CLIENTS | PEOPLE | PERFORMANCE

Porritt Park Complex
BU 0706-001 EQ2

Detailed Engineering Evaluation
Qualitative Report
Version FINAL

845 Avonside Drive, Wainoni

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

Porritt Park Complex
BU 0706-001 EQ2

Detailed Engineering Evaluation
Qualitative Report
Version FINAL

845 Avonside Drive, Wainoni

Christchurch City Council

Prepared By
Alex Baylis

Reviewed By
Stephen Lee

Date
12/09/2013

Contents

Table Index

Figure Index

Appendices

A Photographs

B Existing Drawings / Sketches

C CERA Building Evaluation Form

Qualitative Report Summary

Porritt Park Complex

BU 0706-001 EQ2

Detailed Engineering Evaluation

Qualitative Report - SUMMARY

Version FINAL

845 Avonside Drive, Wainoni

Background

This is a summary of the Qualitative report for 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 and visual inspections on 1st May 2012.

Building Description

The Porritt Park Complex is located adjacent to the hockey fields at Porritt Park, 845 Avonside Road, Wainoni, Christchurch. The building consists of a grandstand facing the western hockey field with changing room and storage facilities situated beneath the grandstand. The eastern half of the building has clubroom facilities on the second floor, backing on to the top half of the grandstand. The building's structure consists of reinforced concrete masonry walls, reinforced concrete frames and a steel framed roof structure over the stand. The building was designed in 1973 based on the information from available structural and architectural drawings.

Key Damage Observed

Moderate cracking was observed to be widespread throughout the building. Elements in which damage was observed include the concrete masonry walls, precast concrete beams, precast concrete seating and flooring units, grandstand stairs and landings, connections between timber rafters and masonry walls and the access ramp on the southern side of the building.

Significant ground damage was observed throughout the Porritt Park site during our inspections. Severe ground cracks and lateral spreading were observed near the river approximately 100m south of the grandstand. Extensive liquefaction and ground settlement was observed throughout the site. Significant amounts of sand and silt were still present on the ground across the site.

Critical Structural Weaknesses

The following potential critical structural weakness has been identified.

- Vertical Irregularity (30% Reduction)
- Site Characteristics (30% Reduction)

9% NBS

Further severe ground damage in the form of liquefaction and resulting settlement has a high probability of reoccurring on site. The foundations of the structure (pads linked with strip footings beneath the concrete masonry walls in both directions) have a low to moderate probability of failing in a manner that will cause premature collapse of the building. In terms of the Detailed Engineering Evaluation it has been assessed as a “significant” site characteristic in accordance with NZSEE guidelines to account for the potential for lateral spreading to cause damage to the structure and the likely damage to some structural elements not effectively linked to the foundation structure of the building (stairs to the grandstand on the western side of the building).

Indicative Building Strength (from IEP and CSW assessment)

Based on the information available, and using the NZSEE Initial Evaluation Procedure, the original capacity of the building has been assessed to be in the order of 9% NBS and post-earthquake capacity in the order of 7% NBS (25% reduction based on damage observed). The building’s post-earthquake capacity excluding critical structural weaknesses is in the order of 14% NBS, allowing for a 25% reduction based on damage observed.

The building has been assessed to have a seismic capacity in the order of 7% NBS and is therefore potentially Earthquake Prone.

Recommendations

As the structure has suffered moderate damage during the recent seismic activity and has been assessed as potentially Earthquake Prone, we recommend that the building remain closed as per Christchurch City Council’s Earthquake Prone Buildings policy until further detailed assessment of the structure and ground conditions is undertaken and if necessary, strengthening options explored.

Given the enclosed information we would recommend a series of additional location specific geotechnical assessments, including testing and investigation, be completed.

1. Background

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

This report is a Qualitative 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 qualitative assessment involves inspections of the building and a desktop review of existing structural and geotechnical information, including existing drawings and calculations, if available.

The purpose of the assessment is to determine the likely building performance and damage patterns, to identify any potential critical structural weaknesses or collapse hazards, and to make an initial assessment of the likely building strength in terms of percentage of new building standard (%NBS).

At the time of this report, no intrusive site investigation, detailed analysis, or modelling of the building structure had been carried out. Construction drawings were not made available. The building description below is based on our visual inspections only.

2. Compliance

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

2.1 Canterbury Earthquake Recovery Authority (CERA)

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

Section 38 – Works

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

Section 51 – Requiring Structural Survey

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

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

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

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

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

2.2 Building Act

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

Section 112 – Alterations

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

Section 115 – Change of Use

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

2.2.1 Section 121 – Dangerous Buildings

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

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

Section 122 – Earthquake Prone Buildings

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

Section 124 – Powers of Territorial Authorities

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

Section 131 – Earthquake Prone Building Policy

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

2.3 Christchurch City Council Policy

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

The 2010 amendment includes the following:

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

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

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

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

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

2.4 Building Code

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

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

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

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

3. Earthquake Resistance Standards

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

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

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

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

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

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

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

Table 1 %NBS compared to relative risk of failure

4. Building Description

4.1 General

The Porritt Park Complex is located adjacent to the hockey fields at Porritt Park, 845 Avonside Road, Wainoni, Christchurch. The building consists of a grandstand facing the western hockey field with changing room and storage facilities situated beneath the grandstand. The eastern half of the building has clubroom facilities on the second floor, backing on to the top half of the grandstand. The building's structure consists of reinforced concrete masonry walls, reinforced concrete frames and a steel framed roof structure over the stand. The building was designed in 1973 based on information from the available structural and architectural drawings.

Comparisons between the original drawings and observations on site suggest that alterations were undertaken at some stage after the construction of the original building. This involved removal of some masonry walls on the second floor of the eastern half of the building and erecting lightweight partition walls. No records of the alterations have been made available.

Key structural details of the building are shown in Figure 2 below.

