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Denton Oval

BU 0770-003 EQ2

Detailed Engineering Evaluation

Quantitative Report

Version FINAL

442 Main South Road, Hornby

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

Denton Oval
BU 0770-003 EQ2

Detailed Engineering Evaluation
Quantitative Report
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442 Main South Road, Hornby

Christchurch City Council

Prepared By
Jay Zeus B. Rivera

Reviewed By
Stephen Lee

Date
07/03/13

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Quantitative Report Summary

Denton Oval

BU 0770-003 EQ2

Detailed Engineering Evaluation

Quantitative Report - SUMMARY

Version FINAL

442 Main South Road, Hornby

Background

This is a summary of the Quantitative report for the structures of the buildings at Denton Oval, and is based in general on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011 and visual inspections on 18th May 2012.

Building Description

The buildings at Denton Oval consist of a steel and reinforced concrete grandstand structure with a one(1) storey unreinforced masonry amenity building beneath it and a two (2) storey masonry Hornby Cycling Clubrooms building. The grandstand and the Hornby cycling clubroom building were separated by a seismic gap. This report focusses on the assessment of the grandstand and the amenity building only.

Building Capacity Assessment

For the purpose of seismic assessment, the grandstand and the amenity building have been analysed separately. This is due to the lack of connection between the walls of the amenity building and the concrete frame of the grandstand. The grandstand is composed of steel frame atop of the concrete frame while the amenity building is composed mainly of unreinforced masonry block wall. The amenity building is then classified into two blocks, Block A serves as an office and changing room for players and officials while Block B is for Men's and Women's toilet.

The grandstand achieved a rating of 35% NBS while the amenity building scored 22% NBS. The rating of the changing area is greatly affected by the unreinforced masonry block walls that support the lightweight roof. The walls do not have sufficient strength to resist lateral loads that are transferred to it by the roof. Therefore the building is considered "Earthquake Prone".

However it should be noted that considering the size and location of the amenity building (the weakest part of the building), it does not pose significant risk to the grandstand even if it collapses.

Key Damage Observed

Key damage observed:

- Minor cracking in concrete masonry walls in the changing room areas of the amenity building.

Recommendations

GHD recommend that further work is undertaken in order to develop a strengthening and repair scheme. This work should involve:

- Developing a strengthening works scheme to increase the seismic capacity of the grandstand and the amenity building to as near as practicable to 100% NBS, or at least 67% NBS.
- The structure should remain unoccupied until such time that strengthening works are completed.

1. Background

GHD has been engaged by the Christchurch City Council (CCC) to undertake a detailed engineering evaluation of Denton Oval.

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

A quantitative assessment involves analysis and checking of all structural members that forms part of the structure that contributes in resisting of horizontal and vertical forces that are subjected to it. Furthermore, it is also used to evaluate the existing conditions of the structure with respect to our prevailing industry codes and standards.

The main purpose of this procedure is to assess how the structure will respond upon application of external forces and to what extend will the damage may be with respect to its existing condition. Evaluating the capacity of the structure versus the applied loads, we can determine the structure’s rating in terms of percentage of New Building Standards (%NBS) as per NZSEE requirements.

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.

CERA now requires 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). The Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011 has been adopted by CERA for evaluations. 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.

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 circumtances
High Risk Building	D or E	High	33 or lower	Unacceptable (Improvement Required)	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

Denton Oval is located in 442 Main South Road, Hornby Christchurch and can be accessed thru Chalmers Road. The site is consisting of a 400 m length concrete velodrome, a rugby playing field, a reinforced concrete grandstand with an amenity building underneath and a two storey concrete masonry Hornby cycling clubrooms building. The stadium was built in 1974 for the Commonwealth games. In 1990, the Hornby cycling clubrooms were constructed at the right side of the grandstand. A seismic gap was created in order to have a separation between the existing grandstand and the new Hornby cycling clubrooms building.

The main structure is composed of a grandstand and the amenity building. For the purpose of analysis, the two (2) are separately assessed and checked.

The grandstand is divided into two major components which are the steel frame and the concrete frame assembly. The grandstand has a base dimension of 11.80 m x 41.40 m and a height of approximately 9.30 m from ground to roof apex. It has a capacity of approximately 2000 people and offers a full view of the whole velodrome and the playing field.

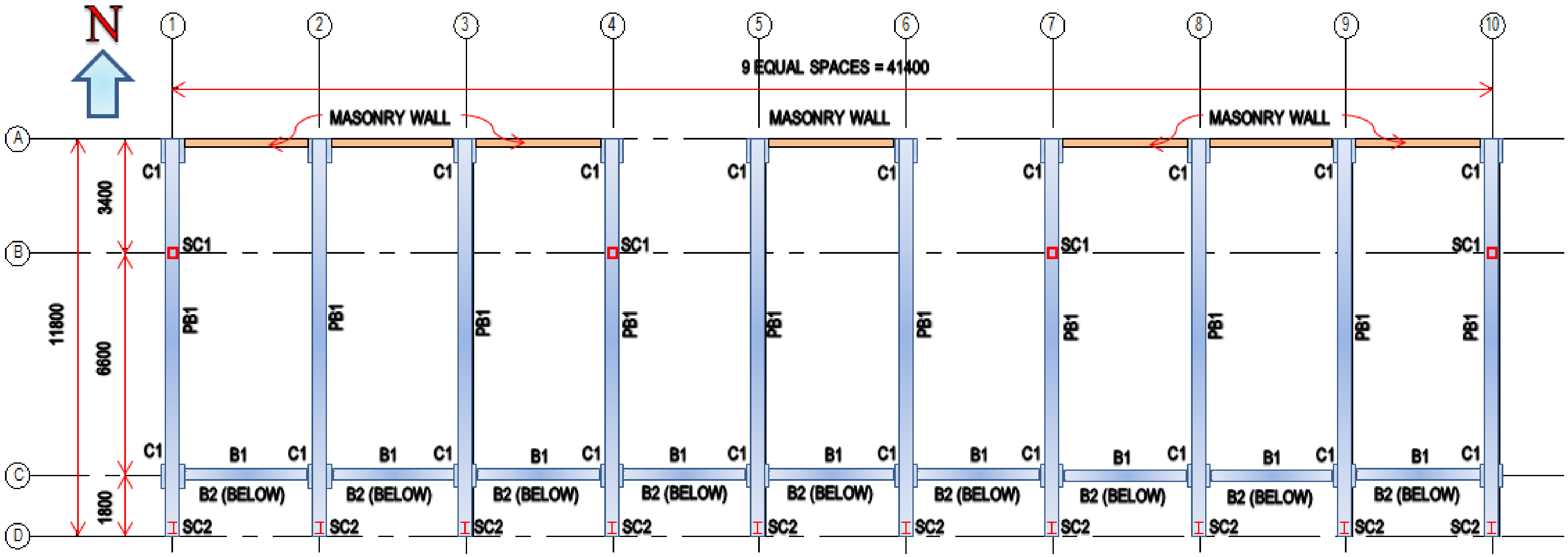
The steel frame assembly serves as the support for the roof of the structure. The roof consists of lightweight metal roof sheeting on light gauge metal purlins supported by a series of 250UB rafters spanning from the back of the structure up to the front. These rafters are then supported by a 460UB longitudinal beam at the front and a series of 250UB columns at the back of the structure. The 460UB longitudinal beam is supported by 4-150x150x6 SHS columns spaced at every 13.80 m. Three (3) layers of 125x75x6 RHS horizontal girt beams are seen at the back of the structure in between 250UB columns. These girt beams are equally spaced up the height of the column and give lateral support to the columns and also supports the cladding for the grandstand. All the columns, both the 250UB and the 150x150x6 SHS, are pinned connected to the concrete frame structure below it.

The concrete frame assembly serves as the support for the steel frame and the bleacher seats. It is composed of a series of 450 mm x 450 mm R.C. columns supporting the 300 mm x 1000 mm pre-stressed concrete raking beams. These raking beams are inclined from the ground by 27°. A series of 225 mm x 550 mm longitudinal beams laterally support the columns in the longitudinal direction. Double T precast concrete units serve as the flooring for the grandstand and are supported by the raking beams. The concrete columns sit on reinforced concrete pad footings that are tied together by R.C. footing tie beams.

The amenity building is located underneath the grandstand. It is divided into two (2) blocks, namely Block A and B. Block A has a base dimension of 9.00 m x 22.90 m and Block B has 5.40 m x 13.80 m. Roof height for both Block A and B is 3.45 m from finished floor level. Block A serves as a changing room and office while Block B is the toilet block. A portion of Block A is under the grandstand while the rest is extended outward. Block B has its back wall at the face of the grandstand columns and extends outward from the grandstand. The roof is made of lightweight metal sheet on 75 mm x 50 mm timber purlins supported by 250 mm x 50 mm timber rafters. The walls are made up of 190 mm thick concrete masonry block. A 100 mm thick concrete ground slab serves as the floor for both Blocks A and B. The masonry block walls are only reinforced horizontally in bond beams at the top of the walls and vertically at corners and edges of openings. The masonry block walls are just butted against the grandstand columns without any structural connection. The block walls sit on reinforced concrete strip footings.

Refer to Figure 3 to 6 for steel, concrete and masonry plans and a typical frame elevation of the structure.

Figure 2   Aerial photograph of Denton Oval



Note:

B1 – 250mm x 550mm R.C. Beam

B2 – 250mm x 550mm R.C. Beam

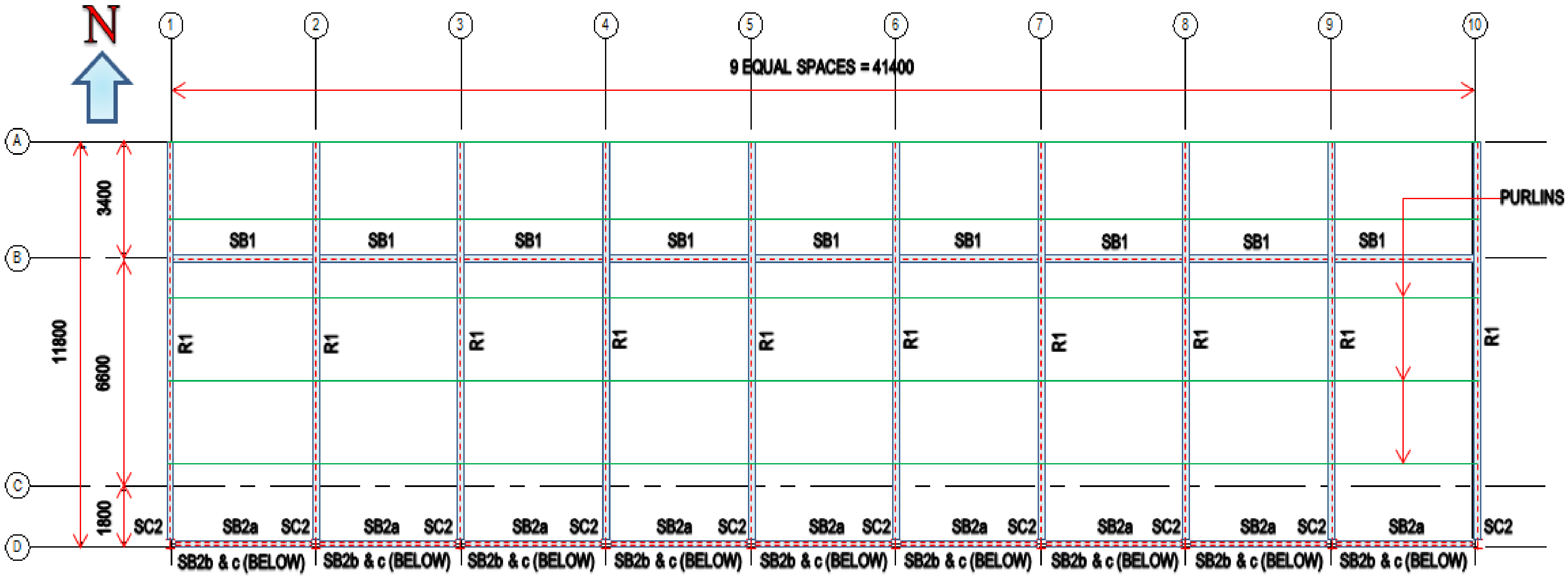
PB1 – 300mm x 1000mm Pre-stress Concrete Beam

