Holmes Consulting Group

REPORT

STRUCTURAL AND CIVIL ENGINEERS

QLDC OFFICES AND LIBRARY

DETAILED SEISMIC ASSESSMENT

PHASES 1 & 2

PREPARED FOR

QUEENSTOWN LAKES DISTRICT COUNCIL

112461.00

OCTOBER 2014

QLDC OFFICES AND LIBRARY – DETAILED SEISMIC ASSESSMENT REPORT

PHASES 1 & 2

Prepared For:
QUEENSTOWN LAKES DISTRICT COUNCIL

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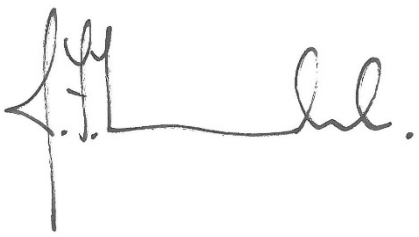

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This report covers the detailed seismic assessment of the QLDC building, 8-10 Gorge Road, Queenstown. The building is currently in use as Council Chambers, Council Offices and Library.

We have based our assessment on the building being classified as an Importance Level 2 (IL2) building. This will require QLDC to re-address storage of any critical records and the current use as the civil defence headquarters.

Our findings of the structural assessment show the building to have a capacity of 35% of the Design Base Earthquake (DBE) for an IL2 building. We consider this to be a lower bound capacity limited by the performance of the building's concrete frames flexural capacity.

The building was originally designed in 1976 with subsequent alterations carried out in 1994, 1998 and 2003. Review of the original calculations shows that it adopted the loadings standard NZS4203:1976 which was released that year. The seismic design load used for the original design was 0.29g. The loadings standard NZS4203:1976 has since been superceded by NZS1170.5:2004. As such, the seismic design load for an IL2 building on the site today would be 0.78g. Therefore, based on a direct code comparison, the original building would be expected to have a capacity of just 37% of current code requirements.

The subsequent alterations would have been carried out under the Building Act (1991). At that time, the definition of 'Earthquake Prone' buildings was limited to only buildings of unreinforced masonry construction. As such, alterations would generally have targeted simply making the building no worse than before the alteration, and consideration of overall seismic strength would not have been specifically required by the Act.

An assessment of liquefaction susceptibility indicated that onset of liquefaction is likely in an earthquake with an annual probability of exceedance (AEP) of 1/100, and liquefaction is fully developed in a 1/200 AEP event. This is roughly equivalent to 50% of current code for an Importance Level 2 building, and that the point at which it is fully developed is approximately equivalent to 70% of current code.

We have completed a probability of exceedance to quantify the risk posed relative to a new building with a 50 year design life. This illustrates that the building with a capacity of 35% DBE represents a relative risk 6.5 times that of a new building. However if occupation of the building was limited to a period of 5 years, this is approximately the same risk as a new building for a 50 year design life.

1. INTRODUCTION

Holmes Consulting Group has been engaged by Queenstown Lakes District Council to complete a detailed seismic assessment of the building at 8-10 Gorge Road Queenstown.

1.1 SCOPE OF WORK

The scope of work for this project included the following:-

- Review available structural drawings for the building to determine the nature of the design, primary structural characteristics, and adequacy of the lateral force-resisting systems.
- Walk around the building to familiarise our engineers with the structure, visually assess its condition, observe important structural and seismic characteristics, and note obvious deficiencies.
- Assess the likely seismic performance of the building, based on general observations, and preliminary analysis where appropriate.
- Carry out further investigations and detailed assessment of critical areas identified in the preliminary analysis (Phase 2).
- Report on our findings and recommendations.

1.2 LIMITATIONS

Findings presented as a part of this project are for the sole use of the Queenstown Lakes District Council in its evaluation of the subject property. The findings are not intended for use by other parties, and may not contain sufficient information for the purposes of other parties or other uses.

Our professional services are performed using a degree of care and skill normally exercised, under similar circumstances, by reputable consultants practicing in this field at this time. No other warranty, expressed or implied, is made as to the professional advice presented in this report.

2.1 BUILDING ACT

When dealing with existing buildings there are a number of relevant sections of the Building Act 2004 [1] that need to be considered in relation to the building's structure and strength.

Section 112 - Alterations to Existing Buildings

Section 112 of the Building Act requires that a building subject to an alteration continue to comply with the relevant provisions of the Building Code to at least the same extent as before the alteration.

Essentially this section means that the building may not be made any weaker than it was, as a result of any alteration.

Section 115 – Change of Use

Section 115 of the Building Act requires that the territorial authority be satisfied that the building in its new use will comply with the relevant sections of the building code "as nearly as is reasonably practicable"

In relation to building earthquake strength, this section is typically interpreted as requiring earthquake strengthening to a minimum level of 67% of that required for an equivalent new building.

Section 122 – Meaning of Earthquake Prone Building

Section 122 of the Building Act 2004 deems a building to be earthquake prone if its ultimate capacity (strength) would be exceeded in a "moderate earthquake" and it would be likely to collapse causing injury or death, or damage to other property.

The Building Regulations (2005) define a moderate earthquake as one that would generate loads 33% as strong as those used to design an equivalent new building.

Section 124 – Powers of Territorial Authorities

If a building is found to be earthquake prone, the territorial authority has the power under section 124 of the Building Act to require strengthening work to be carried out, or to close the building and prevent occupancy.

Section 131 – Earthquake Prone Building Policy

Section 131 of the Building Act requires all territorial authorities to adopt a specific policy on dangerous, earthquake prone, and unsanitary buildings.

2.2 QUEENSTOWN LAKES DISTRICT COUNCIL EPB POLICY

QLDC require buildings identified as earthquake-prone buildings to be strengthened only to the minimum level of 34%; this then takes the building out of the category of earthquake-prone buildings. The Council's Policy on earthquake prone buildings is dated March 2007 and is currently undergoing review. In August 2012 the Otago Councils agreed to investigate the development of a common set of principles around earthquake prone buildings.

In 2007, the QLDC had prioritised both the identification and notification date and the maximum time for strengthening or demolition respectively. Times required for strengthening or demolition commence on the date of issue of formal notice. Specific times were to be assigned for action according to assessment of structural performance and the nature of the concerns.

The order was as indicated below:

a. Buildings with special post-disaster functions as defined in AS/NZS 1170.0:2002 Importance Level 4 (December 2008, 2 years)

b. Buildings that contain people in crowds or contents of high value to the community as defined in AS/NZS 1170.0:2002, Importance Level 3, including all Council owned buildings (December 2009, 3 years)

c. Buildings with a Heritage Classification (December 2012, 5 years)

d. Buildings with an Importance Level of less than 3 as defined in AS/NZS 1170.0:2002 (December 2022, 15 years)

Once each category has been reviewed and the earthquake-prone buildings within it identified, the process of liaising with owner(s) and serving notice on them was to commence.

Identification of buildings in each category was to proceed according to the priorities identified above.

This section discusses the construction history of the building at 8-10 Gorge Road, including its original design and subsequent alterations. A summary of the key changes to the building throughout its history is shown in Table 3-1 below.

Table 3-1: Summary of construction history

Year	Description	Architect	Engineer	Contractor
1977	New building for the Wakatipu Club	N John Blair Architect	Holmes Wood Poole & Johnstone	Unknown
1994	Alterations for the Wakatipu Club	Cockburn van Brandenburg Architects	AS Major	Rilean Construction
1998	Refit for QLDC	Noel Tapp Designs	Holmes Consulting Group Ltd	Amalgamated Builders
2003	Alterations for QLDC	Noel Tapp Designs	Holmes Consulting Group Ltd	Naylor Love

Significant alterations have been made to the building since its original construction and it is therefore important to understand the various modifications that have been made.

3.1 SITE DESCRIPTION

The site at 8-10 Gorge Road is sloping from Gorge Road down towards Horne Creek which runs along the western boundary of the site. The level difference is currently addressed by the building which acts as a retaining structure over the lower level, and a crib wall running away to the east. The photo taken in Figure 3-1 is looking south towards the building from the western bank of Horne Creek.

Construction records indicate that poor foundation conditions were found at the north-east corner of the site, and we understand that some modifications may have needed to be made to the foundations during the 1994 alterations as a result.

The site has been identified as potentially liquefaction prone (Tonkin & Taylor

Queenstown Lakes District 2012 Liquefaction Hazard Assessment, September 2012). Noting the presence of Horne Creek to the north, there was also considered to be a potential for lateral spreading. The site is also identified as flood prone (Otago Regional Council Horne Creek Flood Risk Study 2006) due to swelling of Horne Creek.

Figure 3-1: 8-10 Gorge Road

3.2 ORIGINAL BUILDING FOR WAKATIPU CLUB (1977)

The building at 8-10 Gorge Road was originally built for the Wakatipu Club in 1976. The building comprised an open carpark at ground level with a 120m^2 enclosure at the southern end. Above this was the main 400m^2 suspended first floor, with a further 120m^2 library and managers flat above.

The building was designed by Holmes Wood Poole and Johnston in 1976 and the full structural drawings and specification were available for review. Figure 3-2 shows the general layout of the first floor plan from the original structural drawings.

Reinforced concrete blockwork walls formed a stiff box in the southern third of the building, while the northern portion was relatively open with reinforced concrete frames providing gravity support to the floors. The southern blockwork walls also retain the embankment to the south east of the building.

The suspended floors were constructed of 350mm deep precast concrete double tee flooring units with a 50mm topping. The precast flooring units were supported by the blockwork walls and concrete beams in a manner referred to as 'flange hung' – a type of detail not commonly used until the 1980's. Records show that the double tee units were designed and constructed by Stresscrete but details of the units and flange hung supports have not been able to be found at this time.

The building was founded on shallow strip footings beneath the blockwork walls, and isolated pad footings between the concrete columns forming the frames.

The lightweight roof was formed by timber purlins spanning to timber trusses which are in turn supported on the concrete first floor structure.

