Scarborough Fare Tearooms
BU 1471-001 EQ2

Detailed Engineering Evaluation
Quantitative Report
Version FINAL

147, Esplanade, Sumner

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147 Esplanade
Sumner
Christchurch

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Scarborough Fare Tearooms

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Detailed Engineering Evaluation

Quantitative Report - SUMMARY

Version FINAL

147 Esplanade, Sumner

Background

This is a summary of the Quantitative report for the above 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, NZS 3604:2011 Timber-Framed buildings and a visual inspection and site measure up carried out on the 13th of August 2012.

Brief Description

The Scarborough Fare Tearooms' building is estimated to be constructed in the 1950s with extensions to the northern and eastern faces of the original building added at a later stage. The exact construction date and building plans/drawings were not available.

The building is a single storey timber framed structure on subfloor framing. The roof is pitched and consists of lightweight metal cladding. The roof structure is supported by load bearing timber frame walls and a lintel and timber post system over openings. The roof and floor loads are transferred into the foundations which consist of a concrete perimeter foundation wall and timber piles.

The original structure consists of brick veneer external wall cladding. The extensions to the building are cladded externally with timber weatherboard. A large proportion of the northern face of the building consists of glazing. The internal wall linings consist of plasterboard to both the timber framed walls and ceilings. The building is approximately 13m wide by 17m long with a wall height of 3.0m in the original building, and 2.4m in the extensions.

Key Damage Observed

Key damage noted includes:-

- Minor cracking to the internal plasterboard lining above several internal doorways (See Photograph 6 to Photograph 10).
- Minor movement of the floor in the kitchen area of the building (See Photograph 11).

Critical Structural Weaknesses

The site has a moderate to severe liquefaction potential, however due to the nature of the structure (timber framed, single storey structure), any settlement as a result of liquefaction is not expected to cause premature collapse of the building.

Indicative Building Strength (from DEE and CSW assessment)

Based on the quantitative analysis carried out on the structure using NZS 3604:2011 for Timber-Framed buildings and referencing the New Zealand Society for Earthquake Engineering (NZSEE) guidelines, the building has been assessed to be in the order of 67% NBS along the building and 53% NBS across. Based on this, the overall %NBS for the building is 53%.

Recommendations

As the building has been assessed to have a %NBS greater than 33%NBS, it is not considered to be an Earthquake Prone Building. Therefore, based on the Christchurch City Council's policy for Earthquake Prone Buildings no further action is required.

However the building's seismic capacity was assessed to be less than 67% and it is considered to be an Earthquake Risk building. Therefore GHD recommends that the building be strengthened to at least 67% NBS, which is the strengthening target level adopted by the Christchurch City Council. It is also recommended that more bracing elements be added along the northern face of the building to prevent any potential damage to the glazing in a seismic event.

In addition there are no immediate collapse hazards, or any significant critical structural weaknesses associated with the structure, therefore 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 Scarborough Fare Tearooms.

This report is a Quantitative Assessment of the building structure, and is based in general on NZS 3604:2011 Timber Framed buildings and the New Zealand Society for Earthquake Engineering (NZSEE) guidelines.

A Quantitative Assessment involves a full site measure of the building which is used to determine bracing capacity in accordance with manufacturers' guidelines where available. When the manufacturers' guidelines are not available, values for material strengths are taken from Table 11.1 of the NZSEE guidelines for the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes. The demand for the building is determined in accordance with NZS 3604:2011 and the percentage of new building standard (%NBS) is assessed.

At the time of this report, no modelling of the building structure had been carried out. The detailed analysis consisted of an analysis of the bracing capacity of the structure. No further analysis or calculations were carried out.

2. Compliance

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

2.1 Canterbury Earthquake Recovery Authority (CERA)

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

Section 38 – Works

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

Section 51 – Requiring Structural Survey

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

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

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

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

- The importance level and occupancy of the building
- The placard status and amount of damage
- The age and structural type of the building
- Consideration of any critical structural weaknesses
- 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 building's capacity based on a comparison of loading codes from when the building was designed and currently. It is a quick high-level procedure that can be used when undertaking a Qualitative analysis of a building. The guidelines also provide guidance on calculating a modified Ultimate Limit State capacity of the building which is much more accurate and can be used when undertaking a Quantitative analysis.

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

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

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

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

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

Table 1 %NBS compared to relative risk of failure

4. Building Description

Scarborough Fare Tearooms is located at 147 Esplanade, Sumner. The single storey building is currently used as a café with separate areas for food preparation and customer seating. The original timber frame building was assumed to have been constructed during the 1950s based on information observed in the entrance to the building. Extensions on the northern and southern faces of the original building have been added at a later date. The exact construction dates of the extensions are unknown.

The building is a single storey timber framed structure on subfloor framing. The roof is pitched and consists of lightweight metal cladding. The roof structure is supported by load bearing timber frame walls and a lintel and timber post system over openings. The roof and floor loads are transferred into the foundations which consist of a concrete perimeter foundation wall and timber piles.

The original structure consists of brick veneer external wall cladding. The extensions to the building are clad externally with timber weatherboard. A large proportion of the northern face of the building consists of glazing as shown in Figure 2. The internal wall linings consist of plasterboard to both the timber framed walls and ceilings. A sketch of the building plan showing walls that are capable of providing bracing and any openings such as windows and doors are shown in Figure 2.

The dimensions of the building are approximately 13 m wide by 17 m long and 3.5 m tall. The overall footprint of the building is approximately 200 m^2.

4.1 Gravity Load Resisting System

The gravity loads acting on the structure are resisted by a timber frame system. The original building consists of a dual-pitched roof supported by timber framing. The additions to the north and south of the original building consist of mono-pitched roofs supported by timber framing. The roof is clad with lightweight steel.