C1 – 450mm x 450mm R.C. Column

SC1 – 150x150x6 SHS

SC2 – 250UB

Figure 3 Framing plan of the grandstand



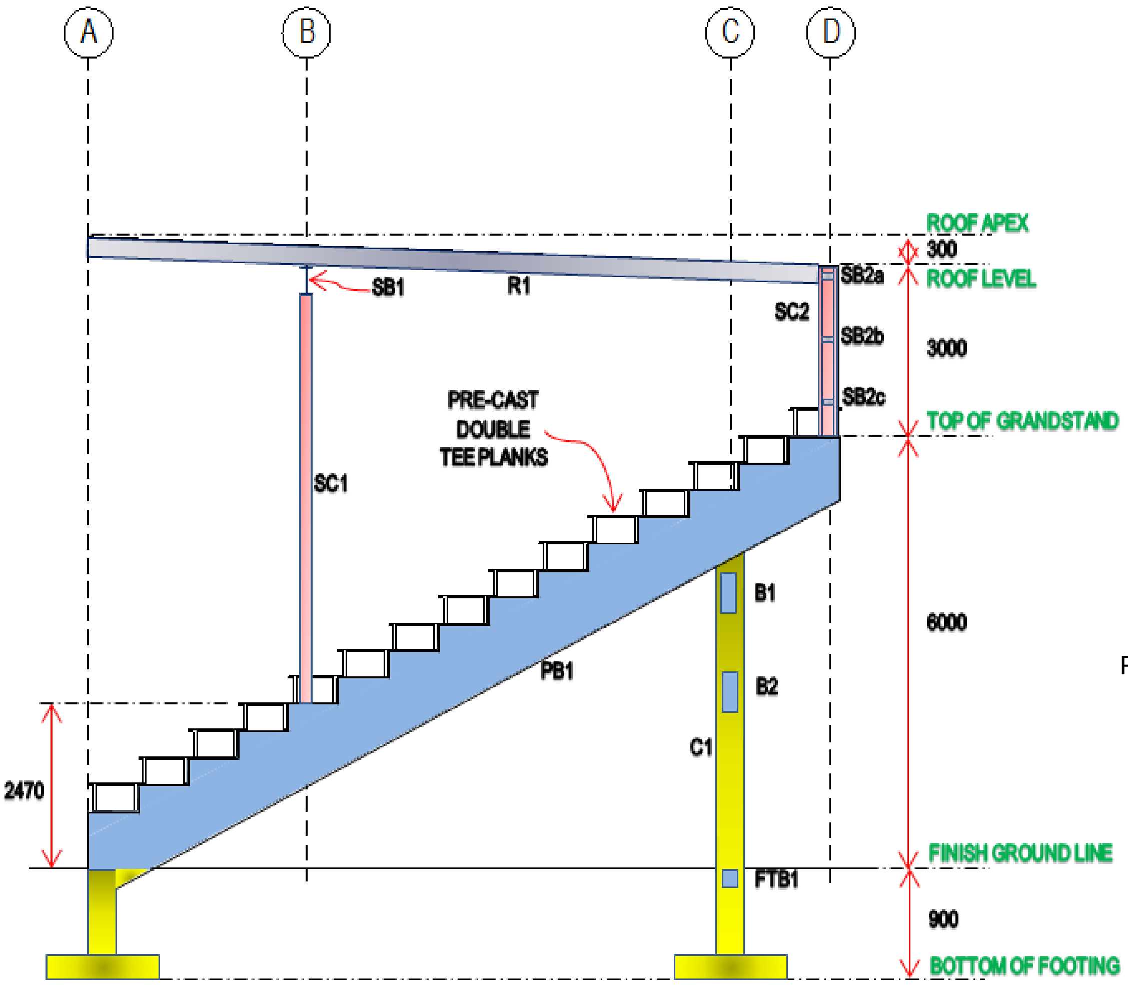
Note:

SB1 – 460UB

SB2(a-c) – 125x75x6 RHS

R1 – 250UB

Figure 4 Roof framing plan of the grandstand



Note:

FTB1 – 300mm x 300mm R.C. Beam

Figure 5 Typical frame elevation

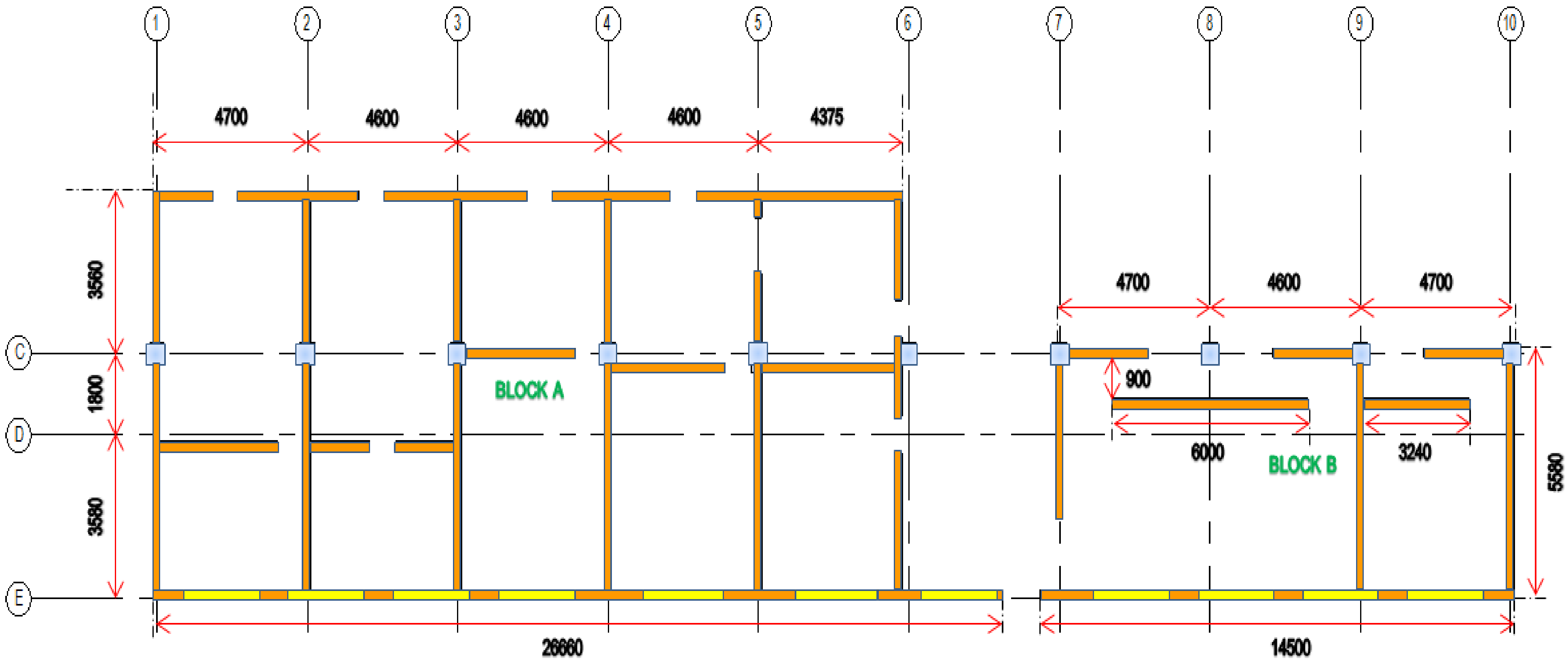


Figure 6 Masonry wall layout for Amenity Building

4.2 Gravity Load Resisting System

4.2.1 Grandstand

The gravity loads for this structure are resisted by the steel frame and concrete frame system.

The roof structure of the grandstand consists of lightweight metal roofing on light gauge metal purlins supported by a series of 250UB steel rafters. These steel rafters are supported by a 460UB longitudinal steel beam along Gridline B and series of 250UB steel columns along Gridline D. The longitudinal steel beam is then supported by 4-150x150x6SHS steel columns. These SHS columns are located along Gridline 1, 4, 7 and 10. Then this steel frame transfer the gravity loads to the pre-stressed concrete beam in which forms part of the concrete frame structure below.

The floor of the grandstand is composed of precast double tee units which are bolted and grouted to the pre-stressed concrete raking beams. These precast double tee units support the wood benches for the spectators. The raking beams then transferred the gravity forces to the R.C. columns along Gridline A and C. These columns then transfer the gravity load to the foundations.

4.2.2 Amenity Building

The gravity loads in the amenity building for both Block A and B are resisted by the unreinforced masonry walls. These gravity loads are transferred by the roof consisting of lightweight metal sheeting on timber purlins supported by timber rafters to the unreinforced concrete masonry walls.

4.3 Lateral Load Resisting System

4.3.1 Grandstand

Lateral loads acting on the structure are resisted by the steel frame and concrete frame system.

The steel frame resists the lateral load for the upper portion of the grandstand. The roof cladding on steel purlins acts as a diaphragm that transfers lateral load to the steel rafters that are supported by steel columns. Lateral loads in longitudinal direction along Gridline D are resisted by series of 250UB columns and RHS girt beams while on Gridline B, the 4-SHS columns and the 460UB beam act together to provide a frame. For the transverse direction, steel frames consisting of a steel rafter and columns resist lateral loads.

The concrete frame resists the lateral load for the lower part of the structure. With the help of the precast concrete double tee units, which acts as diaphragms and as flooring as well for the grandstand, lateral loads are transferred to the pre-stressed raking beams. Lateral loads in longitudinal direction are resisted by reinforced concrete frame consisting of concrete columns and beams. Lateral loads in the transverse direction are resisted by the combination of reinforced concrete columns and pre-stressed concrete raking beams.

4.3.2 Amenity Building

In the changing room areas, lateral loads are resisted by concrete masonry walls in both the long and short directions of the building.

The lateral loads in the amenity building are resisted by the unreinforced masonry walls. The lightweight roof acts as a diaphragm and transfers lateral loads to the walls in plane. Also, the timber rafters act as out-of-plane braces for these walls.

5. Assessment

5.1 Site Inspection

An inspection of the building was undertaken on the 18th of January 2012. Both the interior and exterior of the building were inspected. The building was observed to have a green placard in place. The main structural components of the building were all able to be viewed due to the exposed nature of the structure. The underside of the grandstand is open and the concrete masonry changing rooms and the two storey addition are unlined. No inspection of the foundations of the structure was able to be undertaken.

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

5.2 Investigation & Opening Up Work

Further inspections were carried out on the 17th and 18th of May 2012. The work included drafting of an as-built plan of the structure because there are no available drawings/plans, taking of measurements and dimensions of the structure as well as the key structural elements (i.e. columns, beams and walls). Reinforcement scanning using a Hilti PS200 Ferroscan was also performed. A series of photographs of key structural elements and connections were also taken.

5.3 Analysis and Modelling Methodology

5.3.1 Mathematical Modelling

The three-dimensional frame modelling of the Denton Oval Grandstand structure was performed to realistically simulate the effects of the applied loads on the structure under different conditions such as normal operation, wind, earthquake and combinations thereof.

This modelling approach determines the adequacy of members or sections for the structure under various loading conditions.

