CLIENTS | PEOPLE | PERFORMANCE

PRK_3616_BLDG_006
Duvauchelle Reserve Tennis Club

Detailed Engineering Evaluation

Quantitative Report

Version FINAL

Seafield Road

Duvauchelle, Banks Peninsula

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

PRK_3616_BLDG_006

Duvauchelle Reserve Tennis Club

Detailed Engineering Evaluation
Quantitative Report
Version FINAL

Seafield Road
Duvauchelle, Banks Peninsula

Prepared By
Yifei Li

Reviewed By
Hamish Mackinven

Date
18 June 2013

Contents

Table Index

Figure Index

Appendices

Quantitative Report Summary

PRK_3616_BLDG_006

Duvauchelle Reserve Tennis Club

Detailed Engineering Evaluation

Quantitative Report - SUMMARY

Version FINAL

Duvauchelle Reserve Tennis Club

Seafield Road

Duvauchelle, Banks Peninsula

Background

This is a summary of the Quantitative report for the tennis club building located at Seafield Road, Duvauchelle, Banks Peninsula. The assessment is based in general on the current New Zealand codes, the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on the 19th of July 2011 and the visual inspection on the 10th of May 2013.

Building Description

The Duvauchelle Reserve tennis club building is located on Seafield Road, Duvauchelle. The exact construction date of the tennis club building is unknown; however it is believed to be constructed prior to 1970 based on site observations. An adjacent single storey timber framed building is situated approximately 1.5 m away to the west. The site is predominantly flat with insignificant variations on ground levels throughout.

The building is a single storey timber framed structure on concrete slab-on-grade foundations. The roof is pitched and is made up of timber rafters and lightweight metal cladding externally and plywood sarking internally. The roof of the building extends over the building's eastern walls to form a veranda on the eastern elevation of the building. The rafters are supported on veranda beams on timber posts embedded into concrete footing of unknown depth. The exterior wall cladding to the building comprises of lightweight timber weatherboard and metal cladding. Plywood lining is provided to the walls and ceilings internally.

On the north elevation, a concrete light pole used as a part of the tennis court's lighting system is located directly adjacent to the building. It penetrates through the overhanging roof eaves then extends approximately 4.0 m higher than the building roof.

The dimensions of the building are approximately 11.9 m long by 6.7 m wide and 3.7 m high at the roof apex. The overall footprint of the building is approximately 80 m^2.

Indicative Building Strength

The strength of the building is defined by the lowest percentage of new building standard (%NBS) along the bracing directions. The building's seismic capacity has been assessed to achieve 73% NBS due to the bracing capacity in the along direction. As the building has a capacity of greater than 67% NBS, it is not considered to be an earthquake risk structure in accordance with NZSEE guidelines.

Recommendations

As the building is not considered to be Earthquake Prone or Earthquake Risk in accordance with NZSEE guidelines, no strengthening of the building is recommended.

As the building has been analysed with a seismic capacity greater than 67% NBS with no immediate collapse hazards associated with the structure, general occupancy of the building is permitted.

.

1. Background

GHD has been engaged by the Christchurch City Council to undertake a detailed engineering evaluation of the tennis club building at Seafield Road, Duvauchelle, Banks Peninsula.

The report presents the Quantitative Assessment of the building's seismic capacity, and is based in general on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011 and the New Zealand Society for Earthquake (NZSEE) guidelines.

2. Compliance

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

2.1 Canterbury Earthquake Recovery Authority (CERA)

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

Section 38 – Works

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

Section 51 – Requiring Structural Survey

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

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

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

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

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

2.2 Building Act

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

Section 112 – Alterations

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

Section 115 – Change of Use

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

2.2.1 Section 121 – Dangerous Buildings

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

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

Section 122 – Earthquake Prone Buildings

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

Section 124 – Powers of Territorial Authorities

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

Section 131 – Earthquake Prone Building Policy

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

2.3 Christchurch City Council Policy

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

The 2010 amendment includes the following:

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

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

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

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

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

2.4 Building Code

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

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

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

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

3. Earthquake Resistance Standards

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

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

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

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

Figure 1 NZSEE Risk Classifications 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.

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 Duvauchelle Reserve tennis club building is located on Seafield Road, Duvauchelle. The exact construction date of the tennis club building is unknown; however it is believed to be constructed prior to 1970 based on site observations. An adjacent single storey timber framed building is situated approximately 1.5 m away on the west side. The site is predominantly flat with insignificant variations on ground levels throughout.

Refer to Figure 2 for site plan.

