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Denton Park Pavilion

PRK 0770 BLDG 005 EQ2

Detailed Engineering Evaluation

Quantitative Report

Version FINAL

Denton Park, 446 Main South Road
Hornby, Christchurch

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

Denton Park Pavilion
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Detailed Engineering Evaluation
Quantitative Report
Version FINAL

Denton Park, Hornby, Christchurch

Christchurch City Council

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3/10/2013

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Denton Park Pavilion

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Detailed Engineering Evaluation

Quantitative Report - SUMMARY

Version FINAL (V2.0)

Hornby, Christchurch

Background

This is a summary of the Quantitative report for the Denton Park Pavilion building, and is based in general on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011 and inspections on the 11th of April 2012, 16th of May 2012 and the 22nd of February 2013.

Brief Description

The original date of construction of the Pavilion building at Denton Park is unknown. The building has undergone several alterations throughout its lifetime with changes in 1989 being the most recent. Partial plans were made available through Hornby Cricket Club. The details shown on the structural drawings were confirmed visually during an inspection of the property.

The structure is a two-storey building with a partial roof top balcony and is used as a pavilion and clubrooms. The ground-level pavilion is used by the Hornby Cricket Club as a changing/shower facility. The Hornby Cricket Club also occupies the first floor and balcony at roof level. It is understood that the original pavilion structure was a single-storey concrete masonry building which was altered with the addition of a club room first floor in approximately 1962. Further alterations were made to the building in 1989 when the first floor clubrooms were extended to the north; a concrete masonry stairs core was added to the east and a roof top balcony was added. At the time of the additions, steel framing was added to support the roof system and the overhanging clubrooms.

The roof of the building consists of corrugated steel cladding fixed to timber purlins running the length of the building. The purlins are supported by a combination of timber rafters and trusses. The rafters and trusses are supported by a main steel roof truss at the north end and by external timber framed walls to the south. A timber framed roof top balcony to the north of the building is supported by the timber framed external walls to the north and the steel roof truss to the south. The timber framed walls are lined internally with plasterboard and externally with weatherboards. The steel roof truss is supported by square hollow section (SHS) column at each end which are supported by a concrete pad foundation to the west and a concrete masonry wall to the east. The underside of the timber roof trusses and the timber framed balcony are lined with plasterboard. The first floor consists of 20mm plywood flooring supported by timber floor joists running across the building. The first floor framing is supported by a combination of the original unreinforced concrete masonry walls, the reinforced concrete masonry walls of the stair core and steel framing. The original support framing of the 1962 extension consists of a timber beam supported by seven circular hollow section (CHS) columns. The 1989 extension support framing consists of universal beam (UB) horizontal members supported by SHS columns. The UB beam has web stiffeners over the supporting columns. Foundations are concrete slab on grade of unknown thickness for the most part with pad footings under the columns.

Indicative Building Strength

Following a detailed assessment, the overall building has been assessed as achieving 43 **%**NBS. Under the New Zealand Society for Earthquake Engineering (NZSEE) guidelines the building is considered a moderate Earthquake Risk building.

Recommendations

The recent seismic activity in Christchurch has caused minor damage to the ground floor portion of the building, with cracking in concrete masonry walls the only damage noted along with minor cracking to plasterboard linings of the first floor structure.

The building has been assessed as being a moderate Earthquake Risk building as it has achieved less than 67% NBS, but greater than 34%.

Strengthening

As the %NBS of the building has been assessed at 43% NBS, additional strengthening works are recommended to increase the seismic capacity of the building to achieve at least of 67% NBS to comply with NZSEE guidelines recommendation.

1. Background

GHD has been engaged by the Christchurch City Council (CCC) to undertake a Detailed Engineering Evaluation of the Denton Park Pavilion.

This report is a Quantitative Assessment and is based in general on NZS 1170.5: 2004, NZS 4230: 2004, NZS 3604:2011, NZS 3404: 2009, the New Zealand Society for Earthquake Engineering (NZSEE) guidelines for the Assessment and Improvement of Unreinforced Concrete Masonry Buildings for Earthquake Resistance (02/2011) and the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes (06/2006). Furthermore, the recent publication of “General Procedure for Seismic Assessment of Out-of-plane Loaded URM Wall” issued by Auckland University has been used in the analysis.

The Quantitative Assessment to the building comprises an investigation on in-plane and out-of-plane strength of the masonry block walls, bracing capacity of the upper timber framed walls and the combined axial and bending capacity of the steel members. 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 elements to give the percentage of New Building Standard (%NBS) of each of the structural elements.

Electromagnetic scans have been carried out on site to ascertain the extent of the reinforcement in the masonry walls.

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 original date of construction of the Pavilion at Denton Park is unknown. The building has undergone several alterations throughout its lifetime with changes in 1989 being the most recent. Partial plans were made available through the Hornby Cricket Club. The details shown on the structural drawings were confirmed visually during the inspection of the property. The site is flat with no variation in topography near the building. The site is located at the edge of Denton Park, which contains large amounts of flat land to the north and east of the building, and is closely bordered to the south and west by residential structures. Main South Road runs roughly 40m to the south of the building.

The structure is a two-storey building with a partial third-storey deck, which is vertically irregular but roughly rectangular in plan. The ground-level pavilion is used by the Hornby Cricket Club as a changing facility. The Hornby Cricket Club also occupies the first floor and balcony at roof level. It is understood that the original pavilion structure was a single-storey concrete masonry building which underwent the addition of a first floor club room in approximately 1962. Further alterations were made to the building in 1989 when the first floor clubrooms were extended to the north, a concrete masonry stairs core was added to the east and a roof top balcony was added. At the time of the additions, steel framing was added to support the roof system and the increased clubroom’s size.

The roof of the building consists of corrugated steel cladding fixed to timber purlins running the length of the building. The purlins are supported by a combination of timber rafters and trusses. The rafters and trusses are supported by a main steel roof truss at the north end and by the external timber framed walls to the south. A timber framed roof top balcony to the north of the building is supported by the timber framed external walls to the north and the steel roof truss to the south. The timber framed walls are lined internally with plasterboard and externally with weatherboards. The steel roof truss is supported by square hollow section (SHS) column at each end which are supported by a concrete pad foundation to the west and a concrete masonry wall to the east. The underside of the roof trusses and the timber framed balcony are lined with plasterboard. The first floor consists of 20mm plywood flooring supported by timber floor joists running across the building. The first floor framing is supported by a combination of the original unreinforced concrete masonry walls; the reinforced concrete masonry walls of the stairs core and steel framing. The original support framing of the 1962 extension consists of a timber beam supported by seven circular hollow section (CHS) columns. The 1989 extension support framing consists of universal beam (UB) horizontal members supported by SHS columns. The UB beam has web stiffeners over the supporting columns. Foundations are concrete slab on grade of unknown thickness for the most part with pad footings under the columns.

Internal stairs are made up of steel stringers and a steel framed half landing with timber treads. The external stairs to the west of the building has galvanised steel treads and stringers.

The ground floor plan in Figure 2 illustrates the main structural members of the building.