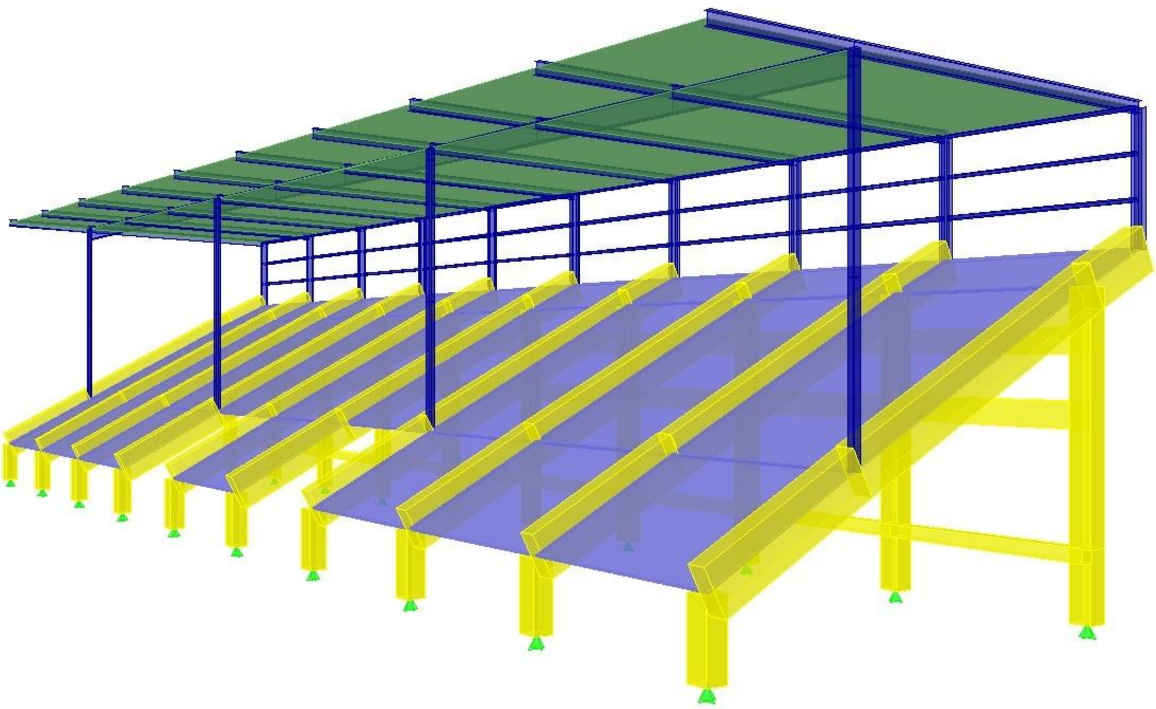


Figure 7 3D Mathematical Model of Denton Oval's Grandstand in Etabs

Each section, member and node of the model was defined using the physical dimensions, material properties and connection details gathered from site inspections as stated in Section 5.2. Using Etabs Version 9.7.2 structural analysis software, a computer model that incorporates all the properties of the steel portal frame and reinforced concrete structure was prepared.

The Amenity Building was analysed separately using manual calculations and spread sheets.

5.3.2 Loadings

Loadings such as permanent actions, imposed actions as well as wind and earthquake actions are considered in the analysis of the structure. Also the different loading combinations and factors of safety are used. New Zealand Standards (NZS) are used for the determination of each of the parameters and values required. The references used are listed in Section 13.

5.3.3 Seismic Design

The Denton Oval structure was checked to the seismic design standards in accordance with the AS/NZ 1170.5, NZBC Clause B1 Structure and New Zealand Society of Earthquake Engineering Guidelines for Assessment and Improvement of the Structural Performance of Buildings in Earthquakes.

5.3.4 Wind Design

As wind had the potential to be a dominant effect, additional action was considered in the analysis and checking of Denton Oval Grandstand structure. Wind action is included in order to take into account its effect to the structure. AS/NZ 1170.2:2011 was used as reference.

5.4 Calculations

5.4.1 Determination of %NBS

After analysing the structure with the use of the mathematical model and spread sheets, all the structural elements that form part for the structure were checked and individual demand capacity ratios were computed. From there the %NBS of each element was determined.

6. Damage Assessment

6.1 Surrounding Buildings

No damage to surrounding buildings was observed during our inspection of the site.

6.2 Residual Displacements and General Observations

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

Minor cracking was noted in the concrete masonry partition wall that was built around one of the beams supporting the stand seating as can be seen in Photograph 6.

Minor cracking was also noted in a number of the concrete masonry walls of the changing room areas. These cracks have typically occurred around the doorways and in the corners of the rooms as can be seen in Photograph 7.

6.3 Ground Damage

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

7. Analysis

7.1 Seismic Parameters

The seismic design parameters based on current design requirements from NZS1170.5:2002, NZS 3604:2011 and the NZBC clause B1 for this building are:

Location	:	Christchurch
Importance Level	:	3
Site Classification	:	D
Seismic Zone Factor	:	0.30
(Table 3.3, NZS 1170.5:2004)		
Annual Probability of Exceedance	:	1/1000 (ULS)
(Table 3.3, NZS 1170.0:2002)		
Annual Probability of Exceedance	:	1/25 (SLS)
(Table 3.3, NZS 1170.0:2002)		
Return Period Factor (Ru)	:	1.30 (ULS)
(Table 3.5, NZS 1170.5:2004)		
Return Period Factor (Ru)	:	0.33 (SLS)
(Table 3.5, NZS 1170.5:2004)		
Ductility Factor,		
Concrete Structure, (μc)	:	1.25
Steel Structure, (μs)	:	2.00
Masonry Wall, (μw)	:	1.25
Performance Factor (Sp)	:	0.925
Gravitational Constant (g)	:	9.81 m/sec^2
Liquefaction Potential	:	Low

7.2 Wind Parameters

The wind design parameters based on current design requirements from NZS1170.2:2011 are:

Location	:	Christchurch
Importance Level	:	3
Terrain Category Definition	:	Category 2
Region Classification	:	A7
Annual Probability of Exceedance	:	1/1000 (ULS)

(Table 3.3, NZS 1170.0:2002)		
Annual Probability of Exceedance	:	1/25 (SLS)
(Table 3.3, NZS 1170.0:2002)		
Wind Direction Multiplier (Md)	:	1.00
Shielding Multiplier (Ms)	:	1.00
Topographic Multiplier (Mt)	:	1.00
Density of Air	:	1.20 kg/m^3

7.3 Structural Ductility Factor

A structural ductility factor of 1.25 has been assumed for the reinforced concrete frame while 2.00 for the steel frame. With this, a structural ductility factor of 1.25 was adopted for the whole structure because the frames are connected to each other.

For the unreinforced masonry wall, a structural ductility factor of 1.25 has been assumed.

8. Geotechnical Consideration

The site is situated within a recreational reserve, 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.

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 property is owned and maintained by the Christchurch City Council.

8.1.1 Published Information on Ground Conditions

8.1.1.1 Published Geology

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

- Holocene alluvial soils of the Yaldhurst Member, sub-group of the Springston Formation, which contains alluvial gravel, sand and silt of historic river flood channels.

8.1.1.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates eight boreholes are located within a 200m radius of the site. Of these boreholes, six of them had lithographic logs of which four are considered for this report. The site geology described in these logs shows the area is predominantly sandy gravels with varying amounts of silt and clay.

Table 2 ECan Bore Log Summary Table

Bore Name	Depth (m bgl)	Log Summary
	0 – 1	Hardfill
(110m SE of site)	1 – 21	Fine to coarse GRAVEL and SAND
	21 – 49	Medium dense to dense GRAVEL, with some sand and clay
	49 – 52	Dense GRAVEL, with sand and clay
	52 – 79	Fine to medium GRAVEL, with traces of clay
	79 – 86	Sandy medium GRAVEL
	86 – 88	PEAT
	88 – 94	Dense GRAVEL, and stiff CLAY
	94 – 102	Dense Sandy GRAVEL, with some yellow clay
	0 – 0.3	
	0.3 – 3.9	

[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	Depth (m bgl)	Log Summary
	3.9 – 40.5	Sandy GRAVEL, with some clay
	40.5 – 49.5	CLAY, with some gravel and peat
	49.5 – 52	Dense GRAVEL, with some clay
	52 – 95.8	Layers of CLAY, SAND and GRAVEL
	0 – 6	Gravelly SAND
	6 – 23.5	Sandy GRAVEL, with traces of silt and clay
	23.5 – 29.5	Sandy GRAVEL
	0 – 1	Clayey GRAVEL
	1 – 9	Sandy GRAVEL, with some clay and silt
	9 – 10.8	Sandy GRAVEL
	10.8 – 12.5	Slightly clayey, fine SAND
	12.5 – 20.7	Clayey GRAVEL and sandy GRAVEL

It should be noted the quality of soil logging descriptions included on the boreholes is unknown and were likely written by the well driller and not a geotechnical professional or to a recognised geotechnical standard. In addition strength data is not recorded.

8.1.1.3 EQC Geotechnical Investigations

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

8.1.1.4 Land Zoning

Canterbury Earthquake Recovery Authority (CERA) has zoned the site as Green, indicating repair and rebuild may take place

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

The site is classified as “not applicable”. This indicates that it is a non-residential properties in an urban area that has not been given a Technical Category. However, nearby land has been classified as Technical category 1 (TC1) which means that liquefaction is unlikely in a future earthquake event.

8.1.1.5 Post February Aerial Photography

Aerial photography taken following the 22 February 2011 earthquake (Figure 8) shows no signs of liquefaction outside the building footprint or adjacent to the site.

Figure 8 Post February 2011 Earthquake Aerial Photography[2]

8.1.1.6 Summary of Ground Conditions from desk study

From the ECan borehole information the ground conditions on Main South Road comprise multiple strata of gravel, sandy gravel and sand, with varying amounts of silt and clay.

8.1.2 Seismicity

8.1.2.1 Nearby Faults

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

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

Known Active Fault	Distance from Site (km)	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	120 NW	8.3	~300 years
Greendale (2010) Fault	13 W	7.1	~15,000 years

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

[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 http://maps.gns.cri.nz/website/af/viewer

Known Active Fault	Distance from Site (km)	Max Likely Magnitude	Avg Recurrence Interval
Hope Fault	100 N	7.2~7.5	120~200 years
Kelly Fault	100 NW	7.2	~150 years
Porters Pass Fault	54 NW	7.0	~1100 years

Recent earthquakes since 4 September 2010 have identified the presence of a previously unmapped fault system underneath the Canterbury Plains including, Christchurch City and the Port Hills. Research and published information on this system is in development and not generally available and average recurrence intervals are yet to be estimated.

8.1.2.2 Ground Shaking Hazard

New Zealand Standard NZS 1170.5:2004 now 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.

The recent seismic activity in Canterbury has produced earthquakes of Magnitude-7.1 (Sept., Darfield), 6.3 (Feb., and June, Christchurch) 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.

8.1.3 Field Investigations

In order to further understand the ground conditions at the site, intrusive testing comprising one piezocone CPT investigation was conducted at the site on 12 April 2012.

The location of the test is tabulated in Table 4.

Table 4 Coordinates of Investigation Locations

Investigation	Depth (m bgl)	Easting (NZMG)	Northing (NZMG)
CPT 001	2.0	2471263	5740467

The CPT investigation was undertaken by McMillan Drilling Service on 12 April 2012, typically to a target depth of 20m below ground level. However, refusal was reached at depth of 2.0m due to the presence of dense gravels. Please refer to the attached CPT results for detail (Appendix A).

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

[5] McMillans Drilling CPT data plots, Appendix A.

8.1.4 Ground Conditions Encountered

8.1.4.1 Summary of CPT-Inferred Lithology

Table 5 Summary of CPT-Inferred Lithology

Depth (m)	Lithology [1]	Cone Tip Resistance q_c (MPa)	Friction Ratio Fr (%)
0 – 2.0	Surface soil	~5	~1
>2.0	Gravel	> 20	~0

8.1.5 Interpretation of Ground Conditions

8.1.5.1 Liquefaction Assessment

Based on an overall assessment of the following, the site is considered unlikely to be susceptible to liquefaction confirming the CERA TC1 classification.

- The identified ground conditions confirmed by CPT;
- The minimal damage to ground (and building) caused by the Canterbury earthquake sequence evidenced by aerial and visual inspection.

8.1.5.2 Slope Failure and/or Rockfall 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.

8.1.5.3 Foundation Recommendations

Following the guidance provided by the Department of Housing and building[6] (DBH) in Section 4 for repairing of foundations for TC1 dwellings, the following geotechnical recommendations are provided.

- A site subsoil class of **D**, Deep or Soft Soil, should be adopted for this sites (in accordance with NZS 1170.5:2004)
- An allowable bearing capacity of 100kPa can be used for any replacement shallow foundations required.
- If a re-build is deemed necessary a shallow investigation specific to the new building footprint should be undertaken. Shallow ground improvement is not required.

9. Results

The following are the results of the structural analysis for Denton Oval structure.