Figure 2 Duvauchelle Reserve Tennis Club Site Plan

The building is a single storey timber framed structure on concrete slab-on-grade foundations. The roof is pitched and is made up of timber rafters and lightweight metal cladding externally and plywood sarking internally. The roof of the building extends over the building's eastern walls to form a veranda on the eastern elevation of the building. The rafters are supported on veranda beams on timber posts embedded into concrete footing of unknown depth. The exterior wall cladding to the building comprises of lightweight timber weatherboard and metal cladding. Plywood lining is provided to the walls and ceilings internally.

On the north elevation, a concrete light pole used as part of the tennis court's lighting system, is located directly adjacent to the building. It penetrates through the overhanging roof eaves then extends approximately 4.0 m higher than the building roof (refer to Photo 5).

The dimensions of the building are approximately 11.9 m long by 6.7 m wide and 3.7 m high at the roof apex. The overall footprint of the building is approximately 80 m^2. A floor plan is presented in Figure 3 below.

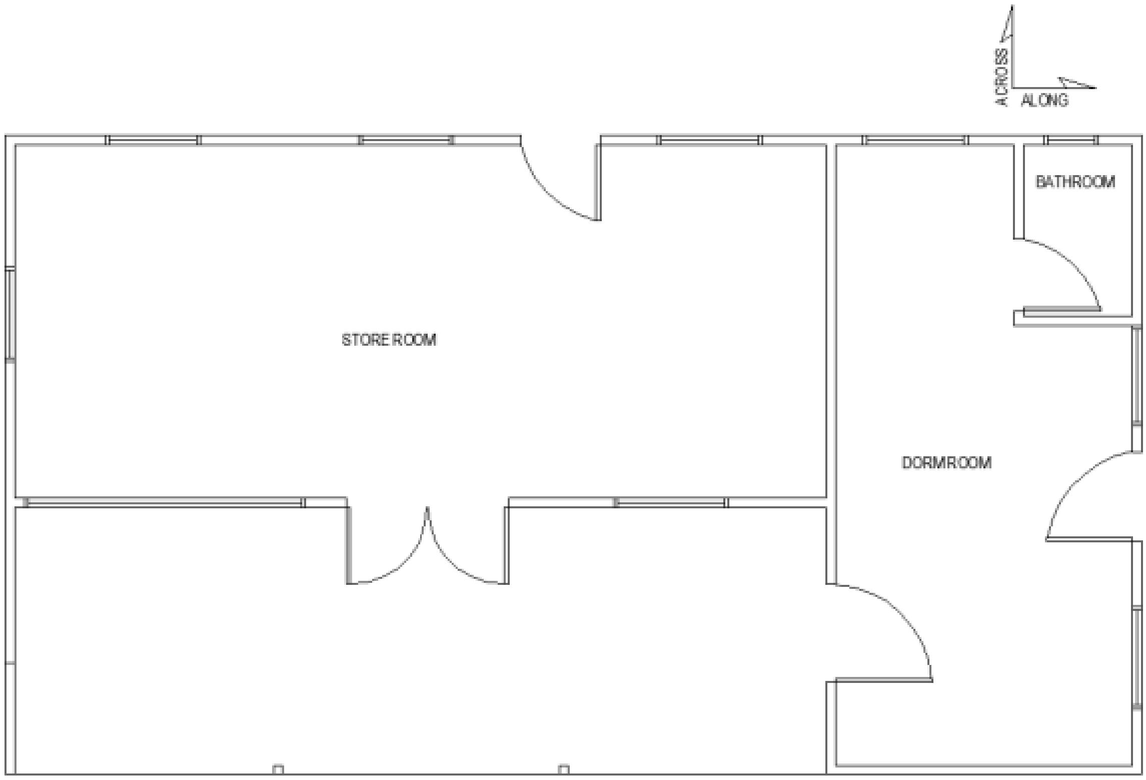

Figure 3 Duvauchelle Reserve Tennis Club Floor Plan

4.2 Gravity Load Resisting System

The gravity load resisting system of the building generally consists of load bearing timber framed walls.

The gravity loads of the lightweight corrugated iron sheeting are supported by timber rafters spanning between exterior timber framed load bearing walls, veranda beams and timber posts. The load bearing walls, veranda beams and timber posts then transfer the gravity loads to the concrete slab-on-grade foundations.

4.3 Lateral Load Resisting System

The lateral load resisting system of the building consists of plywood lined timber framed walls.

Lateral loads acting on the structure in both the along and across directions of the building are resisted by timber framed walls with plywood lining. The seismic forces from the roof structure are distributed to the bracing walls through the diaphragm action of the ceiling and roof sarking. The bracing walls then transfer the seismic loads to the concrete slab-on-grade foundations.