Gravity loads in the original section of the building are transferred from the roof through the timber purlins and timber trusses/rafters. The timber trusses/rafters are supported by timber posts and load bearing walls. The loads are transferred through the load bearing walls and posts into the foundations of the building. The extension to the south of the original building consist of a suspended timber floor supported by a perimeter concrete foundation wall as can be seen in Photograph 12. The foundations of the original building and the eastern extension were unable to be inspected but are assumed to consist of a suspended timber floor supported by timber piles internally, and a perimeter concrete foundation wall.

4.2 Lateral Load Resisting System

Lateral loads acting on the structure in both the long and short directions of the building are resisted by timber framed walls braced with plasterboard lining. The braced walls are distributed throughout the building in both the long and short directions. Seismic forces acting on the building during an earthquake are distributed to the braced walls through diaphragm action of the plasterboard lined ceiling.

The lateral loads in the substructure are expected to be distributed by diaphragm action provided by the floor into the concrete perimeter foundation walls. The concrete perimeter foundation walls provide bracing for the subfloor structure.

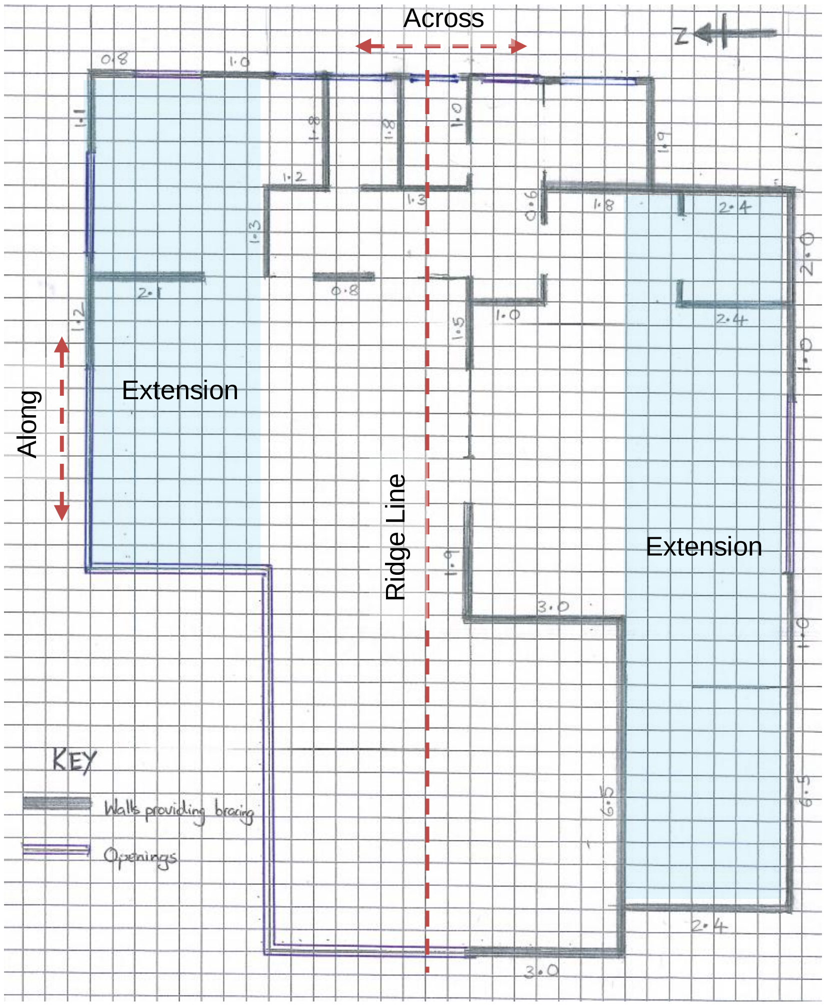

Figure 2 Plan Sketch of building showing bracing walls and openings

5. Damage Assessment

5.1 Surrounding Buildings

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

5.2 Residual Displacements and General Observations

Minor residual displacement of the structure was noted during the inspection of the building. The floor in the kitchen area of the building appears to have undergone movement due to the carpet tiles in this area having separated as can be seen in Photograph 11. This is likely due to some minor settlement of the building foundations.

Minor cracking to the internal plasterboard lining above and around doorways was observed in several locations throughout the building. These are shown in Photograph 6 to Photograph 10. The damage to the plasterboard lining is due to movement of the structure during an earthquake. The ductile timber frame is able to accommodate movement during an earthquake; however the plasterboard lining has behaved in a more brittle manner and has cracked as a result of the movement.

Opening up works were carried out on 13th of February 2012 to determine the extent of damage to the timber structure behind the plasterboard lining where cracking was visible. An area with diagonal cracking in the plasterboard lining above an internal doorway was opened up. The opening up work carried out can be seen in Photograph 13 and Photograph 14. The connection between the lintel and the timber post was checked to ensure load transfer could still be achieved through the connection. The opening up works performed did not reveal any damage to the timber framed structure behind the lining.

5.3 Ground Damage

The site is approximately 50m from the cliffs and evidence of minor rockfall was noted during the site inspection beneath these cliffs.

6. Survey and Investigation

Given the low level of structural damage, and only minor settlement of the foundations noted, a floor level survey has not been completed for the building.

7. Geotechnical Investigation

The site is located adjacent to Pegasus Bay on relatively flat area in Southeast Sumner and approximately 50m from the cliff. The site is bordered to the south by Herebeden Ave with residential properties surrounding the green area.

7.1 Published Information on Ground Conditions

7.1.1 Published Geology

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

- Holocene soils of Christchurch Formation, comprising dominantly sand of fixed and semi-fixed dunes and beaches.

7.1.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that there are six (6) boreholes within 100m of the site, of those three were closely grouped. Four of the boreholes were considered in this study. From the bore logs, the ground conditions are indicated to comprise predominantly sand.