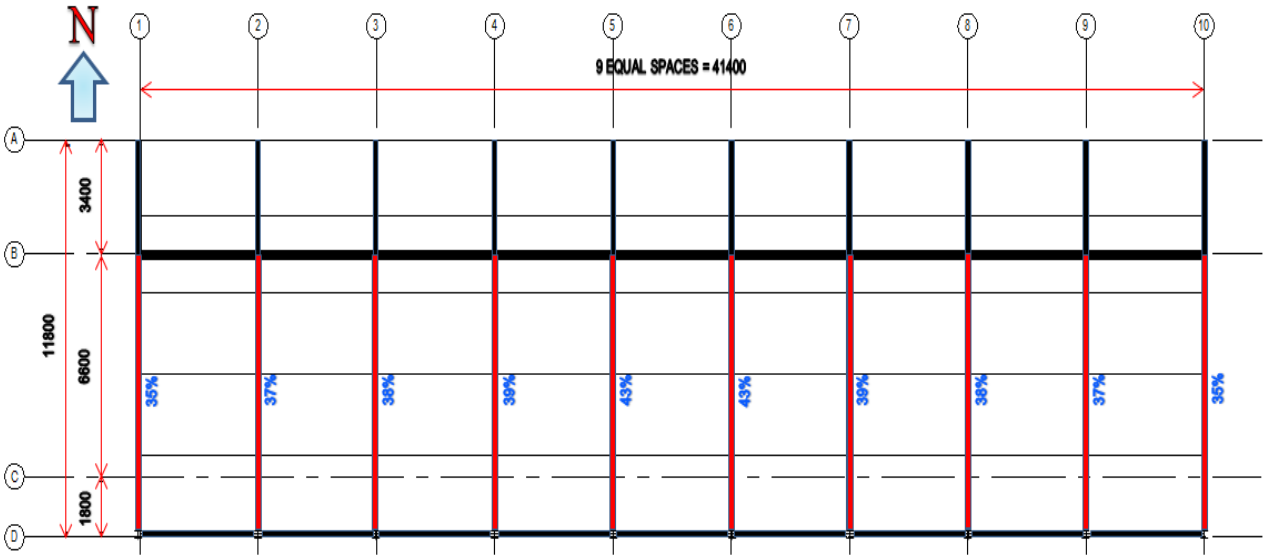
9.1 %NBS

Our analysis showed that seismic effects were most critical.

9.1.1 Grandstand

Steel Rafters

Ten (10) steel rafters rated below 67% NBS and they are highlighted in red in the figure below. The lowest rating achieved is 35% NBS.

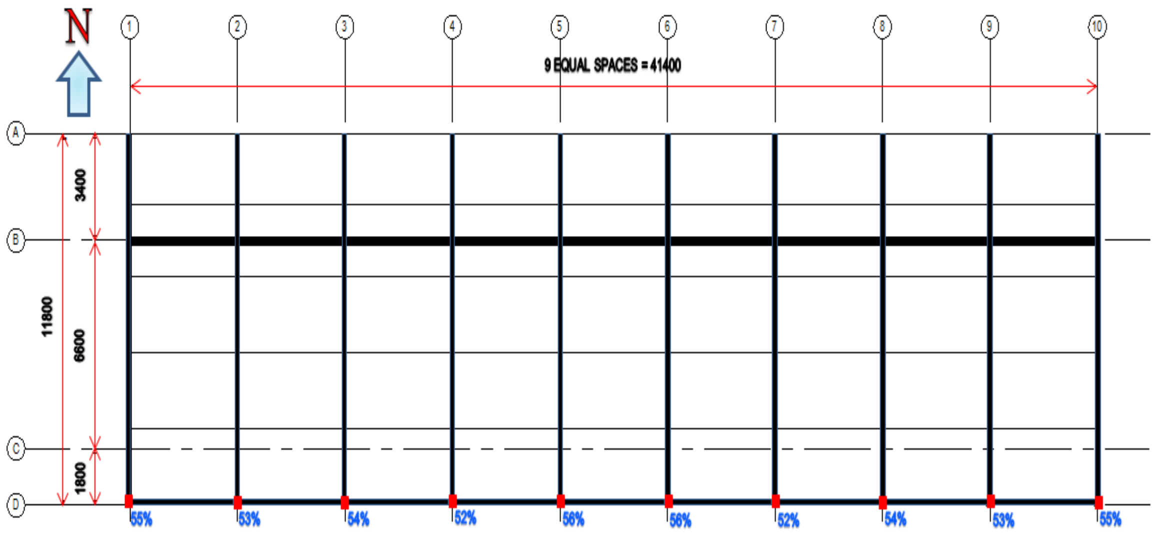


Steel Beams

All steel beams have a rating of greater than 67% NBS.

Steel Columns

Ten (10) steel columns rated below 67% NBS. They are highlighted in red in the figure below.



Pre-stressed Raking Beams

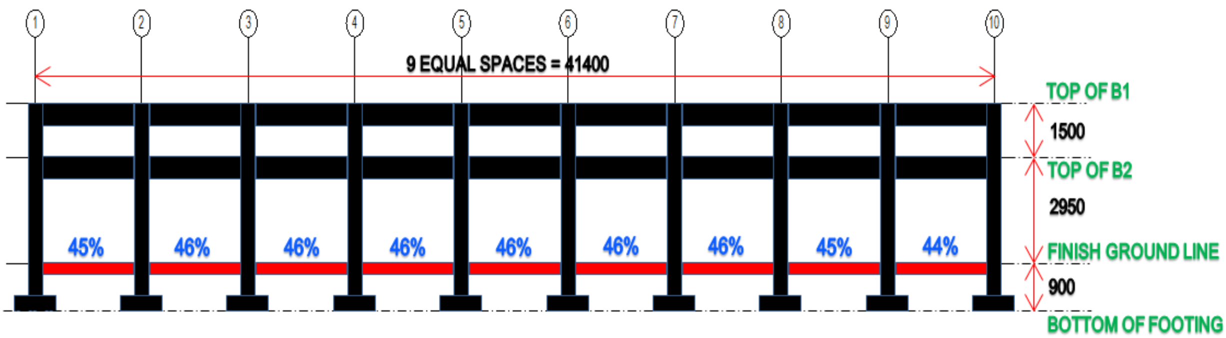
All pre-stressed concrete beams have a rating of greater than 67% NBS.

Reinforced Concrete Beams

All reinforced concrete beams have a rating of greater than 67% NBS.

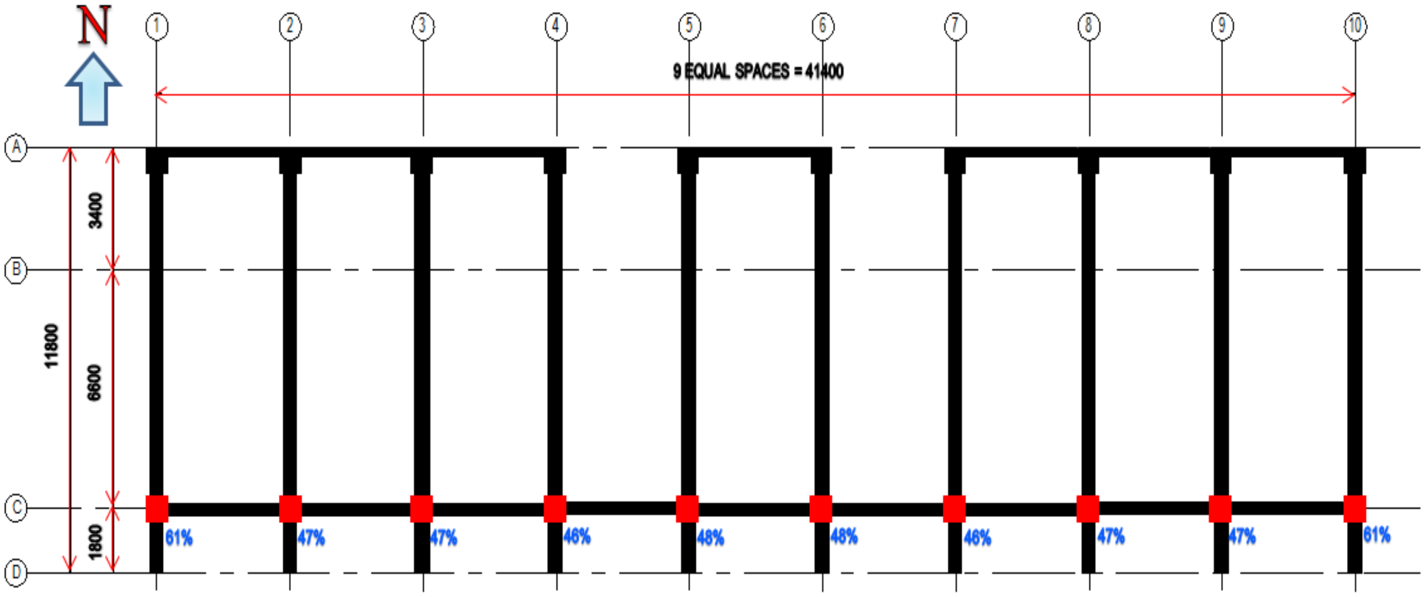
Reinforced Concrete Footing Tie Beams

Nine (9) reinforced concrete footing tie beams rated below 67% NBS and they are highlighted in red in the figure below. The lowest rating achieved is 44% NBS.



Reinforced Concrete Columns

Ten (10) reinforced concrete columns rated below 67% NBS and they are highlighted in red in the figure below. The lowest rating achieved is 52% NBS.

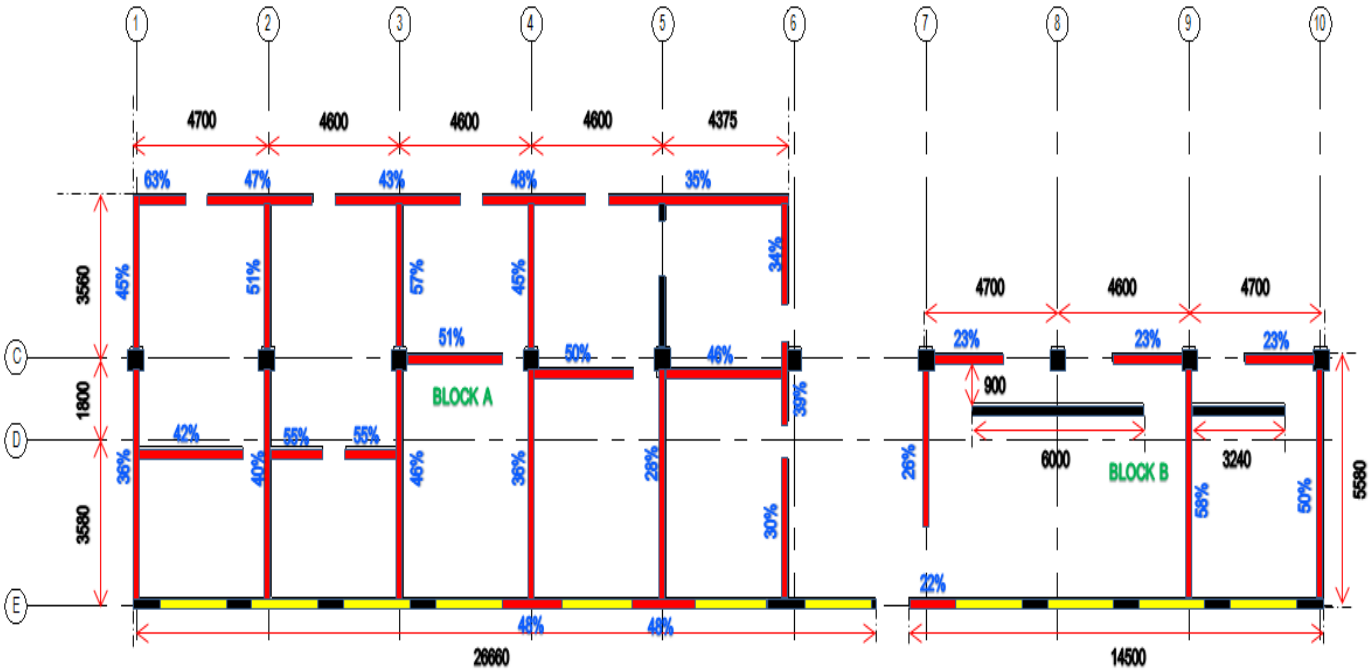


9.1.2 Amenity Building

Unreinforced Masonry Walls

Block A - Three (3) unreinforced masonry wall rated below 34% NBS and twenty two (22) unreinforced masonry walls rated below 67% NBS. They are highlighted in red in the figure below.

Block B - Five (5) unreinforced masonry wall rated below 34% NBS and two (2) unreinforced masonry walls rated below 67% NBS. They are highlighted in red in the figure below.



Reinforced Concrete Masonry Bond Beam

All reinforced concrete masonry bond beams have a rating of greater than 67% NBS.

