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Bower Park Pavilion/Toilet
PRK 1307 BLDG 001

Detailed Engineering Evaluation
Quantitative Report
Version FINAL

479 New Brighton Road, Christchurch

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

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479 New Brighton Road,
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Christchurch City Council

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Date
20th September 2013

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Bower Park Pavilion

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Detailed Engineering Evaluation

Quantitative Report - SUMMARY

Version FINAL

479 New Brighton Road

Background

This is a summary of the Quantitative report for the Bower Park Pavilion building located at 479 New Brighton Road, Christchurch, and is based in part on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, visual inspections on the 7th of August 2013, and seismic capacity calculations.

Building Construction

- Roof: timber rafters and purlins cladded with corrugated lightweight metal sheets and plastic skylights;
- Walls: 20 series unreinforced unfilled masonry walls;
- Floor: reinforced concrete on-grade slab;
- Foundation: perimeter concrete strip footings.

Key Damage Observed

Key damage observed includes:

- Cracking to the top of the eastern rear block wall;
- Shrinkage cracking to the concrete floor slab across the building;
- Cracking to concrete floor under the veranda;
- Evidence of ground movement was observed to the pavement outside the building.

Critical Structural Weaknesses

No critical structural weaknesses have been identified when assessing the structure.

Geotechnical Investigation

The geotechnical assessment is based on a review of the geology and existing ground investigation information, and observations from the Christchurch earthquakes since 4 September 2010.

The site is considered to be susceptible to significant liquefaction. A soil class of D (in accordance with NZS 1170.5:2004) should be adopted for the site.

Quantitative Assessment Summary

The overall seismic capacity for the Bower Park Pavilion building assessed in accordance with NZSEE guidelines is **19%** NBS. The rate of 19% NBS represents the out-of-plane seismic capacity of the cantilevered partition walls (as outlined in grey in Figure 2). The in-plane seismic capacity of the building has been assessed as 52% NBS in the along direction and 58% NBS in the across direction. The out-of-plane seismic capacity of the simply supported walls (as outlined in blue in Figure 2) has been assessed as 60% NBS.

Under the New Zealand Society for Earthquake Engineering (NZSEE) guidelines the building is considered High Earthquake Risk as it achieves less than 34% NBS. The results obtained from the seismic capacity assessment are consistent with those expected for a building of this age and construction type, and combined with the increase in the hazard factor for Christchurch to 0.3, it would be expected that the building would not achieve 100% NBS. Also, the out-of-plane capacity achieves less than 67% NBS, which is consistent with expectations for buildings constructed with unreinforced masonry block walls.

Conclusion and Recommendations

The building has been assessed to have a seismic capacity in the order of **19%** NBS and is therefore deemed to be a High Earthquake Risk Building in accordance with the NZSEE guidelines.

The recent seismic activity in Christchurch has only caused minor damage to the building, with minor cracking in the concrete blockwork masonry walls the only damage noted. As the building suffered insignificant damage that does not compromise the load resisting capacity of the existing structural systems, and has a calculated seismic capacity of less than 34% NBS following a Quantitative Detailed Engineering Evaluation, no further assessment is required. The building should be strengthened to minimum of 34% NBS to comply with Christchurch City Council's "Earthquake Prone, Dangerous and Insanitary Buildings Policy (2010)". However, GHD recommends strengthening options to the blockwork walls should be explored and implemented to bring the %NBS of the building to a minimum of 67% as recommended by the NZSEE guidelines.

All remedial work for damage outlined in Section 5 of this report shall be carried out in accordance with the repair methods given in Section 6.

1. Background

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

This report is a Quantitative Assessment and is based in general on NZS 1170.5: 2004, 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) with the recent supplement from the University of Auckland (05/2013).

This quantitative assessment to the building comprises of an investigation of the in-plane and out-of-plane strengths of the unreinforced masonry block walls. 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 the existing structural elements to resist the seismic forces applied to them. 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 block masonry walls.

At the time of this report, no finite element modelling of the building structure has been carried out.

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; and
- 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).

Table 1: %NBS compared to relative risk of failure

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

4. Building Description

4.1 General

The building is a single-storey rectangular structure, located within Bower Park at 479 New Brighton Road, Christchurch. The date of construction is unknown; however it appears consistent with building construction in 1960s. The building is currently used as a pavilion, including toilets and changing rooms. No alterations to the original structure were obvious during the site inspection.

The building measures approximately 15 m long by 7 m wide by 3.4 m high. It is rectangular in plan, with a gross floor area of approximately 105 m^2. The veranda along the northern side extends 1.8 m out from the building, as shown in Figure 2.

No plans or drawings for the building were made available. This assessment is based on observations, measurements and reinforcing scans from the site inspection.