Table 2 ECan Borehole Summary Table

Bore Name	Log Depth	Groundwater	Distance & Direction from Site
N36/0155	~2.13m	N/A	50m, SE
N36/0160	~4.57m	N/A	20m, S
N36/0161	~4.57m	N/A	70m, W
N36/0172	~6m	N/A	100m, S

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

7.1.3 EQC Geotechnical Investigations

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

7.1.4 Land Zoning

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

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

[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.

The site is classified as "not applicable" N/A. This means that the site is generally suitable for houses to be repaired or rebuilt. **Not applicable** means that non-residential properties in urban areas, properties in rural areas or beyond the extent of land damage mapping, and properties in the Port Hills and Banks Peninsula have not been given a Technical Category.

7.1.5 Post February Aerial Photography

Aerial photography taken following the 22 February 2011 earthquake (Figure 1) show signs of liquefaction with sand boils emminent near the site.

Figure 3 Post February 2011 Earthquake Aerial Photography[2]

7.1.6 Summary of Ground Conditions

From the above information the ground conditions adjacent to the site comprise predominantly of sand.

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

7.2 Seismicity

7.2.1 Nearby Faults

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

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

Known Active Fault	Distance from Site (km)	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	140 NW	8.3	~300 years
Greendale (2010) Fault	30 W	7.1	~15,000 years
Hope Fault	110 N	7.2~7.5	120~200 years
Kelly Fault	110 NW	7.2	~150 years
Porters Pass Fault	75 NW	7.0	~1100 years

Recent earthquakes since 4 September 2010 have identified the presence of a new active fault system / zone 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.

7.2.2 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.

7.3 Field Investigations

In order to further understand the ground conditions at the site, intrusive testing comprising one piezocone/seismic CPT investigation was conducted at the site on 02 April 2012.

The locations of the tests are tabulated in Table 4.

Table 4 Coordinates of Investigation Locations

Investigation	Depth (m bgl)	Easting (NZMG)	Northing (NZMG)
CPT 001	20.0	2491361	5737137

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

[4] GNS Active Faults Database

The CPT investigation was undertaken by McMillans Drilling Ltd on 04 April 2012 to a target depth of 20m below ground level. Please refer to the attached CPT results for detail (Appendix C).

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

7.4 Ground Conditions Encountered

7.4.1 Summary of CPT-Inferred Lithology

Table 5 Summary of CPT-Inferred Lithology

Depth (m)	Lithology [1]	Cone Tip Resistance q_c (MPa)	Friction Ratio Fr (%)	Relative Density Dr (%)
0 – 10.5	Sand to silty sand	6-12	~1	60-100
10.5 – 18.0	Sandy silt to clay silt	~4	~2	(Su = 80 - 200 kPa)
18.0 – 20.0	Clay silt to silty clay	~2	~2	(Su = 60 - 200 kPa)

7.5 Interpretation on Ground Conditions

7.5.1 Liquefaction Assessment

Assumptions made for the analysis process are as follows:

- D50 particle sizes for the site soil (sands) from CPT soil analysis
- Importance Category 2, post seismic event (50-year design life)
- PGA 0.57g; and,
- Groundwater level of 3m

The following equation has been used to approximate soil unit weight from the CPT investigation data: [6]

$$\gamma = \frac{\gamma_w Gs}{2.65}\left(0.27\log Fr \ + \ 0.36\log\left(\frac{qc}{p_{atm}}\right) + 1.236\right)$$

This obtained unit (saturated) unit weight is 16 to 20kN/m^3 (saturated).

The liquefaction analysis process has been conducted using the methodology from Stark & Olson[7], and from the NZGS Guidelines[8].

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

[6] Robertson P.K., & Cabal K.L. 2010: *Estimating soil unit weight from CPT*. Gregg Drilling & Testing Inc.: Signal Hill, California, USA.

7.5.2 Results of Liquefaction Analysis

The results of the liquefaction analysis, as outlined in Table 6, indicate that depths of 3.5-10.5 are considered low to severe liquefiable.

Table 6 Summary of Liquefaction Susceptibility

Depth (m)	Lithology	Triggering Factor F_L	Liquefaction Susceptibility [9]
0 – 3.5	Sand to silty sand	-	*Not Liquefiable*
3.5 – 5.5		1-2	Low to high
5.5 – 10.5		0-1	Severe
10.5 – 18.0	Sandy silt to clayey silt	-	*Not Liquefiable*
18.0 – 20.0	Clayey silt to silty clay	-	*Not Liquefiable*

7.5.3 Interpretation of Analysis

Overall, the site is considered to be of high susceptibility to liquefaction based on:

- No signs liquefaction or settlement was observed during site inspection however, due to sand boils as evidenced from aerial photography following February 22 event, the site is moderately to severely susceptible to liquefaction.
- The ground conditions encountered saturated sand layers considered to be severely liquefiable

7.5.4 Slope Failure and/or Rockfall Potential

The site itself is located adjacent to the beach and within a predominantly flat section of Sumner. However, the site is approximately 50m from the cliffs. Evidence of rockfall was noted during the site inspection beneath these cliffs. However, minor rockfall is not anticipated to impact on the site. An analysis of the slope stability is beyond the scope of this report. Any localised retaining structures and/or embankments installed at or near the cliff should be investigated to determine the site-specific slope failure or rockfall potential.

7.5.5 Foundation Recommendations

Based on the information presented above, we recommend the following for the subject site:

- All foundations be specifically-designed by a suitably qualified and experienced geotechnical engineer;
- These foundations should be deep, and designed to mitigate the impacts of subsoil liquefaction; and,

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[7] Olson, S.M. & Stark, T.D. (2002); "Liquefied strength ratio from liquefaction flow failure case histories". *Canadian Geotechnical Journal*, 39 (3), 629–647pp.

[8] Cubrinovski M., McManus K.J., Pender M.J., McVerry G., Sinclair T., Matuschka T., Simpson K., Clayton P., Jury R. 2010: *Geotechnical earthquake engineering practice: Module 1 – Guideline for the identification, assessment and mitigation of liquefaction hazards*. NZ Geotechnical Society

[9] Table 6.1, NZGS Guidelines Module 1 (2010)

The soil class of D (in accordance with NZS 1170.5:2004) recommended in Section 8 of the DEE/IEP is still believed to be appropriate.