9.1.3 Hornby Cycling Clubrooms

This building was assessed at 65% NBS based on a qualitative assessment completed in February 2012. No further assessment has been carried out.

9.2 Lateral Seismic Drift

The computed drift of the Denton Oval is 34 mm and 60 mm in the longitudinal and transverse direction respectively.

The existing seismic gap between the grandstand and the two (2) storey masonry horny cycling club building is approximately 100 mm in the longitudinal direction.

9.3 Discussion of Results

Based on the quantitative analysis done for the structure, it is found that the lowest rating achieved is 22% NBS. This rating comes from the unreinforced masonry wall in Block B of Amenity building. This is to be expected since there is virtually no vertical or horizontal reinforcement present in the walls. It would appear that the amenity building, considering the materials used and the prevailing codes and standards at the time it was constructed, serves only to carry gravity loads and not lateral loads.

The grandstand as a structure, considered in isolation, achieved a ratings of 35% NBS.

10. Conclusions

10.1 Building Capacity Assessment

Based on the quantitative assessment of the structure, it is found that the overall seismic capacity is 22% NBS. This is a result of the unreinforced masonry walls in Block B of the Amenity building under the grandstand. Therefore, the building is classified as an ‘Earthquake Prone’ building.

11. Recommendations

GHD recommend that further work is undertaken in order to develop a strengthening and repair scheme. This work should involve:

- Developing a strengthening works scheme to increase the seismic capacity of the Denton Oval grandstand and the amenity building to as near as practicable to 100% NBS, or at least 67% NBS. This will need to consider compliance with accessibility and fire requirements.
- The structure should remain unoccupied until such time that strengthening works are completed.

12. Limitations

12.1 General

This report has been prepared subject to the following limitations:

- The only available drawing is for the amenity building with nothing for the grandstand. As a result, the information contained in this report has been inferred from site inspection done on the structure.
- The Hornby cycling clubrooms building was not checked.
- The foundations of the structure were not checked.
- No level or verticality surveys have been undertaken.
- No material testing has been undertaken.

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

12.2 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 in Section 8.

13. References

1. Detailed Engineering Evaluation Qualitative Report for Denton Oval, February 24, 2012, GHD Pty. Ltd.
2. AS/NZS 1170.0:2002 Structural design actions, Part 0: General Principles, New Zealand Standards
3. AS/NZS 1170.0 Supplement 1:2002 Structural design actions - General principles - Commentary
4. AS/NZS 1170.1:2002 Structural design actions, Part 1: Permanent, imposed and other actions, New Zealand Standards
5. AS/NZS 1170.1 Supplement 1:2002 Structural design actions – Permanent, imposed and other actions - Commentary
6. AS/NZS 1170.2:2011 Structural design actions, Part 2: Wind actions, New Zealand Standards
7. AS/NZS 1170.2 Supplement 1:2002 Structural design actions – Wind actions - Commentary
8. NZS 1170.5:2004 Structural design actions, Part 5: Earthquake actions, New Zealand Standards
9. NZS 1170.5 Supplement 1:2004 Structural design actions –Earthquake actions – New Zealand - Commentary
10. NZS 3101:2006 Concrete Structure Standard, Part 1-The design of concrete structures
11. NZSEE 2006, Assessment and Improvement of the Structural Performance of Buildings in Earthquakes, New Zealand Society for earthquake Engineering
12. Compliance Document for New Zealand Building Code Clause B1: Structure, Department of Building and Housing
13. Engineering Advisory Group, Guidance in Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury, Part 2 Evaluation Procedure, Draft, issued by Engineering Advisory Group on 19 July 2011.
14. Australian Steel Institute (ASI), Design capacity tables for structural steel, Volume 1: Open Sections
15. Australian Steel Institute (ASI), Design capacity tables for structural steel, Volume 2: Hollow Sections

Appendix A

Geotechnical Investigation Reports and Analysis

CPT ANALYSIS NOTES

Soil Type

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

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

Liquefaction Screening

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

- high susceptibility
- medium susceptibility
- low susceptibility

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

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

Low susceptibility is all other cases.

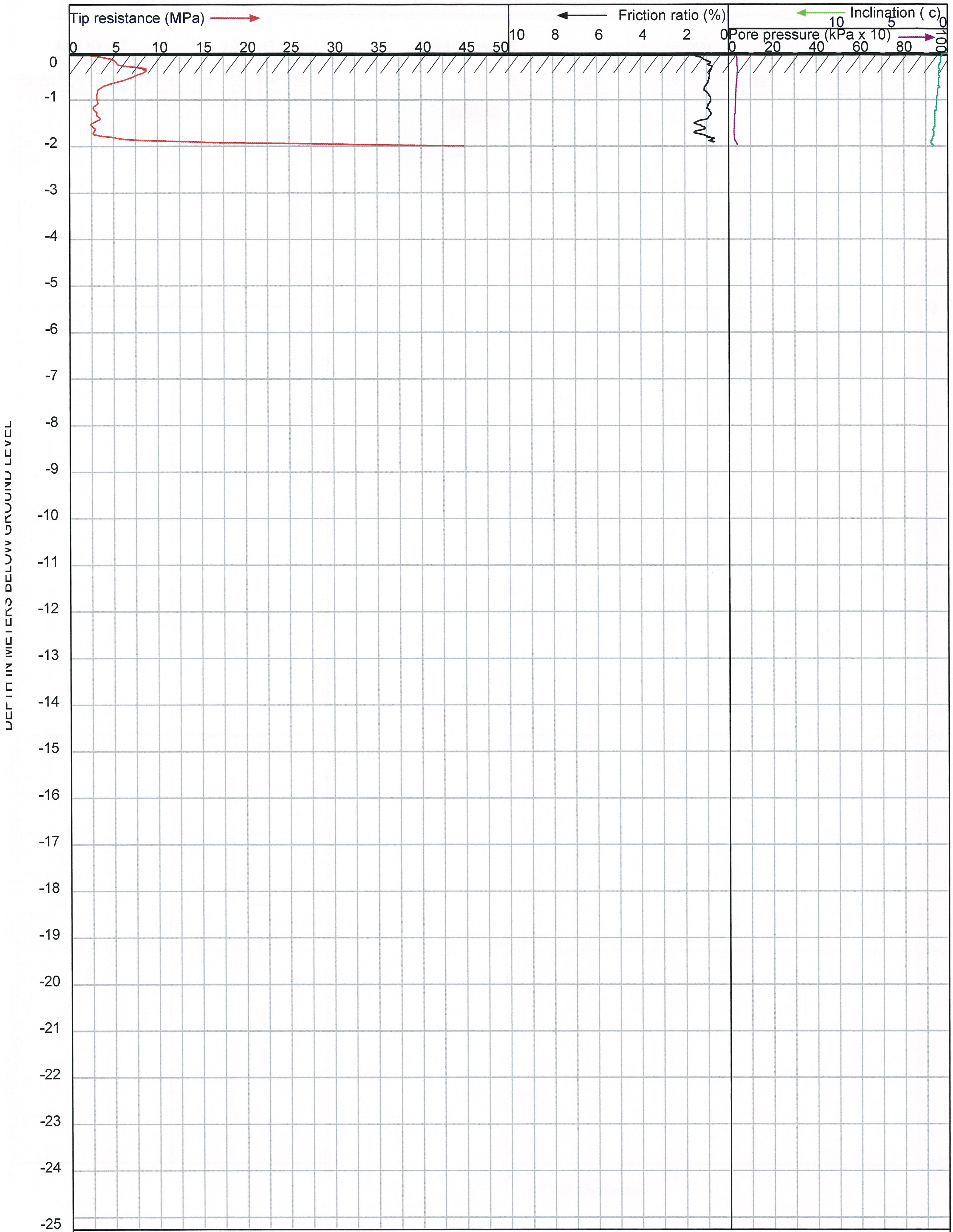
Relative Density (D_R)

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

Undrained Shear Strength (S_U)

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

McMILLAN DRILLING SERVICES

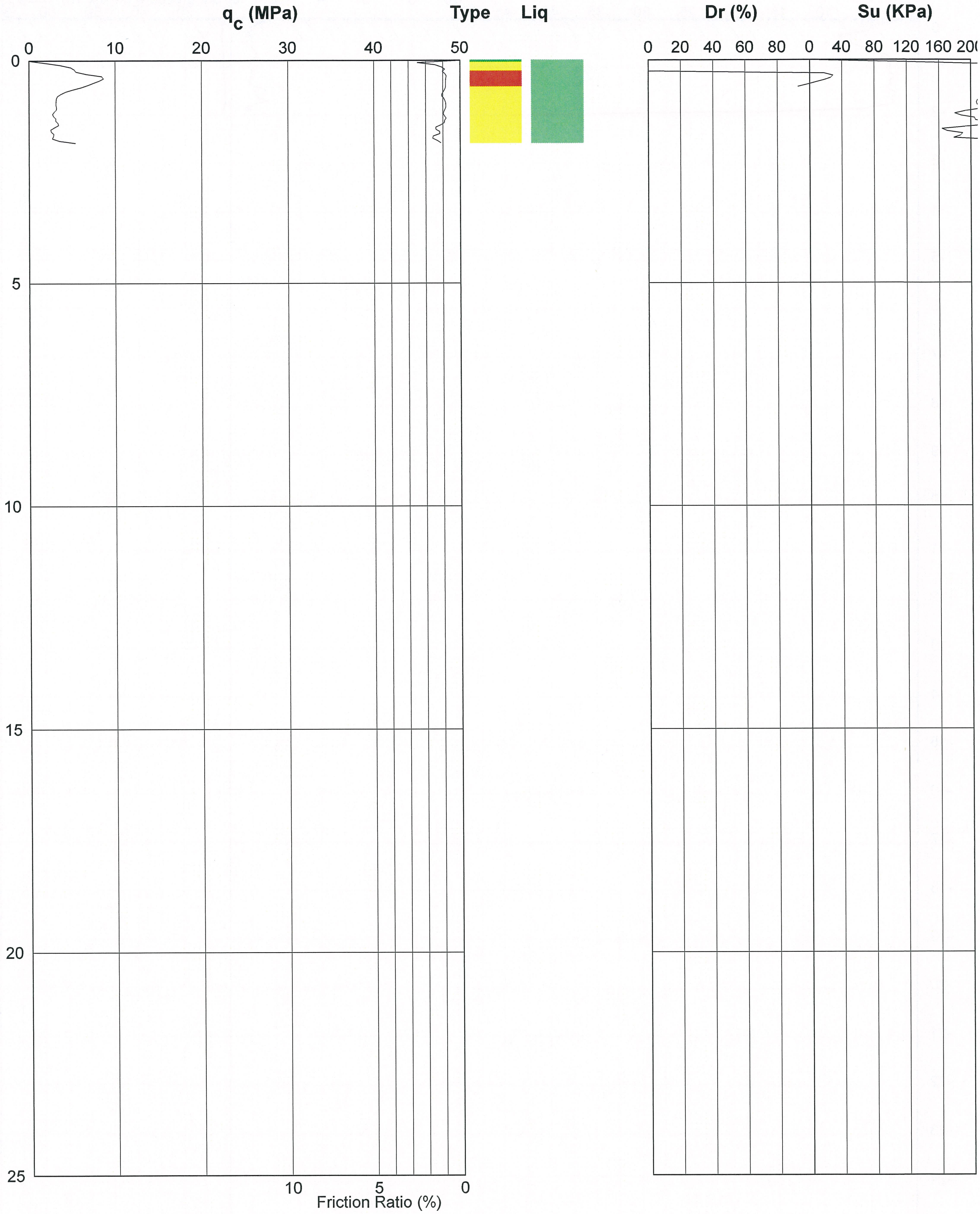


CLIENT : GHD
LOCATION : Christchurch Various (CCC Properties)
DATE : 12-4-2012
OPERATOR : S. Cardona
REMARK 1 : CPTu04
REMARK 2 : Effective Refusal

JOB # : 10386
TEST # : 4

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

PIEZOCONE PENETROMETER TEST (CPTU) INTERPRETIVE REPORT



Job No:	10386	Date:	12-4-2012
CPT No:	CPTu04	Operator:	S. Cardona
Project:	GHD	Remark:	Effective Refusal
Location:	Christchurch Various (CCC Properties)		

McMILLAN DRILLING SERVICES

Appendix B

Photographs

Photograph 1 Two storey concrete masonry extension.