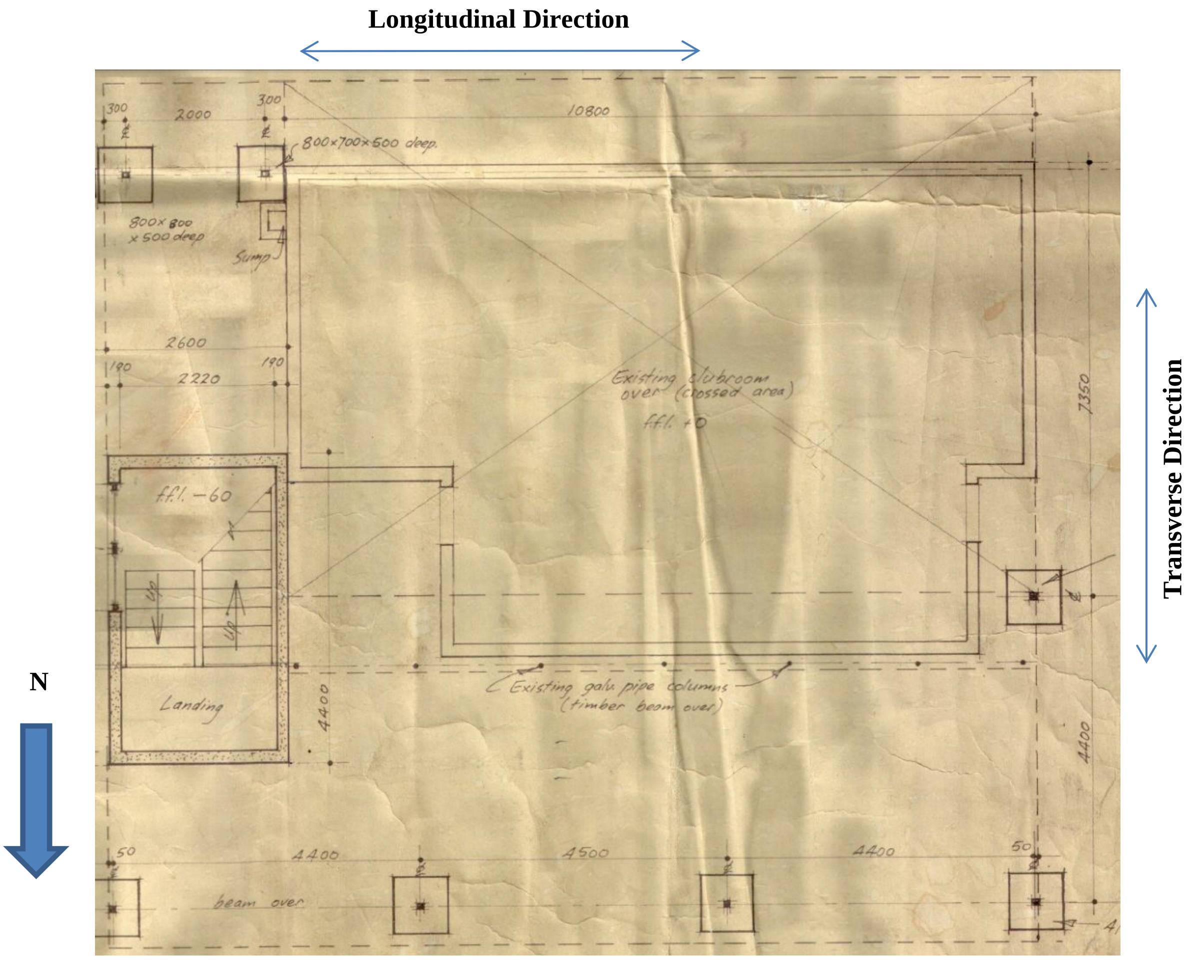


Figure 2 Plan of ground floor of the building showing key structural elements.

4.2 Gravity Load Resisting System

Gravity loads on the roof are carried through the steel roof cladding to the timber purlins and rafters, and out to the external timber framed walls and internal steel truss. Loads acting on the balcony are also transferred through the timber floor joists to the external walls and the internal steel truss. Gravity loads from the roof deck are transferred down through the external walls and onto the cantilevered first floor joists. The gravity loads are then transferred through these joists via shear back to the steel UB beam at the north side of the building, and the steel UB beam and unreinforced concrete masonry bearing wall at the southern end of the building. Gravity loads to the internal steel truss are transferred through the top and bottom chords back to the supporting SHS columns. Gravity loads these members are then transferred down through the columns to the foundations to the west and to the supporting reinforced concrete masonry wall to the east. Loads acting on the UBs and the masonry walls are transferred down through the wall or through the steel SHS support posts, into the concrete foundations, and into the ground via bearing on the founding soils beneath. Gravity loads acting on the first floor are similarly transferred down to the ground below.

4.3 Lateral Load Resisting System

In the longitudinal direction, lateral loads are transferred through the roof cladding and purlins to the rafters. The loads are then transferred through the rafters to the supporting steel truss and the supporting timber framed walls. The lateral loads acting on the steel roof truss are transferred through the top and bottom chords to the supporting SHS columns. The SHS columns provide resistance to the lateral loads through moment connections at the base. Lateral loads acting on the external timber framed walls will be resisted, to some extent, by the plasterboard internal linings. Nominal diaphragm action will be provided at the first floor ceiling level by the plasterboard linings fixed to the underside of the roof trusses and balcony floor joists. Further diaphragm action will be provided at the first floor level by the plywood timber flooring fixed to the top of the floor joists. Lateral loads acting on the external walls will be transferred through the floor diaphragm action to the concrete masonry walls and steel framing below. Lateral loads apportioned to the steel framing will be resisted by the moment connections of the steel framing and the fixed base foundations. Lateral loads apportioned to the concrete masonry walls will be resisted by the in-plane shear strength of the mortar of the original pre-1962 unreinforced masonry walls and through a combination of the steel reinforcing bars and the in plane shear strength of the mortar of the masonry walls in the 1989 stairs core.

In the transverse direction, the roof cladding and framing will transfer loads back to the supporting timber external framed walls and the steel truss framing. Internal plasterboard lined timber framed walls running in the transverse direction at the south end of the building will transfer lateral loads down to the floor diaphragm below. The steel truss will also transfer lateral loads from the roof framing and balcony floor framing down to the fixed base connections below. Transverse lateral loads transferred through the floor diaphragm action will be resisted similarly to those acting in the longitudinal direction.

5. Damage Assessment

5.1 Surrounding Buildings

Denton Park Pavilion is located adjacent to residential properties, a sports pitch and a car park. There are no buildings that are adjoining the pavilion building. During the inspection of the pavilion there was no apparent damage to the surrounding buildings.

5.2 Residual Displacements and General Observations

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

Minor cracking was noted to the plasterboard wall linings of the clubrooms above and below windows. Cracking was also noted along construction joints of an internal masonry partition wall within the ground floor changing rooms. These cracks are believed to have occurred because the partition wall is not an original wall and has not been adequately tied into the original masonry walls. No damage was noted to the stairs and these are not considered to be a critical structural weakness within the building. The strength of the building has not been affected by the damage.

5.3 Ground Damage

No ground damage was observed during our inspection of the site.

6. Geotechnical Consideration

This geotechnical study outlines the ground conditions, as indicated from sources quoted within.

This report is only specific to the Pavilion at Detailed Engineering Evaluations. The park is located between the Main South Railway line to the north and Main South Road (SH1) to the south. It is bound to the east by commercial properties and west by residential properties. The ground floor of the property is owned and maintained by the Christchurch City Council.

6.1 Site Description

The site is situated within a recreational area, within the suburb of Hornby in western Christchurch. It is relatively flat at approximately 30m above mean sea level. It is approximately 2.5km west of the Heathcote River, and 15km west of the coast (Pegasus Bay) at New Brighton.

6.2 Published Information on Ground Conditions

6.2.1 Published Geology

The geological map of the area[1] indicates that the site is close to the boundary of two different units of the Holocene alluvial soils of the Yaldhurst Member, sub-group of the Springston Formation, comprising:

- Alluvial gravel, sand, and silt of historic river flood channels*;* and*,*
- Dominantly alluvial sand, and silt overbank deposits*.*

6.2.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that six boreholes are located within a 200m radius of the site (see Table 2). Of these boreholes, two have lithographical logs that extend beyond 2.5m. The lithology described in the logs indicate the area is predominantly underlain by layers of sand and gravel to depths of approximately 42m bgl where a layer of clay and peat is recorded, below this unit more sand and gravel are indicated with varying proportions of clay.

Table 2 ECan Borehole Summary

Bore Name	Log Depth	Groundwater	Distance & Direction from Site
M35/14764	~1.9m	N/A	~80m SE
M35/12767	~2.4m	N/A	~65m S
M35/12766	~2.4m	N/A	~65m S
M35/3732	~0m	N/A	~195m S
M35/3546	~95.8m	~12.7m bgl	~185m SE

[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.

Bore Name	Log Depth	Groundwater	Distance & Direction from Site
M35/1865	~102m	~14.7m bgl	~200m NE

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

6.2.3 EQC Geotechnical Investigations

The Earthquake Commission has not undertaken geotechnical testing in the area of the subject site.

6.2.4 Additional Investigations

In order to get better understanding of soil conditions, one CPT was undertaken on 12 April 2012 by McMillan Drilling Services, which reach refusal in gravel at 2.0m depth.

The testing results are summarised in Table 3 below:

Table 3 Summary of CPT undertaken on 12 April 2012-Inferred Lithology

Bore Name	Grid Reference	Depth (m)	Lithology
CPT-04	-43.542369 mE	0 – 2.0	Surface Soil
	172.520556 mN	>2.0	Gravel

6.2.5 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 itself is a non-residential properties in urban area of Christchurch, therefore it has not been given a CERA land zoning technical category. However, a property to the west of the subject building has been given a technical category of TC1 (grey) – CERA indicates that this means that future land damage from liquefaction is unlikely and the standard foundations for concrete slabs or timber floors can be used.