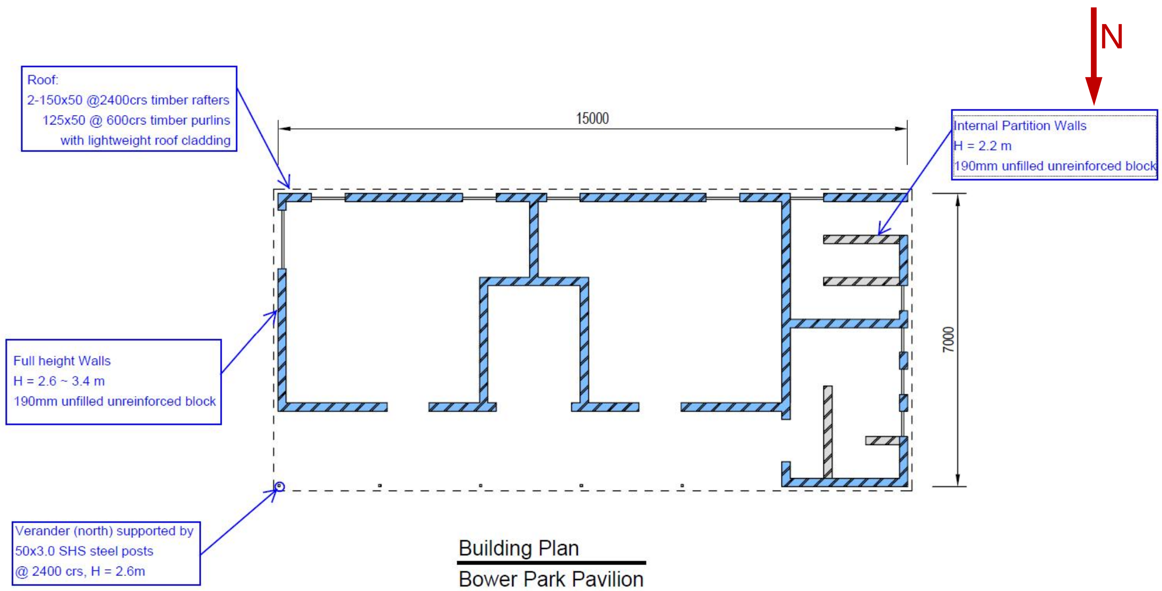

Figure 2: Plan Sketch Showing Key Structural Elements

The roof comprises 2-150x50 built-up timber rafters spanning across the building at approximately 2400 mm centres, clad with lightweight corrugated metal sheets and fibreglass skylight panels on 125x50 timber purlins at 600 mm centres typical.

The walls are predominantly 190 mm thick blockwork masonry, which form the perimeter and lateral load-resisting system of the building. There are a few 190mm thick blockwork masonry walls, 2.2 m cantilevered, forming the partitions in the toilet areas.

Electromagnetic scans have been carried out on site to ascertain the extent of the reinforcement in the block masonry walls. Reinforcing was found at bond beams, each end of walls and the intersection of walls. Some reinforcing bars were also found at edges of all openings. The remaining portion of the wall was found to be unreinforced and appeared to be unfilled.

The floor of the building is a concrete slab on grade, approximately 150 mm above surrounding ground level. Electromagnetic scans detected 6 mm diameter bars at 150 mm centres each way, which are inferred to be 665 mesh.

Perimeter foundations are inferred to be present underneath the blockwork perimeter walls; the thickness of floor and perimeter foundation details could not be confirmed.

The veranda structure is constructed with timber purlins on timber rafters spanning between the blockwork walls and 50x3.0 SHS steel posts. The posts are embedded into concrete foundation.

4.2 Gravity Load Resisting System

The gravity load resisting system of the building consists of external loadbearing masonry walls founded on concrete strip footings and supporting the timber roof rafters and purlins with lightweight metal roof cladding.

4.3 Lateral Load Resisting System

Lateral loads in both the longitudinal and the transverse directions are resisted by the unreinforced masonry walls through in-plane action.

5. Damage Assessment

An inspection of the building was undertaken on 8 August 2013. Both the interior and exterior of the building were inspected. The main structural components of the roof of the building were able to be viewed as most of them are exposed. Foundations were unable to be viewed due to inaccessibility.

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

No level or verticality surveys have been undertaken for this building at this stage as indicated by Christchurch City Council guidelines.

A Hilti PS 200 Ferroscan was used to determine the position, depth and diameter of the reinforcement in the blockwork masonry structure. This scanning equipment uses electro-magnetic fields to determine the size and depth of the reinforcing steel in the building. In the case of conflicting results, the most conservative bar diameter has been chosen for the capacity calculations.

5.1 Surrounding Buildings

Moderate signs of liquefaction were observed on the nearby streets and properties.

5.2 Residual Displacements and General Observations

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

Key damage observed included:

- Cracking to the top of the eastern rear block wall;
- Shrinkage cracking to the concrete floor slab across the building;
- Cracking to concrete floor under the veranda;
- Evidence of ground movement was observed to the pavement outside the building.

This damage observed is not considered to have a significant impact on the seismic capacity of the building. Refer photographs of the damge in Appendix A.

5.3 Ground Damage

Evidence of ground movement was observed to the pavement outside the building.

6. Building Repair Options

6.1 On-grade slab cracking

We recommend that an epoxy injection system (e.g. Sika Sikadur Injectokit TH or similar approved) be used for the repair of the slab cracking greater than 0.2mm in width. It should be noted that epoxy injection cannot be carried out where crack widths are less than 0.2mm in width. Repairs should be carried out in accordance with manufacturer’s instructions.

6.2 Cracking to masonry walls

The repairs to the damaged masonry blockwork can be described as non-structural. The repairs carried out will vary in extend. In some instances localized raking/grinding out of mortar and reinstatement in accordance with good trade practice will be appropriate.

It is recommended that a pourable high flow cementitious grout (e.g. Sika Grout GP or Sika Grout 212 or approved equivalent) be used for repairing for these damages.

7. Critical Structural Weakness

Short Columns

No short columns are present in the structure.

Lift Shaft

The building does not contain a lift shaft.

Roof

No bracing elements were observed in the exposed roof structure. However, as the roof structure is lightweight, roof elements such as timber sheathing, purlins and trusses are expected to provide some degree of bracing to the roof structure. Therefore, lack of roof bracing is not considered to lead to a critical structural weakness.

