REPORT

Christchurch City Council

Robert McDougall Gallery Building Geotechnical site walkover and desktop study report

Report prepared for:

Christchurch City Council

Report prepared by:

Tonkin & Taylor Ltd

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1 Introduction

1.1 General

This report summarises the results of a geotechnical walkover and desk study assessment of the Robert McDougall Art Gallery at 9 Rolleston Avenue, Christchurch, that has been completed by Tonkin & Taylor Ltd (T&T) for the Christchurch City Council (CCC).

The work described in this document was commissioned by CCC and has been completed in accordance with the terms and conditions outlined in T&T's letter of engagement dated 13 August 2012.

The earthquakes of 22 February 2011, and to a lesser extent those of 04 September 2010, 13 June 2011, and 23 December 2011 caused widespread land and structural damage throughout much of the Christchurch area. The purpose of our work was to conduct a site walkover inspection, review readily available building foundation drawings, and provide a preliminary assessment of foundation performance following the Canterbury Earthquake sequence.

The structural assessment of the building has been completed by Holmes Consulting Group (HCG).

1.2 Project description

The site is accessed off Rolleston Avenue and is within the Christchurch Botanic Gardens (Figure 1 in Appendix A). The building is located on an essentially flat site with an approximately rectangular footprint of 25 by 50 m. The site is situated within a meander of the Avon River which runs more than 100 m to the north and south.

The single story reinforced concrete and brick building with a partial single story basement was constructed in the early 1930's. The basement was extended to cover much of the building footprint during the 1980's. T&T understand, based on structural drawings from the 1930's provided by HCG that the building is supported on a combination of strip and pad footings.

T&T have previously completed site specific geotechnical investigations for the Canterbury Museum on the adjacent site of 11 Rolleston Avenue which have been detailed in T&T's report dated March 2012[1]. The Canterbury Museum has given permission for CCC to use the site specific geotechnical data obtained for their site.

1.3 Scope of work

The following scope of work has been completed by T&T for the purposes of this report:

- Compilation of existing geotechnical data from readily available sources near the subject site;
- A geotechnical site walkover inspection of the Robert McDougall Art Gallery building;
- Preparation of a preliminary geotechnical model for the site;
- Engineering analysis of the above model to evaluate liquefaction susceptibility and seismic settlement potential;
- A preliminary assessment of foundation damage due to the Canterbury Earthquake sequence; and,
- Preparation and issue of this geotechnical walkover and desk study assessment report.

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[1] Tonkin & Taylor Ltd (March 2012) *"Canterbury Museum Geotechnical Investigation and Assessment Report."*

2 Field investigations

2.1 Site walkover inspection

A walkover inspection of the site was conducted by T&T geotechnical engineers on 08 October 2012. The results of this work are described in Sections 3, 4 and 5 of this report.

2.2 Machine drilled boreholes[2]

Three machine-drilled boreholes at the adjacent 11 Rolleston Avenue site had been undertaken on 12 to 19 December 2011 at the approximate locations shown on the attached site plan (Figure 1, Appendix A). The boreholes were advanced to depths of between 25.0 and 25.9 m below ground level (bgl) using a sonic direct push drill rig. A detailed description of the drilling conducted, and logs of the boreholes, are presented in Appendix B.

2.3 Cone penetration testing[2]

Three Cone Penetration Tests (CPTs) at the adjacent 11 Rolleston Avenue site had been undertaken on 16 December 2011 at the approximate locations shown on the attached site plan (Figure 1, Appendix A). They were then advanced to refusal at depths of between 3.4 and 5.4 m bgl. A detailed description of the CPT testing conducted, and the CPT investigation results, are presented in Appendix B.

[2] Machine drilled borehole and Cone Penetration Test data has been generously provided for use at the Robert McDougall Art Gallery by Canterbury Museum.

3 Investigation findings

3.1 Site walkover

Relatively little damage was observed within the first floor of the Robert McDougall Art Gallery. Several cracks were present within the floor slab. However, based on conversations with Art Gallery maintenance staff T&T understand that these cracks are likely to have occurred prior to the Canterbury Earthquake sequence.

T&T observed a number of cracks within the walls and floor of the basement. No evidence of building settlement or foundation bearing capacity failure was observed during our site walkover inspection.

No liquefaction ejected sand or silt was observed on the site during T&T's site walkover inspection. Art Gallery maintenance staff said that no ejected sand or silt was present on the site following the 22 February 2011 earthquake.