6.2.6 Post February Aerial Photography

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

Figure 3 Post February 2011 Earthquake Aerial Photography [2]

6.2.7 Summary of Ground Conditions

From the information presented above, the ground conditions underlying the site are anticipated to comprise multiple strata of sand and gravel, with some peat and clay at depth.

6.3 Seismicity

6.3.1 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.

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

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

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	120 km	NW	~8.3	~300 years
Greendale (2010) Fault	13 km	W	7.1	~15,000 years
Hope Fault	100 km	N	7.2~7.5	120~200 years
Kelly Fault	100 km	NW	7.2	~150 years
Porters Pass Fault	54 km	NW	7.0	~1100 years

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

6.3.2 Ground Shaking Hazard

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

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

In addition, anticipation of Holocene alluvial soils comprising alluvial gravel and sand, a 475-year PGA (peak ground acceleration) of ~0.4 (Stirling et al, 2002[4]), and bedrock anticipated to be in excess of 500m deep, ground shaking is likely to be moderate to high.

6.4 Slope Failure and/or Rock-fall Potential

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

6.5 Liquefaction Potential

A soil class of **D** (in accordance with NZS 1170.5:2004) should be adopted for the site.

Due to the anticipated presence of predominantly gravels beneath the site, in addition to the aerial photo evidence of no liquefaction presented above, it is considered that liquefaction is unlikely to occur at this site compared to other areas of Christchurch.

[3] 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.

[4] GNS Active Faults Database

6.6 Recommendations

Additional geotechnical investigations are not considered necessary at this site.

6.7 Conclusions & Summary

This assessment is based on a review of the geology and existing ground investigation information, and observations from the Christchurch earthquakes since 4 September 2010, along with the results of the CPT site test undertaken on 12 April 2012.

The site appears to be situated on predominately gravel alluvial deposits. Associated with this (and based on the above information) the site is expected to have negligible liquefaction potential.

A soil class of **D** (in accordance with NZS 1170.5:2004) should be adopted for the site.

7. Survey

No level or verticality surveys have been undertaken for this building at this stage.

The Hilti PS 200 Ferroscan was used to determine the position, depth and diameter of the reinforcement in the structure. This scanning equipment using electro-magnetic fields allowed for the determination of the capacity of the various reinforcement elements of the building. In the case of conflicting results, the most conservative bar diameter was chosen for the capacity calculations.

8. Assessment Methodology

8.1 Quantitative Assessment

The quantitative assessment to the building comprised of an investigation to the in-plane and out-of-plane strength of the masonry block walls, assessment of the capacity of the structural steel members and a bracing capacity check of the timber framed upper floors of the building. The investigation was based on the analysis of the seismic loads that the structure is expected to be subjected to in a ULS seismic event, 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 them to give the %NBS of each structural elements.

8.2 Timber Framed Walls

The buildings bracing capacity was calculated in accordance with NZS 3604:2011 and the NZSEE guidelines. The demand for the building was calculated in accordance with NZS 3604: 2011 and the percentage of New Building Standard (%NBS) was assessed.

8.2.1 Demand

The demand on the structure was determined in accordance with Section 5 of NZS 3604: 2011. The bracing unit demand per square metre was determined from Table 5.10. The building is located in Christchurch (zone 2) on class D soils. Therefore a multiplication factor of 0.8 is applied in accordance with Table 5.10 of NZS 3604: 2011.

An Importance Level of 2 was used for the calculations. This results in the Return Period Factor, as given by Table 3.5 of NZS 1170.5: 2004 and as prescribed by Table 3.3 of NZS 1170.1: 2004, for the building as 1.0 and therefore no increase or decrease to the demand is necessary.

8.2.2 Wall Bracing Capacity

The original date of construction of the building is unknown and as such, no bracing capacities for the wall linings were available for the calculations. Therefore the capacities are taken in accordance with Table 11.1 of the in NZSEE guidelines Table 11.1.

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

$$\text{Aspect ratio factor} = \frac{\text{2x Width}}{\text{Height}}$$

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

8.2.3 Ceiling Diaphragm

The fixing details of the ceilings could not be determined. Therefore where the ceiling dimensions exceed that specified in NZS 3640: 2011, the capacity is determined by;

$$\%NBS = \frac{\text{Permitted length}}{\text{Actual length}} \text{ x } 100\%$$

Where the permitted length is the maximum dimension for a standard plasterboard lined ceiling (e.g. 7.5m)

8.2.4 % NBS

The bracing capacity both along and across the building were then compared to their respective demands to asses which was the most critical and thus determine the overall %NBS for the building

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

8.3 Concrete Masonry Walls

8.3.1 Seismic Coefficient

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

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

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

$$S_P = 1.3 - 0.3\mu$$

Where µ, the structural ductility factor, was taken as 1.00 for the unreinforced masonry walls and 1.25 for the reinforced masonry walls.

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

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

8.3.2 In-Plane Capacity of the Unreinforced Walls

The in-plane capacity of the unreinforced concrete masonry walls 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 four 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).

8.3.3 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}.

$$\mathrm{V_n} = \min(\mathrm{V_{dt}}, \mathrm{V}_s, \mathrm{V_r}, \mathrm{V_{tc}})$$

8.3.4 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. Furthermore, the recent publication of “General Procedure for Seismic Assessment of Out-of-plane Loaded URM Wall” by Auckland University has been used in the analysis.

8.3.5 Shear capacity of the Reinforced Walls

The shear capacity of the reinforced filled masonry walls was determined using NZS 4230: 2004. As there are no details as to the level of supervision during the construction stage, the Observation Type was classed in accordance with Table 3.1.

8.3.6 Moment capacity of the Reinforced Walls

The moment capacity of the reinforced filled masonry walls was determined using NZS 4230: 2004 and the user’s guide to NZS 4230: 2004. The strength reduction factor, ϕ, for flexure with or without axial tension or compression was taken as 0.85 in accordance with Cl 3.4.7.

8.4 Steel Framing

8.4.1 Capacity of Steel Members

The strength reduction factor Φ for steel elements was taken from Table 3.3 (1) of NZS 3404:1997. The loads from the analyses were then checked against the structural member capacities derived from NZS 3404:1997 Steel Structures Standard. The following checks were completed:

- Determine if the element needs be checked for combined action (bending and axial force), Cl 8.1.4 (a) (ii)
- Section bending capacity of the steel member;
- Uniaxial bending capacity about the major principle x-axis, CL 8.3.2.1;

$$\phi\ \mathbf{M_{rx}} = \mathbf{M_{sx}}\left(\mathbf{1} - \frac{\boldsymbol{N^*}}{\phi\ \mathbf{N_s}}\right)\phi$$

- Uniaxial bending capacity about the minor principle y-axis, CL 8.3.3;

$$\phi\ \mathbf{M_{ry}} = \mathbf{M_{sy}}\left(\mathbf{1} - \frac{\boldsymbol{N^*}}{\phi\ \mathbf{N_s}}\right)\phi$$

- Biaxial bending capacity, CL 8.3.4;

$$\frac{\boldsymbol{N^*}}{\phi\ \mathbf{N_s}} + \frac{\boldsymbol{M_x^*}}{\phi\ \mathbf{M_{sx}}} + \frac{\boldsymbol{M_y^*}}{\phi\ \mathbf{M_{sy}}} \leq \mathbf{1.0}$$

- In-plane capacity, Cl 8.4.2.2.1;

$$\phi\ \mathbf{M_i} = \mathbf{M_s}\left(\mathbf{1} \pm \frac{\boldsymbol{N^*}}{\phi\ \mathbf{N_c}}\right)\phi$$

- Out-of-plane bending capacity, CL 8.4.4.1;

$$\phi\ \mathbf{M_{ox}} = \mathbf{M_{bx}}\left(\mathbf{1} - \frac{\boldsymbol{N^*}}{\phi\ \mathbf{N_{cy}}}\right)\phi$$

- Biaxial bending capacity, CL 8.4.5.1;

$$\left(\frac{\boldsymbol{M_x^*}}{\phi\ \mathbf{M_{cx}}}\right)^{\mathbf{1.4}} + \left(\frac{\boldsymbol{M_y^*}}{\phi\ \mathbf{M_{iy}}}\right)^{\mathbf{1.4}} \leq \mathbf{1.0}$$

- Shear capacity of the steel column and chord, Cl 5.11.4.1;

$$\phi\ V_v = 0.6 f_y\ A_w \phi$$

8.4.2 Capacity of Steel Welds

A fillet weld subject to a design force per unit length of weld shall 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

Φ = the strength reduction factor from Table 3.3 of NZS 3404: 1997

v_w = the nominal weld shear capacity as specified in Cl 9.7.1.10.3

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);

$$v_w = 0.6f_{uw}t_tk_r$$

Where

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

t_t = design throat thickness

k_r = reduction factor given in table 9.7.3.10 (2)

9. Initial Capacity Assessment

9.1 Timber Framed Wall

9.1.1 Seismic Loading Investigation

In accordance with Table 5.10 of NZS 3604: 2011, for a 2 storey building, light roof, light wall cladding, concrete slab floor with a pitch between 0°-25° then a bracing demand of 9 BU/m^2 is taken for the upper walls.