8. Seismic Capacity Assessment

8.1 Qualitative Assessment

An initial Qualitative Assessment has been completed by GHD for the Scarborough Fare Tearooms building. This included a visual inspection of the building which was undertaken on 20th January 2012. Both the interior and exterior of the building were inspected.

The Qualitative Assessment 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 non-structural elements.

The %NBS score determined for the building has been based on the IEP procedure described by the NZSEE based on the information obtained from opening up works and visual observations of the building only. The lack of braced walls in the shop front area of the building was treated as a 'plan irregularity' Critical Structural Weakness (CSW) due to the seismic bracing elements being distributed irregularly throughout the building. This has been accounted for in the IEP calculations by reducing the %NBS by 30%. The site is also highly susceptible to liquefaction and therefore a 30% reduction factor for this CSW was also applied to the %NBS. Following the Qualitative Assessment, an initial capacity of the building was assessed to be 39% NBS. This %NBS is now superseded by the capacity of the building assessed through a more detailed Quantitative Assessment outlined below.

8.2 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 13th of August 2012. From this information, the building's bracing capacity was determined in accordance with NZS 3604:2011 and the NZSEE guidelines. The demand for the building was calculated in accordance with NZS 3604:2011 and the percentage of New Building Standard (%NBS) was assessed.

8.2.1 Building demand

The demand on the structure was determined in accordance with Section 5 of NZS 3604:2011. The bracing unit demand per square metre was determined from Table 5.8. In accordance with Table 5.8 of NZS 3604:2011 (for a single storey building with light roof, medium single-storey cladding on heavy subfloor framing) a bracing demand of 20 BU/m^2 for the subfloor structure and 13BU/m^2 for the single storey walls is taken. As the building is located in Christchurch (earthquake zone 2) on Class D soils, a multiplication factor of 0.8 is applied to reduce the demand in accordance with Table 5.8 of NZS 3604:2011. Therefore the total bracing demand for the building is;

$$\text{Single storey walls } \mathrm{BU_{demand}} = (0.8 \text{ x } 13 \ \mathrm{BU/m^2} \text{ x } 200\mathrm{m^2})$$

$$= 2080 \text{ BU}$$

$$\text{Subfloor structure } \mathrm{BU_{demand}} = (0.8 \text{ x } 20 \ \mathrm{BU/m^2} \text{ x } 200\mathrm{m^2})$$

$$= 3200 \text{ BU}$$

8.2.2 Wall bracing capacity

The plasterboard linings evident throughout the building appear to be relatively new when compared with the building age. The original building is estimated to be constructed in the 1950's with extensions to the northern and southern sides added later. This suggests that a bracing design of the whole building was undertaken during the alterations and extensions to the building. However no specific drawings or design calculations were available containing the bracing capacity. Therefore the capacity of the existing wall linings was determined in accordance with Table 11.1 of the NZSEE guidelines and the "3604 Fix List Bracing Elements" publication by BRANZ in 1992.

For this purpose, the strength value of gypsum wall board given in Table 11.1 of the NZSEE guidelines (3kN/m each side) was converted to equivalent bracing units (1kN = 20BU) and then multiplied by the strength reduction factor of 0.7. This value was used for all walls with plasterboard lining on one side only. Therefore the bracing capacity for walls with plasterboard lining on only one side is taken as;

$$\mathrm{BU_{equivalent}} = \left(0.7\,\mathrm{x}\,\frac{3\mathrm{kN}}{\mathrm{m}}\,\mathrm{x}\,\frac{20\mathrm{BU}}{\mathrm{kN}} = 42\mathrm{BU/m\ each\ side}\right)$$

For walls that are lined with plasterboard on both sides, the value calculated from Table 11.1 of NZSEE guidelines will be 84 BU/m . However this value is judged to be high considering modern wall bracing systems have lower bracing ratings. Therefore the bracing capacity for walls with plasterboard lining on both sides is taken as 60 BU/m from the "3604 Fix List Bracing Elements" publication by BRANZ in 1992.

Due to the height of some walls in the original building being greater than 2.4m, a reduction factor was applied to the wall capacity in accordance with Cl 8.3.1.4 of NZS 3604: 2011 as follows;

$$\text{Height difference factor} = \frac{2.4\mathrm{m}}{\text{Height}}$$

Section 11.4 of the NZSEE guidelines states that shear panels can utilise their full bracing capacity for aspect ratios (height-to-width) up to 2:1. Shear panels with aspect ratios greater than 3.5:1 are not expected to provide adequate bracing capacity in accordance with NEHRP Recommended Provisions (BSSC,2000) as follows. Therefore, any sections of wall with an aspect ratio greater than 3.5:1 were not included for the purpose of the bracing calculations. The walls in this building are 2.4m and 3.0m in height, and as such any wall less than 0.7m and 0.85m in length respectively was not considered for the bracing calculations.

The subfloor bracing capacity is provided by the reinforced concrete perimeter foundation wall. The bracing capacity rating for this was determined as 300BUs/m in accordance with Table 5.11 of NZS 3604:2011. As the bracing capacity rating is very high, the bracing capacity will far exceed the bracing demand for the subfloor structure. As such no bracing analysis was carried out for the subfloor structure as the single-storey wall bracing capacity is more critical.

The calculated wall bracing capacities along and across the building are shown in Table 7.

Table 7 Bracing Units Provided

Direction	Bracing Units Provided
Along the building	1402 BU's
Across the building	1094 BU's

8.2.3 %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 value is calculated as follows;

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

The %NBS for both along and across the building is presented in Table 8.