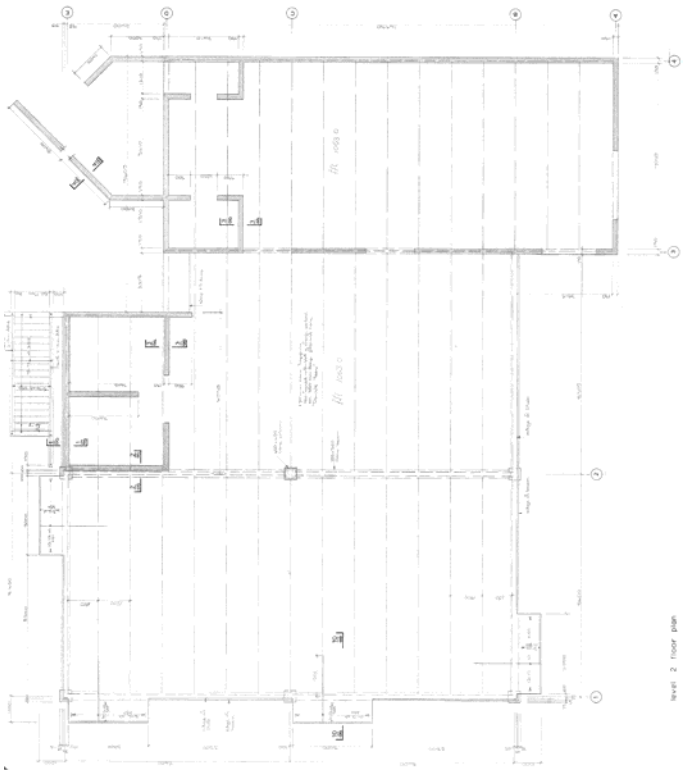

Figure 3-2: New Building for the Wakatipu Club (1976) – First Floor Plan

3.3 ALTERATIONS FOR WAKATIPU CLUB (1994)

The alterations carried out in 1994 comprised a significant extension to the building floor area. The first floor was extended predominantly to the north with an associated increase in floor area from 400m^2 to 770m^2. A portion of the ground floor carpark also appears to have been closed in at this time.

The alterations were designed by AS Major, although no formal records of the construction have been able to be sourced at this time. QLDC do not appear to have any records of the consent documentation, and requests made to AS Major and Rilean Construction also did not return any results.

As such, significant uncertainty currently exists with regards to the construction of the various elements in the 1994 alterations.

Figure 3-3 highlights the approximate magnitude of the first floor extension carried out in the 1994 alterations.

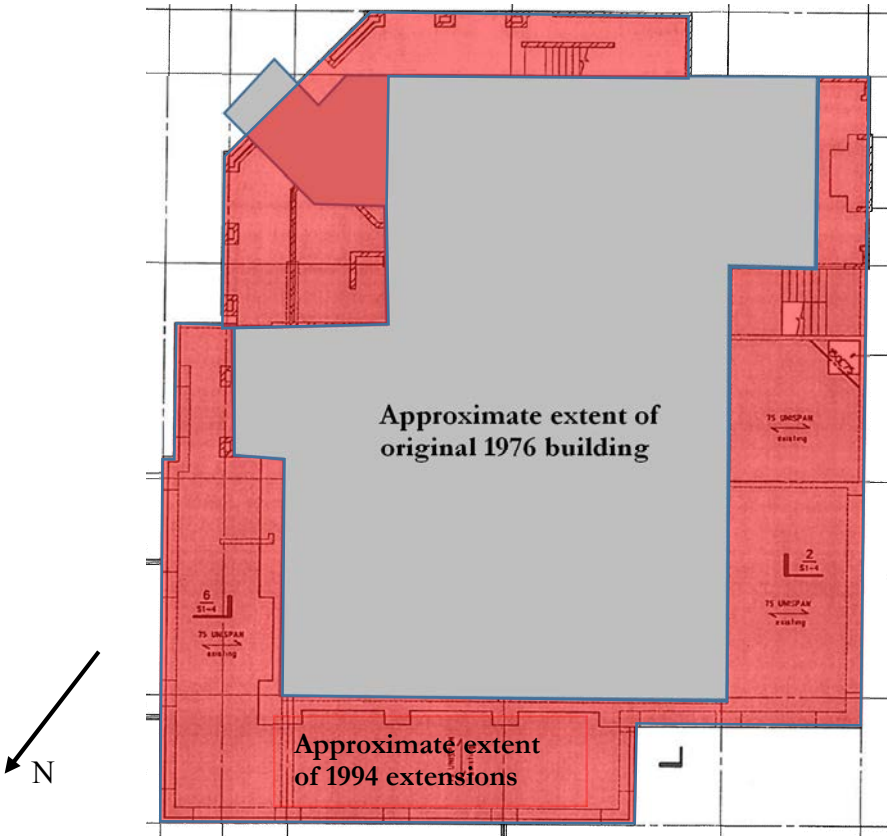

Figure 3-3: Approximate extent of first floor extension for Wakatipu Club (1994) (Inferred from 1998 drawings)

As part of these alterations, a substantial portion of the northern-most concrete blockwork wall at ground floor level appears to have been removed. Additional blockwork walls appear to have been constructed further north of this point, although it is not clear whether they were tied into the structure above, or constructed as non-structural partitions.

A perimeter concrete beam appears to have been constructed around the western, northern and eastern elevations to support the new areas of concrete flooring. These beams appear to be seated on hollow concrete masonry piers. However, again the construction of these elements is largely uncertain.

3.4 REFIT FOR QLDC (1998)

The alterations carried out in 1998 were primarily to convert the building into office space for QLDC and to create more floor area by closing in the ground floor carpark.

The alterations were designed by Holmes Consulting Group Ltd and full drawings of the alterations were available for review. Figure 3-4 highlights the primary alterations at first floor level.

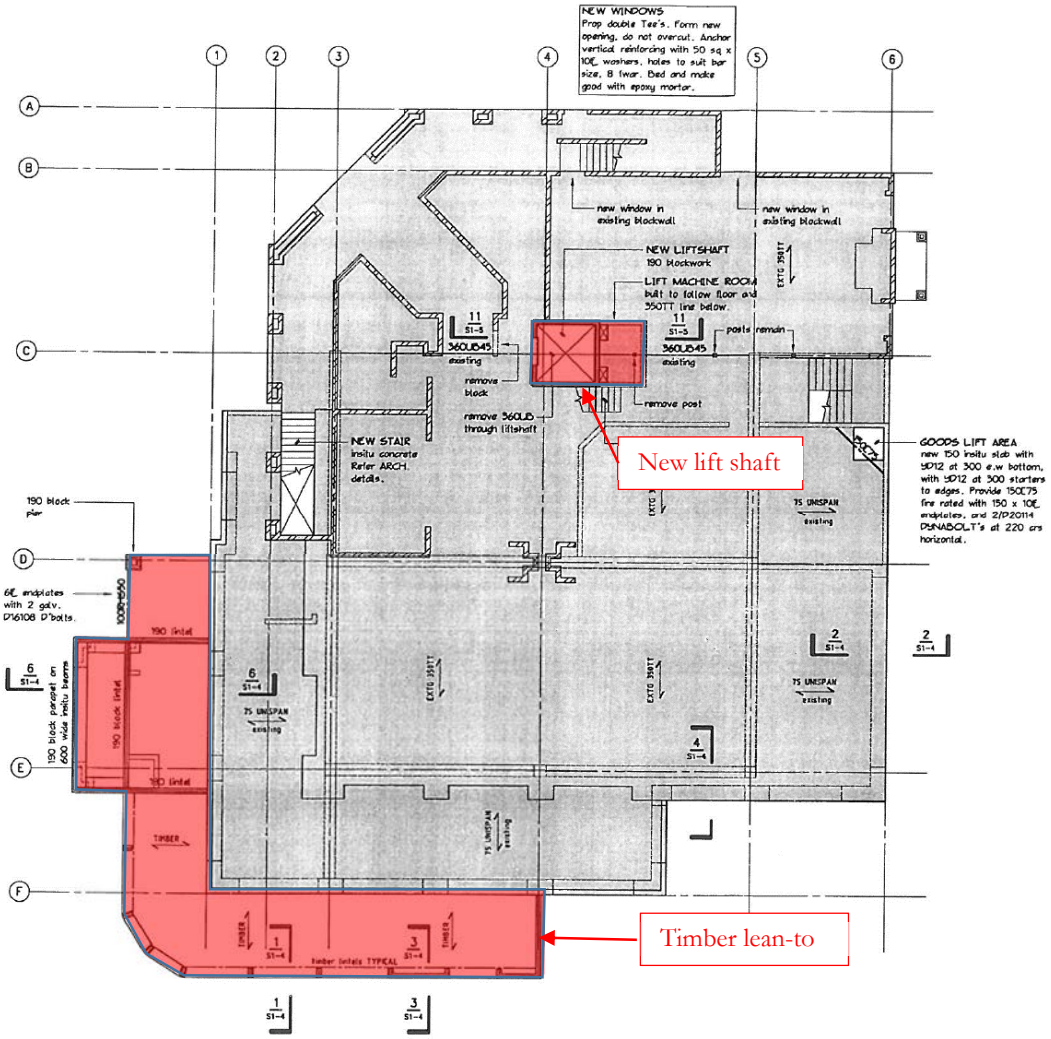

Figure 3-4: Extent of first floor alterations for QLDC (1998)

The alterations generally comprised the installation of a lift, the closing in of the remainder of the ground floor carpark to create office space, and the associated construction of a single level timber lean-to structure in the north-east corner of the building. The roof to the second floor area was also replaced as part of the renovations for the new council chambers.

3.5 ALTERATIONS FOR QLDC (2003)

Alterations carried out by QLDC in 2003 included the closing in of an external terrace, and relocation of internal partitions to expand the available office space. The alterations were largely non-structural and few records of these alterations were available for review.

4.1 CURRENT USE

The building at 8-10 Gorge Road currently houses the Queenstown Public Library on ground floor, with the Queenstown Lakes District Council offices on first floor. The council chambers are located in a smaller footprint on the second floor.

Based on this current use, the building could be considered a normal, Importance Level 2 (IL2) structure in accordance with AS/NZS1170.0:2002. This is based on the assumption that no more than 300 people can congregate in the library.