In accordance with Table 5.10 for Earthquake Zone 2 which covers Christchurch and for soil class D, the bracing demand is reduced by a factor of 0.8 and so the total demand for the building is;

$$\mathrm{BU_{demand}} = (0.8 \text{ x } 9 \ \mathrm{BU/m^2} \text{ x Floor area })$$

= 1238 BU

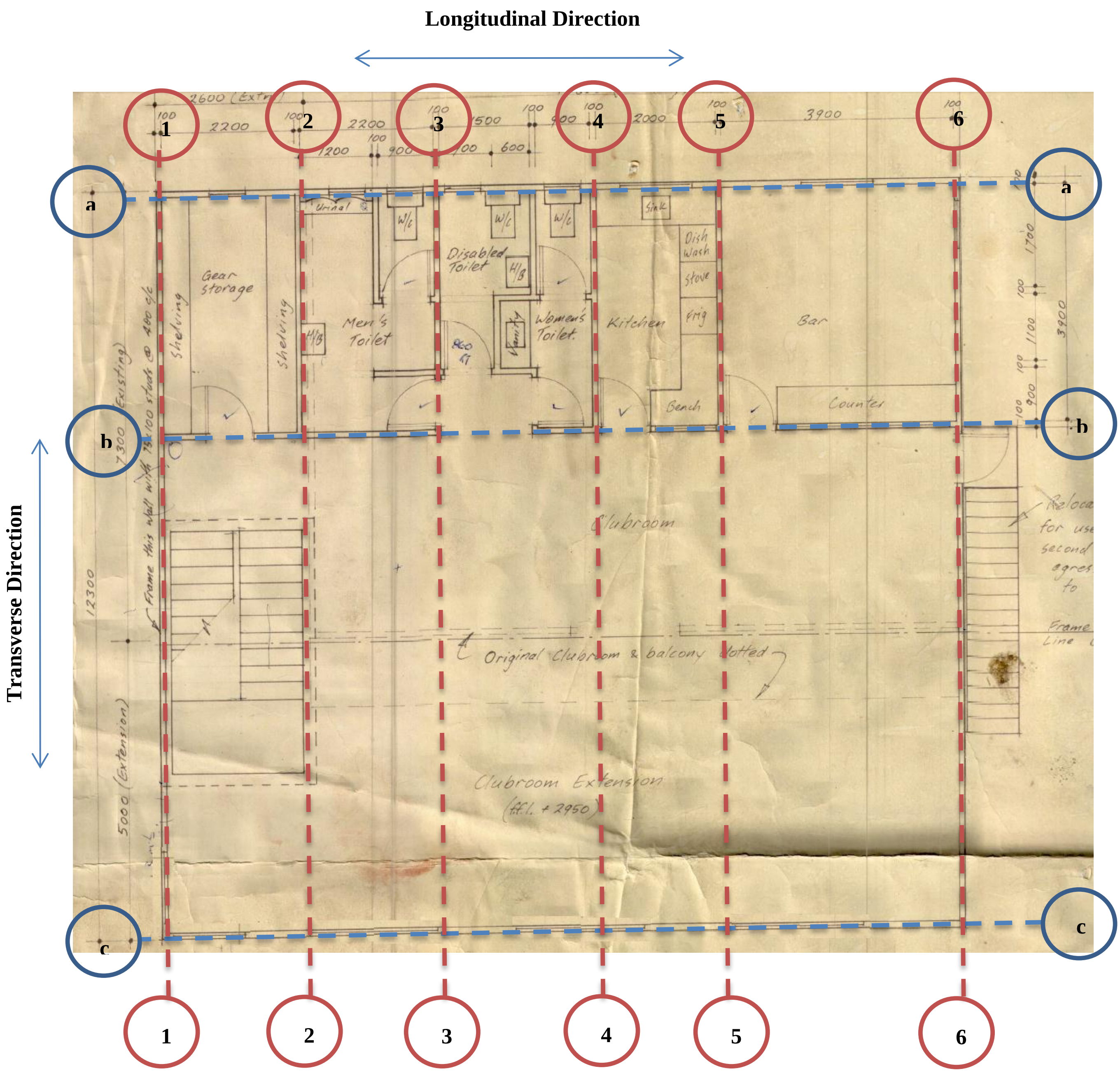
9.1.2 Wall Bracing Capacity – Upper Floor

Table 5 Upper Floor Wall Bracing Capacity – Longitudinal Direction

Bracing Line	Bracing Capacity (BU)
a	293
b	205
c	227
Total bracing capacity =	725 BU

Table 6 Upper Floor Wall Bracing Capacity – Transverse Direction

Bracing Line	Bracing Capacity (BU)
1	402
2	160
3	160
4	160
5	160
6	246
Total bracing capacity =	1287 BU



9.1.3 % NBS Assessment

Following detailed calculations being carried out, the buildings % NBS from the bracing calculations have been assessed across and along the building and the results are shown in Table 7.

Table 7 Upper Storey (Timber Framed Structure) %NBS

Direction	%NBS
Across	100
Along	59

Following a detailed assessment the timber framed portion of the building, the upper floor has been assessed as achieving 59% New Building Standard (NBS).

9.2 Concrete Masonry – Ground Floor

9.2.1 Wall Investigation

The position of the ground level concrete masonry walls are as indicated in the plans in Figure 4 below.

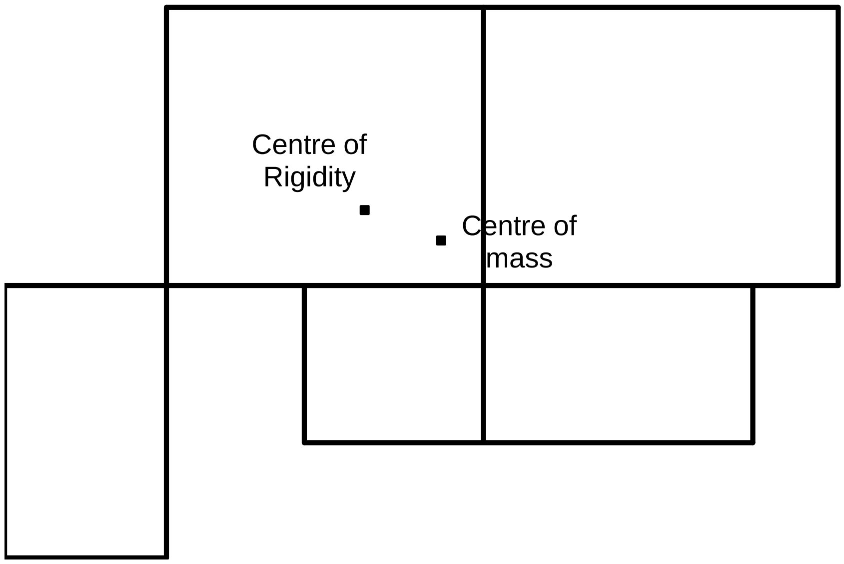


Figure 4 Plan details and wall locations of the ground floor concrete masonry walls

9.2.2 Concrete Masonry Capacity

The analysis results of unfilled unreinforced masonry walls is shown in Table 8.

Table 8 Unreinforced Concrete masonry % NBS

Bracing Direction	% NBS
Transverse Longitudinal	43 % NBS 56 % NBS
Out-of-plane Capacity	67 % NBS
Critical %NBS	***43 % NBS***

9.3 Steel Framing

A 2-D structural analysis using "Robot Structure Analysis Professional" engineering software was undertaken to model the steel structure framing for 100% NBS seismic loads. Loads were applied in both the longitudinal and the transverse directions of the building.