Table 8 %NBS

Direction	%NBS
Along the building	67%
Across the building	53%

Following a detailed assessment the building has been assessed as having a seismic capacity of 53% New Building Standard (NBS). Under the NZSEE guidelines the building is not considered to be an Earthquake Prone building as it achieves above 33% NBS. The building however is considered to be Earthquake Risk as it achieves below 67% NBS.

8.3 Discussion of Results

The results obtained are consistent with the amount of bracing present in the building. A number of lengths of wall were discounted due to their length being less than the minimum required to achieve a width: height ratio of less than 3.5:1. The high liquefaction potential at the site was considered insignificant as any liquefaction induced settlement is not expected to cause a premature collapse of a single storey, timber framed structure. Along the northern face, very little bracing is provided due to the high proportion of glazing present compared to bracing walls. The building is likely to be very flexible along this face, and therefore there is potential for damage to the glazing in a seismic event.

The building's strength is less than 67% NBS so it is deemed to be potentially earthquake risk. However, the building has a strength greater than 33% NBS and therefore is not deemed to be potentially earthquake prone. Based on the Christchurch City Council's policy for earthquake prone buildings no further action is required.

8.4 Occupancy

As the building has been assessed to have a %NBS greater than 33% NBS, it is not considered to be an Earthquake Prone Building. In addition there are no immediate collapse hazards, or critical structural weaknesses associated with the structure that could cause the collapse of the structure. Therefore general occupancy of the building is permitted.

9. Recommendations and Conclusions

The building has been assessed to have a %NBS greater than 33% NBS and is not considered to be an Earthquake Prone building. Therefore, based on the Christchurch City Council's policy for earthquake prone buildings no further action is required.

However the building's seismic capacity was assessed to be less than 67% and it is considered to be an Earthquake Risk building. Therefore GHD recommends that the building be strengthened to at least 67% NBS, which is the strengthening target level adopted by the Christchurch City Council. It is also recommended that more bracing elements be added along the northern face of the building to prevent any potential damage to the glazing in a seismic event.

There are no immediate collapse hazards, or critical structural weaknesses associated with the structure, therefore general occupancy of the building is permitted.

10. Limitations

10.1 General

This report has been prepared subject to the following limitations:

- No intrusive structural investigations have been undertaken other than the opening up works described in this report.
- No level or verticality surveys have been undertaken.
- No material testing has been undertaken.
- No calculations, other than the wall bracing calculations included in this report, have been carried out on the structure.

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

10.2 Scope and Limitations of Geotechnical Investigation

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

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

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

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

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

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

Appendix A

Photographs

Photograph 1 View of extension (yellow section) to the northern side of the building.

Photograph 2 View of minor cracking between the building foundations and external concrete steps.

Photograph 3 View of extension to southern side of building.

Photograph 4 Cliffs to the south-east of the building.

Photograph 5 Brick veneer cladding.

Photograph 6 Cracking to plasterboard lining above internal doorway.

Photograph 7 Diagonal cracking to plasterboard lining above internal doorway.

Photograph 8 Further cracking to plasterboard lining above internal doorway.

Photograph 9 Cracking to plasterboard lining between external door and window.

Photograph 10 Cracking to plasterboard lining between external door and window.

Photograph 11 Minor residual displacement of floor shown by displaced carpet floor tiles.

Photograph 12 Substructure and foundations of extenison to southern side of original building.

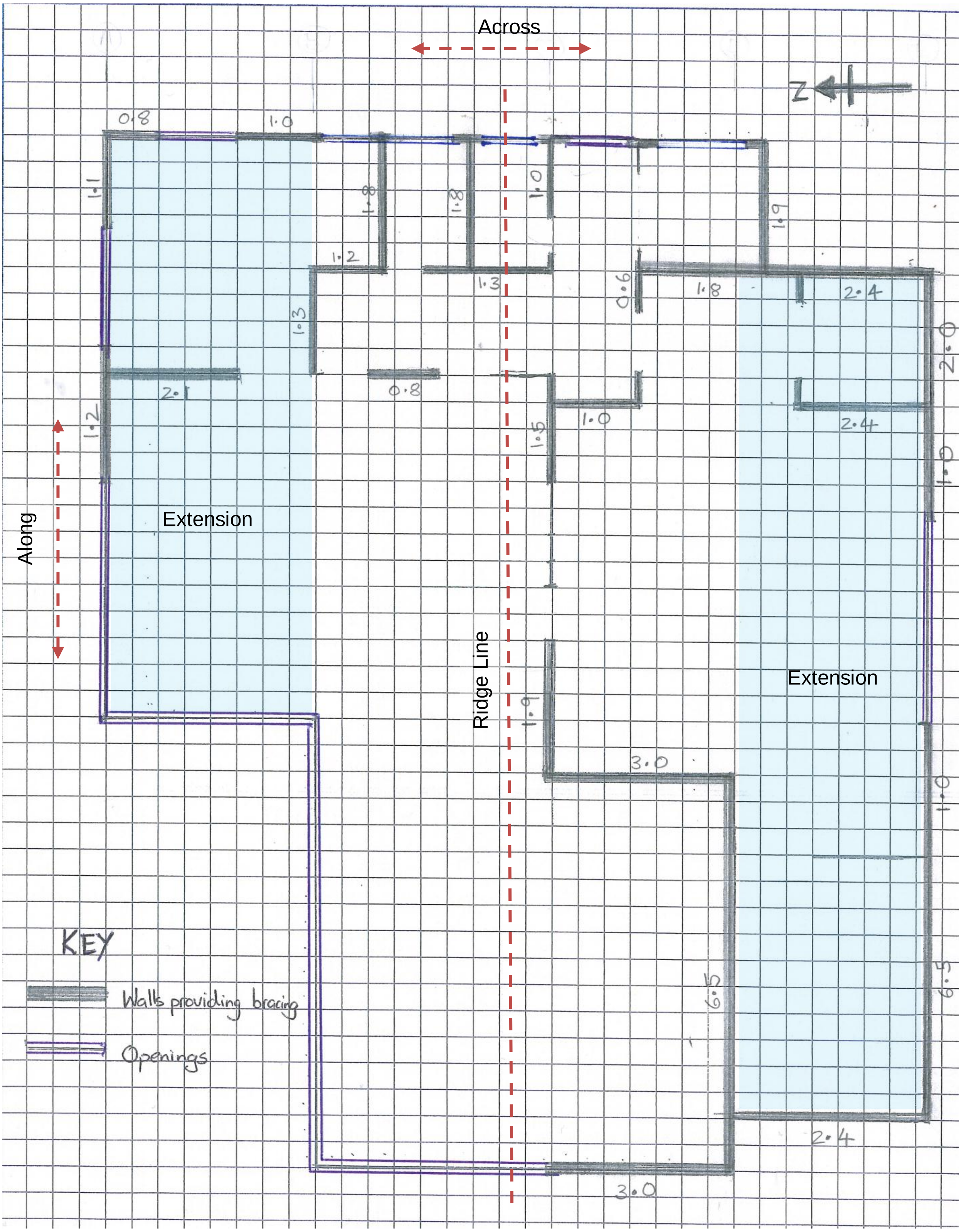
Photograph 13 Opening up works to determine extent of damage to timber structure.