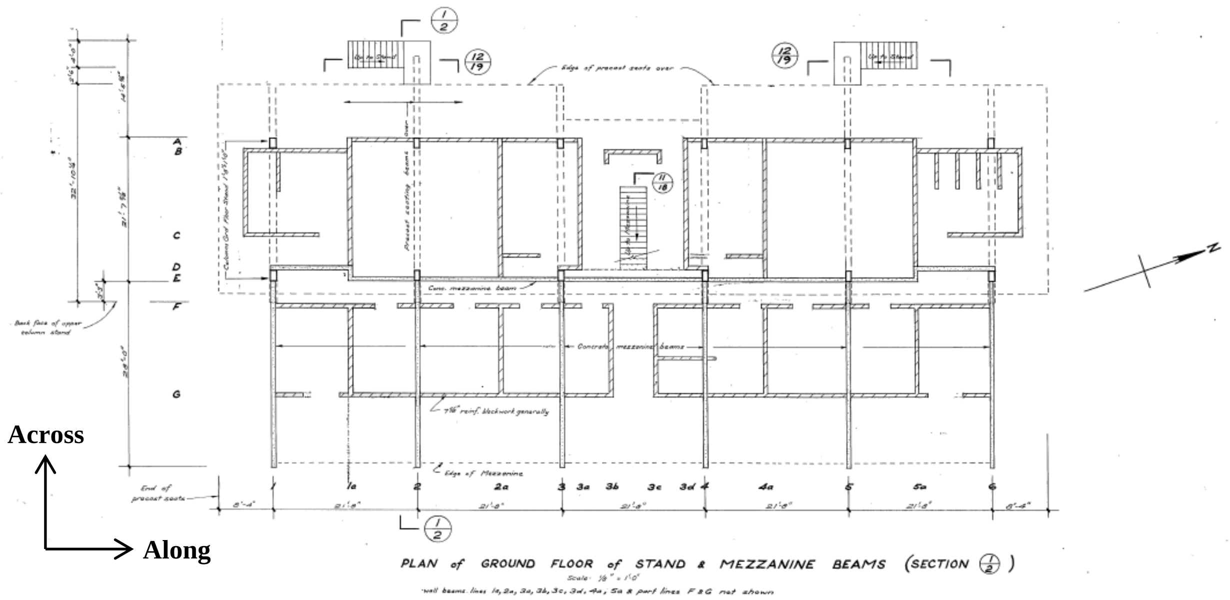


Figure 2 Plan sketch showing key structural elements

The dimensions of the rectangular building are approximately 15.9 m wide by 33.0 m long and 10.5 m tall at the top of the grandstand. The overall footprint of the building is approximately 525 m^2.

Full architectural and structural drawings (excluding the later alterations) of the Porritt Park Complex have been made available by Christchurch City Council.

4.2 Gravity Load Resisting System

The gravity loads acting on the structure are resisted by a combination of frame systems and load bearing walls.

4.2.1 Grandstand Roof

Gravity loads from the lightweight steel roof cladding are supported by steel universal beams (UB) and UB rafters as shown below in Figure 3. The UB rafters are supported by reinforced concrete columns along the eastern side of the roof and are connected by bolted plate connections welded to plates set in the concrete columns. Precast concrete panels span between the columns. These panels have been detailed to allow movement and are not expected to stiffen the structure and provide resistance to lateral loads. The UB rafters are supported near western edge of the roof by a deep UB (along Line 8 as shown in Figure 3) which sits on four RHS posts. The RHS posts are bolted at the base to the precast concrete seating units which are supported directly beneath by the precast grandstand beams. No cross bracing has been provided in the roof structure.

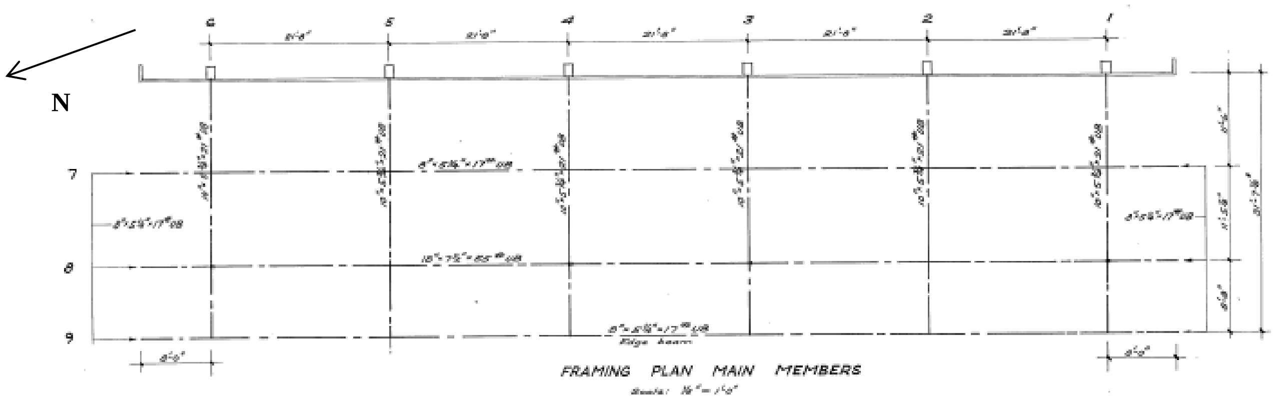


Figure 3 Steel roof framing plan.

4.2.2 Grandstand

Gravity loads from the grandstand roof and the grandstand seating area are supported by reinforced concrete frames consisting of precast reinforced concrete beams and cast-in-situ reinforced concrete columns. This is shown in Figure 4. The precast concrete beam supports loads from the roof that are transferred through the concrete columns at the rear of the roof and the RHS posts towards the front. The precast beams also support the precast concrete seating units that span between adjacent beams. The precast stand beams are supported by reinforced concrete columns. The toe of the precast beams overhangs the supporting columns and support the gravity loads from the concrete landing. Load-bearing concrete masonry walls between the precast stand beams also support gravity loads from the precast seating units.