3.2 Subsurface conditions

3.2.1 Published geology and geotechnical information

Published geological information[3] describes the subject site as being underlain by predominantly alluvial sand and silt overbank deposits of the Springston Formation. The Springston Formation generally consists of Holocene fluvial channel and overbank sediments composed of well-sorted gravel, sand and silt.

Environment Canterbury (ECan) bore logs are available for a number of wells in the area. A review of this data indicates that the site is likely to be underlain by:

- 1.5 to 5.0 m of sandy silt and silty sand;
- 7.0 to 8.5 m of sandy gravel;
- 10.0 to 13.0 m of interbedded sand and silt; and,
- An unknown thickness of gravel from between 20.0 to 25.0m bgl.

It should be noted that the ECan well logs were not made for geotechnical purposes, and as such, should only be used to provide a general indication of subsurface conditions.

3.2.2 T&T geotechnical investigations

T&T undertook three machine drilled boreholes and three Cone Penetration Tests (CPTs) at the Canterbury Museum site adjacent to the Robert McDougall Art Gallery site. These investigations indicate that the site is generally underlain by the following general succession of strata:

- 0.0 to 3.5 m bgl of stiff silt and loose silty sand; overlying,
- 3.5 to 4.25 m bgl of medium dense gravel;
- 4.25 to 5.25 m bgl of medium dense sand;
- 5.25 to 10.0 m bgl of dense to very dense gravel;
- 10.0 to 15.0 m bgl of dense to very dense sand;
- 15.0 to 21.0 m bgl of interbedded stiff silt and medium dense silty sand;

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[3] L.J Brown et al, *Geology of the Christchurch Urban Area*, Institute of Geological and Nuclear Sciences Ltd, New Zealand, 1992.

- 21.0 to 23.5 m bgl of firm to very stiff silt; and,
- More than 2.5 m of very dense gravel from approximately 23.5 m bgl.

3.2.3 Generalised subsurface profile

The inferred generalised site subsurface profile is summarised in Table 1. This profile has been derived from the results of our borehole and CPT investigations undertaken at the adjacent Canterbury Museum site and is supplemented by information which is available on the ECan database and published geological information.

Table 1: Generalised subsurface profile

Soil layer No.	Soil Description	Geologic Member	Approximate depth to top of layer (m)	Approximate layer thickness (m)	Typical CPT cone tip resistance qc (MPa)	Typical SPT N_{60} value (blows/300mm)
1a	SILT, stiff and silty SAND, loose	Yaldhurst Member, Springston Formation	0.0	3.5	1 - 3	6 - 10
1b	GRAVEL, medium dense		3.5	0.75	N/A	14
1c	SAND, medium dense		4.25	1.0	-	14
1d	GRAVEL, dense to very dense		5.25	4.75	N/A	30 – 50+
2a	SAND, dense to very dense	Christchurch Formation	10.0	5.0	-	30 – 50+
2b	Interbedded SILT, stiff and silty SAND, medium dense		15.0	6.0	-	4 - 18
2c	SILT, firm to very stiff		21.0	2.5	-	7 - 28
3a	GRAVEL, very dense	Riccarton Gravel	23.5	2.5 +	N/A	>50

3.2.4 Groundwater

Groundwater was identified at a depth of approximately 3.5 m bgl within the machine drilled borehole and CPT investigations. Groundwater levels were found to be lower within borehole and CPT 03. This is inferred to be due to the higher elevation of the ground surface at this location. Groundwater levels at the site may fluctuate over time due to variations in rainfall, runoff conditions, levels in the nearby water courses and other factors.

4 Liquefaction assessment

4.1 Seismicity

The Christchurch region can be considered as a seismically active area. Recent significant earthquakes that resulted in moderate to strong ground shaking at the project site include:

- A Moment Magnitude (M_w) 7.1 event on 04 September 2010, located approximately 37 km west of the site;
- A M_w 6.2 event on 22 February 2011, located approximately 7 km southeast of the site;
- A M_w 6.0 event on 13 June 2011, located approximately 10 km east southeast of the site; and,
- A M_w 5.9 event on 23 December 2011, located approximately 10 km east of the site.

Recent research indicates that east Canterbury could experience an extended period of heightened seismic activity relative to the past century. Additionally, there is a relatively high probability of a large earthquake occurring on the Alpine Fault at some point within the next 50 years. Such an earthquake is anticipated to produce relatively strong ground shaking in the Christchurch area. Consequently, we judge that the site is likely to be subjected to moderate to strong earthquake shaking during the life of any structure located on the site.

4.2 Design earthquake scenarios

The earthquake scenarios which were used in our liquefaction analyses of the site are summarised in Table 2. The peak horizontal ground accelerations are in line with the Department of Building and Housing guidelines.

