.4 [-7.4]	ML	SILT with some sand; grey. Firm; wet; low plasticity; sand, fine.	W	F
17.0 [-9.0]	ML	Sandy SILT; grey. Firm; wet; low plasticity.	W	F
17.2 [-9.2]	ML	SILT with some fine sand; grey. Firm; wet; low plasticity.	W	F
17.9 [-9.9]	SP	Fine SAND; grey. Medium dense; moist; uniformly graded.	M	MD
18.1 [-10.1]	ML	Sandy SILT; grey. Stiff to very stiff; wet; low plasticity.	W	St
18.6 [-10.6]	ML	SILT with some sand; greenish grey. Firm; moist; low plasticity.	M	'F'
19.0 [-11.0]	ML	Clayey SILT; green. Very Stiff; dry to moist; low plasticity.	M	'VSt'
19.5 [-11.5]		Termination Depth = 19.5m, Target Depth		

Drilling Method	Core Run / Recovery (%)	Support / Casing (m)	Geological Fm
	80	None	
Dual Tube	100	None	Christchurch Formation
	67		
	100		
	100		
	100		
	100		

Water

Weathering

Estimated Rock Strength: EW VW W MS S VS ES

RQD (%)

Defect Spacing (mm): 20 60 200 600 2000

TESTS & SAMPLES / ROCK MASS DEFECTS: Depth, Type, Inclinations, Roughness, Texture, Aperture, Coating	
SPT	11,11, 13,15, 16,18, [>50]
SPT	7,9, 8,8, 6,5, [27]
SPT	1,1, 1,8, 10,10, [29]
SPT	2,3, 5,8, 7,6, [26]
SPT	1,0, 2,1, 2,2, [7]
SPT	0,0, 1,4, 6,8, [19]
SPT	2,2, 2,6, 6,5, [19]

Appendix F

Basis of Design

Basis of Design

General

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

Codes, Standards and Design manual

New Zealand Standard

- NZS 1170.0:2002 Structural Design Actions Part 0: General Principles
- NZS 1170.1:2002 Structural Design Actions Part 1: Permanent, Imposed and Other Actions
- NZS 1170.5:2004 Structural Design Actions Part 5: Earthquake Actions – New Zealand and the NZBC Clause B1 Structure
- NZS 3404:1997 Steel Structures Standard
- New Zealand Society of Earthquake Engineering Guidelines for Assessment and Improvement of the Structural Performance of Buildings in Earthquakes
- New Zealand Society for Earthquake Engineering Guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Performance

Materials

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

Seel (f_y):	275 MPa (assumed)
Concrete (f'm)	12 MPa (assumed)

Assessment Load Criteria

Basic Assessment Information:

Properties of the structure that were used in the structural assessment are:

Height of building:	3.66 m
Dimensions of walls and structural members	Variable
Site Location:	7 Rolleston Avenue, Christchurch, New Zealand
Importance level:	2 (Workplace)

Dead Loads

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

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

Steel, galvanised standard corrugated sheeting (1mm thick)	0.09 kN/m^2
Timber	4.6 kN/m^3
Concrete masonry walls	11.05 kN/m^3
Steel	77.01kN/m^3

Live Loads

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

Roof Live Load	0.25 kN/m^2

Snow Load

Snow Load is not considered in the analysis.

Wind Load

Wind loading is not considered in the analysis.

Seismic Load

Earthquake loads shall be calculated using New Zealand Code.

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

Elastic Site Hazard Spectrum	0.9g

Building Mass

Total weight of building	868 kN
Horizontal base shear	546 kN
Orthogonal horizontal base shear (30%)	163.7 kN
Fundamental period of the building	0.113 seconds
Design Action Coefficient for ULS	0.694g

Site Description

The site is located within The Botanic Gardens located in Christchurch Central.

GHD

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