The precast concrete stairs at the front of the grandstand are supported by the concrete landing at the bottom of the stand and separate foundations at the foot of the stairs.

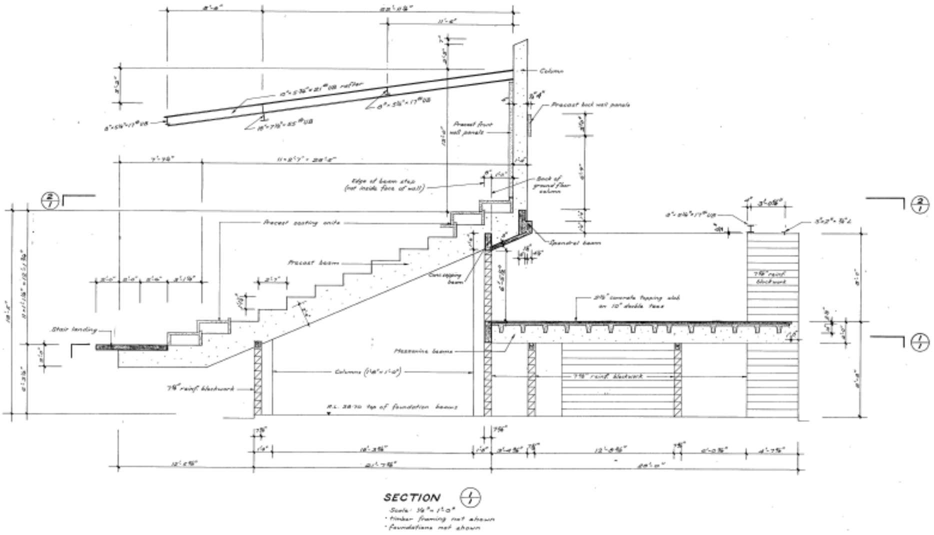


Figure 4 Building cross-section.

4.2.3 Clubroom Area

Gravity loads from the roof in the clubrooms area (second storey of the eastern half of the building) are supported by timber rafters (Photograph 18) connected at the western end of the rafters by steel brackets to precast panels spanning between the reinforced concrete columns on the back of the stand (Photograph 19). The eastern end of the rafters is connected by steel brackets to braced steel frames which are supported by concrete masonry walls. The layout of this section of the roof is shown in Figure 5.

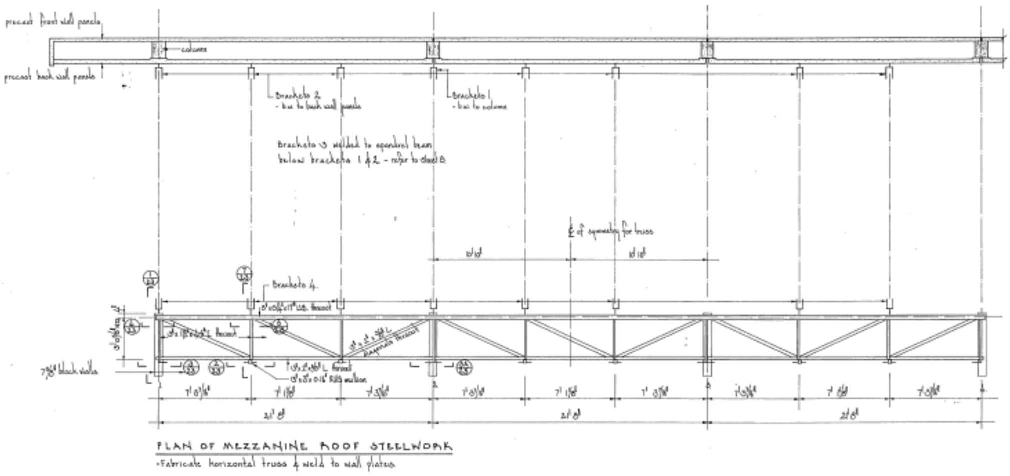


Figure 5 Roof framing plan in clubroom area.

The floor of the second storey clubroom area consists of precast concrete double tee flooring units. The double tee units span along the building between precast concrete beams that sit on top of load-bearing concrete masonry walls.

4.2.4 Foundations

The foundations of the building consist of reinforced concrete pads beneath the columns and strip footings beneath the reinforced concrete masonry walls. The pads are linked by reinforced concrete strip footings beneath the masonry walls. All elements of the main foundation structure are linked to adjacent elements. The foundations of the stairs at the front of the grandstand are isolated from the foundation structure for the main building.

4.3 Lateral Load Resisting System

Lateral loads acting on the structure of the building are resisted by concrete masonry walls along and across the building.

In the eastern half of the building, it is expected that lateral forces are distributed to the concrete masonry walls by diaphragm action of the plasterboard ceiling lining spanning between the timber rafters and diaphragm action of the precast double tee flooring units. In the western half of the building, diaphragm action of the roof cladding and the precast seating units is expected to distribute lateral forces into the concrete masonry walls.

Along the building on gridlines A/B and D/E (see Figure 6 on the following page), the reinforced concrete masonry walls span between the reinforced concrete columns. The concrete masonry wall along gridline G is connected to the precast beams and concrete masonry walls running across the building that support the double tee units.

Across the building, the lateral load-resisting concrete masonry walls along gridlines 1a, 2a, 3a, 3b, 3c, 3d, 4a and 5a (see Figure 6 on the following page) span between the masonry walls running in the long direction. In the western half of the building, the concrete masonry walls have in-situ concrete on top of the walls cast against the precast seating units and are connected through starter bars. The starter bars are welded to the precast seating units beneath the grandstand section of the building. This enables lateral forces in the precast seating units to be transferred into the masonry walls

In the eastern half of the building the concrete masonry walls have reinforced beams cast above them which support the precast double tee flooring units. Starter bars from the reinforced concrete masonry wall allow lateral forces from the double tee units to be transferred through the supporting reinforced beam and into the concrete masonry walls and concrete strip foundations.