Table 2: Summary of the earthquake scenarios used in the liquefaction assessment

	Serviceability Limit State (SLS)	Ultimate Limit State (ULS)	22 February 2011 earthquake
Return period (years)	25	500	-
Moment Magnitude, M_w	7.5	7.5	6.2
Peak horizontal ground acceleration, PGA	0.13g	0.35g	0.45g[(1)] 0.32g[(2)]

1 Peak ground acceleration averaged from the University of Cornell interpretations near the site.
2 Peak ground acceleration corrected using the magnitude scaling factor to derive an equivalent pga for a 7.5 magnitude earthquake

<u>NZS 1170 Scenarios</u>

Two design earthquake scenarios were derived from "NZS 1170 – Structural Design Actions" assuming an Importance Level 2 building with a 50 year design life. These scenarios represent the following design performance requirements:

- Serviceability Limit State (SLS) – to avoid damage that would prevent the structure from being used as originally intended, without structural repair (though maintenance may be required e.g. patching of cracks); and,
- Ultimate Limit State (ULS) – to avoid collapse of the structural system (though significant loss of serviceability may result).

In terms of NZS 1170, a site sub-soil class of D (deep soils) was assumed due to the unknown, but typically great depth to bedrock (more than 60m) in the Christchurch area.

<u>22 February 2011 Earthquake</u>

Our liquefaction analysis also included a ground motion from the 22 February 2011 earthquake interpreted by the University of Cornell and averaged from locations near to the site. The results of this analysis were used to help calibrate the estimates of liquefaction settlement under future SLS and ULS ground shaking. The peak ground acceleration has been corrected using the appropriate magnitude scaling factor to convert the M_w 6.2 earthquake to the design M_w 7.5 earthquake. This indicates that the peak ground accelerations experienced during the 22 February 2011 earthquake were slightly less than the ULS acceleration adopted for design.

4.3 Liquefaction analysis

Seismic liquefaction occurs when excess pore pressures are generated in loose, saturated, generally cohesionless soil during earthquake shaking, causing the soil to undergo a partial to complete loss of shear strength. Such a loss of shear strength can result in settlement and/or horizontal movement (lateral spreading) of the soil mass. The occurrence of liquefaction is dependent on several factors, including the intensity and duration of ground shaking, soil density, particle size distribution, and elevation of the groundwater table.

Analyses were undertaken to evaluate the liquefaction potential of the loose to medium dense sands and non-plastic/low plasticity silts found in the boreholes and CPT soundings utilising the methods recommended by Idriss and Boulanger (2008)[4]. The three earthquake scenarios described in Table 2, and a ground water level of 3.5 to 4.5 m bgl, were assumed in the analyses. A summary of the liquefaction analysis is presented in Appendix C.

The results of the analyses are presented in Table 3.

[4] Idriss, I. And Boulanger, R. (2008). "Soil liquefaction during earthquakes," Earthquake Engineering Research Institute.

Table 3: Liquefaction susceptibility under SLS and ULS seismic shaking

Soil layer No.	Soil Description	Approximate depth to top of layer (m)	Approximate layer thickness (m)	Liquefiable under SLS level earthquake	Liquefiable under ULS level earthquake
1a	SILT, stiff and silty SAND, loose	0.0	3.5	No(1)	No(1)
1b	GRAVEL, medium dense	3.5	0.75	No	No
1c	SAND, medium dense	4.25	1.0	Discrete lenses	Yes
1d	GRAVEL, dense to very dense	5.25	4.75	No	No
2a	SAND, dense to very dense	10.0	5.0	No	No
2b	Interbedded SILT, stiff and silty SAND, medium dense	15.0	6.0	Unlikely(2)	Unlikely(2)
2c	SILT, firm to very stiff	21.0	2.5	No	No
3a	GRAVEL, very dense	23.5	2.5 +	No	No

1 Potentially liquefiable if ground water level raises within this layer.
2 Unlikely to liquefy due to depth of soil layer.

4.3.1 Settlement

Estimates of settlement induced by liquefaction of the subsurface materials are presented in Table 4. These estimates were made using the methodology developed by Zhang, Robertson and Brachman (2002)[5].

It should be noted that the settlement values presented in Table 4 are total, free field settlement estimations. This describes the settlement of ground not occupied by a building, occurring due to dissipation of excess pore water pressure generated during earthquake shaking. An additional component of building settlement may also occur due to yield of the liquefied soils under foundation loading. This component of settlement is very difficult to predict and depends on the interaction of the building and the soil it is founded on.