Staircases

The building does not contain a staircase.

Site Characteristics

Refer to geotechnical consideration in Section 7.

Plan Irregularity

The building is rectangular; no plan irregularity is present.

Vertical irregularity

The building is single-storey, with a constant ceiling height; no vertical irregularity is present.

Pounding effect

No adjacent buildings; pounding is not applicable.

Height difference offset

Not applicable.

8. Geotechnical Consideration

This desktop geotechnical study outlines the ground conditions, as indicated from sources quoted within, for inclusion in the subject structure's DEE Quantitative Assessment. This desktop study report includes observations from a site walkover undertaken by GHD Geotechnical personnel.

This report is specific to the Bower Park Pavilion at Bower Park. The site is surrounded by residential properties, and is owned by the Christchurch City Council.

8.1 Site Description

The site is situated in the suburb of New Brighton, in east Christchurch. It is relatively flat at approximately 1 m above mean sea level. It is approximately 80 m northeast of the Avon River, and 2 km west of the coast (Pegasus Bay).

8.2 Published Information on Ground Conditions

8.2.1 Published Geology

Brown & Weeber, 1992[1] describes the site geology as:

- Dominantly alluvial sand and silt overbank deposits, being alluvial soils of the Yaldhurst Member, sub-group of the Springston Formation, Holocene in age;
- Underlying sediments (younger than 6500 years) are surface alluvial silt and sand, subsurface marine sand and alluvial silt and sand, and some peat. No interbedded gravel;
- The Riccarton gravels are located approximately 35 m bgl; and
- Groundwater is likely within 1 m of ground level.

8.2.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that five boreholes with lithographic logs are located within 200 m of the site. Four ECan borehole logs have been summarised in Table 2.

These indicate the area is underlain by interbedded layers of sand, silt and clay with some peat.

Groundwater was recorded between 1.5 m and 3.0 m bgl in the borehole logs.

Table 2 ECan Borehole Summary

Bore Name	Log Depth	Groundwater	From Site	Log Summary	
M35/12112	6.1 m	1.8 m	200 m S	0.0 to 1.2 m	Sand
				1.2 to 1.8 m	Silt
				1.8 to 4.9 m	Sandy silt
				4.9 to 6.1 m	Sand

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

Bore Name	Log Depth	Groundwater	From Site	Log Summary	
M35/12260	3.0 m	N/A	150 m E	0.0 to 0.3 m	Fill
				0.3 to 1.0 m	Sand
				1.0 to 1.4 m	Sand with trace peat
				1.4 to 1.8 m	Clay
				1.8 to 3.0 m	Sand
M35/12261	4.2 m	3.0 m	150 m SE	0.0 to 0.6 m	Topsoil and silt
				0.6 to 1.0 m	Clay
				1.0 to 1.5 m	Clay with some peat
				1.5 to 2.2 m	Clay and sandy silt
				2.2 to 3.0 m	Clay
				3.0 to 4.0 m	Sand
				4.0 to 4.2 m	Clay
M35/12761	9.1 m	1.5 m	180 m SW	0.0 to 0.3 m	Sandy silt
				0.3 to 1.8 m	Silty sand
				1.8 to 9.1 m	Sand

It should be noted that the logs may have been written by the well driller and not a geotechnical professional or to a standard. In addition strength data is not recorded.

8.2.3 EQC Geotechnical Investigations

The Earthquake Commission has undertaken geotechnical testing in the area of the site. Information pertaining to this investigation is included in the Tonkin & Taylor Report for New Brighton[2]. One investigation point was undertaken within 200 m of the site, as summarised below in Table 3.

Table 3 EQC Geotechnical Investigation Summary Table

Bore Name	Orientation from Site	Depth (m bgl)	Log Summary[3]
CPT-NBT-05	110 m NW	0.0 – 1.2	Pre-drilled
		1.2 – 1.6	Silty CLAY; firm
		1.6 – 6.7	Silty SAND; loose
		6.7 – 18.1	SAND; medium dense to dense
			(WT at 2.0 m bgl)

Initial observations of the CPT results indicate the site is underlain by silty clay to 1.6 m bgl, underlain by loose, silty sand to 6.7 m bgl, underlain by medium dense to dense sand to 18.1 m bgl. The occasional silty clay layer is also present below 16 m bgl.

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

[3] Log Summary for CPT's interpreted from Soil Behavior Type Robertson *et al*. 2010

8.2.4 CERA Land Zoning

Canterbury Earthquake Recovery Authority (CERA) has indicated the site is situated in the Green Zone, indicating that repair and rebuild may take place. However, neighbouring residential properties have been red zoned.

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

The site has been categorised as "N/A" – Urban Non-residential"[4]. However, neighbouring residential properties have either been categorised as "Red zone", indicating that land repair would be prolonged and uneconomic, and TC3 (blue), indicating moderate to severe land damage from liquefaction is possible in future significant earthquakes.

8.2.5 Historic Land Use

The Listed Land Use Register (LLUR)[5] indicates that no hazardous activities have occurred at the site.

The Black Maps[6] shows that the area was historically "swamp".

The CCC historic landfill map[7] shows that no historic landfills exist in the vicinity of the site.

Historical aerial photography shows that the site was previously farm land (1946 and 1955).