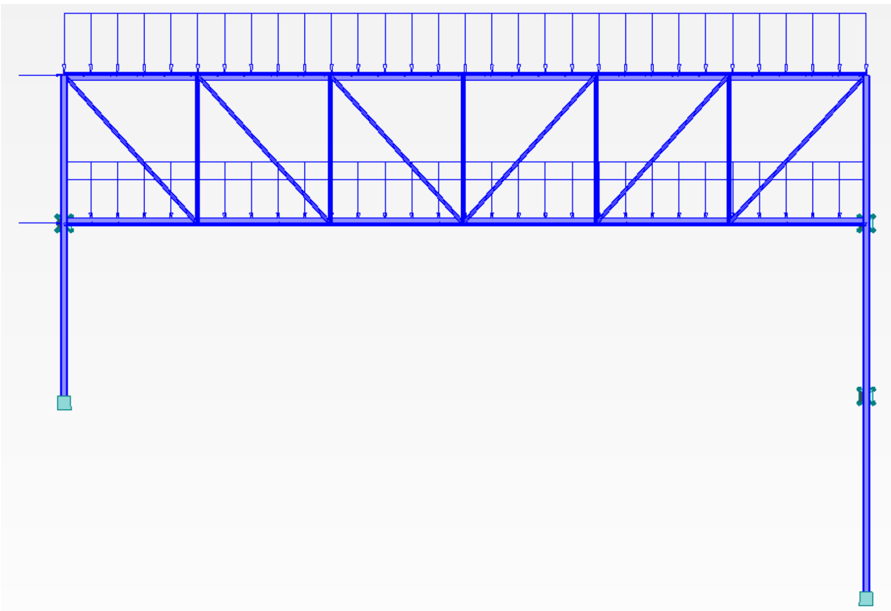


Figure 5 Steel roof truss and frame

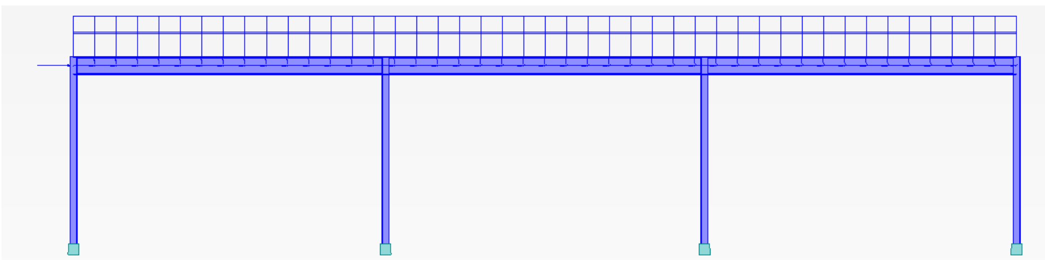


Figure 6 1989 steel support frame at the northern end of the building

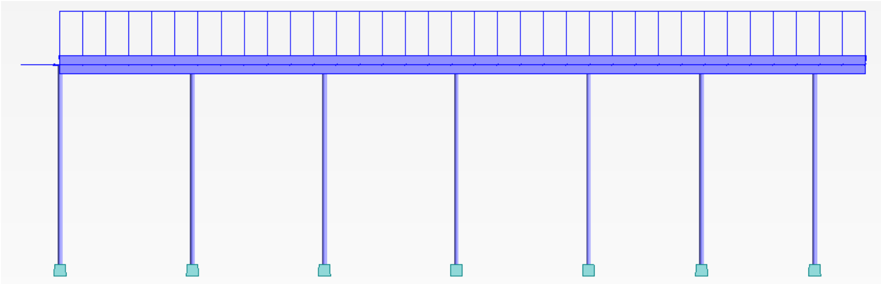


Figure 7 1960's timber and steel support frame in centre of the building

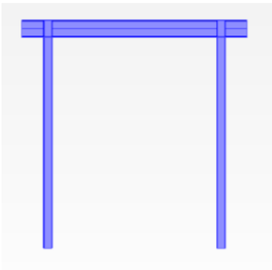


Figure 8 1989 steel support frame at the southern end of the building

9.3.1 % NBS Assessment

Each of the different structural sections used in the construction of the building were checked in accordance with NZS 3404: 1997. Procedures for structural checks are outlined in Section 8.4. The %NBS results for the critical members are shown below in Table 9.

Table 9 Steel member % NBS

Location	Member Use	Section Size/Type	Critical Member Number	Ratio (Combined Bending and Axial)	% NBS
First Floor	**Truss Top & Bottom Chords**	2 x 100 x 10 EA	43	2.24	45% NBS
	Truss Web Chords	50 x 50 x 5 SHS	48	0.25	100% NBS
	Truss Columns	100 x 100 x 9 SHS	24	2.12	47% NBS
Ground Floor	**Northern and Southern Beam**	UB 250 x 37.3	56	0.09	100% NBS
	Northern and Southern Columns	100 x 100 x 4.9 SHS	2	1.25	80% NBS
	Central Columns	60.3 x 4.9 CHS	76	1.35	74% NBS

9.4 Discussion of Results

The concrete masonry ground floor portion of the building was designed pre-1962 and likely designed for the loading standard current at the time, NZSS 95:1955 (or earlier). Other portions of the building were designed in the early 1960s and were likely designed to the same loading standard. The portion of the building designed in 1989 would have been designed to the loading standard current at the time, NZS 4230: 1984. The design loads used in these codes 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 as a whole has been assessed as achieving 43**%**NBS. Under the New Zealand Society for Earthquake Engineering (NZSEE) guidelines the building is considered to be a moderate Earthquake Risk building.

10. Recommendations

The recent seismic activity in Christchurch has caused minor damage to the ground floor portion of the building, with cracking in concrete masonry walls the only damage noted. The building has been assessed as being a moderate Earthquake Risk building as it has achieved 43% NBS. As such, GHD recommends that strengthening options be explored in order to increase the %NBS of the building ideally to 67% NBS as recommended by the NZSEE Guidelines.

11. Limitations

11.1 General

This report has been prepared subject to the following limitations:

- No access to the roof space was available and as such roofing details shown in plans were assumed to be accurate.
- 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 8 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 accepts no responsibility for any other party or person who relies on the information contained in this report.

11.2 Geotechnical Limitations

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

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

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

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

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

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

Appendix A

Photographs

Photograph 1 West elevation of the pavilion.

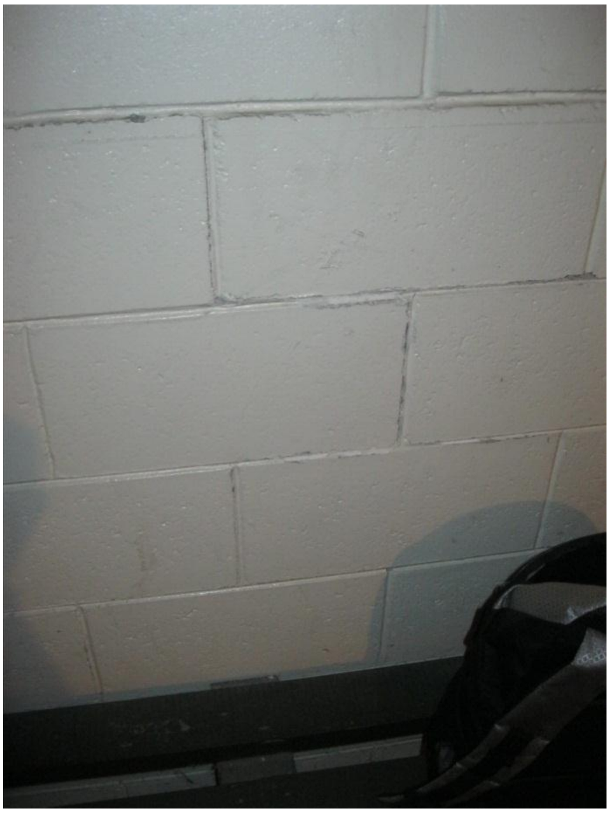
Photograph 2 North elevation of the pavilion.

Photograph 3 East face of pavilion.