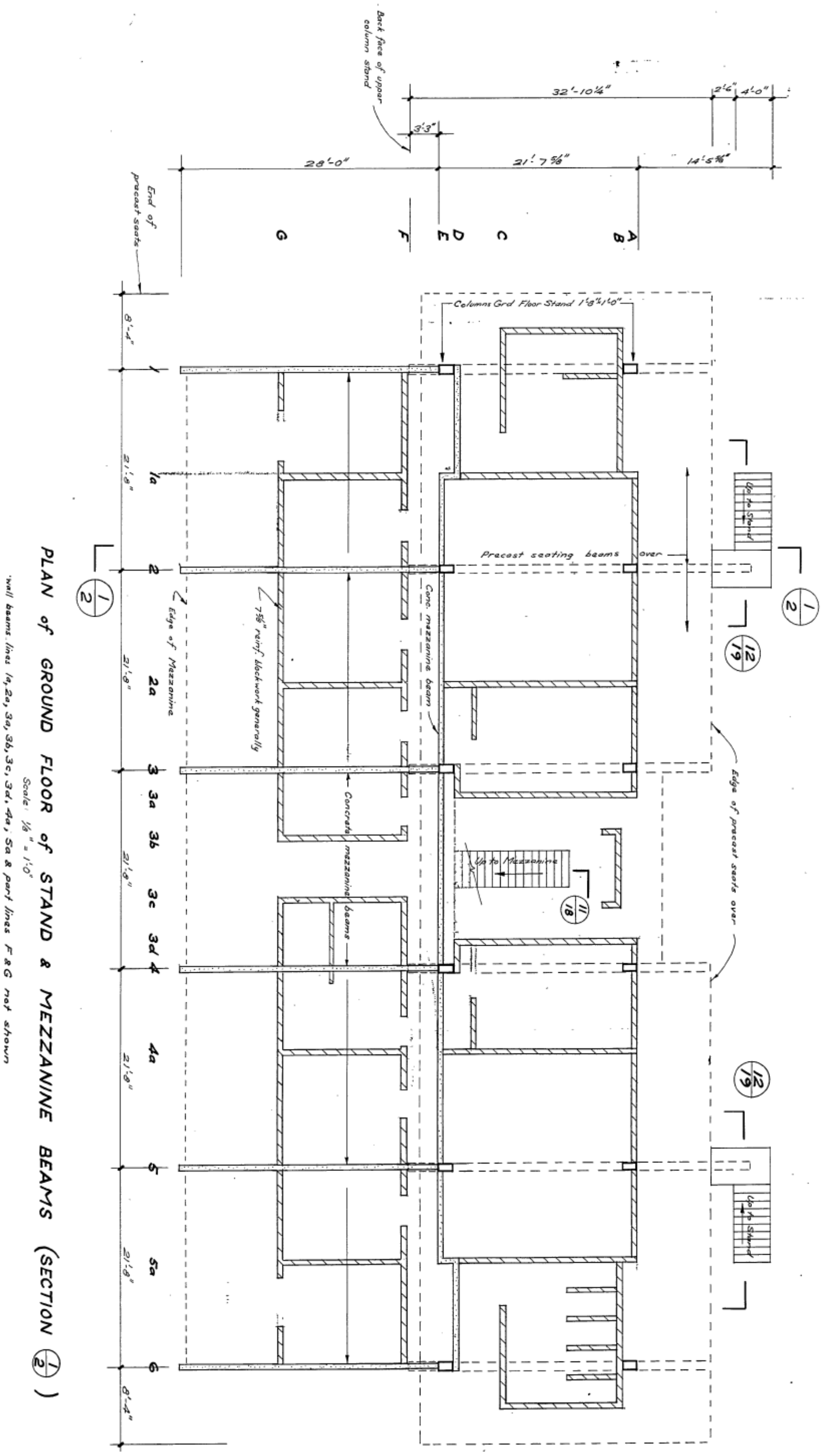


Figure 6 Plan showing positions of concrete masonry walls.

5. Assessment

An inspection of the building was undertaken on the 1st of May 2012. The exterior, interior and surrounding area was inspected. 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 type observed and noting general damage observed throughout the building in both structural and non-structural elements.

The %NBS score determined for the building has been based the IEP procedure described by the NZSEE based on the information obtained from visual observation of the building and a review of the available drawings.

6. Damage Assessment

6.1 Surrounding Buildings

The Porritt Park Garage to the north of the Porritt Park Grandstand Complex has suffered some damage during the recent earthquakes. The lightweight roof over the yard adjacent to the garage has detached from the main concrete masonry section of the building and has collapsed. The garage is approximately 10m from the grandstand complex and also suffered significant and extensive ground damage in the form of liquefaction and ground settlement.

6.2 Residual Displacements and General Observations

Moderate cracking was observed to be widespread throughout the reinforced concrete frames and concrete masonry structural elements in the building. Cracking was observed in the concrete masonry walls both along and across the building. Photographs 8, 23 and 24 show cracking observed on the eastern and western faces of the building. A number of these cracks extend through to the foundations.

Minor cracking in the double tee precast flooring units was also noted in this area. The cracking in the double tees was generally located near the supporting beams (see Photograph 22).

Minor cracking was observed in the precast concrete beams supporting the grandstand seating units on the western side of the building and in the beams supporting the double tee flooring units on the eastern side of the building. Photograph 21 shows an example of the cracking observed in the beams supporting the precast double tee units.