Photograph 14 Close up of opening up work to cracking above internal doorway.

Appendix B

Existing Drawings / Sketches

No structural or architectural drawings have been made available for this building. Shown below is a sketch of the building showing key structural elements which include bracing walls and openings.



Appendix C

Geotechnical Information

CPT ANALYSIS NOTES

Soil Type

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

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

Liquefaction Screening

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

- high susceptibility
- medium susceptibility
- low susceptibility

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

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

Low susceptibility is all other cases.

Relative Density (D_R)

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

Undrained Shear Strength (S_U)

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

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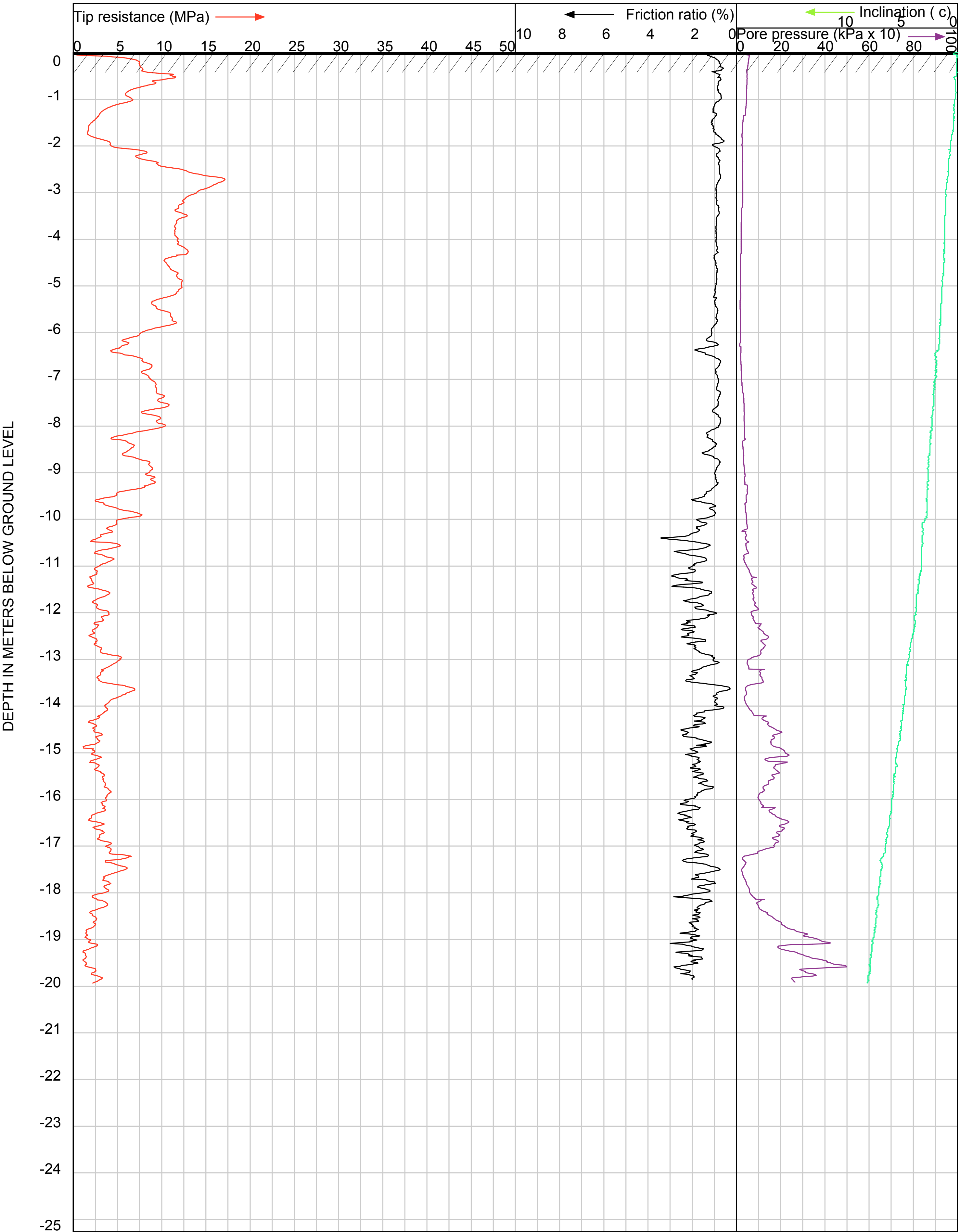