However, it could be argued that the public records held at the QLDC comprise contents of high value to the community. If these records were deemed to be sufficiently important, the building could be considered an Importance Level 3 (IL3) structure.

The building is also currently designated as the civil defence headquarters for QLDC. If this remains the case, the building would be considered an essential post-disaster facility, termed to be an Importance Level 4 (IL4) structure.

The different importance levels referred to above relate to the seismic demands that the building must be assessed against. Table 4-1 describes the various importance levels mentioned and provides the associated seismic design parameters for each. From this it can be seen that IL3 and IL4 structures are respectively required to be resist 1.3 and 1.8 times the seismic loads as that required for an IL2 structure. Furthermore, an additional serviceability limit state, SLS2, is also required for IL4 structures whereby the building should be suitable for continued occupation immediately following a 1 in 500 year event.

Table 4-1: Importance Levels (from AS/NZS1170.0:2002)

Importance Level	Description	Design Basis Earthquake (DBE)	Factor on seismic design loads
IL2	Normal structures	1 in 500 year	1.0
IL3	Structures that contain contents of high value to the community	1 in 1000 year	1.3
IL4	Structures with special post-disaster functions	1 in 2500 year	1.8

Preliminary assessment showed that the structure was unlikely to achieve IL3 or IL4 capacities. As such, we have based this assessment on the assumption that the building is an IL2 structure. This assumes that any critical records could be relocated elsewhere for storage, and that another location is designated as the QLDC's essential post-disaster response facility.

4.2 COMPARISON OF PREVIOUS WORKS WITH CURRENT CODE

The building was originally designed in 1976 and review of the original calculations shows that it adopted the loadings standard NZS4203:1976 which was released that year. The seismic design load used for the original design was 0.29g. The loadings standard NZS4203:1976 has since been superceded by NZS1170.5:2004. As such, the seismic design load for an IL2 building on the site today would be 0.78g. Therefore, based on a direct code comparison, the original building would be expected to have a capacity of just 37% of current code requirements.

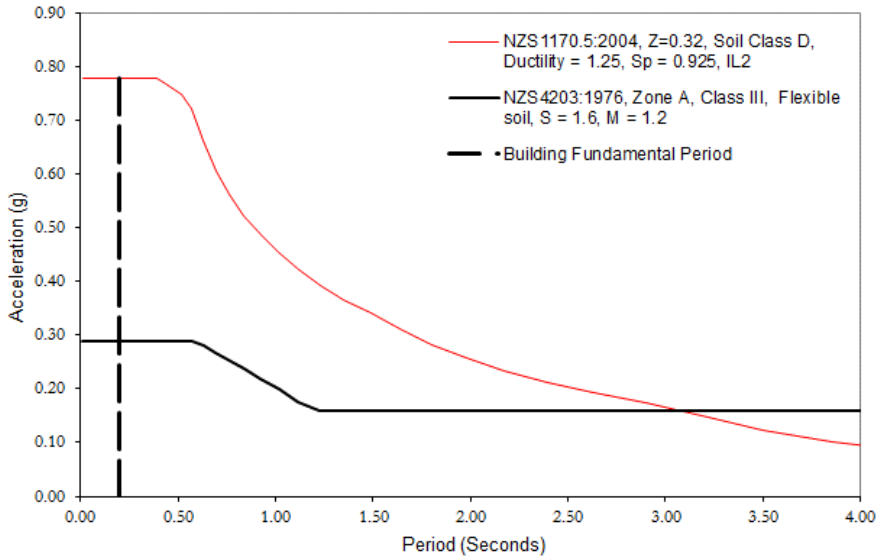

Figure 4-1: Comparison of Design Codes

The alterations carried out in 1994, 1998 and 2003 would have been carried out under the Building Act (1991). At that time, the definition of 'Earthquake Prone' buildings was limited to only buildings of unreinforced masonry construction. As such, alterations would generally have targeted simply making the building no worse than before the alteration, and consideration of overall seismic strength would not have been specifically required by the Act.

4.3 PREVIOUS ASSESSMENT

In 2011 QLDC engaged Hadley Consultants Ltd to carry out an Initial Evaluation Procedure (IEP) to provide a preliminary assessment of the seismic capacity of the building. We note that the IEP is a coarse screening tool typically used to identify potentially earthquake prone buildings.

Hadley Consultants IEP Assessment Report dated 7 July 2011 concluded that the building achieved a score of 53% New Building Standard (NBS), indicating that the building was unlikely to be considered 'Earthquake Prone'.

It also reported that preliminary calculations indicated the overall stability of the building could be substantially higher and recommended that a detailed structural analysis be carried out.

4.4 CURRENT EVALUATION

In order to carry out a detailed seismic assessment of the building capacity a number of assumptions were required with regards to unknown information. This section describes the key assumptions made, and outlines the results of the assessment.

4.4.1 Geotechnical Assessment

A geotechnical investigation was carried out by Tonkin and Taylor Ltd during August 2014, and is the subject of the Tonkin and Taylor Geotechnical Assessment Report dated September 2014. The following is a summary of this geotechnical report.

The soils at the site can be broadly divided into three different layers:

1. Valley infill – Loose to medium dense to Sandy GRAVEL and Gravelly SAND. Only present at BH03/CPT03 to the south of the site from 0-6m depth.
2. Lake sediments – Interbedded SILT, SAND and silty SAND. Present in all boreholes, generally 10-12m thick.
3. Older Till or Lake Sediments – 2-5m thick Dense GRAVEL and SAND layer overlying hard silt.

Groundwater levels were measured at 2.75m to 2.8m depth.

An assessment of liquefaction susceptibility indicated that onset of liquefaction is likely in an earthquake with an annual probability of exceedance (AEP) of 1/100, and liquefaction is fully developed in a 1/200 AEP event. The expected consequences of liquefaction include small ground cracks, some sand boils, differential settlements of up to 100mm and lateral displacements of up to 100mm.

We note that the point of onset of liquefaction (1/100 AEP) is roughly equivalent to 50% of current code for an Importance Level 2 building, and that the point at which it is fully developed (1/200 AEP) is roughly equivalent to 70% of current code.

4.4.2 Description of Lateral System and Key Assumptions

The building in its current configuration comprises a large floor plate at first floor level below the significantly smaller second floor council chambers at the southern end of the building. These floor plates comprise precast concrete double tee and flat slab flooring with a 50mm insitu topping diaphragm which is required to distribute seismic loads to the vertical lateral load resisting elements.

The primary lateral system comprises fully filled reinforced concrete blockwork walls concentrated around the southern end of the building, as well as perimeter frames formed by reinforced concrete masonry piers and insitu concrete beams wrapping around the western, northern, and eastern elevations.

The foundations typically comprise shallow strip footings beneath the typical concrete masonry walls and the perimeter frames. The internal concrete gravity frames are founded on isolated pad footings and there is a lift pit in the southern portion of the building.

Due to the irregularity of the lateral systems and lack of particular detailing to develop a ductile mechanism, the building was considered to behave in a nominally ductile manner.

Beyond the assessment of liquefaction potential outlined in Section 4.4.1 above, no further geotechnical information has been provided for the site. As such, the following assumptions have been made for this quantitative assessment:

- Any retaining loads on the southern portion of the structure have been neglected
- A bearing pressure of 100kPa has been assumed for the assessment of foundation capacity

The effects of liquefaction on the structure are difficult to quantify, and are considered relatively binary. As such, we have neglected the effects of liquefaction in the structural analysis in Section 4.4.3 below, but attempted to make a qualitative assessment of the potential effects of liquefaction on the building in Section 4.4.4.

4.4.3 Modal Response Spectrum Analysis

A detailed assessment has been carried out for the building, using the software ETABS to perform a response spectrum analysis. A rendering of the computer model (excluding the lightweight upper roof level which has not been evaluated at this time) is shown in Figure 4-2.

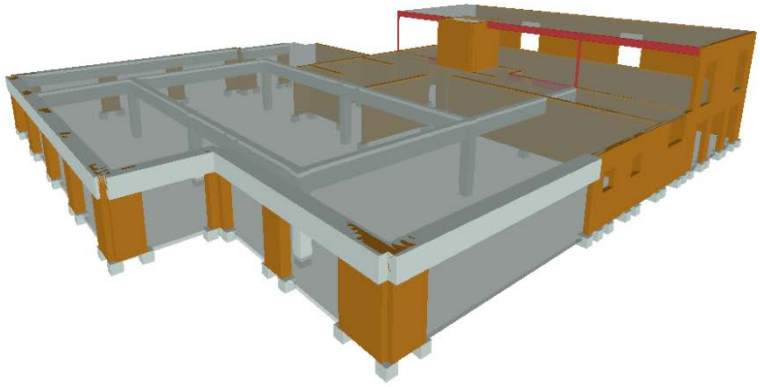

Figure 4-2: 8-10 Gorge Road – analysis model

Based on this analysis, the building is considered to have a capacity of 35% of the DBE for an Importance Level 2 building.

The capacity of the building is limited by the concrete masonry frames around the northern portion of the building. These frames reach their flexural capacity at 35% of the DBE and yielding of their foundation beams is expected to commence. The concrete masonry lift walls between first floor level and the council chambers also reach their capacity at this level of load.

The current analysis model shows the ground floor level concrete masonry walls on the south-western elevation to have their flexural capacity exceeded at 33% DBE. However, this is due to the modelling limitations of the software for these walls with small punctured openings. We believe that further analysis would show that these walls have a capacity in excess of 33% of the DBE.

The support of the precast double tee flooring units was identified as a potential critical structural weakness. This is due to the brittle behaviour typically associated with these types of connections in the past. This brittle failure is typically induced by rotation of the flooring supports during lateral movement of the building. However, the analysis model of the building showed it to be very stiff with inter-storey displacements of less than 5mm expected at 33% of the DBE. Although the actual detailing of the double tee units remains unknown, it is not expected to pose a collapse hazard, given the stiff behaviour of the building. However, it must be noted that this relies on the assumption that liquefaction does not occur (see Section 4.4.4 for further discussion).