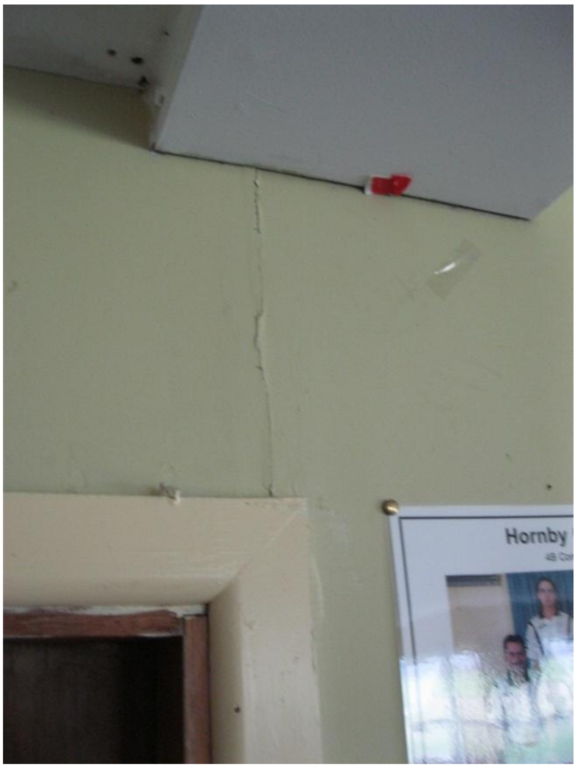
Photograph 4 Underside of the first floor framing supported by steel framing to the north and concrete masonry walls to the south.

Photograph 5 Steel framing with web stiffener above SHS column.

Photograph 6 Cracking between original concrete masonry wall (left) and later concrete masonry internal wall (right).



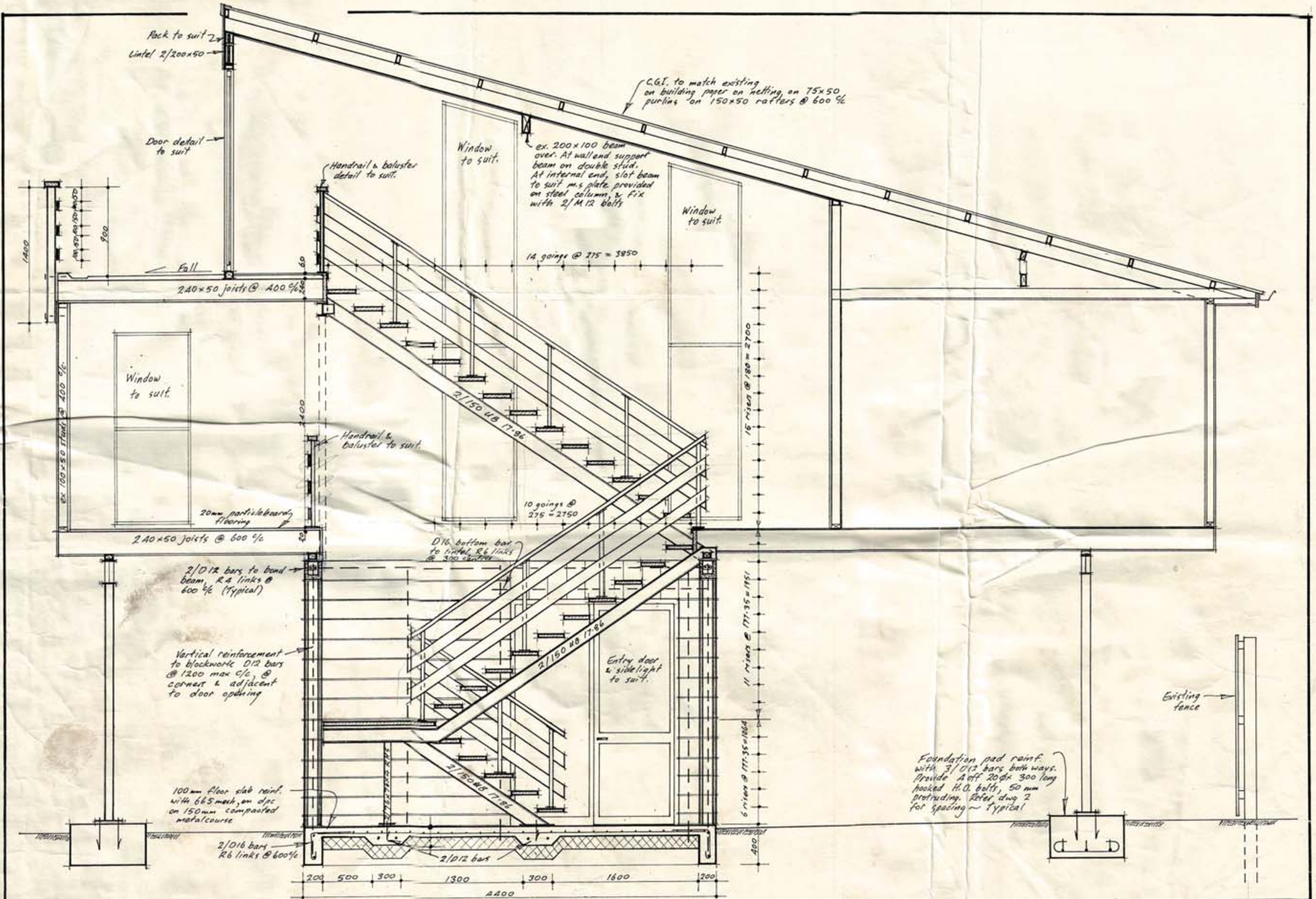
Photograph 7 Cracking along mortar line of concrete masonry walls.



Photograph 8 Example of typical plasterboard cracking above windows and doors of clubrooms

Appendix B

Existing Drawings

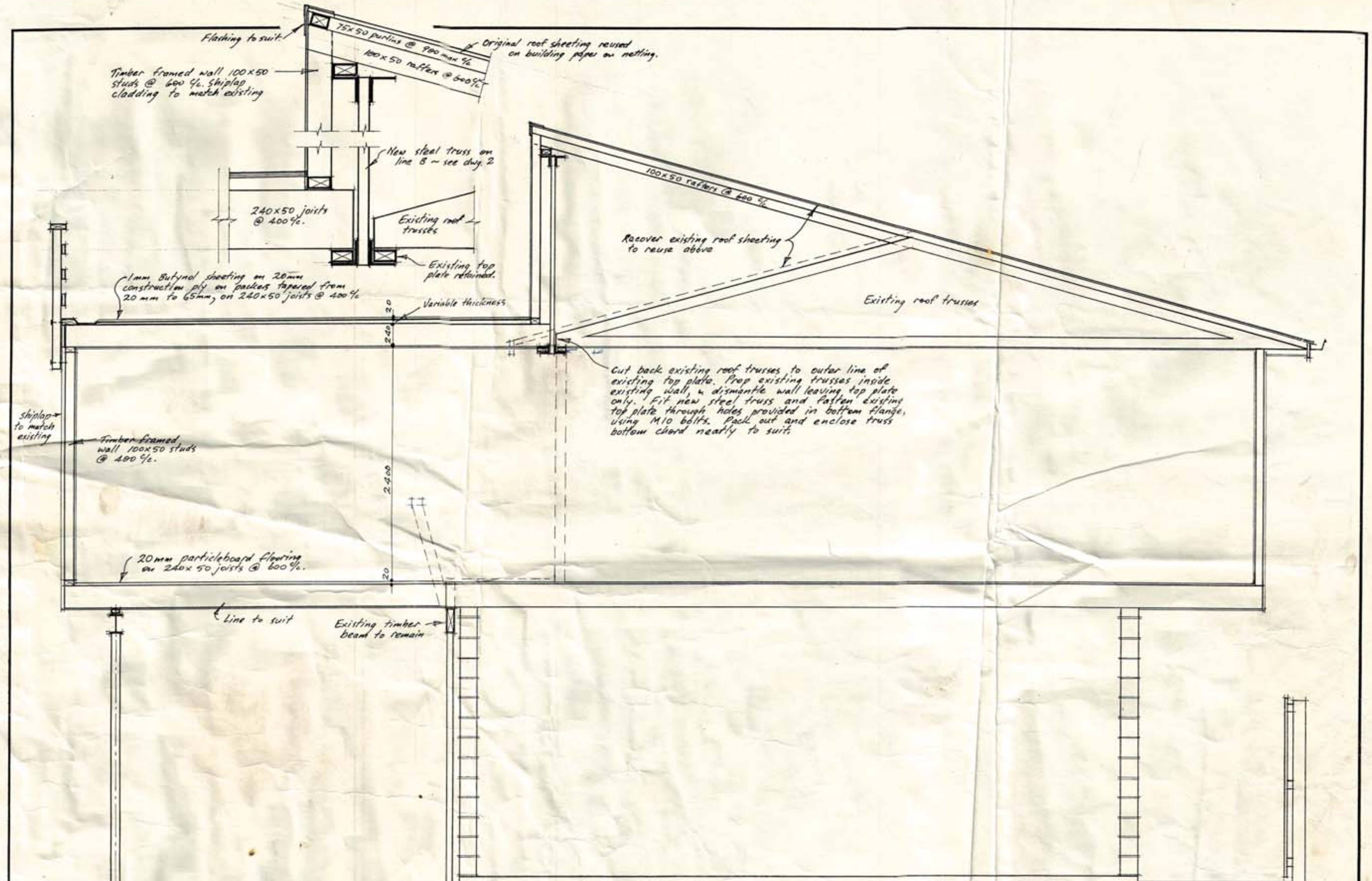


STAIRWELL SECTION (1:25)