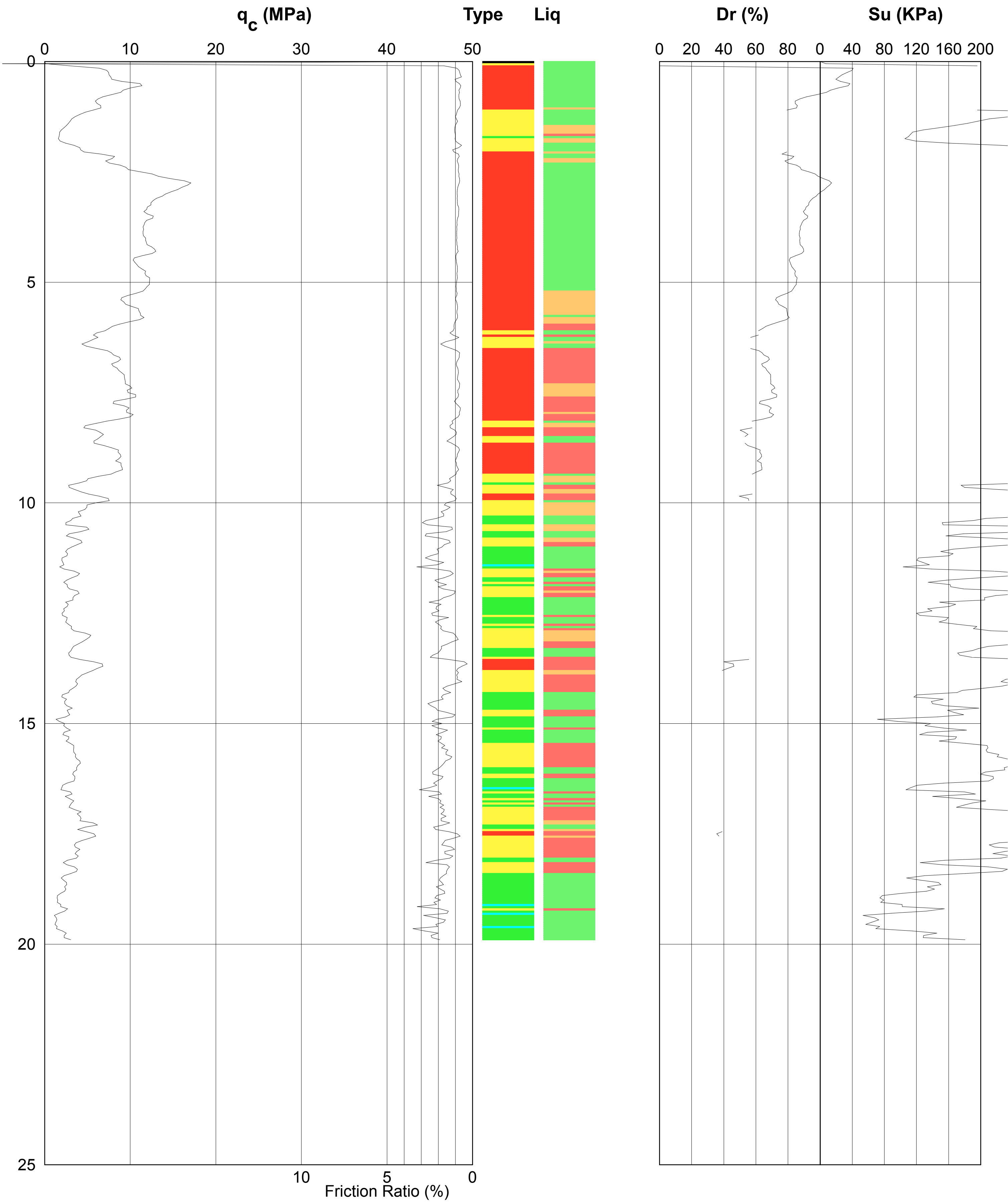
CLIENT : GHD
LOCATION : Christchurch Various (CCC Properties)
DATE : 4-4-2012
OPERATOR : H. Pardoe
REMARK 1 : CPTu31
REMARK 2 : Target Depth

JOB # : 10386
TEST # : 31

McMILLAN
DRILLING SERVICES

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

PIEZOCONE PENETROMETER TEST (CPTU) INTERPRETIVE REPORT



Job No:	10386	Date:	4-4-2012
CPT No:	CPTu31	Operator:	H. Pardoe
Project:	GHD	Remark:	Target Depth
Location:	Christchurch Various (CCC Properties)		