A preliminary assessment of floor diaphragm capacity was carried out and indicated that the floor diaphragms are likely to have a capacity in excess of 33% of the DBE. However, the brittle nature of the hard drawn wire mesh reinforcing means that they are likely to provide little resistance to any tearing action resulting from liquefaction issues as noted below. Furthermore, details of the connection of the first floor extension to the original structure are unknown. These should be reviewed in further detail once details of these alterations can be obtained.

4.4.4 Consequences of Liquefaction

We understand that the site may be subject to liquefaction at between 50% and 70% of current code as outlined in Section 4.4.1 above. Liquefaction can result in a loss of bearing capacity and/or lateral spreading towards a free edge.

Should liquefaction occur, it is not expected to pose a significant hazard for the southern portion of the building which has a relatively large number of structural walls with which to force the structure to settle somewhat in one piece. However, the presence of isolated pad foundations beneath the existing gravity columns and the perimeter strip foundations wrapping around the northern portion of the building are unlikely to provide much resistance to the effects of liquefaction. Of primary concern, is the ability of these structural elements to maintain support to the floors via the inherently brittle seating details.

Loss of bearing capacity under the internal gravity columns would impose rotations on the concrete gravity frames that could lead to failure of the flange hung double tee seating. Similarly lateral spreading of these foundations towards the river could have a similar result.

Depending on the extent of liquefaction expected, it may be possible to mitigate these brittle collapse mechanisms. Steel support angles could be provided beneath the webs of the double tee units to prevent collapse of the floors due to localised bearing failure, and new foundation beams could be cast beneath the ground floor slab to tie the columns back in to the rest of the structure to mitigate the effects of lateral spreading. We would note that this type of work is very intrusive likely requiring at the least potential closure of the building. Likely costs associated with work of this nature would be prohibitive.

4.5 CONSIDERATION OF RISK

With regards to quantifying the risk posed, it is worth considering the relative risk when compared to a new building. Table 4-2 compares the risk posed over a typical 50 year design life in relation to that expected for a new building. This illustrates that a building with a capacity of 35% DBE represents a relative risk of 6.5 times that of a new building.

Table 4-2: Relative risk compared to a new building

% DBE	Annual probability of exceedance	Probability of exceedance in 50 years	Relative risk compared to a new building
35%	1/50 yr	65%	6.5 times
50%	1/100 yr	40%	4 times
100%	1/500 yr	10%	-

The risk also depends on the time that the building is occupied. While a new building is typically designed for a 50 year life, it may only be intended to occupy the current building for another 5 or 10 years. Table 4-3 shows the probability of exceedance for the range of performance levels being considered. This demonstrates that occupation of the building at 8-10 Gorge Road for 5 years poses approximately the same level of risk as occupying a new building for a typical 50 year life.

Table 4-3: Accumulation of risk with time

% DBE	Probability of exceedance in 5 years	Probability of exceedance in 10 years	Probability of exceedance in 50 years
35%	10%	18%	65%
50%	5%	10%	40%
100%	1%	2%	10%

4.6 OTHER POTENTIAL ISSUES

4.6.1 Retaining Loads

Due to a lack of geotechnical information, we have not included any retaining loads in the current assessment. We do not believe that the effects of these are likely to significantly affect the results reported herein, due to the concentration of retaining loads in the corner of the building with the greatest length of wall available to resist these actions directly. Following completion of a further geotechnical investigation, these effects could be included in the assessment.

Should such an investigation be carried out, the extent and type of retaining walls present should be determined, as well as the degree of tie-in between the existing first floor slab and the elevated ground.

4.6.2 Flooding

The QLDC offices and library is located in an identified flood hazard zone. The effects of flooding should be considered when assessing the use of the ground floor spaces.

4.6.3 Tree Hazard

A large tree is located in the north-west corner of the site. Noting that the site is subject to liquefaction in events exceeding 50% of current code as above, instability of the ground caused by loss of bearing capacity or lateral spreading towards Horne Creek could compromise the root system for this tree. Given the lightweight roof structure at 8-10 Gorge Road, the tree has the potential to cause a significant collapse if it were to fall on the building. We recommend others with appropriate expertise carry out an assessment of the tree.

Figure 4-3: Large tree at the north-west corner of the building

4.6.4 Double Tee Seatings

The construction details of the double tee flooring units is currently unknown. As such, we have been unable to assess the adequacy of these units to support imposed loads. In a seismic event, the concrete masonry walls supporting the double height fireplace in the lobby will be required to resist overturning loads. It appears that these walls may sit on the ends of double tee units, and the capacity of these units to support overturning loads should be considered once construction details of the double tees are known.

4.6.5 Construction of Concrete Masonry Wall at First Floor Level

A wall between first and second floors has been assumed to resist much of the lateral loads from the floor of the council chambers. However, this wall appears to sit on the precast flooring units from level 1, and its connection to the lift core walls is unknown due to the construction sequence of these (refer to Figure 4-4). Given the importance of this loadpath, we recommend investigation of the connection of this wall to the exterior wall and the walls forming the lift shaft.

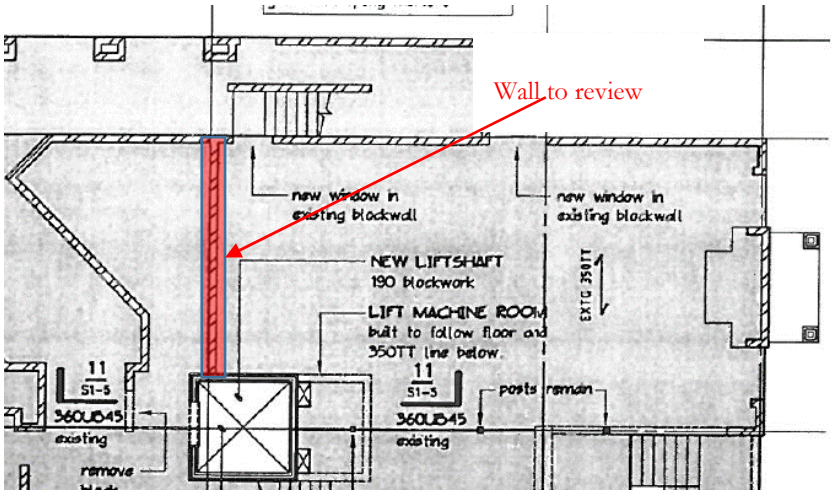

Figure 4-4: First Floor Wall

4.6.6 Stone Veneer to Fireplace in Lobby

A double height fireplace extends from the QLDC reception area up to roof level as seen in Figure 4-5. We have no specific details of the fireplace construction, or fixing of the stone veneer to the concrete masonry units. Failure of the fireplace could cause localised collapse of the precast flooring and we therefore recommend further investigation of the construction of this fireplace.

Figure 4-5: Fireplace with stone veneer

4.6.7 Suspended Ceilings in Library

While not likely to be a life safety hazard, if continued occupation of the library area is desired following an emergency, a review of the suspended ceiling would be recommended, due to the susceptibility of these types of systems to earthquake induced accelerations.

4.7 FURTHER INVESTIGATIONS

As outlined above, the several items could not be determined within the scope of this review. These following actions could be taken should further information be required:

- Source drawings of 1994 alterations
- Confirm double tee hanger details
- Confirm construction of concrete blockwork wall at first floor level
- Confirm connection of floor extensions to existing diaphragms.

It is noted that given the lack of documentation available, confirmation of the as-built construction of the elements identified could require destructive investigation.

The following items have been excluded from the current seismic assessment as their effects are not considered to be significant with regards to limiting the performance of the building. However, they may be considered should further work be desired in relation to the building:

- Assessment of seismic retaining loads to south of structure
- Analysis of timber roof structure

The capacity of the building is considered to be equivalent to 35% of current code, with onset of liquefaction at 50%.

While a detailed strengthening scheme has not been developed, the following items would likely need to be addressed in order to strengthen the capacity of the structure to 50% of current code:

- Addition of reinforced concrete masonry walls to the northern and western elevations with associated foundations
- Addition of reinforced concrete masonry walls within the lift core area extending from ground to second floor
- Investigation and tie-in of first floor diaphragms at connection to 1994 extension
- Confirmation of the items identified in Section 4.7

Realistic strengthening options beyond 50% of current code are somewhat limited, as this would require substantial ground remediation works such as piling or ground improvement before addressing the structural performance of the building above, due to the nature of the liquefaction expected.

6. CONCLUSIONS

The building at 8-10 Gorge Road was initially constructed in 1976 and has a long history of alterations.

The building comprises a suspended concrete floor supported largely on reinforced concrete masonry walls and frames, with a second suspended concrete level over the southern portion of the floor plate. The roof is of primarily lightweight construction and the building is generally founded on isolated shallow foundations. An embankment exists to the south east and is partially retained by the structure, with Horne Creek flowing along the western boundary of the site.

The building currently houses both the Queenstown Lakes District Council offices and the Queenstown Public Library. For the purpose of this assessment, the building has been assumed to be an Importance Level 2 (IL2) structure in accordance with AS/NZS1170. We note that this may require some relocation of essential records and facilities (such that the building is not designated an essential post-disaster emergency facility and does not house contents of high value to the community).

Current code requires an equivalent new IL2 building constructed on the site to be designed for an earthquake with a 500 year return period, defined to be the design basis earthquake (DBE).

On the basis of a modal response spectrum analysis, the overall lateral load resisting capacity of the building is considered to be equivalent to 35% of the DBE. The capacity of the building is limited by the reinforced concrete masonry frames around the northern portion of the building, and the lift walls between first floor level and the council chambers.

A geotechnical assessment showed that liquefaction is expected to be triggered in earthquakes beyond approximately 50% of the DBE. The impacts of liquefaction include differential settlements and lateral displacements of up to 100mm, which could lead to loss of support of the precast flooring supported by concrete frames on isolated pad footings, located in the northern portion of the building.

The assessed capacity of the building is considered to be a lower-bound. At load levels above 35% of the DBE, limited failure is expected to occur until the shaking exceeds 50% DBE. At this point, the onset of liquefaction causes significant uncertainty with regards to the structural performance of the building, and loss of floor seating is a significant concern.