Photograph 2 View of the stand form the south.

Photograph 3 View of the stand from the west.

Photograph 4 Double Tee units supported by reinforced concrete beams.

Photograph 5 Bolting and grouting of Double Tee units.

Photograph 6 Cracking in concrete masonry wall where beam has moved relative to the wall.

Photograph 7 Cracking in the corner between concrete masonry walls in changing rooms.

Photograph 8 Steel girts between steel posts in the roof.

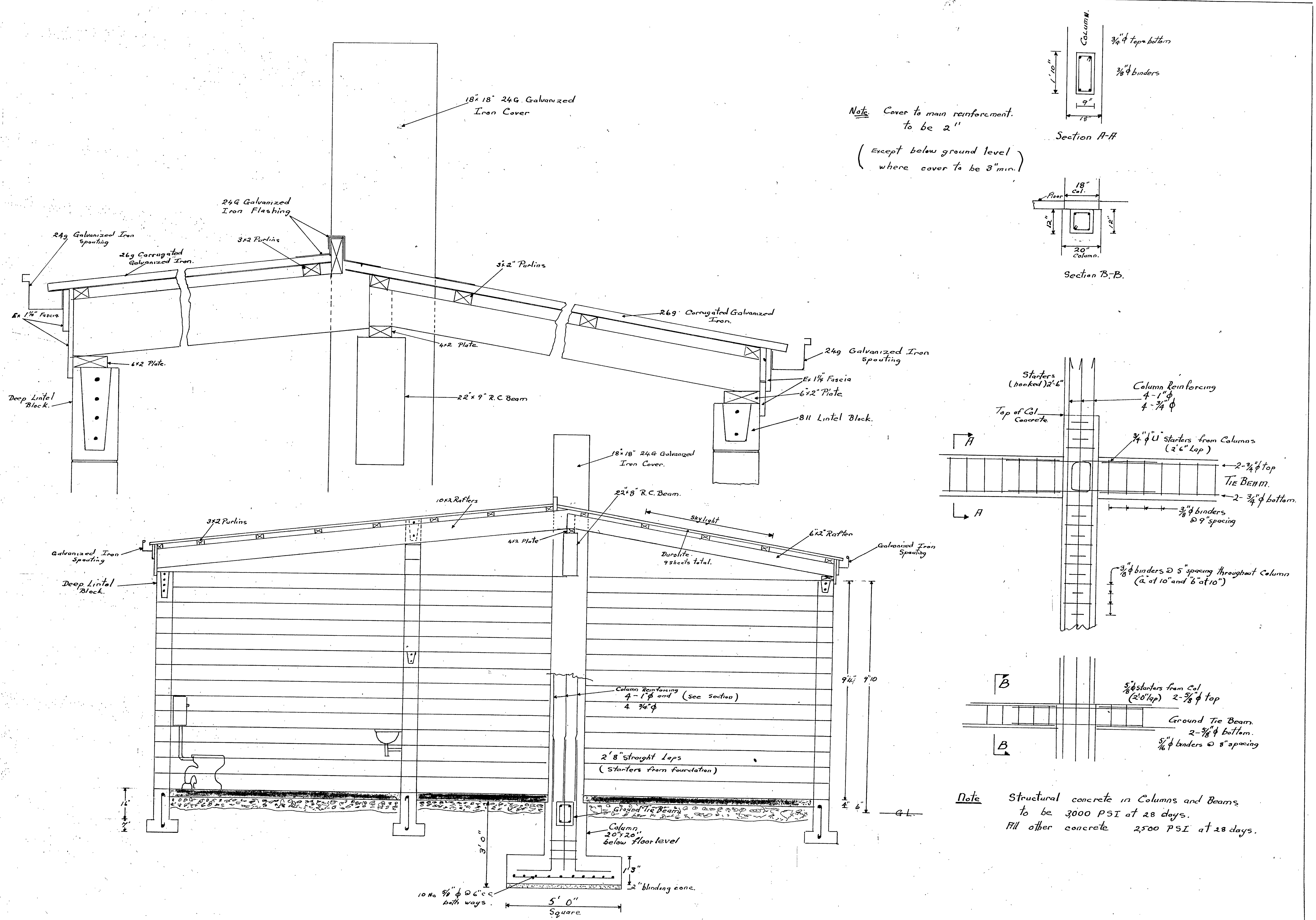
Photograph 9 View of the stand from the north.

Photograph 10 Connection of RHS posts to concrete beams.

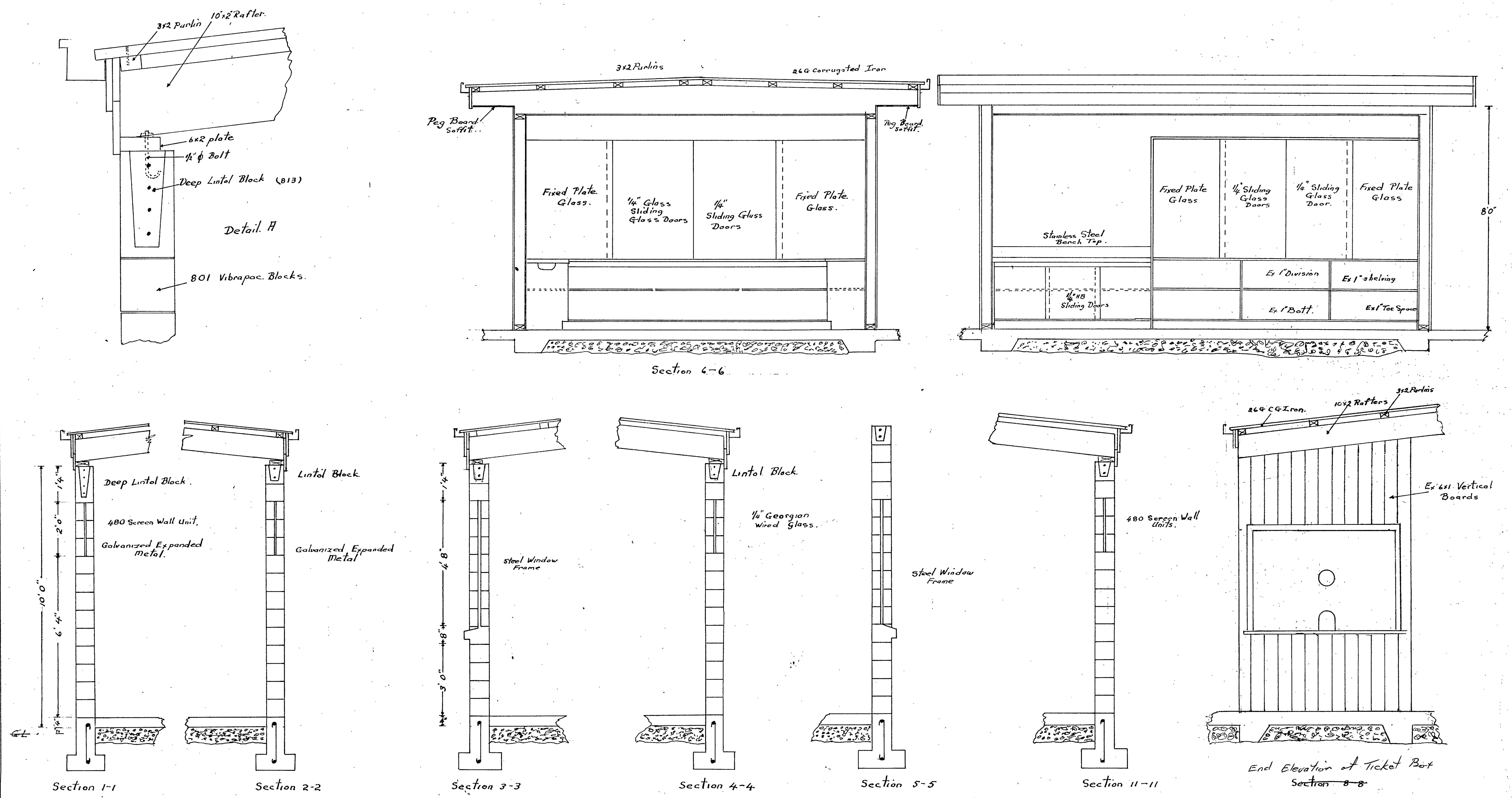
Photograph 11 Beam-column joints in frame running along building. Note the short column between the beams.

Appendix C

Existing Drawings

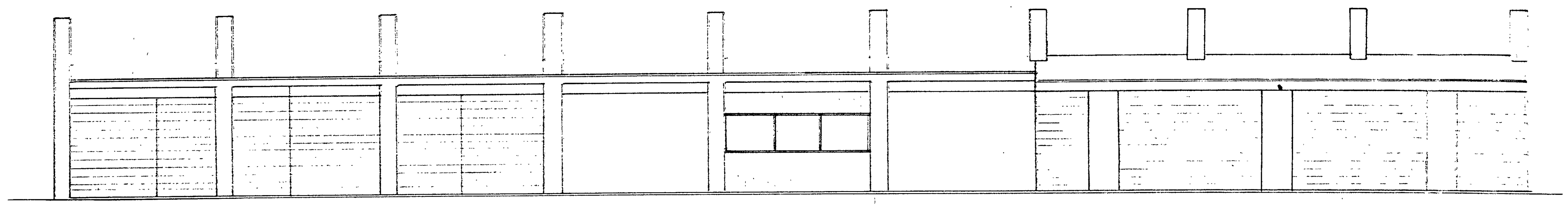


Drawn T.J.B Oct 67 Traced " " Checked	Scales ½" = 1 foot 1/8 Full size.	Proposed Amenity Building Denton Park. for Paparua County Council.	Approved [signature] MNZIE County Engineer	Reference No B25/4

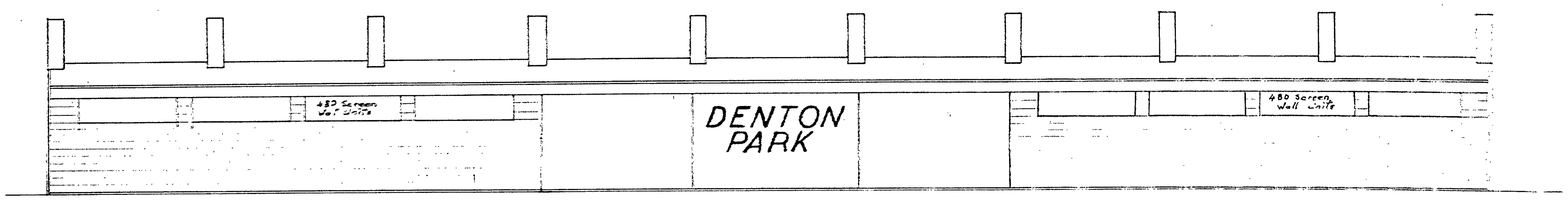


__Note__ All openings, ends of free standing walls and corners to be reinforced vertically with 2/1/2" ϕ Rods.

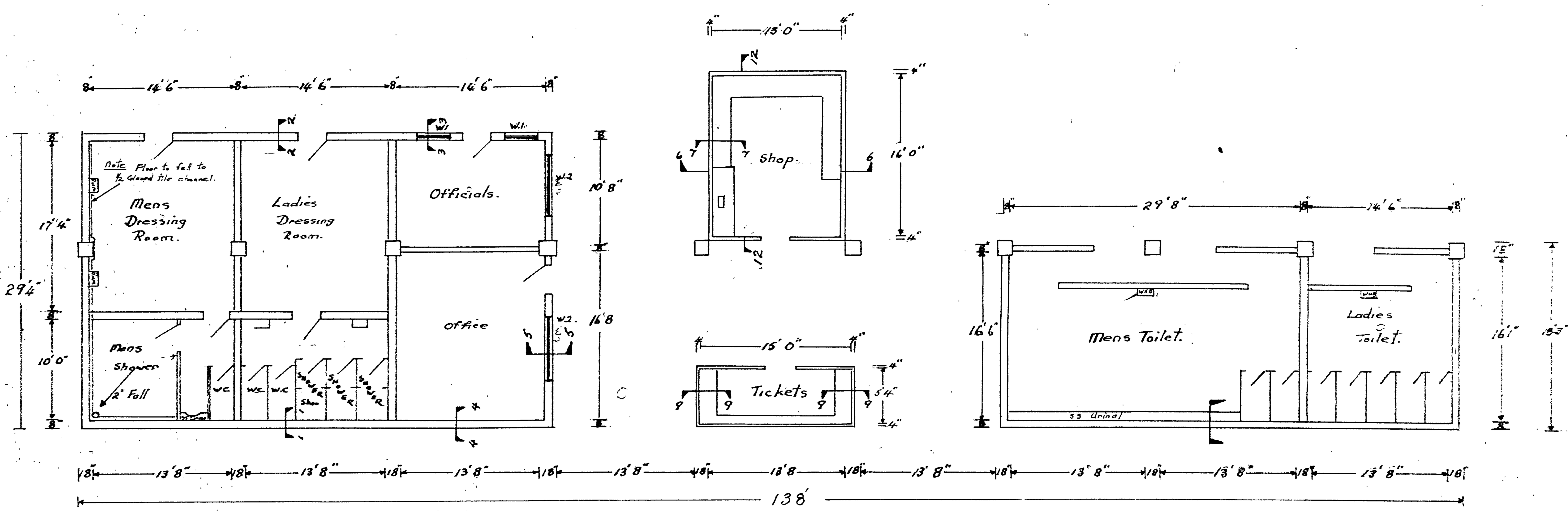
Drawn T.J.B Oct 67 Traced " " Checked	Scale 1/2" = 1foot.	Proposed Amenity Building Denton Park. for Paparua County Council.	Approved Dyhrberg MNZIE County Engineer	Reference No. B25/5



North Elevation.