8.2.6 Post-Earthquake Land Observations

Aerial photography[8] taken following the 22 February 2011 earthquake shows moderate signs of liquefaction in Bower Park, in the carpark adjacent to the site, and on nearby streets, as shown in Figure 3. No new signs of liquefaction occurred following the 13 June 2011 and the 23 December 2011 earthquakes. Aerial photography taken following the 4 September 2010 earthquake shows minor signs of liquefaction in the adjacent carpark to the east.

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[4] CERA Landcheck website, http://cera.govt.nz/my-property

[5] Environmental Canterbury Regional Council: *Listed Land Use Register,* retrieved 16/07/2013 from http://llur.ecan.govt.nz/

[6] Waterways, Swamps and Vegetation Cover in 1856 Compiled from "Black Maps", Source: Christchurch City Council retrieved 20 September 2013, http://resources.ccc.govt.nz/files/blackmap-environmentecology.pdf

[7] Christchurch City Council (1993): *Christchurch Landfill Sites* and accompanying key *Old Landfills within Christchurch*.

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

Figure 3 Post February 2011 Earthquake Aerial Photography

The Canterbury Geotechnical database shows cracks between 10 mm and 50 mm occurred within 100 m of the site[9].

8.2.7 Summary of Ground Conditions

From the information presented above, the ground conditions underlying the site are anticipated to comprise interbedded layers of sand, silt and clay to 1.2 m bgl, underlain by silty clay to 1.6 m bgl, underlain by loose, silty sand to 6.7 m bgl, underlain by medium dense to dense sand to 18.1 m bgl, with varying amounts of clay and peat.

Groundwater is considered to vary between 1.5 m and 3.0 m bgl.

8.3 Seismicity

8.3.1 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.

[9] Canterbury Geotechnical Database (2012) "Observed Ground Crack Locations", Map Layer CGD0400 - 23 July 2012, retrieved [20/09/2013] from https://canterburygeotechnicaldatabase.projectorbit.com/

Table 4 Summary of Known Active Faults[10,11]

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	130 km	NW	~8.3	~300 years
Greendale Fault (2010)	28 km	W	7.1	~15,000 years
Hope Fault	100 km	N	7.2~7.5	120~200 years
Porters Pass Fault	65 km	NW	7.0	~1100 years
Port Hills Fault (2011)	9 km	S	6.3	Not Estimated

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

8.3.2 Ground Shaking Hazard

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.

The recent seismic activity has produced earthquakes of Magnitude 6.3 with significant peak ground accelerations (PGA) across large parts of the city.

Conditional PGA's from the CGD[12] indicate the PGA to be 0.17 g during the 4 September 2010 earthquake, 0.39 g on 22 February 2011, and 0.25 g on 13 June 2011.

8.4 Global Land Movement

Given the site's proximity to the Avon River, and evidence from the recent earthquakes, the site may be susceptible to lateral spreading. In addition, any retaining structures or embankments nearby should be further investigated to determine the site-specific local slope instability potential.

According to table 12.3 in section 12.2.1 of the MBIE guidance[13], the site may be assumed to be susceptible to major global lateral spread, as the edge of building is within 200 m of the bank of the Avon River.

[10] 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, June 2002, pp. 1878-1903.

[11] GNS Active Faults Database, http://maps.gns.cri.nz/website/af/viewer

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

[13] Ministry of Business, Innovation & Employment – Building & Housing (2012): Repairing and Rebuilding Houses affected by the Canterbury Earthquakes; Version 3, Dec 2012. MBIE: Wellington, NZ.

8.5 Liquefaction Potential

The site is considered to be susceptible to significant liquefaction, due to:

- Moderate signs of liquefaction observed following the 22 February 2011 earthquake on nearby streets and properties;
- Neighbouring properties categorised as being TC3 or in the Red Zone;
- Nearby CPT investigations indicate loose silty sand is present to 6.7 m bgl;
- A shallow water table between 1.5 m and 3.0 m bgl.

Further investigation is recommended to better determine subsoil conditions. From this, a more comprehensive liquefaction assessment could be undertaken.

8.6 “Sufficiently Tested at SLS”

Site observations of recent earthquake damage can be correlated to the likely performance of the site at serviceability limit state (SLS) by comparing the PGA observed with design values. This methodology is outlined in the MBIE guidance on Liquefaction Methodology .

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

8.7 Summary & Recommendations

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.

The site appears to be situated on interbedded layers of sand, silt and clay to 1.6 m bgl, underlain by loose, silty sand to 6.7 m bgl, underlain by medium dense to dense sand to 18.1 m bgl, with varying amounts of clay and peat. Associated with this, the site also has a severe liquefaction potential, in particular where sands and/or silts are present.

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

Should a more comprehensive liquefaction and/or ground condition assessment be required, it is recommended that intrusive investigation be conducted.

9. Seismic Capacity Assessment

9.1 Seismic Parameters

The seismic design parameters based on current design requirements from NZS 1170:2002 and the NZBC clause B1 for this building are:

- Site soil class: D
  (NZS 1170.5:2004, Clause 3.1.3, "Deep or Soft Soil");
- Site hazard factor, Z = 0.3
  (NZBC, Clause B1 Structure, Amendment 11 effective from 1 August 2011);
- Return period factor R_u = 1.0
  (NZS 1170.5:2004, Table 3.5: Importance Level 2, 50 year design life).

An increased Z factor of 0.3 for Christchurch has been used in line with requirements from the Department of Building and Housing resulting in a reduced % NBS score.