[5] Zhang, Robertson and Brachman (2002). "Estimating liquefaction-induced ground settlements from CPT for level ground," Canadian Geotechnical Journal.

Table 4: Summary of estimates of liquefaction induced free-field settlement

Description	SLS Seismic Event (M=7.5, PGA=0.13g)	ULS Seismic Event (M=7.5, PGA=0.35g)	22 February 2011 Seismic Event (M=6.2, PGA=0.45g)
Estimated liquefaction induced total settlement	5 - 25 mm	60 - 70 mm	60 - 70 mm
Estimated liquefaction induced differential settlement at surface	Less than 15 mm	Up to 40 mm	Up to 40 mm

The above settlements were calculated using simplified methods based largely on empirical data from homogenous soil sites. It must also be noted that while estimates of settlements are provided above for SLS and ULS level earthquake events, settlement can occur before a SLS level event has occurred and significant settlement (similar to the above ULS values) can occur before a ULS level earthquake has been reached.

Based on T&T's post-earthquake observations, the above predicted settlements may be conservative (an over-estimate) for some sites in Christchurch. In addition, subsurface conditions and soil properties may vary substantially across the site making accurate predictions of future seismic settlement extremely difficult. Therefore, engineering judgment should be applied when interpreting the computed settlements presented above.

In terms of the New Zealand Geotechnical Society[6] (NZGS) guidelines, the level of liquefaction estimated to occur at the site under ULS loading can be considered to correspond to a *Liquefaction Performance Level* of L2 ("moderate") – "Liquefaction occurs in layers of limited thickness (small proportion of the deposit); ground deformation results in differential settlements."

Under SLS loading, the site can be classified as *Liquefaction Performance Level* of L1 ("mild") – "Limited excess pore water pressures without complete liquefaction; relatively small deformation of the ground with relatively small settlements (few tens of millimetres)".

The observed damage at the site following the 22 February 2011 earthquake was generally consistent with an NZGS performance level of L1 ("mild").

4.3.2 Sand Boils

Sand boils occur when liquefied soils at depth break through to the ground surface through fissures, cracking and/or weak crustal soils. This phenomenon can lead to bearing capacity failure and the creation of voids in subsoil zones, and is a significant cause of differential settlement beneath foundations, slabs, roads, etc.

Empirical correlations have been developed by Ishihara[7] to quantify the thickness of non-liquefiable surface crust required to prevent the formation of sand boils resulting from the liquefaction of underlying soil layers. These correlations indicate that for a given thickness of liquefiable soil, as the peak ground acceleration increases a greater surface crust thickness of non-liquefiable soil is required to prevent liquefaction damage from manifesting on the surface.

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[6] New Zealand Geotechnical Society, *Geotechnical earthquake engineering practice, Module 1 – Guideline for the identification, assessment and mitigation of liquefaction hazards*, July 2010

[7] Ishihara, K. (1985). "Stability of natural deposits during earthquakes," Theme lecture, Proc. 11th Int. Conf. On Soil Mechanics and Foundation Engineering, San Francisco, 2, 321-376pp.

The results from CPT soundings and the Ishihara correlations indicate that the thickness of the non-liquefiable surface crust overlying the liquefiable layers at the site may be sufficient to prevent sand boils and abrupt differential settlement at the ground surface during a SLS, ULS and 22 February 2011 earthquake event.

Sand boils were not observed at the site during our site walkover of 08 October 2012. Additionally, Robert McDougall Art Gallery maintenance staff said no sand boils were observed onsite following the 22 February 2011 earthquake. Aerial photos taken shortly after the 22 February 2011 earthquake[8] do not show any evidence of sand boils in the immediate area surrounding the site. The potential for loss of foundation ground support on the site due to sand boiling during a future SLS or ULS seismic event is judged to be low.

4.3.3 Lateral Spreading

Lateral spreading is generally defined as the horizontal displacement of surficial blocks of soil towards an open slope face as a result of liquefaction of the underlying soils. The occurrence of lateral spreading generally requires the presence of a relatively continuous liquefiable layer extending to an open slope face such as a river bank or open channel. Displacements can range from a few centimetres to a metre or more.

The nearest open slope or river channel is the Avon River, which runs more than 100 m to the north and south of the site.

Lateral spreading was not observed following the 04 September 2010, 22 February, and 13 June 2011 earthquakes. This is likely due to the distance to an open slope face. The risk of lateral spreading in a future SLS and ULS earthquake event is considered to be low.