5. Damage Assessment

5.1 Surrounding Buildings

Due to limited access to surrounding buildings, there was no damage observed during the site inspections.

5.2 Residual Displacements and General Observations

Minor cracking and joint opening were observed to the internal plywood linings in several locations throughout the building, primarily above window and door openings. The damage to the linings is due to the movement of the building during an earthquake. Ductile timber wall frames are able to accommodate movement during an earthquake; however, the plywood linings behave in a more brittle manner and crack as a result of lateral movement.

Cracking to the concrete slab floor in the veranda on the east side of building was observed. Some of these appear to be existing shrinkage cracks that have opened up during the recent seismic activity.

5.3 Geotechnical Assessment

A specific geotechnical assessment or investigation was not undertaken as there was no evidence of liquefaction or lateral spreading clearly visible in the aerial photography taken following the September 2010, February 2011, June 2011 or December 2011 earthquakes.

A soil class of D (in accordance with NZS 1170.5:2004) has been adopted for the site due to the following reasons:

- No evidence of liquefaction following earthquakes;
- Anticipated deep sand or sand based soil beneath loess or loess colluvium.

6. Seismic Capacity Assessment

6.1 Site Investigation

A visual inspection of the building was undertaken on the 10th of May 2013. Both the interior and the exterior of the building were inspected.

The site inspection consisted of a visual inspection of 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 no structural elements. Some structural elements of the building were not able to be viewed due to wall and floor linings. It should be noted that inspection of the foundations of the structure was limited due to restricted access to the foundation structure below ground.

No construction drawings for the building were available. A full site measure of the building was undertaken to gather required dimensions of structural elements relevant to this Quantitative Assessment of the building.

A plan sketch of the building has been produced from the site measure up and is attached in Appendix C.

6.2 Quantitative Assessment

A Quantitative Assessment of the building was carried out using the information gathered from a full site measure of the building on the 10th of May 2013. From this information, the building's seismic capacity was determined in accordance with BRANZ publication in 1992. The demand for the building was calculated in accordance with NZS3604:2011 and NZS1170.5:2004 and the percentage of New Building Standard (%NBS) was assessed.

6.3 Seismic Demand

The demand on the structure was determined in accordance with Section 5 of NZS 3604:2011. The bracing unit demand per square meter was determined from Table 5.10. In accordance with this table from NZS 3604:2011 the seismic bracing demand calculated for this building is shown in Table 2 below.

Table 2 Seismic Demand from NZS3604:2011

	Un-factored Seismic Demand (BU/m^2)	**Multiplication Factor (Earthquake Zone 2 & Class D Soil)**	**Seismic Demand (BU/m^2)**
Tennis Club Building	6	0.8	**4.8**

The above bracing demand was also multiplied by a ductility correction factor of 2.39 to account for the difference in structural ductility between NZS3604:2011(μ=3) and the ductility assumed for this building (μ=2). The final seismic demand is shown in Table 3.

Table 3 Final Seismic Demand

Building	**Seismic Demand From NZS3604:2011 (BU/m^2)**	**Multiplication Factor**	**Seismic Demand (BU/m^2)**
Ground Floor	4.8	2.39	**11.5**

6.4 Capacity of the Wall Elements

The building was constructed prior to 1970, which predates available standards that use bracing ratings determined by modern day test methods. Therefore the bracing capacity of the plywood linings was determined in accordance with the "3604 Fix List Bracing Elements" publication by BRANZ in 1992.

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

$$\text{Aspect Ratio Factor} = \frac{2 \text{ x Length of Wall}}{\text{Wall Height}}$$

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

6.5 Calculation of %NBS

The bracing capacity both along and across the building are compared to the demand to determine the critical direction, and therefore the overall %NBS for the building. The %NBS for both along and across the building is presented in Table 4.

Table 4 Summary of Seismic Assessment Analysis

Building	**Bracing Direction**	**%NBS**
Tennis Club Building	Along	73%
	Across	89%
Overall %NBS		**73%**

6.6 Discussion of Results

The building was constructed prior to 1970 and the loading standard it was designed to is unknown. The seismic design loads it was designed to are likely to be significantly less than those required by the current loading standard. As a result, it would be expected that a building of this age would not achieve 100% NBS.

As the building has a capacity of greater than 67% NBS, it is not considered to be Earthquake Prone or Earthquake Risk in accordance with the NZSEE guidelines.

6.7 Occupancy

As the building has been analysed with a seismic capacity of greater than 67% NBS with no immediate collapse hazards associated with the structure, the general occupancy of the building is permitted.

.