Other potential hazards were identified for the site, including the presence of a large tree located between the river and the building. The construction of a number of items was unable to be verified from the documentation available, and areas for further investigation have been identified.

7. REFERENCES

1. *Building Act 2004*, New Zealand Government

2. *New Building for the Wakatipu Club (Inc) – Structural Drawings,* Holmes Wood Poole & Johnstone, 1977

3. *Queenstown Lakes District Council Offices and Library – Structural Drawings,* Holmes Consulting Group Ltd, 1998

4. *QLDC Offices & Library Earthquake IEP Assessment*, Hadley Consultants Ltd, July 2011.

5. *Queenstown Lakes District Council Building Geotechnical Report,* Tonkin and Taylor Ltd, September 2014

6. *Horne Creek Flood Risk Study*, Otago Regional Council, 2006

7. *Queenstown Lakes District 2012 Liquefaction Hazard Assessment*, Tonkin & Taylor, September 2012

REPORT

Holmes Consulting Group

Queenstown Lakes District Council
Building
Geotechnical Assessment

Tonkin & Taylor

ENVIRONMENTAL AND ENGINEERING CONSULTANTS

REPORT

Holmes Consulting Group

Queenstown Lakes District Council
Building
Geotechnical Assessment

Report prepared for:

Holmes Consulting Group

Report prepared by:

Tonkin & Taylor Ltd

Distribution:

Holmes Consulting Group	**1 copy**
Tonkin & Taylor Ltd (FILE)	**1 copy**

September 2014

T&T Ref: 53614

Table of contents

1 Introduction

This report summarises the results of a geotechnical investigation and liquefaction assessment for the Queenstown Lakes District Council (QLDC) building at 10 Gorge Road, Queenstown. The work described in this document was commissioned by Holmes Consulting Group and has been completed in accordance with the terms and conditions outlined in T&T's letter of engagement dated 14 August 2014, reference number 53614.

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

- Desktop study of nearby geotechnical information
- Geotechnical investigation
- Liquefaction and lateral spreading assessment

Assessment of the foundation capacity, retaining wall stability and slope stability are beyond the scope of this report.

We understand that Holmes Consulting Group are undertaking a structural assessment of the building. the work described above is intended as an input into the structural assessment.

The QLDC Building is located at 10 Gorge Road, Queenstown, north of the CBD. The building houses the council chambers and a library. The triangular shaped site is bounded by a stream and playing fields to the west, Gorge Road to the north and east and by commercial buildings to the south. The site slopes from north-east to south-west; Gorge Road is approximately 3m higher than the stream. Gorge Road is supported by a concrete crib retaining wall.

The building is three storeys high; with ground level access to the second storey from Gorge Road. The building is of concrete construction with shallow foundations.

2 Ground and groundwater conditions

2.1 Geology and previous investigations

The geology of the site is complex. The soil units at the site are a mixture of lake sediments, glacial till, outwash gravel and valley infill. No geotechnical investigations results in the immediate vicinity of the building were available at the time of writing this report. Investigations in the Queenstown CBD, 200 – 300m south of the site, indicate variable soil units such as soft silt, loose sand and gravel layers.

2.2 August 2014 geotechnical investigation

A geotechnical investigation at the site was completed during the period 20 – 23 August 2014. The geotechnical investigation comprised:

- Three machine drilled boreholes to a maximum depth of 20m with Standard Penetration Tests (SPT) testing at 1.5m centres.
- Three Cone Penetration Tests (CPTs) to a maximum depth of 12m. The CPTs were all predrilled for the upper 1.5m, and CPT02 was also predrilled from 3 to 5.2m depth to push through a gravel layer.

The drilling and CPTs were undertaken by McMillan Drilling Ltd.

Figure 1 – Site investigation location plan

2.3 Geotechnical model

The soils at the site can be broadly divided into three different layers:

1. Valley infill – Loose to medium dense to Sandy GRAVEL and Gravelly SAND. Only present at BH03/CPT03 to the south of the site from 0 – 6m depth. Likely to have been deposited in the last 5000 years.
2. Lake sediments – Interbedded SILT, SAND and silty SAND. Present in all boreholes, generally 10 – 12m thick. Likely to have been deposited between 5000 – 15 000 years ago.
3. Older Till or Lake Sediments – 2-5m thick Dense GRAVEL and SAND layer overlying hard SILT. likely to have been deposited at least 25 000 years ago.

2.4 Groundwater

Groundwater levels were measured at 2.75m depth in BH01 on 20 August 2014 and at 2.8m depth in BH03 on 21 August 2014.

3 Liquefaction assessment

3.1 Susceptibility

The liquefaction susceptibility assessment indicates that:

- The gravel layers in the valley infill layer are not liquefiable.
- The sand, silty sand and sandy silt layers in the lake sediments layers are liquefiable given sufficient earthquake shaking.
- The silt layers in the lake sediments layers are considered too 'clay-like' to liquefy.
- The dense gravel and hard silt layers are not liquefiable.

3.2 Trigger

A liquefaction triggering assessment was undertaken on the CPT and SPT results using the methods of Boulanger and Idriss (2014) [1]. Peak horizontal ground accelerations (PGAs) and earthquake magnitudes used in the liquefaction triggering assessment have been based on the NZTA Bridge Manual[2].

For susceptible soil layers, the liquefaction triggering assessment indicates:

- Onset of liquefaction is at M6.3, PGA=0.15g (1/100 AEP event)
- Liquefaction is fully developed at M6.3, PGA=0.25g (1/200 AEP event)

3.3 Liquefaction consequences

The consequences of liquefaction at the site are expected to be 'moderate', which means that some liquefaction induced ground damage is expected in earthquakes greater than the 1/100 AEP event. This damage is expected to comprise small ground cracks, some sand boils, differential settlements and lateral displacements.

For the structural assessment of the existing building, the following displacements are expected to occur across the building footprint (i.e. a distance of 25m) due to liquefaction:

- differential settlements of up to 100mm
- lateral displacements of up to 100mm

Three methods were used to determine the likelihood of liquefaction induced land damage, the methods and their results are summarised in Table 1.

[1] Boulanger, R.W. & Idriss, I.M. (2014). CPT and SPT Liquefaction Triggering Procedures, University of California
[2] NZ Transport Agency's Bridge Manual, SP/M/022, Third edition, Amendment 0, Effective from May 2013

Table 1 – Liquefaction consequences summary

Method	Results	Interpretation	Implications for this site
Crust Thickness (m) (1)	H_1 = 2.8m @ BH1 H_1 = 3m @ BH2 H_1 = 6m @ BH3	Observations from Christchurch and other earthquakes are that the greater the depth to liquefied soils (crust thickness) the less damage is likely to be reflected at the ground surface. Examples of sand boils and damaging differential settlement are few for sites with a crust thickness H_1>3.5m	Minor ground surface damage including sand boils and settlement expected
Calculated Free Field Settlements (mm) (2)	Range: 100 – 150mm	Calculated settlements generally show a poor (and often conservative) correlation with actual settlements measured after earthquakes. They are best considered as an indicator of potential damage, rather than an accurate prediction of liquefaction induced settlements. Moderate land damage is expected for sites with calculated settlements of 100 – 300mm	Moderate damage, for example small ground cracks, oozing of sand. Differential settlements of up to 100mm could occur across the building footprint
Liquefaction Severity Number (LSN) (3)	Range: 15 to 30 Average: 20	LSN is a parameter calculated on the basis of investigation data considering liquefaction potential and its depth. This parameter has been correlated with evidence of surface ground damage in Christchurch. A higher LSN value indicates a likely greater degree of surface ground damage (3). LSN = 5-20 indicates minor expression of liquefaction, with some sand boils; LSN = 20-40 indicates moderate damage	Moderate damage – ground deformation can result in differential settlements, sand boils possible
Lateral Spread	Stream is shallow relative to the depth of liquefiable layers.	Lateral spreading towards the stream is not expected to occur However, general EQ induced downslope displacements of up to 100mm may be expected due to liquefaction	Lateral displacements of up to 100mm could occur across the building footprint

(1) Bowen, H.J. and Jacka, M.E (2013) Liquefaction induced ground damage in the Canterbury Earthquake: Predictions versus reality. Proceedings of the 19th NZGS Geotechnical Symposium. Editor CY Chin. Queenstown, New Zealand.

(2) Ministry of Business, Innovation & Employment (MBIE) Guidance - Repairing and rebuilding houses affected by the Canterbury earthquakes, Version 3, December 2012

(3) van Ballegory, S., Lacrosse, V., Jacka, M. and Malan, P. (2013) LSN – a new methodology for characterising the effects of liquefaction in terms of relative land damage severity. Proceedings of the 19th NZGS Geotechnical Symposium. Editor CY Chin. Queenstown, New Zealand.

4 Further work

The purpose of this report is to provide a liquefaction and lateral spreading assessment in order to inform the structural assessment of the existing building.

Should the structural assessment indicate that foundation strengthening or ground improvement be required, further geotechnical work will be required, including:

- Analysis of the seismic response of the foundation system, including impact of any retaining wall and slope movements on the structure
- Assessment of foundation strengthening options
- Assessment of ground improvement options
- Discussion with QLDC, Holmes and T&T regarding the pros and cons of each option
- Detailed design of the foundation strengthening/ground improvement once a preferred option is decided

5 Applicability

This report has been prepared for the benefit of Holmes Consulting Group Ltd with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose without our prior review and agreement.

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 recommended that the final foundation design, drawings and specification are undertaken by an appropriately qualified and experienced Geotechnical Chartered Professional Engineer, who is familiar with the contents of this report.

The recommendations and opinions which are contained in this report are based upon data from the geotechnical investigations as described in this report, 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.