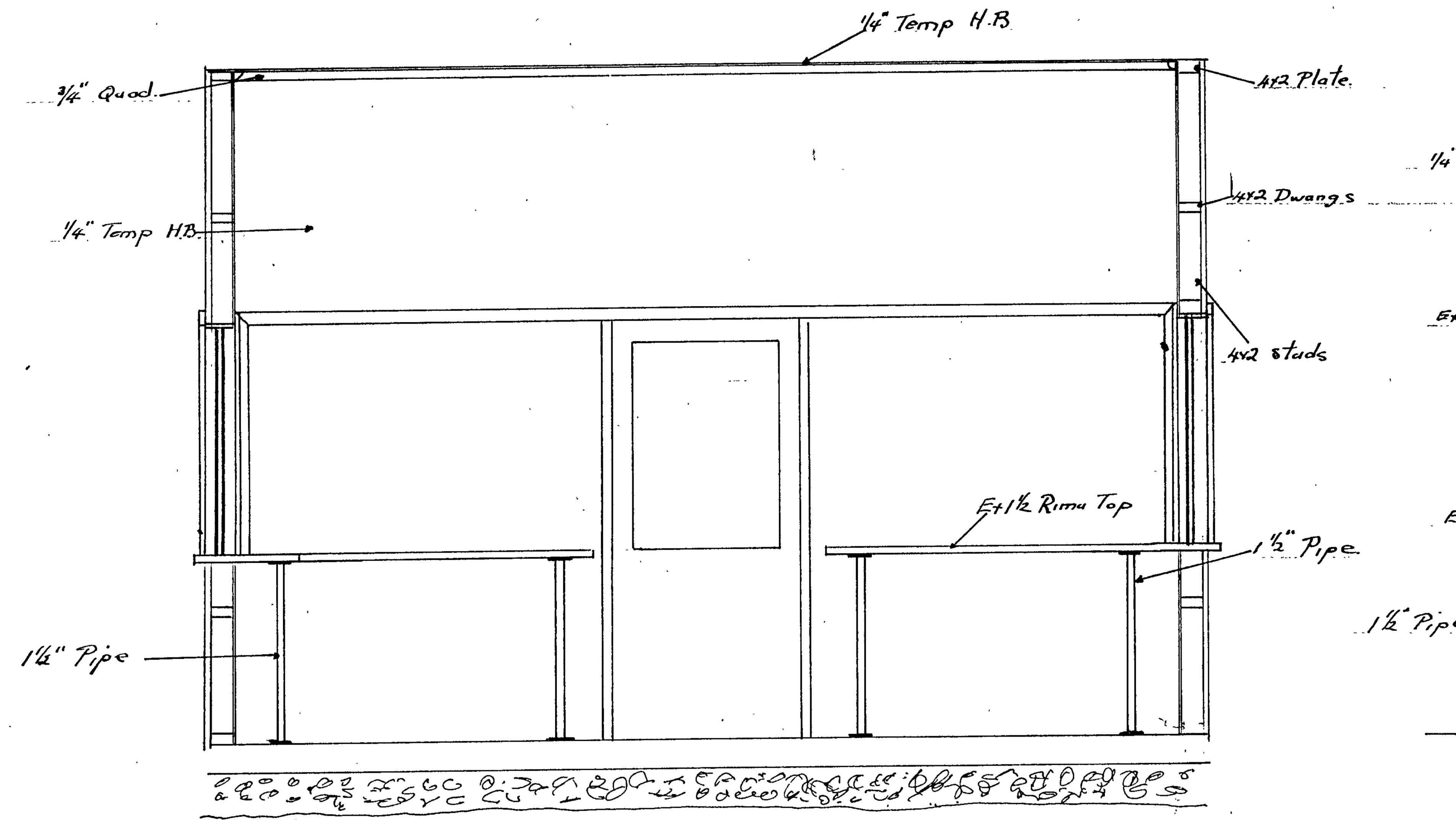


South Elevation.

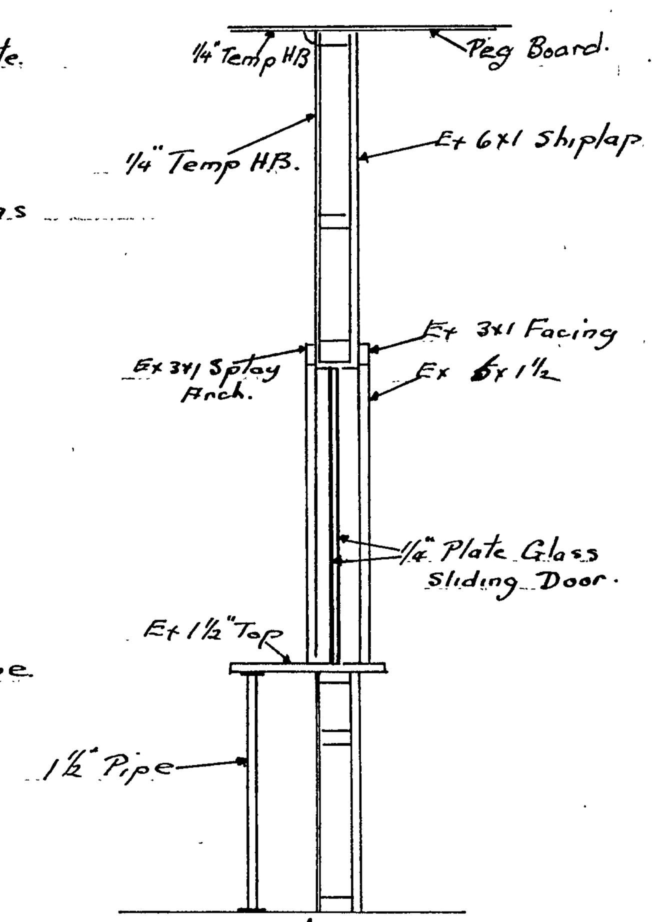


Plan.

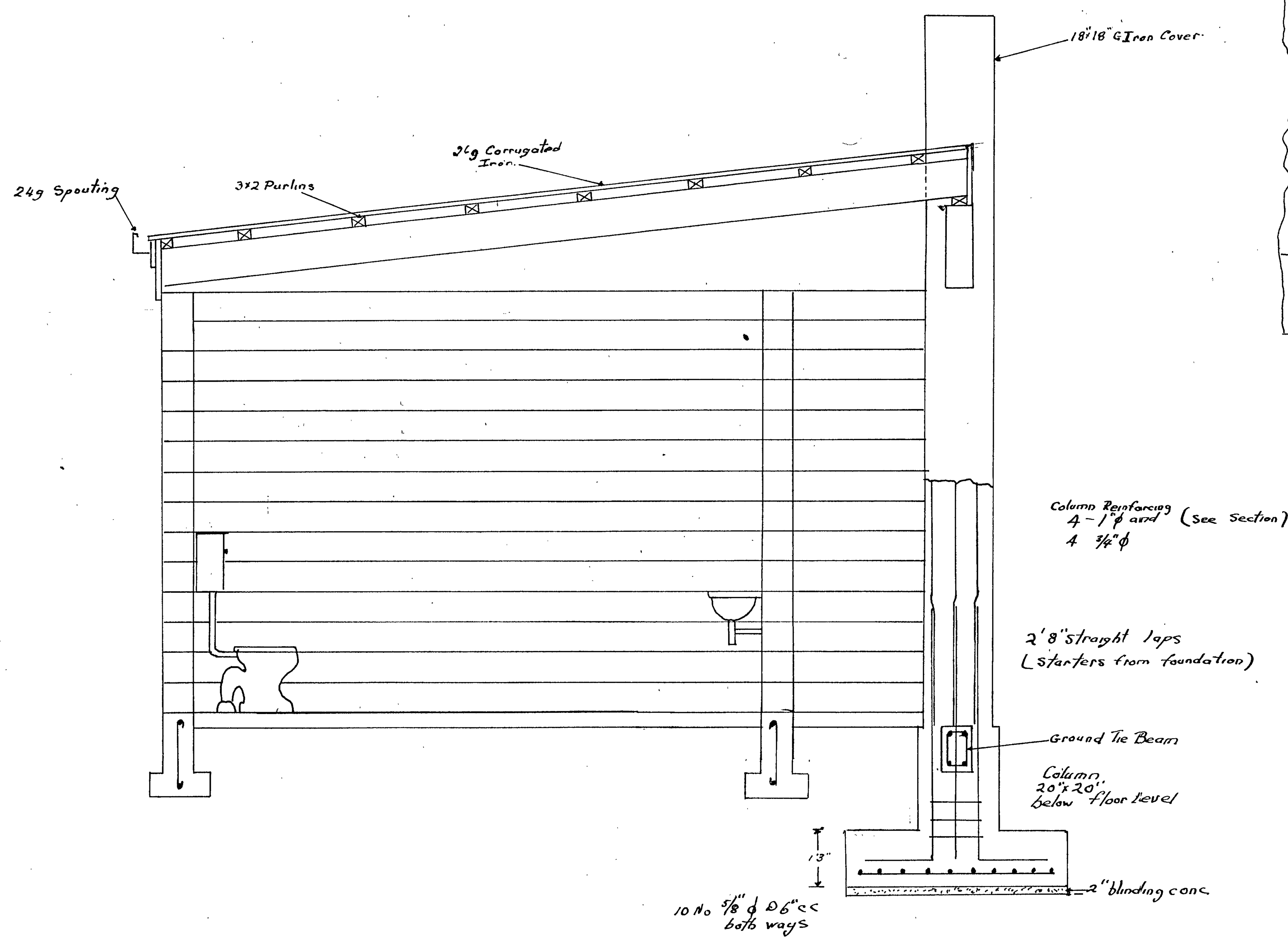
Drawn T.J.B Oct 67 Traced " " Checked	Scale 1/8" = 1 foot.	Proposed Amenity Building for Denton Park. Paparua County Council.	Approved Hjhsberg MNZIE County Engineer	Reference No B 25/2.