The two sets of precast concrete stairs leading to the grandstand seating area on the western side of the building have suffered damage as a result of settling differentially to the main structure. The foundations of the stairs are not linked to the foundation system of the grandstand structure and as a result they have moved relative to each other during the earthquakes, damaging the stairs and the landings. Photographs 12, 13, and 14 show the damage caused to the stairs and landing. The landings of the stairs have lifted off the supports at one end suggesting the stair foundations have been driven up by liquefaction, resulting in the stairs twisting and buckling. Photograph 15 shows cracking in the in-situ slab section of the grandstand where the stair landings connect to the grandstand structure.

Cracking in the precast concrete seating units was observed approximately along the lines of the supporting precast beams (see Photograph 25). A large number of these cracks had been filled suggesting that they may have occurred in the September earthquake and had been repaired.

Damage was observed to a connection between the timber rafters and the concrete masonry wall in the 2nd storey area of the eastern half of the building. This is shown in Photograph 20. The damage has been caused by movement of the rafters away from the concrete masonry wall during earthquake shaking, causing the concrete masonry to fracture.

The ramp providing access to the southern side of the building has suffered damage as a result of the ground supporting the lower end of the ramp moving laterally during the earthquakes. Photograph 16 shows the ramp has moved approximately 300mm away from the path. The connection between the top of the ramp and the building has also been damaged as a result of the movement.

No significant overall residual displacement of the main structure was observed during our inspection. The linked foundation structure appears to have prevented significant differential settlement of the structure. Due to the extensive liquefaction that has occurred at the site, it is expected that some minor overall settlement of the structure has occurred.

A check of the building level and verticality was undertaken with a spirit level during our inspection. The checks indicated that the building is still relatively level and that the building had not significantly settled differentially.

6.3 Ground Damage

Significant ground damage was observed throughout the Porritt Park site during our inspections. Severe ground cracks and lateral spreading were observed near the river approximately 100m south of the grandstand. Extensive liquefaction and ground settlement was observed throughout the site. Significant amounts of sand and silt were still present on the ground across the site.

7. Critical Structural Weakness

7.1 Short Columns

No significant short columns were observed in the building.

7.2 Roof / Plan Irregularity

There is no bracing in the steel roof structure over the seating area in the grandstand building. The lack of bracing along the roof provides no clear load path for distributing lateral load to the supporting structure. The reinforced concrete columns supporting the steel rafters at the rear of the stand are expected to be stiffer in the long direction of the building than the four RHS posts supporting the steel beam and rafters towards the front of the stand. The lack of bracing in the roof structure may result in torsional actions being induced and a potential skewing of the roof causing a premature or rapid collapse of the roof structure. For the purposes of the IEP assessment of the building and the determination of the %NBS score, the effects of the lack of bracing in the roof structure on the performance of the building has been assessed as a ‘significant’ plan irregularity in accordance with the NZSEE guidelines.

7.3 Staircases

The two sets of precast concrete stairs leading to the grandstand seating area on the western side of the building have suffered damage as a result of settling differentially to the main structure. The foundations of the stairs are not linked to the foundation system of the grandstand structure. No provision for movement has been made in the detailing of the stairs. As a result, any differential movement between the foundations of the stairs and the grandstand structure during an earthquake is likely to cause further damage to the stairs which may compromise egress from the grandstand.

7.4 Site Characteristics

Further extensive, severe ground damage in the form of liquefaction and resulting settlement has a high probability of reoccurring on site. The foundations of the structure (pads linked with strip footings beneath the concrete masonry walls in both directions) have a low to moderate possibility of failing in a manner that will cause premature collapse of the building. In terms of the Detailed Engineering Evaluation it has been assessed as a “significant” site characteristic in accordance with NZSEE guidelines to account for the likely damage to some structural elements not effectively linked to the foundation structure of the building (stairs to the grandstand on the western side of the building).

8. Geotechnical Consideration

8.1 Site Description

Porritt Park Complex is located in Wainoni, and is accessed from Avonside Drive. The Avon River presently flows north immediately to the west of the site. However, the river previously meandered around the southern, eastern and northern boundaries of the reserve until realigned and widened to allow for increased recreational use in the early twentieth century. The previous course of the river still contains water, and is connected to the main branch at both ends.

The subject area is low lying and topographically is typically flat. It is approximately 2m above sea level, and approximately 4km west of the coast.

8.2 Published Information on Ground Conditions

8.2.1 Published Geology

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

A north-south oriented band of marine sand of fixed and semi-fixed dunes (Christchurch Formation) is located nearby to the east.

8.2.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that numerous boreholes are located within a 200m radius of the site (see Table 2), although none are within the property itself. Those with deeper lithographic logs indicate the area to be underlain by layers of alternating layers of sand and clay, with gravel below ~25m.

Bore Name	Log Depth	Groundwater	Distance & Direction from Site
M35/5362	92.7m	-	150m S
M35/12097	3.0m	-	100m S
M35/12098	3.3m	-	100m S
M35/12099	3.3m	-	100m S
M35/12647	6.1m	-	200m E

Table 2 ECan Borehole Summary

It should be noted that the purpose of the boreholes the well logs are associated with, were sunk for groundwater extraction and not for geotechnical purposes. Therefore, the amount of material recovered and available for interpretation and recording will have been variable at best and may not be

[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.

representative. The logs have been written by the well driller and not a geotechnical professional or to a standard. In addition strength data is not recorded.

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 Wainoni[2]. Four CPT investigations were conducted along the eastern boundary of the park across the river, approximately 200m from the subject buildings, as summarised below in Table 3.