McMILLAN DRILLING SERVICES

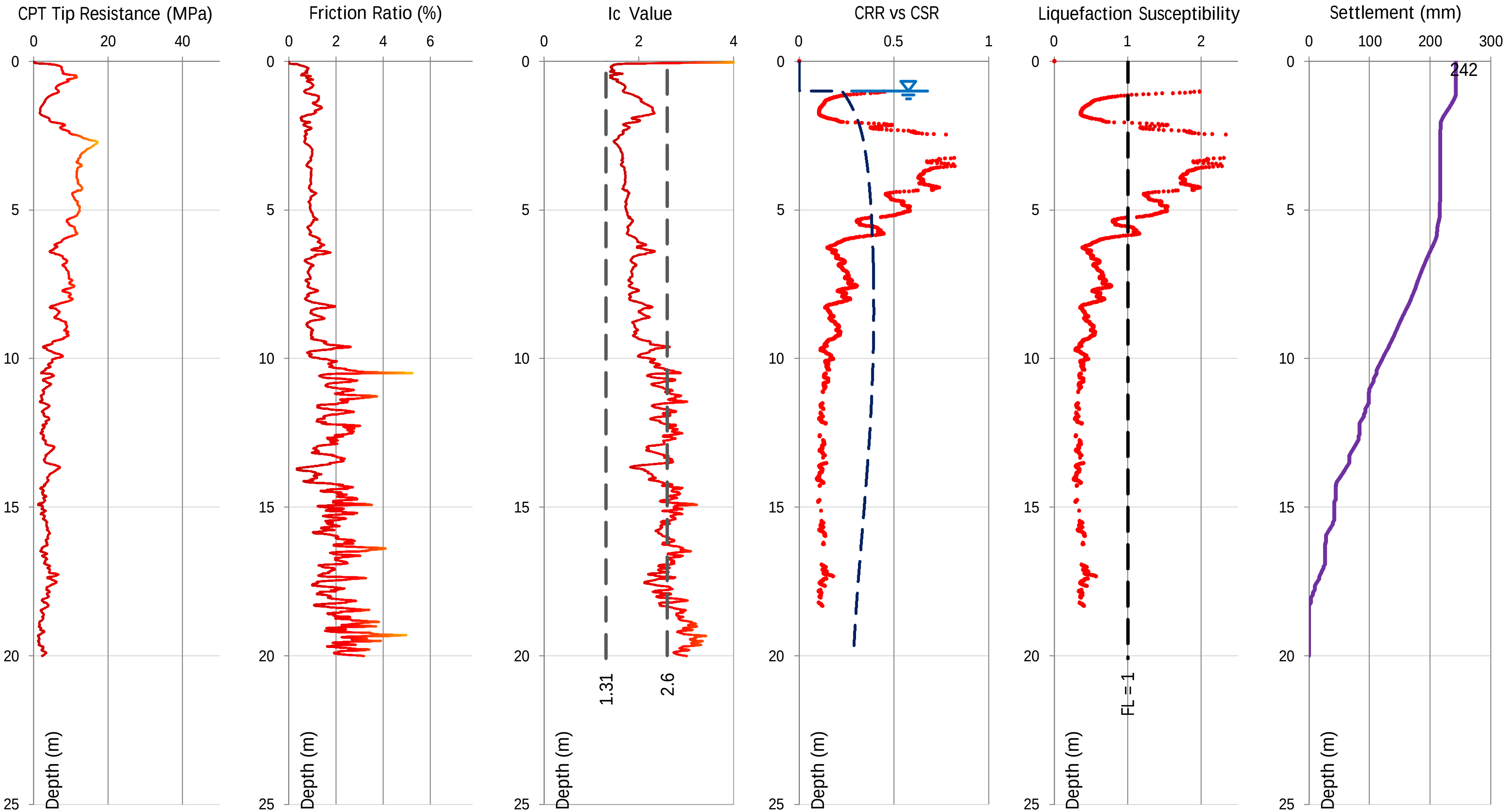
SOIL LIQUEFACTION SUSCEPTIBILITY ASSESSMENT

LOCATION :	CPT 11A	SHEET :	1
PROJECT :	Scarborough Fare Tearooms	CALCULATED :	DBS
JOB NO :	51 30596 11	CHECKED BY :	
TEST DATE :	4 Apr 2012	DATE :	23/8/2012

PGA (a_{max}):	0.35 g	Groundwater Level (m bgl):	1.0	Bore depth (m):	20
EQ Magnitude:	7.5	Atmospheric Pressure (kPa):	101	Test data step (m):	0.01

Total Estimated Settlement (mm)

242



N:\NZ\Wellington\Projects\51\30596\11 Scarborough Fare Tearooms\Investigation\Geotech Desk Study\Liquefaction\Liquefaction and Settlement Analysis 11A

Appendix D

CERA Building Evaluation Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Scarborough Fare Tearooms	Reviewer:	Derek Chinn
	Unit / No: 147 / Street: Esplanade	CPEng No:	177243
Building Address:	147 Esplanade	Company:	GHD
Legal Description:		Company project number:	51/30596/23
		Company phone number:	04 472 0799
	Degrees / Min / Sec		
GPS south:		Date of submission:	
GPS east:		Inspection Date:	18/1/12
		Revision:	1
Building Unique Identifier (CCC):		Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	sandy silt	Soil Profile (if available):	
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):	20	If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):	50	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:	timber piles	if Foundation type is other, describe:	
Building height (m):	3.50	height from ground to level of uppermost seismic mass (for IEP only) (m):	3.5
Floor footprint area (approx):	200		
Age of Building (years):	60	Date of design:	1976-1992
Strengthening present?	yes	If so, when (year)?	
		And what load level (%g)?	
Use (ground floor):	commercial	Brief strengthening description:	Plasterboard bracing of timber walls.
Use (upper floors):	commercial		
Use notes (if required):			
Importance level (to NZS1170.5):	IL2		

Gravity Structure

Gravity System:	frame system		
Roof:	timber framed	rafter type, purlin type and cladding	
Floors:	timber	joist depth and spacing (mm)	
Beams:	timber	type	
Columns:	timber	typical dimensions (mm x mm)	
Walls:	non-load bearing		0

Lateral load resisting structure

Note: Define along and across in detailed report!

Lateral system along:	lightweight timber framed walls		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:	plaster system	describe	
Roof Cladding:	Metal	describe	
Glazing:	aluminium frames		
Ceilings:	fibrous 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:	No ground damage observed.	Describe damage:	
Settlement:	none observed	notes (if applicable):	
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	none apparent	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	none apparent	notes (if applicable):	

Building:

	Current Placard Status:	green		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	
	Describe (summary):	Minor damage. Less than 5% if at all.		
Across	Damage ratio:	0%		
	Describe (summary):	Minor damage. Less than 5% if at all.		
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	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:	minor structural	Describe:	more bracing needed (see report)
Building Consent required:	yes	Describe:	
Interim occupancy recommendations:	full occupancy	Describe:	

Along	Assessed %NBS before:	67%	#### %NBS from IEP below	If IEP not used, please detail assessment methodology:	Bracing schedule as detailed in report
	Assessed %NBS after:	67%			
Across	Assessed %NBS before:	53%	#### %NBS from IEP below		
	Assessed %NBS after:	53%			

GHD

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Document Status

Rev No.	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
FINAL	Shashank Kumar	Derek Chinn		Nick Waddington		02/10/2012