[8] Publicly available from koordinates.com

5 Foundations

5.1 General

T&T understand, based on the original structural drawings from the 1930's, that the Robert McDougall Art Gallery is supported on pad footing at column locations with strip footings connecting the pad footings. The dimensions of the existing strip and pad footings are shown to vary over the building footprint. Limited information is available on the drawings regarding the width, depth and thickness of the footings. The original section of the basement appears to have the same foundation type as the rest of the structure.

The building's basement has been extended in the 1980's to encompass much of the building footprint. Structural drawings showing the extensions to the basement have not been available for review. Observations during T&T's walkover inspection indicate that the extensions to the basement are likely founded on strip and pad footings beneath the columns and walls in a similar layout to the original 1930's drawings. However, the depth, width and thickness of these foundations are unknown.

5.2 Foundation performance

No evidence of significant foundation settlement or bearing capacity failure was observed during T&T's walkover inspection. It is unlikely that significant voids have been created beneath the building's floor slabs or foundations due to seismic settlement or the formation of sand boils.

T&T conclude that it is unlikely that damage to the Robert McDougall Art Gallery building following the Canterbury Earthquake sequence was due to geotechnical foundation issues.

6 Conclusions and recommendations

6.1 General

Excess pore water pressures for the recent earthquakes and associated aftershocks are expected to have dissipated. The strength of the soil underlying the site is expected to have returned close to the pre-earthquake levels.

The inferred generalised site subsurface profile is summarised as:

- 3.5 m of stiff silt and loose sand; overlying, (Layer 1a)
- 0.75 m of medium dense gravel; (Layer 1b)
- 1.0 m of medium dense sand; (Layer 1c)
- 4.75 m of dense to very dense gravel; (Layer 1d)
- 5.0 m of dense to very dense sand; (Layer 2a)
- 6.0 m of interbedded stiff silt and medium dense silty sand; (Layer 2b)
- 2.5 m of firm to very stiff silt; (Layer 2c)
- Very dense gravel from approximately 23.5 m bgl for at least 2.5 m. (Layer 3a)

Much of the medium dense sand (Layer 1c) is likely to liquefy under ULS earthquake shaking with portions of this layer likely to liquefy under SLS earthquake shaking. Additionally, the stiff silt and loose silty sand (Layer 1a) may liquefy under ULS and SLS earthquake shaking if it is below the ground water table.

In terms of the New Zealand Geotechnical Society[9] (NZGS) guidelines, the level of liquefaction estimated to occur at the site under ULS loading can be considered to correspond to a *Liquefaction Performance Level* of L2 ("moderate") – "Liquefaction occurs in layers of limited thickness (small proportion of the deposit); ground deformation results in differential settlements."

Under SLS loading, the site is classified in term of the NZGS guideline as Performance Level L1 ("mild") – "Limited excess pore water pressures without complete liquefaction; relatively small deformation of the ground with relatively small settlements (few tens of millimetres)".

6.2 Foundations

T&T understand that the Robert McDougall Art Gallery building is supported on strip and pad footings. The width, depth and thickness of the building's foundations are currently unknown. It is recommended that foundation dimension be confirmed in order to assess likely foundation capacities.

No evidence of significant foundation settlement or bearing capacity failure was observed during T&T's walkover inspection. It is unlikely that significant voids have been created beneath the building's floor slabs or foundations due to seismic settlement or the formation of sand boils.

T&T conclude that it is unlikely that damage to the Robert McDougall Art Gallery building following the Canterbury Earthquake sequence was due to geotechnical foundation issues.

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[9] New Zealand Geotechnical Society, *Geotechnical earthquake engineering practice, Module 1 – Guideline for the identification, assessment and mitigation of liquefaction hazards*, July 2010

7 Applicability

This report was prepared for the benefit of Christchurch City Council with respect to the particular brief given to us and it may not be relied upon in any other context or for any other purpose without our prior review and written agreement.

The recommendations and opinions which are contained in this report are based upon data from the currently available geotechnical investigations, and observations of surface features. The nature and continuity of sub-surface conditions away from the investigation locations is inferred, and it must be appreciated that the actual conditions may vary from the assumed model.

The susceptibility analyses carried out represent probabilistic analyses of empirical liquefaction databases under various earthquakes. Earthquakes are unique and impose different levels of shaking in different directions on different sites. The results of the liquefaction susceptibility analyses and the estimates of consequences presented within this document are based on regional seismic demand and published analysis methods, but it is important to understand that the actual performance may vary from that calculated.

It is important that Tonkin & Taylor Ltd be immediately contacted if there is any variation in subsoil conditions from those which are described in this report.

Tonkin & Taylor Ltd

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