Tonkin & Taylor Ltd

Environmental and Engineering Consultants

Report prepared by:	Authorised for Tonkin & Taylor Ltd by:
	
Hayden Bowen	Mike Jacka
Geotechnical Engineer	Project Director

HJB

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Appendix A: Geotechnical investigation

- **Borehole logs**
- **CPT results**

TONKIN & TAYLOR LTD

BOREHOLE LOG

BH No: BH1
Hole Location: Upper Carpark

PROJECT: QLDC Building		LOCATION: 10 Gorge Road, Queenstown	JOB No: 53614
CO-ORDINATES:	5004668.74 mN 1258225.53 mE	DRILL TYPE: Roto-Sonic	HOLE STARTED: 20/8/14
		DRILL METHOD: PQDT/Auto SPT	HOLE FINISHED: 20/8/14
R.L.:	96.18 m		DRILLED BY: McMillan Drilling
DATUM:		DRILL FLUID: LP2000	LOGGED BY: KPS CHECKED: BMCD

GEOLOGICAL | ENGINEERING DESCRIPTION

GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.	FLUID LOSS	WATER	CORE RECOVERY (%)	METHOD	CASING	TESTS	SAMPLES	R.L. (m)	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	SHEAR STRENGTH (kPa)	COMPRESSIVE STRENGTH (MPa)	DEFECT SPACING (mm)	SOIL DESCRIPTION / ROCK DESCRIPTION
			0	HAND DIG				96.0	0								0 to 1.52m- pre-clear for services.
								95.5	0.5								0.5
								95.0	1.0								1.0
									1.5								1.5
			80	SPT		*SPT@1.52 1/1//2/1/1/1 N=5		94.5			ML	W	F				SILT with trace sand, grey. Wet, no plasticity, rapid dilatancy; sand is fine.
									2.0								2.0
								94.0									1.88 to 1.97m- no recovery.
			28	PQDT					2.5								2.27 to 3.04m- no recovery. 2.5
		▼						93.5									2.7m- becoming brown; trace rootlets.
									3.0								3.0
			100	SPT		*SPT@3.04 0/0//0/0/0/0 N=0		93.0					VS				
									3.5								3.61m- trace fibrous organics (wood pieces up to 3mm length). Soft, low to moderate plasticity. 3.5
								92.5									4.08 to 4.11m- some fibrous organics (contains buried wood pieces); dark brownish black.
			85	PQDT					4.0								4.2m- minor fine to medium sand. 4.0
								92.0									4.26m- 25mm lense of fine to medium SAND, grey, wet, poorly graded.
									4.5								4.4 to 4.56m- no recovery. 4.5
			100	SPT		*SPT@4.56 0/0//0/3/6/17 N=26		91.5					VSt				
									5								4.9m- becoming greyish brown.

T+T DATATEMPLATE-SPT.GDT kps

TONKIN & TAYLOR LTD

BOREHOLE LOG

BH No: BH1
Hole Location: Upper Carpark

PROJECT: QLDC Building | LOCATION: 10 Gorge Road, Queenstown | JOB No: 53614

CO-ORDINATES: 5004668.74 mN, 1258225.53 mE
R.L.: 96.18 m
DATUM:

DRILL TYPE: Roto-Sonic
DRILL METHOD: PQDT/Auto SPT
DRILL FLUID: LP2000

HOLE STARTED: 20/8/14
HOLE FINISHED: 20/8/14
DRILLED BY: McMillan Drilling
LOGGED BY: KPS CHECKED: BMCD

GEOLOGICAL | ENGINEERING DESCRIPTION

GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.	FLUID LOSS	WATER	CORE RECOVERY (%)	METHOD	CASING	TESTS	SAMPLES	R.L. (m)	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	SHEAR STRENGTH (kPa)	COMPRESSIVE STRENGTH (MPa)	DEFECT SPACING (mm)	SOIL DESCRIPTION / ROCK DESCRIPTION
			100	PQDT				91.0			ML	W	S				Fine to medium SAND with minor silt, grey. Wet, poorly graded.
											SP		MD				5.29 to 5.37m- coarse gravel with minor sand and minor silt, light yellowish grey. Dense, wet, well graded; angular to subrounded, slightly weathered; sand is fine to coarse. (contains trace clay?). 5.5
								90.5	5.5		ML/SP		VSt/MD				Interbedded SILT and SAND; beds 35mm to 70mm thick. (SILT with trace fine to medium sand, grey, wet, low to moderate plasticity, slow dilatancy. Sand lenses finger into massive silt). 6.0
			80	SPT		*SPT@6.08 1/0//1/2/3/2 N=8		90.0	6.0				F/L				6.08m- 40mm heave, flushed prior to SPT.
									6.5								6.44 to 6.53m- no recovery. 6.5
			100	PQDT				89.5									7.0
								89.0	7.0								
									7.5		SP		MD				Fine to medium SAND with minor silt, grey.Wet, poorly graded. 7.5
			80	SPT		*SPT@7.60 0/2//4/4/4/5 N=17		88.5									
									8.0								7.96 to 8.05m- no recovery. 8.0
			100	PQDT				88.0									8.14m- trace silt.
									8.5								8.5
								87.5									8.61m- some silt. (contains clay?)
																	8.74 to 8.88m- some gravel; fine to coarse, angular to subangular.
								87.0	9.0								8.95m- minor silt. 9.0
			100	SPT		*SPT@9.12 0/0//0/0/0/1 N=1					ML	S	VS				9.12m- 100mm heave, flushed prior to SPT. SILT with trace sand, bluish grey. Saturated, moderate to high plasticity, very slow to no dilatancy; sand is fine to coarse.
								86.5	9.5								9.45 to 9.70m- minor to some gravel, minor sand; gravel is fine to coarse, angular to subrounded; sand is fine to coarse. 9.5
			100	PQDT					10								9.8m- soft.

T+T DATATEMPLATE-SPT.GDT kps

Log Scale 1:25

TONKIN & TAYLOR LTD

BOREHOLE LOG

BH No: BH1
Hole Location: Upper Carpark

PROJECT: QLDC Building		LOCATION: 10 Gorge Road, Queenstown	JOB No: 53614
CO-ORDINATES:	5004668.74 mN 1258225.53 mE	DRILL TYPE: Roto-Sonic	HOLE STARTED: 20/8/14
		DRILL METHOD: PQDT/Auto SPT	HOLE FINISHED: 20/8/14
R.L.:	96.18 m		DRILLED BY: McMillan Drilling
DATUM:		DRILL FLUID: LP2000	LOGGED BY: KPS CHECKED: BMCD

GEOLOGICAL | ENGINEERING DESCRIPTION

GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.

FLUID LOSS | WATER | CORE RECOVERY (%) | METHOD | CASING | TESTS | SAMPLES | R.L. (m) | DEPTH (m) | GRAPHIC LOG | CLASSIFICATION SYMBOL | MOISTURE CONDITION / WEATHERING | STRENGTH/DENSITY CLASSIFICATION | SHEAR STRENGTH (kPa) | COMPRESSIVE STRENGTH (MPa) | DEFECT SPACING (mm)

SOIL DESCRIPTION
Soil type, minor components, plasticity or particle size, colour.
ROCK DESCRIPTION
Substance: Rock type, particle size, colour, minor components.
Defects: Type, inclination, thickness, roughness, filling.

Core Recovery (%)	Method	Tests	Depth (m)	Classification Symbol	Moisture Condition	Strength/Density Classification	Description
100	PQDT		0	ML	S	VS	SILT with trace sand, bluish grey. Saturated, moderate to high plasticity, very slow to no dilatancy; sand is fine to coarse.
			10.19				10.19m- very soft, low plasticity, quick dilatancy.
			10.5				
90	SPT	*SPT@10.64 0/0//1/2/3/2 N=8	11.0			F	
			11.05 to 11.09m- no recovery.				
100	PQDT		11.14				11.14m- firm, no plasticity, no dilatancy.
			11.5		M		
			11.54				11.54m- moist.
			11.6				11.6m- some gravel, minor sand, bluish grey mottled light grey; gravel is fine to coarse, angular to subrounded; sand is fine to coarse.
70	SPT	*SPT@12.16 6/10//10/15/20/25 N>50 SOLID	12.0			VD	
			12.5	SM			Silty fine to coarse SAND with some gravel, grey. Very dense, saturated, well graded; gravel is fine to coarse, angular to subrounded.
			12.48 to 12.61m- no recovery.				
100	PQDT		13.0	GW	S		Sandy fine to coarse GRAVEL with minor silt, grey. Saturated, well graded; angular to subrounded; sand is fine to coarse.
				SW			Gravelly fine to coarse SAND with minor silt, grey. Saturated, well graded; gravel is fine to coarse, angular to subrounded.
			13.5				
0	SPT	*SPT@13.68 12/30//42 N>50 SOLID	13.68 to 14.13m- sample obtained from overcore.				
			14.0	GW			Sandy fine to coarse GRAVEL with some silt, bluish grey. Saturated, well graded; angular to subrounded; sand is fine to coarse.
100	PQDT		14.5				
			15	ML	D/M	H	

T+T DATATEMPLATE-SPT.GDT kps

TONKIN & TAYLOR LTD

BOREHOLE LOG

BH No: BH1
Hole Location: Upper Carpark

PROJECT: QLDC Building | LOCATION: 10 Gorge Road, Queenstown | JOB No: 53614

CO-ORDINATES: 5004668.74 mN, 1258225.53 mE
R.L.: 96.18 m
DATUM:

DRILL TYPE: Roto-Sonic
DRILL METHOD: PQDT/Auto SPT
DRILL FLUID: LP2000

HOLE STARTED: 20/8/14
HOLE FINISHED: 20/8/14
DRILLED BY: McMillan Drilling
LOGGED BY: KPS CHECKED: BMCD

GEOLOGICAL | ENGINEERING DESCRIPTION

GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.	FLUID LOSS	WATER	CORE RECOVERY (%)	METHOD	CASING	TESTS	SAMPLES	R.L. (m)	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	SHEAR STRENGTH (kPa)	COMPRESSIVE STRENGTH (MPa)	DEFECT SPACING (mm)	SOIL DESCRIPTION / ROCK DESCRIPTION
			100	PQDT							ML	D/M	H				15.0m- minor gravel; medium to coarse, angular to subrounded.
			0	SPT		*SPT@15.20 40/20 for 5mm. N>50 SOLID		81.0			SM	W	VD				Sandy SILT with trace gravel, bluish grey. Dry to moist, low plasticity, slow dilatancy; sand is fine to coarse; gravel is fine, subangular to subrounded. (contains clay?)
								15.5									Silty fine to medium SAND with some gravel, grey. Wet, well graded, low plasticity; sand is fine to coarse; gravel is fine, subangular to subrounded. (contains clay?) 15.2 to 15.35m- sample obtained from overcore.
								80.5									End of Borehole @ 15.35mbgl (refusal).
								16.0									
								80.0									
								16.5									
								79.5									
								17.0									
								79.0									
								17.5									
								78.5									
								18.0									
								78.0									
								18.5									
								77.5									
								19.0									
								77.0									
								19.5									
								76.5									
								20									