HORNBY CRICKET CLUBROOM EXTENSIONS

DENTON PARK, HORNBY

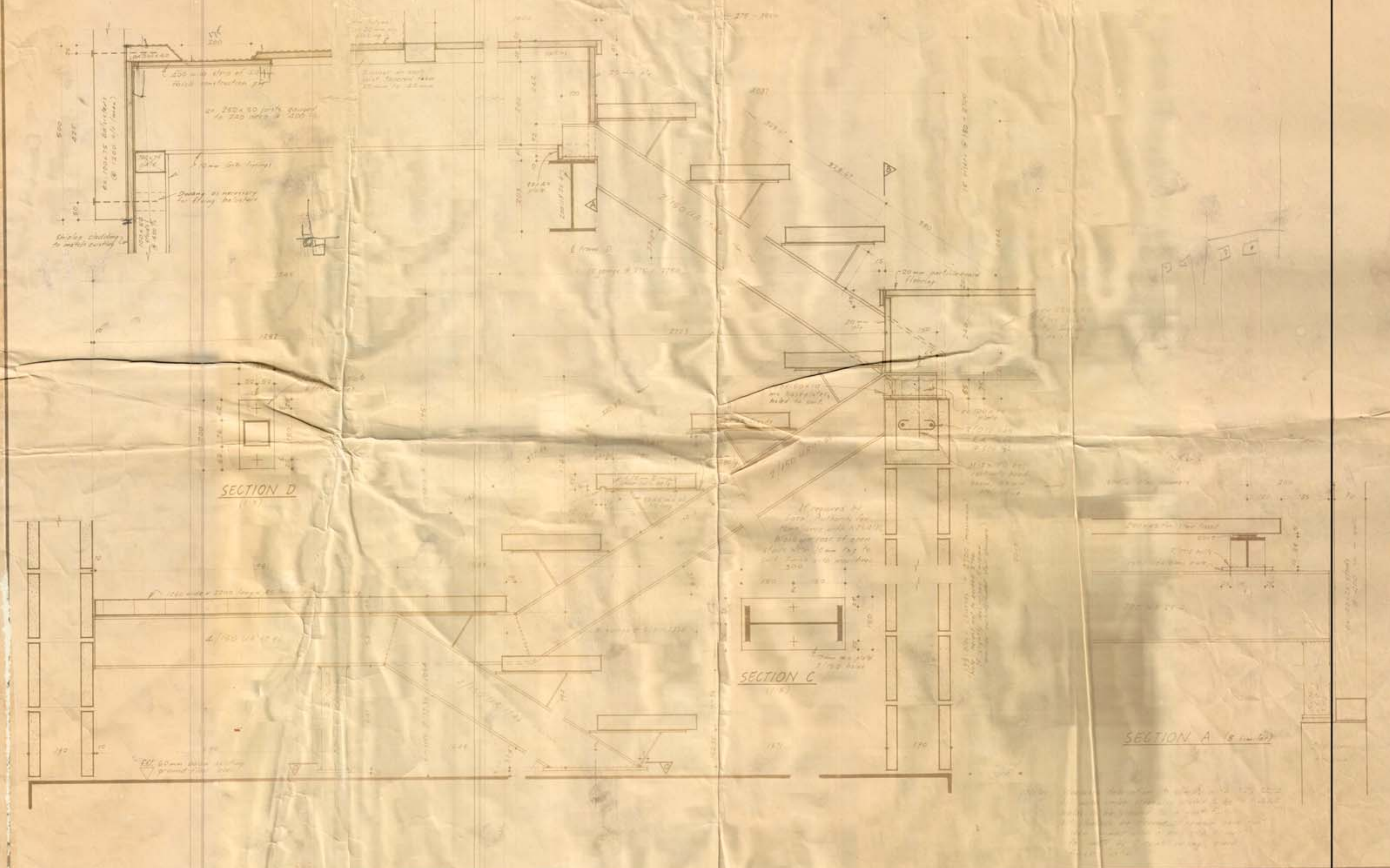
Drawn /M.	Checked	Scales	Sheet 4A
Traced	Date	As shown	Series of 5 shts Ref.



TYPICAL SECTION THROUGH CLUBROOM
(1:25)

HORNBY CRICKET CLUBROOM EXTENSIONS	DENTON PARK, HORNBY.			SHEET 3
	DRAWN [signature]	CHECKED	SCALES As shown	SERIES OF 5 shts
	TRACED	DATE Feb. 1989		REF