Bore Name	Grid Reference	Depth (m bgl)	Log Summary
CPT WAI 36	2484689 mE	0 – 1	Surface Soil
	5743611 mN	1 – 3.6	SILT mixtures (silty clay to sandy silt)
		3.6 – 17.7	Medium dense to dense SAND
			(WT at 1.6m bgl)
CPT WAI 37	2484773 mE	0 – 1	Surface Soil
	5743521 mN	1 – 2.4	SILT mixtures (silty clay to sandy silt)
		2.4 – 19	Dense SAND, with occasional silt/clay lenses
			(WT at 2.6m bgl)
CPT WAI 38	2484755 mE	0 – 1	Surface Soil
	5743469 mN	1 – 4	SILT mixtures (silty clay to sandy silt)
		4 – 22.9	Dense SAND, with occasional silt/clay lenses
CPT WAI 39	2484671 mE	0 – 2.2	Surface Soil and Clays
	5743354 mN	2.2 – 8.8	Loose to medium dense SAND
		8.8 – 9.8	SILT mixtures (silty clay to sandy silt)
		9.8 – 19.5	Medium dense to dense SAND
			(WT at 1.5m bgl)

Table 3 EQC Geotechnical Investigation Summary Table

Initial observations of the CPT results indicate the soils typically comprise a surface layer of fines (clay and silt), underlain by sand with occasional silt/clay lenses.

8.2.4 Land Zoning

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

The site is situated within the Green Zone. Within this, it is classified Technical Category Not Applicable, as the property is considered non-residential.

[2] Tonkin and Taylor . September 2011: Christchurch Earthquake Recovery, Geotechnical Factual Report, Wainoni

8.2.5 Post February Aerial Photography

Aerial photography taken following the 22 February 2011 earthquake shows signs of major liquefaction (including lateral spreading) across the site, and throughout surrounding streets, as shown in Figure 7. Significant liquefaction-induced damage is also evident across the hockey turf surfaces.

Figure 7 Post February 2011 Earthquake Aerial Photography [3]

8.2.6 Summary of Ground Conditions

From the information presented above, the ground conditions underlying the site are anticipated (near the surface) to comprise silts, clays, and loose to dense sand, underlain by multiple strata of gravel, clay and sand.

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

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.

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 (2010) Fault	25 km	W	7.1	~15,000 years
Hope Fault	100 km	N	7.2~7.5	120~200 years
Kelly Fault	100 km	NW	7.2	~150 years
Porters Pass Fault	60 km	NW	7.0	~1100 years

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

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

8.3.2 Ground Shaking Hazard

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

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

The site has a 475-year PGA (peak ground acceleration) of ~0.4 (Stirling et al, 2002[4]). Combining this with the anticipated geology and estimated bedrock depths in excess of 500m, the ground shaking hazard is expected to be relatively high.

8.4 Slope Failure and / or Rockfall Potential

The topography surrounding the site is typically flat, and hence rockfalls are not considered to be a hazard at this site. However, given the site’s proximity to the Avon River, it is considered possible that lateral spreading may occur in the area.

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

[5] GNS Active Faults Database

In addition, any localised retaining structures should be investigated to better establish the site-specific slope instability.

8.5 Liquefaction Potential

The site is considered to be at major risk from liquefaction during further earthquakes as evidenced by:

- Significant liquefaction at the site following the events of 4th September 2010 (Mw 7.1), 22nd February (Mw 6.3, 2.0g) and 13th June 2011 (Mw 5.6-6.3, 1.5g); and,
- Anticipated ground conditions comprising sand and silt layers considered to be highly liquefiable.

Lateral spreading also occurred following the September and February earthquakes. Due to the property being an island, this spreading propagated in all directions. The surface of the ground across the park was significantly cracked.

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

8.6 Recommendations

Given the anticipated ground conditions, we recommend that further investigation is undertaken. Specifically, we recommend two to four CPT investigations and one machine-drilled borehole be conducted to target depths of 20m bgl.

A soil class of **E** (in accordance with NZS 1170.5:2004) should be adopted for the site. However, this is subject to confirmation following the assessment of intrusive ground investigation results.

8.7 Conclusions & Summary

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

The site appears to be situated on stratified alluvial deposits, comprising layers of silt, clay, loose to dense sand, and gravel (gravel at depth). Associated with this the site also has major liquefaction potential. The proximity to the Avon River highlights the potential for lateral spreading, observed on 4th September 2010, and 22nd February 2011.

It is recommended that intrusive investigation comprising two to four piezocone CPT's be conducted. This will allow a more comprehensive liquefaction and/or ground condition assessment to be made.

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

9. Survey

A check of the building level and verticality was undertaken with a spirit level at several locations during our inspection. The checks indicated that the building is still relatively level and that the building had not significantly settled differentially. A detailed level survey has not been undertaken.

10. Initial Capacity Assessment

10.1 % NBS Assessment

The building has had its capacity assessed using the Initial Evaluation Procedure based on the information available. The buildings capacity excluding critical structural weaknesses and the capacity of any identified weaknesses are expressed as a percentage of new building standard (%NBS) and are in the order of that shown below in Table 5. These capacities are subject to confirmation by a more detailed quantitative analysis.

Item	%NBS
Building excluding CSW's and Damage	18
Vertical Irregularity (30% Reduction) Site Characteristics (30% Reduction)	9
Damage to Building (25% Reduction)	7

Table 5 Indicative Building and Critical Structural Weaknesses Capacities based on the NZSEE Initial Evaluation Procedure

Following an IEP assessment, the building has been assessed as achieving 7% New Building Standard (NBS). The building is therefore considered potentially Earthquake Prone as it achieves less than 33% NBS. This score has been reduced by 25% when considering damage to the structure as the damage observed was reasonably widespread throughout the structure, particularly in the concrete masonry walls.

10.2 Seismic Parameters

The seismic design parameters based on current design requirements from NZS1170:2002 and the NZBC clause B1 for this building are:

- Site soil class: E, NZS 1170.5:2004, Clause 3.1.3, Soft Soil
- Site hazard factor, Z = 0.3, NZBC, Clause B1 Structure, Amendment 11 effective from 1 August 2011
- Return period factor R_u = 1.3, NZS 1170.5:2004, Table 3.5, Importance Level 3 structure with a 50 year design life.