T+T DATATEMPLATE-SPT.GDT kps

Log Scale 1:25

TONKIN & TAYLOR LTD

BOREHOLE LOG

BH No: BH2
Hole Location: Driveway Entrance

PROJECT: QLDC Building | LOCATION: 10 Gorge Road, Queenstown | JOB No: 53614

CO-ORDINATES: 5004637.08 mN, 1258254.23 mE
R.L.: 98.71 m
DATUM:

DRILL TYPE: Roto-Sonic
DRILL METHOD: PQDT/Auto SPT
DRILL FLUID: LP2000

HOLE STARTED: 22/8/14
HOLE FINISHED: 23/8/14
DRILLED BY: McMillan Drilling
LOGGED BY: KPS CHECKED: BMCD

GEOLOGICAL | ENGINEERING DESCRIPTION

GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.	FLUID LOSS	WATER	CORE RECOVERY (%)	METHOD	CASING	TESTS	SAMPLES	R.L. (m)	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	SHEAR STRENGTH (kPa)	COMPRESSIVE STRENGTH (MPa)	DEFECT SPACING (mm)	SOIL DESCRIPTION / ROCK DESCRIPTION
			0	HAND DIG				98.5	0								0 to 1.52m- pre-clear for services.
								98.0	0.5								0.5
								97.5	1.0								1.0
			50	SPT		*SPT@1.52 3/3//1/1/1/1 N=4		97.0	1.5		SM	M	L				1.5 Silty fine to coarse SAND with minor gravel, orangish brown. Moist, well graded; gravel is fine to medium, angular to subrounded.
								96.5	2.0			W					1.75 to 1.97m- no recovery. 2.0
			25	PQDT				96.0	2.5								Silty fine to coarse SAND with some gravel, orangish brown mottled greenish grey, bluish grey, reddish brown and dark brown. Wet, well graded; gravel is fine to medium, angular to subrounded, slightly to moderately weathered. (contains clay?) 2.5 2.24 to 3.04m- no recovery.
			0	SPT		*SPT@3.04 5/8//6/7/5/3 N=21 SOLID		95.5	3.0		SP		MD				3.0 3.04 to 3.49m- sample obtained from overcore.
			100	PQDT				95.0	3.5								Fine to medium SAND with minor to some silt, minor gravel, and trace cobble; grey. Wet, poorly graded; gravel is medium to coarse, subangular to subrounded; cobble is 70mm, subrounded. 3.5 3.3m- gravel and cobble absent. 3.7m- 25mm lense of organic silt with some fine to medium sand, dark brown. 3.9m- trace silt.
								94.5	4.0								4.0
			100	SPT		*SPT@4.56 0/1//2/1/1/2 N=6		94.0	4.5				L				4.5 4.5m- minor silt.
									5								

T+T DATATEMPLATE-SPT.GDT kps

Log Scale 1:25

TONKIN & TAYLOR LTD

BOREHOLE LOG

BH No: BH2
Hole Location: Driveway Entrance

PROJECT: QLDC Building | LOCATION: 10 Gorge Road, Queenstown | JOB No: 53614

CO-ORDINATES: 5004637.08 mN, 1258254.23 mE
R.L.: 98.71 m
DATUM:

DRILL TYPE: Roto-Sonic
DRILL METHOD: PQDT/Auto SPT
DRILL FLUID: LP2000

HOLE STARTED: 22/8/14
HOLE FINISHED: 23/8/14
DRILLED BY: McMillan Drilling
LOGGED BY: KPS CHECKED: BMCD

GEOLOGICAL | ENGINEERING DESCRIPTION

GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.	FLUID LOSS	WATER	CORE RECOVERY (%)	METHOD	CASING	TESTS	SAMPLES	R.L. (m)	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	SHEAR STRENGTH (kPa)	COMPRESSIVE STRENGTH (MPa)	DEFECT SPACING (mm)	SOIL DESCRIPTION / ROCK DESCRIPTION
			100	PQDT				93.5	5.5		SP	W	L				Fine to medium SAND with minor to some silt, minor gravel, and trace cobble; grey. Wet, poorly graded; gravel is medium to coarse, subangular to subrounded; cobble is 70mm, subrounded.
								93.0	6.0								5.5
			100	SPT		*SPT@6.08 0/0//0/0/0/0 N=0		92.5	6.5		SM		VL				Silty fine to medium SAND, grey. Wet, poorly graded.
																	6.5
			100	PQDT				92.0	7.0		ML		VS				SILT with minor sand, brownish grey. Wet, moderate plasticity, slow dilatancy; sand is fine to medium. 6.8m- trace sand, fine to medium. 6.8 to 6.83m- trace rootlets; dark grey.
								91.5	7.5								7.0 7.12 to 7.45m- 2 to 10mm thick lenses of sand with minor silt; sand is fine to medium. 7.4m- trace fibrous organics.
			90	SPT		*SPT@7.60 1/1//2/3/3/4 N=12		91.0	8.0		SP		MD				7.5 Fine to medium SAND with some silt, grey. Wet, poorly graded.
								90.5	8.5								8.0 8.01 to 8.05m- no recovery. 8.3m- minor silt.
			100	PQDT				90.0	9.0		ML		St				8.5 SILT with minor sand, bluish grey. Wet, moderate to high plasticity, very slow dilatancy; sand is fine to medium.
			100	SPT		*SPT@9.12 0/1//4/3/3/6 N=16		89.5	9.5		SM SP		MD				9.0 Silty fine to medium SAND with minor gravel, bluish grey. Wet, poorly graded; gravel is fine to medium, subangular to subrounded. Fine to medium SAND with some silt, bluish grey. Wet, poorly graded.
											ML		St				9.5 Sandy SILT with some gravel, grey. Wet, no plasticity, no to very slow dilatancy; sand is fine to medium; gravel is fine to medium, angular to subrounded.
			100	PQDT				89.0	10		SM		MD				

T+T DATATEMPLATE-SPT.GDT kps

TONKIN & TAYLOR LTD

BOREHOLE LOG

BH No: BH2
Hole Location: Driveway Entrance

PROJECT: QLDC Building | LOCATION: 10 Gorge Road, Queenstown | JOB No: 53614

CO-ORDINATES: 5004637.08 mN, 1258254.23 mE
R.L.: 98.71 m
DATUM:

DRILL TYPE: Roto-Sonic
DRILL METHOD: PQDT/Auto SPT
DRILL FLUID: LP2000

HOLE STARTED: 22/8/14
HOLE FINISHED: 23/8/14
DRILLED BY: McMillan Drilling
LOGGED BY: KPS CHECKED: BMCD

GEOLOGICAL | ENGINEERING DESCRIPTION

GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.	FLUID LOSS	WATER	CORE RECOVERY (%)	METHOD	CASING	TESTS	SAMPLES	R.L. (m)	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	SHEAR STRENGTH (kPa)	COMPRESSIVE STRENGTH (MPa)	DEFECT SPACING (mm)	SOIL DESCRIPTION / ROCK DESCRIPTION
			100	PQDT				88.5	10.5		SM	W	MD				Silty fine to medium SAND with some gravel, grey. Wet, well graded; gravel is fine to coarse, subangular to subrounded.
			60	SPT		*SPT@10.62 8/12//13/15/13/6 N=47		88.0					D				
									11.0		ML		H				SILT with minor sand and minor gravel, dark grey. Wet, low plasticity, moderate dilatancy; sand is fine to coarse; gravel is fine to medium, subangular to subrounded. 10.91 to 11.09m- no recovery. 11.28m- gravel absent; grey.
			100	PQDT				87.5	11.5								
								87.0	12.0								12.02m- trace sand, fine to medium; low to moderate plasticity; very slow dilatancy.
			80	SPT		*SPT@12.16 3/4//5/6/7/10 N=28		86.5					VSt				
									12.5								12.52 to 12.61m- no recovery.
			100	PQDT				86.0	13.0								
								85.5	13.5								13.49m- brownish grey. 13.5 to 13.56m- minor fibrous organics.
			100	SPT		*SPT@13.68 4/5//6/8/9/10 N=33		85.0					H				
									14.0			M					14.0m- grey; moist, low plasticity, rapid dilatancy.
			100	PQDT				84.5	14.5								
								84.0									
								15									

T+T DATATEMPLATE-SPT.GDT kps

TONKIN & TAYLOR LTD

BOREHOLE LOG

BH No: BH2
Hole Location: Driveway Entrance

PROJECT: QLDC Building		LOCATION: 10 Gorge Road, Queenstown	JOB No: 53614
CO-ORDINATES: 5004637.08 mN 1258254.23 mE		DRILL TYPE: Roto-Sonic	HOLE STARTED: 22/8/14 HOLE FINISHED: 23/8/14
R.L.: 98.71 m		DRILL METHOD: PQDT/Auto SPT	DRILLED BY: McMillan Drilling
DATUM:		DRILL FLUID: LP2000	LOGGED BY: KPS CHECKED: BMCD