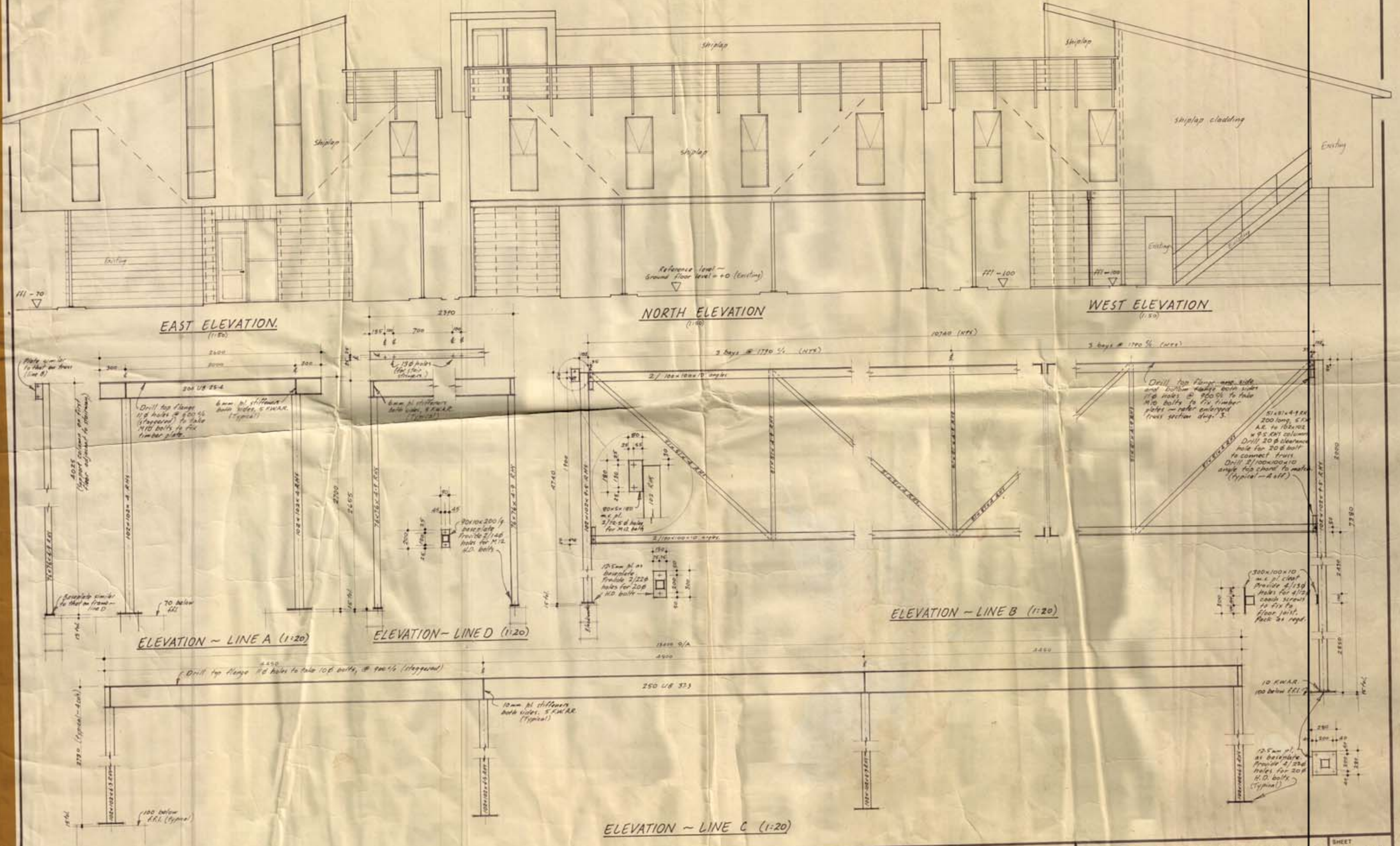
95
50
145



HORNBY CRICKET CLUBROOM EXTENSIONS

DENTON PARK, HORNBY

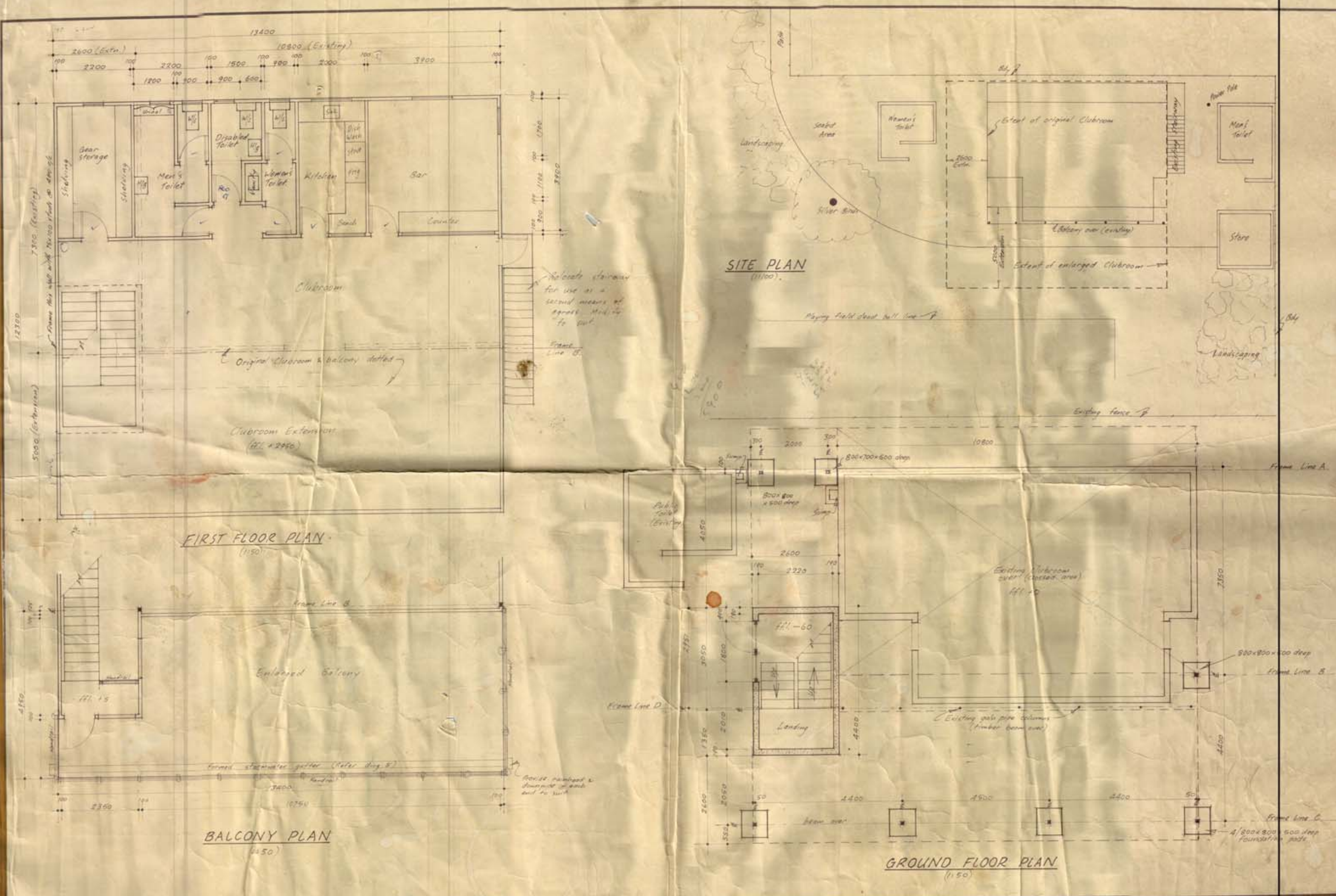
5 A



HORNBY CRICKET CLUBROOM EXTENSIONS.

DENTON PARK, HORNBY.

DRAWN Nichols BE.	CHECKED	SCALES As Shown	SHEET 2A
TRACED	DATE Feb. 1989.		SERIES OF 5 shts
			REF



HORNBY CRICKET CLUBROOM EXTENSIONS

DENTON PARK, HORNBY

DRAWN M	CHECKED	SCALES As Shown	SHEET 1A
TRACED	DATE Sept 1989		SERIES OF 5 shts

Appendix C

CERA Building Evaluation Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Denton Park Pavilion			Reviewer:	H D Mackinven
	Unit	No:	Street	CPEng No:	1003941
Building Address:		446	Main South Road	Company:	GHD Ltd
Legal Description:	RS 41304			Company project number:	5130596-94
				Company phone number:	03 3780900
	Degrees	Min	Sec		
GPS south:	43	32	36.55	Date of submission:	10/3/2013
GPS east:	172	31	10.23	Inspection Date:	2/22/2013
				Revision:	FINAL
Building Unique Identifier (CCC):	PRK 0770 BLDG 005 EQ2			Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	0
Soil type:	mixed	Soil Profile (if available):	
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):		If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	30.00

Building

No. of storeys above ground:	2	single storey = 1	Ground floor elevation (Absolute) (m):	30.00
Ground floor split?	no		Ground floor elevation above ground (m):	0.00
Storeys below ground	0			
Foundation type:	mat slab		if Foundation type is other, describe:	Pads under columns
Building height (m):	7.00		height from ground to level of uppermost seismic mass (for IEP only) (m):	5
Floor footprint area (approx):	172			
Age of Building (years):	51		Date of design:	1935-1965
Strengthening present?	no		If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):	other (specify)		Brief strengthening description:	
Use (upper floors):	other (specify)			
Use notes (if required):	Changing rooms & Clubrooms			
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	frame system		
Roof:	timber truss	truss depth, purlin type and cladding	2m, timber, metal
Floors:	timber	joist depth and spacing (mm)	238 @ 460mm
Beams:	steel non-composite	beam and connector type	250 UB 31.4 Welded
Columns:	structural steel	typical dimensions (mm x mm)	100 x 100 x 9 SHS
Walls:	partially filled concrete masonry	thickness (mm)	200

Lateral load resisting structure

Lateral system along:	other (note)	**Note: Define along and across in detailed report!**		URM Walls & RM walls
Ductility assumed, μ:	1.00		describe system	
Period along:	0.10	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	other (note)			Steel Framing, Timber Framed Walls, URM Walls & RM walls
Ductility assumed, μ:	1.00		describe system	
Period across:	0.10	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

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

Non-structural elements

Stairs:	steel	describe supports	
Wall cladding:	plaster system	describe	
Roof Cladding:	Metal	describe	
Glazing:	aluminium frames		
Ceilings:	plaster, fixed		
Services(list):			

Available documentation

Architectural	partial	original designer name/date	Unknown, Sept 1989
Structural	partial	original designer name/date	Unknown, Sept 1989
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:	0%	Describe how damage ratio arrived at:	
	Describe (summary):	Cracking along mortar lines		
Across	Damage ratio:	0%		
	Describe (summary):	Cracking along mortar lines		
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	no	Describe:	

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

Recommendations

Level of repair/strengthening required:	significant structural and strengthening	Describe:	Minor repair and significant strengthening of URM walls
Building Consent required:	yes	Describe:	
Interim occupancy recommendations:	full occupancy	Describe:	

Along	Assessed %NBS before e'quakes:	56%	##### %NBS from IEP below	If IEP not used, please detail assessment methodology:	NZS 3604, NZEE Guidelines, NZS 3404
	Assessed %NBS after e'quakes:	56%			
Across	Assessed %NBS before e'quakes:	43%	##### %NBS from IEP below		
	Assessed %NBS after e'quakes:	43%			

GHD

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Document Status

Rev No.	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
Draft	Peter O'Brien	David Lee	DLee	Nick Waddington		04/03/13
Draft (V2.0)	Yifei Li	Hamish Mackinven		Donna Bridgman		22/08/13
FINAL	Yifei Li	Hamish Mackinven		Donna Bridgman		3/10/13