9.1.1 Expected Structural Ductility Factor

In accordance with NZSEE Guideline (02/2011), a displacement ductility of 2 is used in the in-plane assessment of unreinforced masonry elements. This corresponds to a structural performance factor S_p of 0.7 and k_μ is 1.2 as recommended in the guideline.

9.1.2 Material strength

The masonry unit is assumed to be Observation type B with a compressive strength of 12 MPa in accordance with NZS 4230:2004.

Average masonry compressive strength, f'_m is calculated using the average masonry compressive strength and average mortar compressive strength in accordance with NZSEE Guideline (02/2011).The compressive strength of masonry was assumed to be 9.9 MPa which was determined as follows:

$$f'_m = 0.7 f'^{0.75}_b f'^{0.3}_j$$

Where,

f'_b = 21.5 MPa, hardness of masonry assumed to be medium,

f'_j = 3.2 MPa, hardness of mortar assumed to be medium.

9.2 Quantitative Assessment Procedure

A quantitative assessment of the building was carried out using the information gathered from a full site measure of the building on the 7th August 2013. From this information, the building's seismic capacity was determined in accordance with New Zealand Society for Earthquake Engineering (NZSEE) guidelines with the recent supplement from the University of Auckland (05/2013). The seismic demand for the building was calculated in accordance with NZS 1170.5:2004 and the percentage of New Building Standard (%NBS) was assessed.

The lateral load resisting system of the building was modelled as in-plane and out-of-plane shear walls. For further details on the assessment, please refer to Appendix C.

9.2.1 %NBS

The in-plane capacity of the walls and the out of plane moment capacities were then compared to their respective demands to assess which was the most critical and thus determine the overall %NBS for the structure as follows:

$$\%NBS = \frac{\emptyset S_n\ (\text{Capacity})}{S^*\ (\text{Demand})} \times 100\%$$

9.3 % NBS Assessment

The overall seismic capacity for the Bower Park Pavilion building assessed in accordance with NZSEE guidelines is **19%** NBS. The rate of 19% NBS represents the out-of-plane seismic capacity of the cantilevered partition walls (as outlined in grey in Figure 2). The in-plane seismic capacity of the building has been assessed as 52% NBS in the along direction and 58% NBS in the across direction. The out-of-plane seismic capacity of the simply supported walls (as outlined in blue in Figure 2) has been assessed as 60% NBS.

Table 5: Indicative Building Seismic Capacities based on NZS 1170.5:2004 and NZSEE Guidelines

Element / Direction Assessed	%NBS Achieved
Along direction – in-plane shear	52%
Across direction – in-plane shear	58%
190mm simply supported block wall – out-of-plane	60%
190mm cantilevered partition wall – out-of-plane	19%
Critical %NBS for building	**19% NBS**

The overall seismic capacity for the Bower Park Pavilion building assessed in accordance with NZSEE guidelines is **19%** NBS.

This score has not been adjusted when considering damage to the structure as all damage observed was relatively minor and considered unlikely to adversely affect the load carrying capacity of the structural systems.

9.4 Discussion of Results

Under the New Zealand Society for Earthquake Engineering (NZSEE) guidelines the building is considered High Earthquake Risk as it achieves less than 34% NBS. The results obtained from the seismic capacity assessment are consistent with those expected for a building of this age and construction type, and combined with the increase in the hazard factor for Christchurch to 0.3, it would be expected that the building would not achieve 100% NBS. Also, the out-of-plane capacity achieves less than 67% NBS, which is consistent with expectations for buildings constructed with unreinforced masonry block walls.

10. Conclusions and Recommendations

The building has been assessed to have a seismic capacity in the order of **19%** NBS and is therefore deemed to be a High Earthquake Risk Building in accordance with the NZSEE guidelines.

The recent seismic activity in Christchurch has only caused minor damage to the building, with minor cracking in the concrete blockwork masonry walls the only damage noted. As the building suffered insignificant damage that does not compromise the load resisting capacity of the existing structural systems, and has a calculated seismic capacity of less than 34% NBS following a Quantitative Detailed Engineering Evaluation, no further assessment is required. The building should be strengthened to minimum of 34% NBS to comply with Christchurch City Council's "Earthquake Prone, Dangerous and Insanitary Buildings Policy (2010)". However, GHD recommends strengthening options to the blockwork walls should be explored and implemented to bring the %NBS of the building to a minimum of 67% as recommended by the NZSEE guidelines.

All remedial work for damage outlined in Section 5 of this report shall be carried out in accordance with the repair methods given in Section 6.

11. Limitations

11.1 General

This report has been prepared subject to the following limitations:

- No intrusive structural investigations have been undertaken;
- No intrusive geotechnical investigations have been undertaken;
- Visual inspections of the roof space were limited to the vicinity of the access hatch and due to its non-central location, the entirety of the roof space could not be inspected visually;
- No level or verticality surveys have been undertaken;
- No material testing has been undertaken; and
- No modelling of the building for structural analysis purposes has been performed.

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

11.2 Geotechnical Limitations

This report presents the results of a geotechnical appraisal prepared for the purpose of this commission, and for prepared solely for the use of Christchurch City Council and their advisors. 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.

The advice tendered in this report is based on a visual geotechnical appraisal. No subsurface investigations have been conducted. An assessment of the topographical land features have been made based on this information. It is emphasised that Geotechnical conditions may vary substantially across the site from where observations have been made. Subsurface conditions, including groundwater levels can change in a limited distance or time. In evaluation of this report cognisance should be taken of the limitations of this type of investigation.