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

10.3 Expected Structural Ductility Factor

A structural ductility factor of 1.25 has been assumed both along and across the building based on the concrete masonry wall lateral load resisting system observed and the construction date of 1973. The concrete masonry walls are expected to govern the response of the structure during an earthquake. The masonry walls generally consist of fully filled masonry blocks with 12mm vertical reinforcing bars at 600mm centres. No horizontal reinforcement is shown on the drawings. As a result, the structural ductility factor has been chosen based on the nominal ductility likely to be achieved for the concrete masonry walls.

10.4 Discussion of Results

The results obtained from the initial IEP assessment are consistent with those expected for a building of this age and construction type founded on Class E soils. The lack of bracing in the steel framed roof structure over the grandstand seating has been treated as a ‘plan irregularity’ Critical Structural Weakness. It has been assessed as significant in accordance with NZSEE guidelines. The increase in the hazard factor for Christchurch to 0.3 further reduces the %NBS score.

The grandstand complex has been assessed as an Importance Level 3 structure as the grandstand can seat over 300 people in one area.

The %NBS score has been reduced by 25% to account for the moderate damage to the structure, particularly the concrete masonry walls, which is likely to reduce the overall lateral load resisting capacity of the structure.

10.5 Occupancy

The building has suffered moderate damage during the recent seismic activity and has been assessed as being potentially Earthquake Prone. As a result, it is recommended that the building remain unoccupied pending further detailed assessment and strengthening if required, as per Christchurch City Council’s policy regarding the occupancy of potentially Earthquake Prone buildings.

11. Initial Conclusions

The building has been assessed to have a seismic capacity in the order of 7% NBS and is therefore potentially Earthquake Prone.

12. Recommendations

The recent seismic activity in Christchurch has caused moderate structural damage throughout the building and extensive ground damage.

The structure has been assessed as potentially Earthquake Prone, due to the damage observed, age of the building, site ground conditions and plan irregularity of the grandstand. As a result, we recommend that the building remain closed as per Christchurch City Council's Earthquake Prone Buildings policy until further detailed assessment of the structure and ground conditions is undertaken and if necessary, strengthening options explored.

Given the land damage observed, we recommend a series of location specific geotechnical assessments, including testing and investigation, be completed.

13. Limitations

13.1 General

This report has been prepared subject to the following limitations:

- The foundations of the building were unable to be inspected.
- No intrusive structural investigations have been undertaken.
- No level or verticality surveys have been undertaken.
- No material testing has been undertaken.
- No calculations, other than those included as part of the IEP in the CERA Building Evaluation Report, have been undertaken. 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 report.

13.2 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 Ground cracks near the river south of the grandstand.

Photograph 2 Southern face of the Porritt Park Complex.

Photograph 3 Northern face of the Porritt Park Complex.

Photograph 4 Rear section of northern face of the Porritt Park Complex.

Photograph 5 **Eastern face of the Porritt Park Complex.**

Photograph 6 **Sand and silt on the ground east of the building due to extensive liquefaction.**

Photograph 7 **Precast double tee flooring units.**

Photograph 8 **Cracking in a concrete masonry wall along the eastern face of the building.**

Photograph 9 **View from hockey field of eastern face of the Porritt Park Complex.**

Photograph 10 **Concrete masonry wall in the south-east corner of the building.**

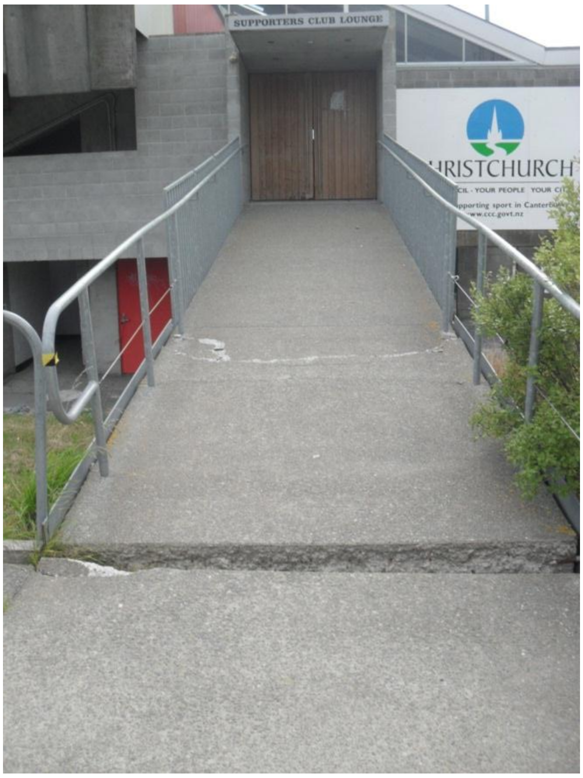
Photograph 11 **Steel framed roof structure.**

Photograph 12 **Cracking in grandstand stairs along the western side of the grandstand.**

Photograph 13 **Separation of the seating between the concrete beams supporting the grandstand and the southern concrete steps.**

Photograph 14 **Separation of the seating between the concrete beams supporting the grandstand and the northern concrete steps.**

Photograph 15 Cracking in the in-situ floor slab in the middle of the grandstand.



Photograph 16 Residual displacement of the access ramp south of the building.

Photograph 17 **In-situ beam supporting precast double tee flooring units.**

Photograph 18 **Timber rafters supporting the roof along eastern half of the building.**

Photograph 19 **Connection between in-situ concrete beam and timber rafters.**

Photograph 20 **Damaged connection between a concrete masonry wall and timber rafter.**

Photograph 21 Cracking in concrete beam supporting precast flooring units.

Photograph 22 Cracking in precast double tee flooring unit near supporting beam.

Photograph 23 **Cracking in concrete masonry wall on eastern side of the building.**

Photograph 24 **Cracking in concrete masonry wall on western side of the building.**

Photograph 25 Cracking in precast concrete seating units.