7. Recommendations and Conclusions

7.1 Strengthening

As the building is not considered to be an Earthquake Prone or Earthquake Risk structure in accordance with NZSEE guidelines, no strengthening of the building is recommended.

7.2 Conclusions

As the building has been assessed to have a seismic capacity greater than 67% NBS, with no immediate collapse hazards associated with the structure, the general occupancy of the building is permitted.

8. Limitations

8.1 General

This report has been prepared subject to the following limitations:

- No intrusive structural investigations have been undertaken.
- No desktop geotechnical investigations have been undertaken.
- No intrusive geotechnical investigations have been undertaken.
- No material testing has been undertaken.

This report for the property at Seafield Road has been prepared by GHD Ltd ("GHD") for Christchurch City Council, hereto known as the client; may only be used and relied on by the client; must not be copied to, used by, or relied on by any person other than the client without the prior written consent of GHD; may only be used for the purpose of structural assessment for the property at Seafield Road and must not be used for any other purpose.

GHD and its servants, employees and officers otherwise expressly disclaim responsibility to any person other than the client arising from or in connection with this Report.

To the maximum extent permitted by law, all implied warranties and conditions in relation to the services provided by GHD and the Report are excluded unless they are expressly stated to apply in this Report.

The opinions, conclusions and any recommendations in this Report are based on assumptions made by GHD when undertaking services and preparing the Report, including (but not limited to) those specifically detailed in section 1 above.

GHD expressly disclaims responsibility for any error in, or omission from, this Report arising from or in connection with any of the Assumptions being incorrect.

Subject to the paragraphs in this section of the Report, the opinions, conclusions and any recommendations in this Report are based on conditions encountered and information reviewed at the time of preparation and may be relied on until 12 months from day of writing, after which time, GHD expressly disclaims responsibility for any error in, or omission from, this Report arising from or in connection with those opinions, conclusions and any recommendations.

Appendix A
Photographs

Photo 1 West Elevation of the Building

Photo 2 North Elevation of the Building

Photo 3 South Elevation of the Building

Photo 4 East Elevation of the Building

Photo 5 Light Pole Adjacent to North Elevation of the Building

Photo 6 Interior of the Building

Photo 7 Interior of the Building

Photo 8 Opening to Joint of Interior Linings

Photo 9 Cracking to Interior Linings

Photo 10 Cracking on Veranda Concrete Slab On-Grade

Photo 11 Cracking on Veranda Concrete Slab On-Grade

Appendix B

CERA Building Evaluation Form

Detailed Engineering Evaluation Summary Data V1.14

Location

Building Name:	Duvauchelle Reserve Tennis Club			Reviewer:	H D Mackinven
	Unit	No:	Street	CPEng No:	1003941
Building Address:			Seafield Road, Akaroa	Company:	GHD Limited
Legal Description:	Lot 7, DP 4974			Company project number:	51 31526 12
				Company phone number:	03 378 0900
	Degrees	Min	Sec		
GPS south:	43	45	14.00	Date of submission:	
GPS east:	172	56	37.00	Inspection Date:	10-May-13
				Revision:	0
Building Unique Identifier (CCC):	PRK_3616_BLDG_006			Is there a full report with this summary?	yes

Site

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

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):	
Ground floor split?	no		Ground floor elevation above ground (m):	
Storeys below ground	0			
Foundation type:	mat slab		if Foundation type is other, describe:	concrete slab on-grade
Building height (m):	3.70		height from ground to level of uppermost seismic mass (for IEP only) (m):	3.7
Floor footprint area (approx):	80			
Age of Building (years):	45		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):				
Use notes (if required):	storage room			
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber framed	rafter type, purlin type and cladding	lightweight roof cladding & timber frame roof
Floors:	concrete flat slab	slab thickness (mm)	unknown
Beams:	timber	type	
Columns:	timber	typical dimensions (mm x mm)	
Walls:			

Lateral load resisting structure

Lateral system along:	lightweight timber framed walls	**Note: Define along and across in detailed report!**	note typical wall length (m)	
Ductility assumed, μ:	2.00			
Period along:	0.40	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	lightweight timber framed walls		note typical wall length (m)	
Ductility assumed, μ:	2.00			
Period across:	0.40	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:	other light	describe	lightweight weatherboard
Roof Cladding:	Metal	describe	lightweight corrugated matel cladding
Glazing:			
Ceilings:	plaster, fixed		
Services(list):			

Available documentation

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

Damage

Site:
(refer DEE Table 4-2)

Site performance:		Describe damage:	
Settlement:	none observed	notes (if applicable):	
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	none apparent	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	none apparent	notes (if applicable):	

Building:

	Current Placard Status:			
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	
	Describe (summary):			
Across	Damage ratio:	0%		
	Describe (summary):			
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	no	Describe:	