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

Appendix A

Photographs

Photograph 1: North elevation (front)

Photograph 2: South elevation

Photograph 3: East elevation

Photograph 4: West elevation

Photograph 5: Cracking to the top of the external blockwork walls on the eastern side

Photograph 6: Veranda canopy

Photograph 7: Cracking to the concrete pavement under veranda canopy

Photograph 8: As above

Photograph 9: Typical shrinkage cracks to the cocnrete floor slab thoughout the building

Photograph 10: Damage to the pavement outside the building

Photograph 11: Typical interior view

Photograph 12: As above

Appendix B

CERA Building Evaluation Form

Detailed Engineering Evaluation Summary Data V1.14

Location

Building Name:	Bower Park Pavilion			Reviewer:	David Lee
	Unit	No:	Street	CPEng No:	112052
Building Address:		479	New Brighton Road	Company:	GHD Ltd
Legal Description:	RES 4670			Company project number:	51/31526/29
				Company phone number:	64 03 378 0000
	Degrees	Min	Sec		
GPS south:	43	30	11.00	Date of submission:	8/27/2013
GPS east:	172	42	29.00	Inspection Date:	8/7/2013
				Revision:	FINAL
Building Unique Identifier (CCC):	PRK 1307 BLDG 001			Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	sandy silt	Soil Profile (if available):	
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):	80	If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	1.00

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):	1.15
Ground floor split?	yes		Ground floor elevation above ground (m):	0.15
Storeys below ground	0			
Foundation type:	strip footings		if Foundation type is other, describe:	
Building height (m):	2.60		height from ground to level of uppermost seismic mass (for IEP only) (m):	3.4
Floor footprint area (approx):	105			
Age of Building (years):	50		Date of design:	1935-1965
Strengthening present?			If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):	public		Brief strengthening description:	
Use (upper floors):				
Use notes (if required):	pavilion			
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber framed	rafter type, purlin type and cladding	2-150x50 bult-up timber rafters @ 2400crs and 125x50 timber purlins @600crs with corrugated lightweight metal sheets
Floors:	concrete flat slab	slab thickness (mm)	unknown
Beams:			
Columns:			
Walls:			

Lateral load resisting structure

Lateral system along:	other (note)		Note: Define along and across in detailed report!	describe system	URM - block
Ductility assumed, μ:	2.00				
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)			describe system	URM - block
Ductility assumed, μ:	2.00				
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:			
Wall cladding:	exposed structure	describe	URM walls
Roof Cladding:	Metal	describe	
Glazing:			
Ceilings:			
Services(list):			

Available documentation

Architectural		original designer name/date	
Structural		original designer name/date	
Mechanical		original designer name/date	
Electrical		original designer name/date	
Geotech report		original designer name/date	

Damage

Site:
(refer DEE Table 4-2)

Site performance:	minor	Describe damage:	
Settlement:	none observed	notes (if applicable):	
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	0-2 m³/100m²	notes (if applicable):	from the aerial photograph (2011)
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	0-20mm/20m	notes (if applicable):	cracking to the the concrete slab
Damage to area:	slight	notes (if applicable):	damage to the pavement

Building:

	Current Placard Status:	green	
Along	Damage ratio:	0%	Describe how damage ratio arrived at:
	Describe (summary):		
Across	Damage ratio:	0%	
	Describe (summary):		
Diaphragms	Damage?:		Describe:
CSWs:	Damage?:		Describe:
Pounding:	Damage?:		Describe:
Non-structural:	Damage?:		Describe:

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

Recommendations

	Level of repair/strengthening required:	minor non-structural	Describe:	crack repair to slab and blockwork walls
	Building Consent required:	no	Describe:	
	Interim occupancy recommendations:		Describe:	
Along	Assessed %NBS before e'quakes:	19%	##### %NBS from IEP below	If IEP not used, please detail assessment methodology: Quantitative Assessment
	Assessed %NBS after e'quakes:	19%		
Across	Assessed %NBS before e'quakes:	19%	##### %NBS from IEP below	
	Assessed %NBS after e'quakes:	19%		

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

Period of design of building (from above): 1935-1965

h_n from above: 3.4m

Seismic Zone, if designed between 1965 and 1992:

not required for this age of building

not required for this age of building

	along	across
Period (from above):	0.1	0.1
(%NBS)nom from Fig 3.3:		

Note:1 for specifically design public buildings, to the code of the day: pre-1965 = 1.25; 1965-1976, Zone A =1.33; 1965-1976, Zone B = 1.2; all else 1.0	1.00
Note 2: for RC buildings designed between 1976-1984, use 1.2	1.0
Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0)	1.0

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

2.2 Near Fault Scaling Factor

Near Fault scaling factor, from NZS1170.5, cl 3.1.6: 1.00

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

2.3 Hazard Scaling Factor

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

2.4 Return Period Scaling Factor

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

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.2)	1.00	1.00
Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:		
Ductiity Scaling Factor, **Factor D**:	0.00	0.00

2.6 Structural Performance Scaling Factor:

Sp:	1.000	1.000
Structural Performance Scaling Factor **Factor E**:	1	1

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

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

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

3.1. Plan Irregularity, factor A:		1
3.2. Vertical irregularity, Factor B:		1
3.3. Short columns, Factor C:		1
3.4. Pounding potential	Pounding effect D1, from Table to right	1.0
	Height Difference effect D2, from Table to right	1.0
	Therefore, Factor D:	1
3.5. Site Characteristics		1

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

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

3.6. Other factors, Factor F

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

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

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

3.7. Overall Performance Achievement ratio (PAR)	**0.00**	**0.00**
4.3 PAR x (%NBS)b: PAR x Baselline %NBS:	**#DIV/0!**	**#DIV/0!**
4.4 Percentage New Building Standard (%NBS), (before)		**#DIV/0!**

Appendix C

Assessment Result Summary

IN-PLANE ANALYSIS - Across

Dower Park Pavilion

51-31526-29

$V^* = 110.8$ kN.