Appendix B

Existing Drawings / Sketches


Porritt Park Complex

Appendix C

CERA Building Evaluation Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Porritt Park Complex			Reviewer:	Stephen Lee
	Unit	No:	Street	CPEng No:	1006840
Building Address:		845	Avonside Drive	Company:	GHD
Legal Description:	Pt Lot 3 DP 14075			Company project number:	51/30596/23
				Company phone number:	04 472 0799
	Degrees	Min	Sec		
GPS south:	43	30	57.00	Date of submission:	12/9/2013
GPS east:	172	41	3.00	Inspection Date:	1/5/2012
				Revision:	FINAL
Building Unique Identifier (CCC):	BU 0706-001 EQ2			Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	mixed	Soil Profile (if available):	
Site Class (to NZS1170.5):	E		
Proximity to waterway (m, if <100m):	50	If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	2.00

Building

No. of storeys above ground:	2	single storey = 1	Ground floor elevation (Absolute) (m):	
Ground floor split?	no		Ground floor elevation above ground (m):	
Storeys below ground	0			
Foundation type:	pads with tie beams		if Foundation type is other, describe:	
Building height (m):	10.50		height from ground to level of uppermost seismic mass (for IEP only) (m):	10.5
Floor footprint area (approx):	525			
Age of Building (years):	39		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):	Grandstand and Clubrooms			
Importance level (to NZS1170.5):	IL3			

Gravity Structure

Gravity System:	frame system		
Roof:	steel framed	rafter type, purlin type and cladding	
Floors:	precast concrete with topping	unit type and depth (mm), topping	
Beams:	cast-insitu concrete	overall depth x width (mm x mm)	
Columns:	cast-insitu concrete	typical dimensions (mm x mm)	
Walls:	fully filled concrete masonry	#N/A	

Lateral load resisting structure

Lateral system along:	fully filled CMU	**Note: Define along and across in detailed report!**	note total length of wall at ground (m):	
Ductility assumed, μ:	1.25		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:	fully filled CMU		note total length of wall at ground (m):	
Ductility assumed, μ:	1.25		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):		

Non-structural elements

Stairs:	cast insitu	notes	
Wall cladding:	exposed structure	describe	
Roof Cladding:	Metal	describe	
Glazing:	aluminium frames		
Ceilings:	fibrous plaster, fixed		
Services(list):			

Available documentation

Architectural	full	original designer name/date	
Structural	full	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:	Poor	Describe damage:	Extensive and severe ground damage
Settlement:	25-100m	notes (if applicable):	
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	5-10 m²/100m³	notes (if applicable):	
Lateral Spread:	250-500mm	notes (if applicable):	
Differential lateral spread:	0-1:400	notes (if applicable):	
Ground cracks:	more than 200mm/20m	notes (if applicable):	
Damage to area:	widespread to major (in in 3 to most)	notes (if applicable):	

Building:

	Current Placard Status:	red	
Along	Damage ratio:	25%	Describe how damage ratio arrived at:
	Describe (summary):	Moderate damage. Approximately 25%.	
Across	Damage ratio:	25%	
	Describe (summary):	Moderate damage. Approximately 25%.	
Diaphragms	Damage?:	yes	Describe:
CSWs:	Damage?:	yes	Describe:
Pounding:	Damage?:	no	Describe:
Non-structural:	Damage?:	yes	Describe:

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

Recommendations

	Level of repair/strengthening required:	significant structural and strengthening		Describe:
	Building Consent required:	no		Describe:
	Interim occupancy recommendations:	do not occupy		Describe:
Along	Assessed %NBS before:	9%	9% %NBS from IEP below	If IEP not used, please detail assessment methodology:
	Assessed %NBS after:	7%		
Across	Assessed %NBS before:	9%	9% %NBS from IEP below	
	Assessed %NBS after:	7%		

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

Period of design of building (from above): 1965-1976

h_n from above: 10.5m

Seismic Zone, if designed between 1965 and 1992: B

not required for this age of building

not required for this age of building

	along	across
Period (from above):	0.4	0.4
(%NBS)nom from Fig 3.3:	5.0%	5.0%

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 $(\%NBS)_{nom}$:	**5%**	**5%**

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:	0.30	
	Z_{1992}, from NZS4203:1992		
	Hazard scaling factor, **Factor B:**	3.333333333	
2.4 Return Period Scaling Factor	Building Importance level (from above):	3	
	Return Period Scaling factor from Table 3.1, **Factor C**:	0.80	
		along	across
2.5 Ductility Scaling Factor	Assessed ductility (less than max in Table 3.2)	1.25	1.25
	Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:	1.25	1.25
	Ductiity Scaling Factor, **Factor D**:	1.25	1.25
2.6 Structural Performance Scaling Factor:	Sp:	0.925	0.925
	Structural Performance Scaling Factor **Factor E**:	1.081081081	1.081081081
2.7 Baseline %NBS, $(NBS\%)_b = (\%NBS)_{nom}$ x A x B x C x D x E	**$\%NBS_b$:**	**18%**	**18%**

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

3.1. Plan Irregularity, factor A:	insignificant	1
3.2. Vertical irregularity, Factor B:	significant	0.7
3.3. Short columns, Factor C:	insignificant	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	significant	0.7

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

		Along	Across
3.6. Other factors, Factor F	For ≤ 3 storeys, max value =2.5, otherwise max valule =1.5, no minimum	1.0	1.0
	Rationale for choice of F factor, if not 1		

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

List any: Lacl of bracing in front of grandstand — 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)		**0.49**	**0.49**
4.3 PAR x (%NBS)b:	PAR x Baselline %NBS:	**9%**	**9%**
4.4 Percentage New Building Standard (%NBS), (before)			**9%**

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

Document Status

Rev No.	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
DRAFT	Alex Baylis	Jenny Stevenson		Stephen Lee		29/05/2012
FINAL	Alex Baylis	Stephen Lee		Nick Waddington		12/09/2013