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

Recommendations

	Level of repair/strengthening required:	minor non-structural	Describe:	see report
	Building Consent required:	no	Describe:	
	Interim occupancy recommendations:	full occupancy	Describe:	see report
Along	Assessed %NBS before e'quakes:	73% ##### %NBS from IEP below	If IEP not used, please detail assessment	see report

	Assessed %NBS after e'quakes:	73%
Across	Assessed %NBS before e'quakes:	89% ##### %NBS from IEP below
	Assessed %NBS after e'quakes:	89%

methodology:

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: 3.7m

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):	0.4	0.4
(%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:

	along	across
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

	along	across
$\%NBS_b$:	**#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

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**

Alignment of floors not within 20% of H	0.4	0.7	0.8

Therefore, Factor D: 1

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

3.5. Site Characteristics 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

	Along	Across
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

Plan Sketch

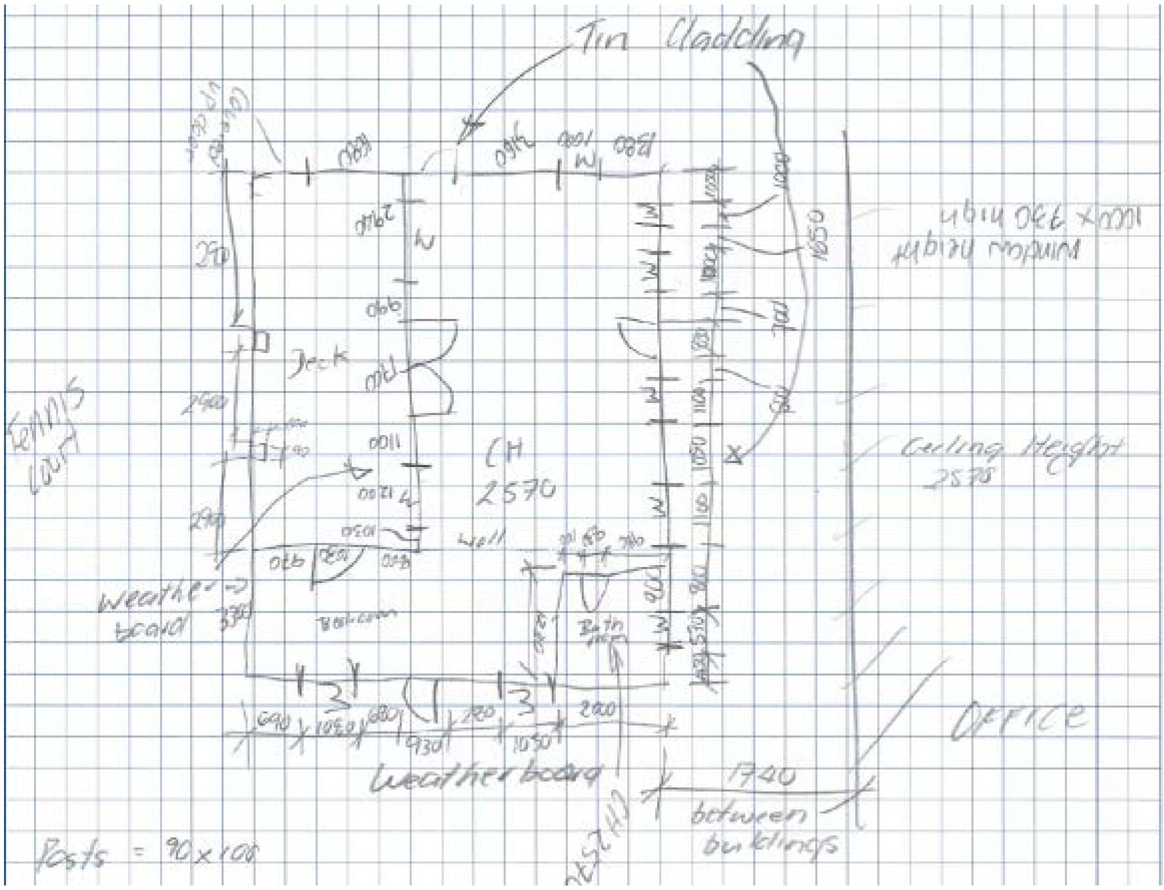

Figure 4 Plan Sketch of Building from Site Inspection

GHD

226 Antigua Street, Christchurch 8011
T: 64 3 378 0900   F: 64 3 377 8575   E: chcmail@ghd.com

Document Status

Rev No.	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
Final	Yifei Li	Hamish Mackinven		Nick Waddington		18/6/13