$IL = 2.$

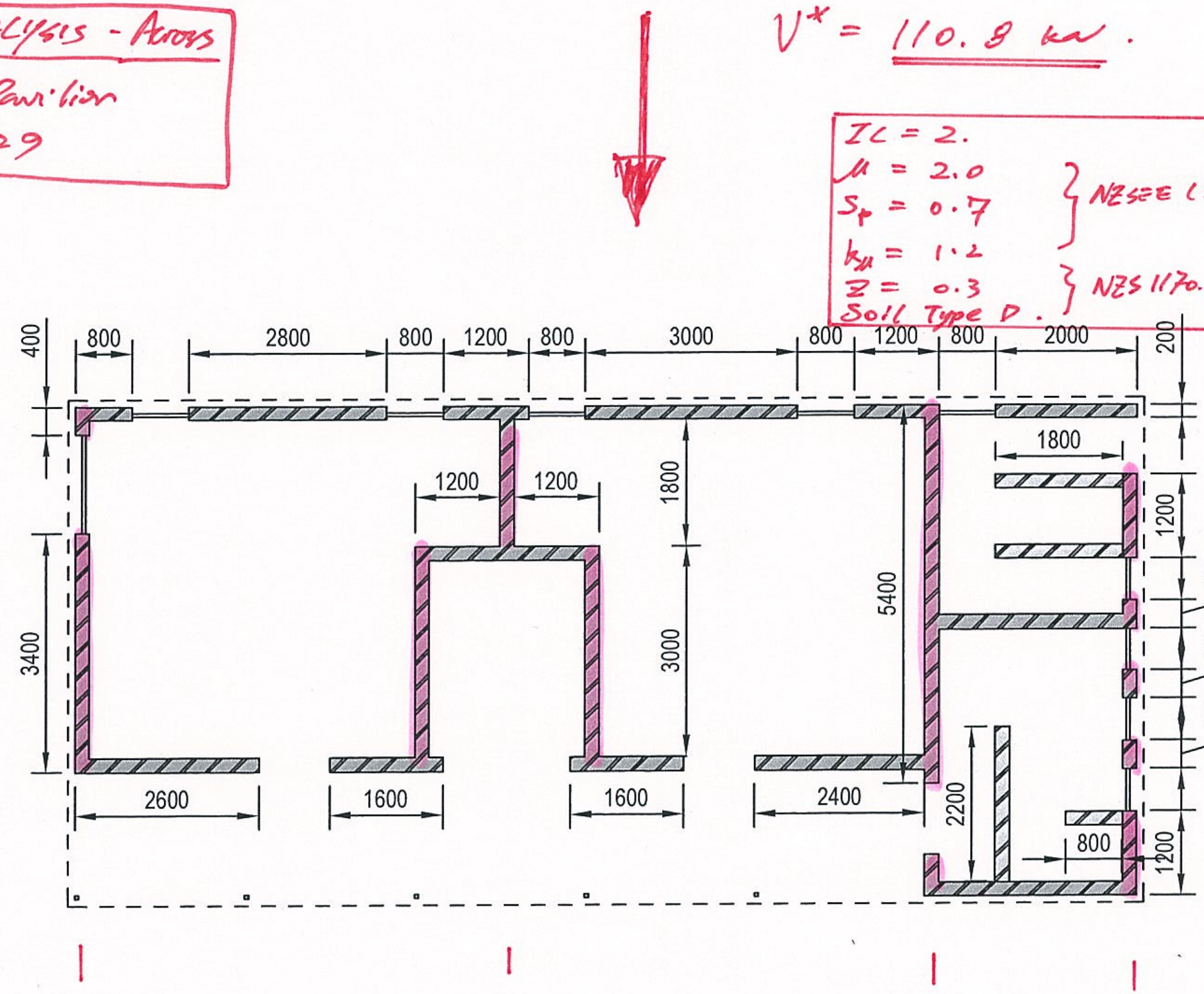
$\mu = 2.0$

$S_p = 0.7$ } NZSEE (2011)

$k_\mu = 1.2$

$Z = 0.3$ } NZS 1170.5

Soil Type D.



Brace Line Z: $V_Z = 11.5$ kN.

Brace Line Y: $V_Y = 20.8$ kN

Brace Line X: $V_X = 28.9$ kN

Brace Line W: $V_W = 3.4$ kN

Across the building.

$V^* = 110.8$ kN

$\Sigma V_n = 64.6$ kN

therefore,

%NBS = 58%

IN-PLANE ANALYSIS - ALONG

Bower Park Pavilion

51-31526-29.

$V^* = 110.8$ kN.