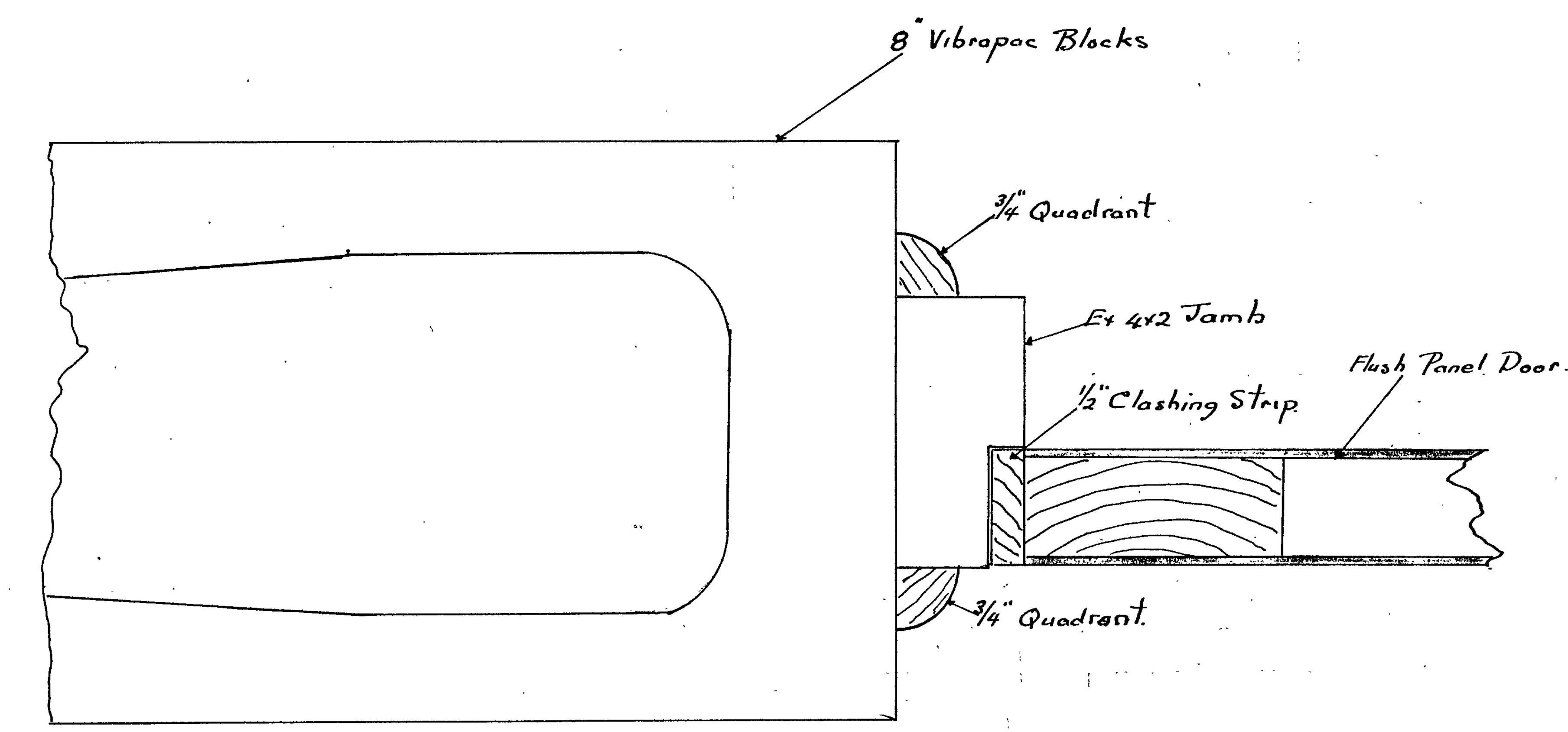


Elevation of Inside of Ticket Box.

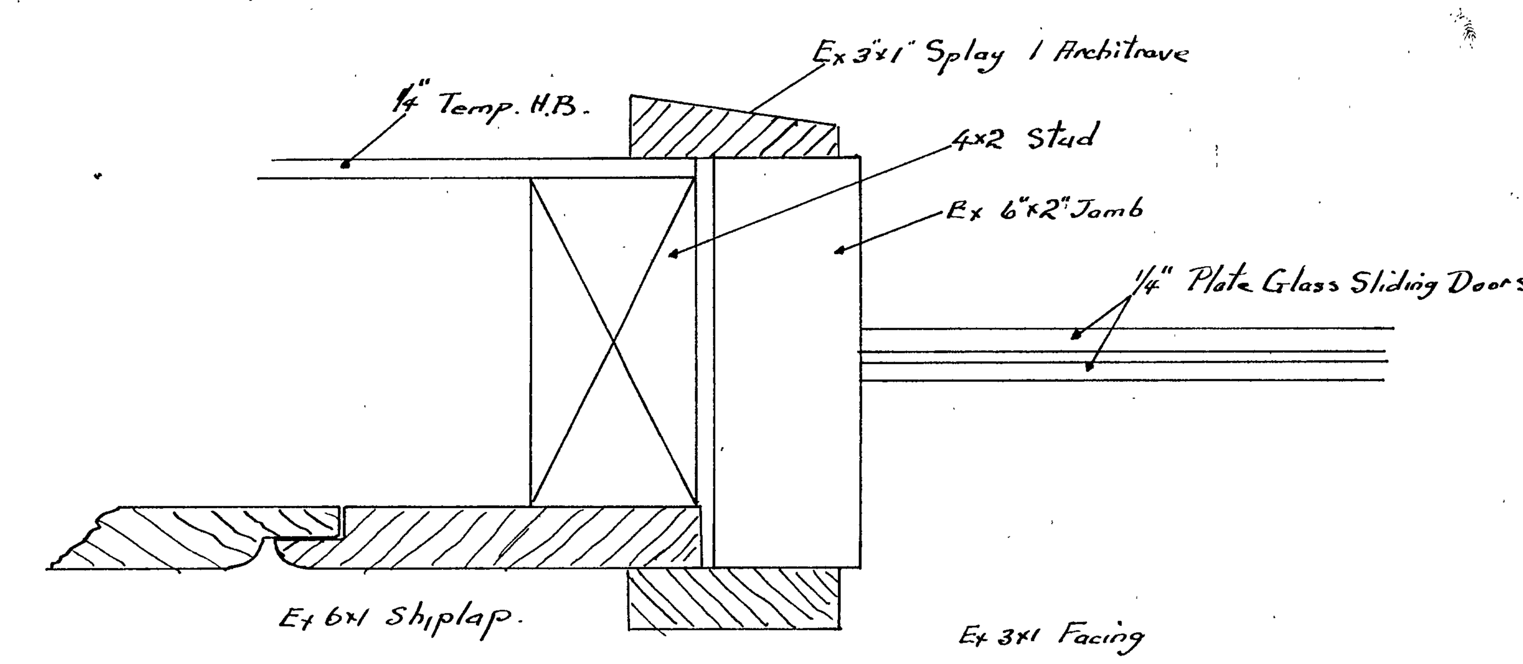


Section 9-9.





1/2 Full Size Section



1/2 Full Size Section of Ticket Box & Shop Walls.

Drawn TJB Oct 67 Traced " " Checked	Scales 1/2" = 1 foot 1/2 Full Size.	Proposed Amenity Building Denton Park. for Paparua County Council.	Approved [signature] MNZIE. County Engineer	Reference No B25/3

Appendix D

CERA Report Forms

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Denton Oval - Stand	Reviewer:	Stephen Lee
	Unit / No: 442 / Street: Main Road South	CPEng No:	1006840
Building Address:	442 Main Road South	Company:	GHD
Legal Description:	RS 41304	Company project number:	51/30596/04
		Company phone number:	04 472 0799
	Degrees / Min / Sec		
GPS south:	43 / 32 / 31.98	Date of submission:	7/03/2013
GPS east:	172 / 31 / 14.11	Inspection Date:	18/1/12
		Revision:	FINAL
Building Unique Identifier (CCC):	BU 0770-003 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:	1	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:	isolated pads, no tie beams		if Foundation type is other, describe:	Foundations are assumed.
Building height (m):	11.00		height from ground to level of uppermost seismic mass (for IEP only) (m):	11
Floor footprint area (approx):	900			
Age of Building (years):	38		Date of design:	1965-1976
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):	Sports stand with changing facilities.			
Importance level (to NZS1170.5):	IL3			

Gravity Structure

Gravity System:	frame system		
Roof:	steel framed	rafter type, purlin type and cladding	
Floors:	precast concrete toppingless	unit type and depth (mm), diaphragm	
Beams:	cast-insitu concrete	overall depth x width (mm x mm)	
Columns:	cast-insitu concrete	typical dimensions (mm x mm)	
Walls:	partially filled concrete masonry	thickness (mm)	

Lateral load resisting structure

Lateral system along:	ductile concrete moment frame	**Note: Define along and across in detailed report!**	note typical bay length (m)	5.3
Ductility assumed, μ:	2.00			
Period along:	0.54	0.54 from parameters in sheet	estimate or calculation?	calculated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	ductile concrete moment frame		note typical bay length (m)	12
Ductility assumed, μ:	2.00			
Period across:	0.54	0.00	estimate or calculation?	calculated
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):	150	
south (mm):		
west (mm):		

Non-structural elements

Stairs:	other (specify)	describe	none
Wall cladding:	exposed structure	describe	
Roof Cladding:	Metal	describe	
Glazing:	other (specify)		none
Ceilings:	none		
Services(list):			

Available documentation

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

Damage

Site:
(refer DEE Table 4-2)

Site performance:	No ground damage noted.	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:	3%	Describe how damage ratio arrived at:	
	Describe (summary):	Minor, non-structural cracking. Less than 5%.		
Across	Damage ratio:	3%		
	Describe (summary):	Minor, non-structural cracking. Less than 5%.		

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

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

Recommendations

Level of repair/strengthening required:	significant structural and strengthening	Describe:	Strengthening to 67% Recommended
Building Consent required:	yes	Describe:	
Interim occupancy recommendations:	do not occupy	Describe:	

Along	Assessed %NBS before:	35%	##### %NBS from IEP below	If IEP not used, please detail assessment methodology:	Qunatitative Assessment
	Assessed %NBS after:	35%			
Across	Assessed %NBS before:	35%	##### %NBS from IEP below		
	Assessed %NBS after:	35%			

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Denton Oval - Masonry Extension	Reviewer:	Stephen Lee
Building Address:	Unit: / No: 442 / Street: Main Road South	CPEng No:	1006840
Legal Description:	RS 41304	Company:	GHD
		Company project number:	51/30596/04
		Company phone number:	04 472 0799
GPS south:	Degrees 43, Min 32, Sec 31.98	Date of submission:	7/03/2013
GPS east:	Degrees 172, Min 31, Sec 14.11	Inspection Date:	18/1/12
		Revision:	FINAL
Building Unique Identifier (CCC):	BU 0770-003 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:	strip footings		if Foundation type is other, describe:	Foundations are assumed.
Building height (m):	6.00		height from ground to level of uppermost seismic mass (for IEP only) (m):	6
Floor footprint area (approx):	300			
Age of Building (years):	25		Date of design:	1976-1992
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):	Sports stand with changing facilities.			
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber framed	rafter type, purlin type and cladding	
Floors:	concrete flat slab	slab thickness (mm)	
Beams:	cast-insitu concrete	overall depth x width (mm x mm)	
Columns:	cast-insitu concrete	typical dimensions (mm x mm)	
Walls:	partially filled concrete masonry	thickness (mm)	

Lateral load resisting structure

Lateral system along:	partially filled CMU	**Note: Define along and across in detailed report!**	note total length of wall at ground (m):	5.3
Ductility assumed, μ:	1.50		wall thickness (m):	
Period along:	0.40	##### enter height above at H31	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	partially filled CMU		note total length of wall at ground (m):	12
Ductility assumed, μ:	1.50		wall thickness (m):	
Period across:	0.40	##### enter height above at H31	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

north (mm):		leave blank if not relevant
east (mm):		
south (mm):		
west (mm):	150	

Non-structural elements

Stairs:	other (specify)	describe	none
Wall cladding:	exposed structure	describe	
Roof Cladding:	Metal	describe	
Glazing:	other (specify)		none
Ceilings:	none		
Services(list):			

Available documentation

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

Damage

Site:
(refer DEE Table 4-2)

Site performance:	No ground damage noted.	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:	4%	Describe how damage ratio arrived at:	
	Describe (summary):	Minor cracking.		
Across	Damage ratio:	4%		
	Describe (summary):	Minor cracking.		
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:	Strengthening to 67% NBS
Building Consent required:	yes	Describe:	
Interim occupancy recommendations:	do not occupy	Describe:	

Along	Assessed %NBS before:	22%	##### %NBS from IEP below	If IEP not used, please detail assessment methodology:	
	Assessed %NBS after:	22%			
Across	Assessed %NBS before:	22%	##### %NBS from IEP below		
	Assessed %NBS after:	22%			

GHD

Level 11, Guardian Trust House
15 Willeston street, Wellington 6011
T: 64 4 472 0799   F: 64 4 472 0833   E: wgtnmail@ghd.com

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Document Status

Rev No.	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
FINAL	Jay Zeus Rivera	Stephen Lee		Nick Waddington		07/03/2013