$IL = 2$

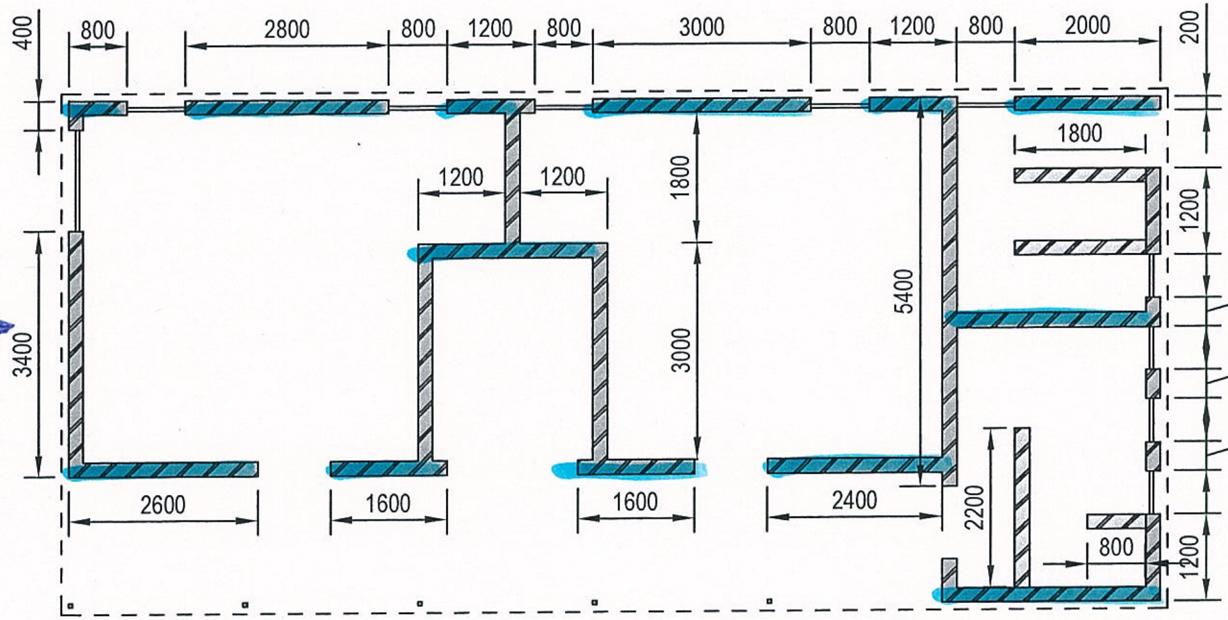
$\mu = 2.0$

$S_p = 0.7$ } NZSEE (2011)

$k_\mu = 1.2$

$Z = 0.3$

Soil Type D. } NZS1170.5.



Brace Line A

$V_A = 18$ kN

Brace Line B

$V_B = 14.3$ kN

Brace Line C

$V_C = 25$ kN.

Along the building.

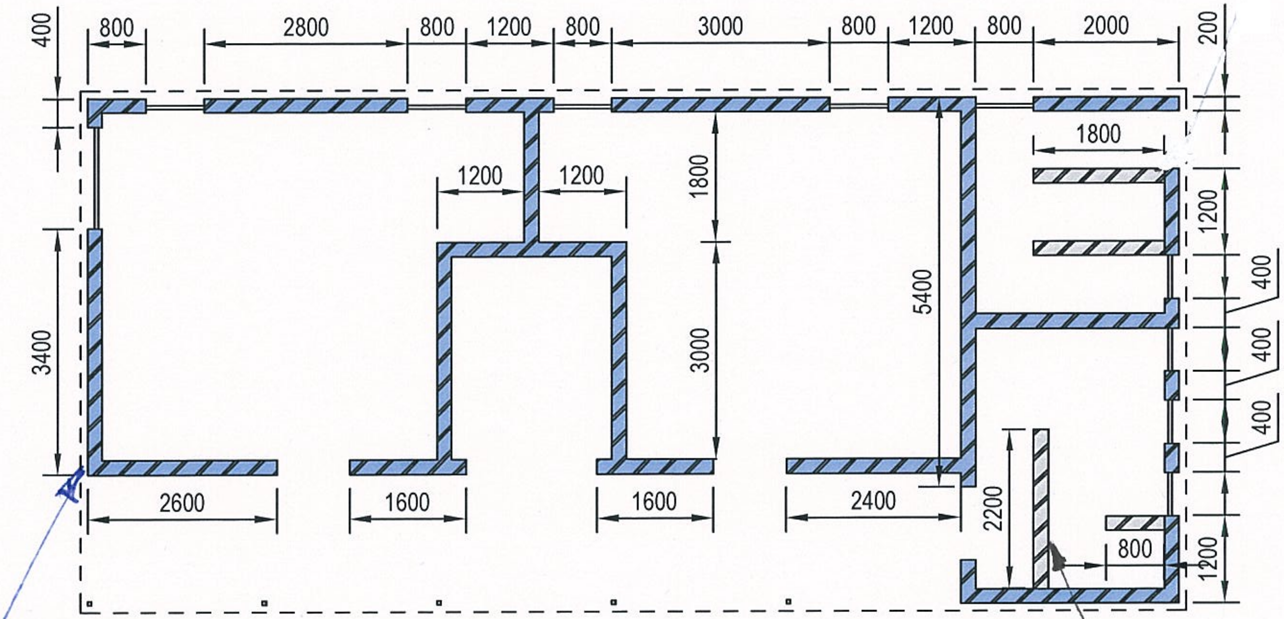
$V^* = 110.8$ kN

$\Sigma V_n = 57.3$ kN

therefore,

%NBS = 52% NBS.

Out-of-plane Analysis
Bower Park Pavilion
51-31526-29.



Full height Block Wall
H = 2.6m ~ 3.4 m (to apex).
unreinforced, unfilled.
Simply supported.
% NBS = 60 %

Cantilevered Block Wall
H = 2.2 m.
unreinforced. unfilled.
% NBS = 19%

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