GEOLOGICAL | ENGINEERING DESCRIPTION

GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.	FLUID LOSS	WATER	CORE RECOVERY (%)	METHOD	CASING	TESTS	SAMPLES	R.L. (m)	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	SHEAR STRENGTH (kPa) 10 25 50 100 200	COMPRESSIVE STRENGTH (MPa) 1 5 20 50 100 200	DEFECT SPACING (mm) 60 200 600 2000	SOIL DESCRIPTION: Soil type, minor components, plasticity or particle size, colour. ROCK DESCRIPTION: Substance: Rock type, particle size, colour, minor components. Defects: Type, inclination, thickness, roughness, filling.
			100	PQDT				83.5			ML	M	H				15.2m- casing advanced twice per run (material expanding up tube).
			100	SPT		*SPT@15.20 5/6//7/8/9/13 N=37		15.5									
								83.0									
			100	PQDT				16.0									
								82.5									
								16.5									
			100	SPT		*SPT@16.72 4/5//8/8/10/10 N=36		82.0									
								17.0									
								81.5									
			100	PQDT				17.5									
								81.0									
								18.0									
			100	SPT		*SPT@18.24 4/6//8/8/11/11 N=38		80.5									18.24 to 19.45m- saturated.
								18.5									
								80.0									
			100	PQDT				19.0									
								79.5									
								19.5									
			100	SPT		*SPT@19.76 4/5//8/11/11/13 N=43		79.0									
								20									

T+T DATATEMPLATE-SPT.GDT kps

TONKIN & TAYLOR LTD

BOREHOLE LOG

BH No: BH2
Hole Location: Driveway Entrance

PROJECT: QLDC Building | LOCATION: 10 Gorge Road, Queenstown | JOB No: 53614

CO-ORDINATES: 5004637.08 mN, 1258254.23 mE
R.L.: 98.71 m
DATUM:

DRILL TYPE: Roto-Sonic
DRILL METHOD: PQDT/Auto SPT
DRILL FLUID: LP2000

HOLE STARTED: 22/8/14
HOLE FINISHED: 23/8/14
DRILLED BY: McMillan Drilling
LOGGED BY: KPS CHECKED: BMCD

GEOLOGICAL | ENGINEERING DESCRIPTION

GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.	FLUID LOSS	WATER	CORE RECOVERY (%)	METHOD	CASING	TESTS	SAMPLES	R.L. (m)	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	SHEAR STRENGTH (kPa) 10 25 50 100 200	COMPRESSIVE STRENGTH (MPa) 1 5 20 50 100	DEFECT SPACING (mm) 60 200 600 2000	SOIL DESCRIPTION: Soil type, minor components, plasticity or particle size, colour. ROCK DESCRIPTION: Substance: Rock type, particle size, colour, minor components. Defects: Type, inclination, thickness, roughness, filling.
			100	SPT				78.5			ML	M	H				
																	End of Borehole @ 20.21mbgl (target).
								20.5									
								78.0									
								21.0									
								77.5									
								21.5									
								77.0									
								22.0									
								76.5									
								22.5									
								76.0									
								23.0									
								75.5									
								23.5									
								75.0									
								24.0									
								74.5									
								24.5									
								74.0									
								25									

T+T DATATEMPLATE-SPT.GDT kps

Log Scale 1:25

TONKIN & TAYLOR LTD

BOREHOLE LOG

BH No: BH3
Hole Location: Lower Carpark

PROJECT: QLDC Building | LOCATION: 10 Gorge Road, Queenstown | JOB No: 53614

CO-ORDINATES: 5004614.38 mN, 1258219.53 mE
R.L.: 96.70 m
DATUM:

DRILL TYPE: Roto-Sonic
DRILL METHOD: PQDT/Auto SPT
DRILL FLUID: LP2000

HOLE STARTED: 21/8/14
HOLE FINISHED: 21/8/14
DRILLED BY: McMillan Drilling
LOGGED BY: KPS CHECKED: BMCD

GEOLOGICAL | ENGINEERING DESCRIPTION

GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.	FLUID LOSS	WATER	CORE RECOVERY (%)	METHOD	CASING	TESTS	SAMPLES	R.L. (m)	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	SHEAR STRENGTH (kPa)	COMPRESSIVE STRENGTH (MPa)	DEFECT SPACING (mm)	SOIL DESCRIPTION / ROCK DESCRIPTION
			0	HAND DIG				96.5	0.5								0 to 1.52m- pre-clear for services.
								96.0	1.0								
								95.5	1.5								
			60	SPT		*SPT@1.52 4/2//3/2/2/2 N=9		95.0	2.0		GW ML	W M	L St				Sandy fine to coarse GRAVEL with minor silt, orangish brown. Angular to subangular; wet, poorly graded; sand is fine to coarse. (contains clay?)
			25	PQDT				94.5	2.5		GW	W	L				Sandy SILT with minor gravel, yellowish orangish brown. Moist, no plasticity, moderate dilatancy; sand is fine to coarse; gravel is fine to medium, angular to subangular. (contains clay?) 1.7 to 1.97m- no recovery. Sandy fine to coarse GRAVEL with minor to some silt, brown. Angular to subangular; wet, well graded; sand is fine to coarse. (contains clay?) 2.15m- some silt. 2.24 to 3.04m- no recovery.
			55	SPT		*SPT@3.04 4/3//3/2/3/3 N=11		94.0	3.0				MD				
								93.5	3.5								3.2m- reddish brown. 3.29 to 3.49m- no recovery.
			100	PQDT				93.0	4.0		SW	M					Gravelly fine to coarse SAND with trace silt, yellowish brown. Moist, well graded; gravel is fine to coarse, angular to subrounded, slightly weathered. (contains clay?)
								92.5	4.5			W					4.5m- minor silt, reddish brown, wet; gravel is moderately weathered.
			60	SPT		*SPT@4.56 3/2//3/4/3/4 N=14		92.0	5			M					4.6m- 60mm thick lense of fine to coarse sand with minor fine gravel, grey. 4.7m- light yellowish orangish brown, moist. 4.83 to 5.01m- no recovery.

TONKIN & TAYLOR LTD

BOREHOLE LOG

BH No: BH3
Hole Location: Lower Carpark

PROJECT: QLDC Building		LOCATION: 10 Gorge Road, Queenstown	JOB No: 53614
CO-ORDINATES:	5004614.38 mN 1258219.53 mE	DRILL TYPE: Roto-Sonic	HOLE STARTED: 21/8/14
		DRILL METHOD: PQDT/Auto SPT	HOLE FINISHED: 21/8/14
R.L.:	96.70 m		DRILLED BY: McMillan Drilling
DATUM:		DRILL FLUID: LP2000	LOGGED BY: KPS CHECKED: BMCD

GEOLOGICAL | ENGINEERING DESCRIPTION

GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.

FLUID LOSS | WATER | CORE RECOVERY (%) | METHOD | CASING | TESTS | SAMPLES | R.L. (m) | DEPTH (m) | GRAPHIC LOG | CLASSIFICATION SYMBOL | MOISTURE CONDITION / WEATHERING | STRENGTH/DENSITY CLASSIFICATION | SHEAR STRENGTH (kPa) | COMPRESSIVE STRENGTH (MPa) | DEFECT SPACING (mm)

SOIL DESCRIPTION
Soil type, minor components, plasticity or particle size, colour.
ROCK DESCRIPTION
Substance: Rock type, particle size, colour, minor components.
Defects: Type, inclination, thickness, roughness, filling.

CORE RECOVERY (%)	METHOD	TESTS	R.L. (m)	DEPTH (m)	CLASSIFICATION SYMBOL	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	ENGINEERING DESCRIPTION
100	PQDT		91.5	5.5	SW	M	MD	5.5 to 6.0m- greyish brown.
			91.0	6.0				5.92m- some silt, brown, wet.
70	SPT	*SPT@6.08 1/2//1/1/2/1 N=5	90.5	6.5			L	6.4 to 6.53m- no recovery.
100	PQDT		90.0	7.0		W		7.02m- brownish grey mottled orange. 7.05m- 10mm lense of silt, grey mottled orange; no plasticity, rapid dilatancy.
			89.5	7.5				Fine to coarse SAND with minor to some gravel and minor silt, grey. Wet, well graded; gravel is fine to medium, angular to subangular. 7.43m- 30mm thick lense of sandy fine to coarse gravel with minor silt, grey; subangular to subrounded, well graded; sand is fine to coarse.
70	SPT	*SPT@7.60 2/3//2/2/1/2 N=7	89.0	7.5				7.6m- 40mm heave observed prior to SPT. 7.65m- orangish brown.
			88.5	8.0				7.92 to 8.05m- no recovery. 8.05m- sand is fine to medium. 8.2m- some silt, brownish grey.
100	PQDT		88.0	8.5	ML		S	SILT with trace sand, grey. Wet, low plasticity, slow dilatancy; sand is fine to medium. 8.63m- 100mm lense of sandy fine to coarse gravel with minor silt, reddish orangish brown; angular to subangular; wet, well graded; sand is fine to coarse.
100	SPT	*SPT@9.12 0/0//0/0/1/2 N=3	87.5	9.0				9.12m- bluish grey, moderate plasticity, moderate dilatancy. 9.3 to 9.6m- no plasticity, rapid dilatancy.
100	PQDT		87.0	9.5				9.83m- 15mm lense of silty fine to medium sand.
				10				

T+T DATATEMPLATE-SPT.GDT kps

TONKIN & TAYLOR LTD

BOREHOLE LOG

BH No: BH3
Hole Location: Lower Carpark

PROJECT: QLDC Building | LOCATION: 10 Gorge Road, Queenstown | JOB No: 53614

CO-ORDINATES: 5004614.38 mN, 1258219.53 mE
R.L.: 96.70 m
DATUM:

DRILL TYPE: Roto-Sonic
DRILL METHOD: PQDT/Auto SPT
DRILL FLUID: LP2000

HOLE STARTED: 21/8/14
HOLE FINISHED: 21/8/14
DRILLED BY: McMillan Drilling
LOGGED BY: KPS CHECKED: BMCD

GEOLOGICAL | ENGINEERING DESCRIPTION

GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.	FLUID LOSS	WATER	CORE RECOVERY (%)	METHOD	CASING	TESTS	SAMPLES	R.L. (m)	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	SHEAR STRENGTH (kPa)	COMPRESSIVE STRENGTH (MPa)	DEFECT SPACING (mm)	SOIL DESCRIPTION / ROCK DESCRIPTION
			100	PQDT				86.5	10.5		ML	W	S				9.9 to 10.0m- bluish grey mottled brown, trace rootlets. 10.38m- 3mm lense of rootlets, dark greyish brown. 10.5
			80	SPT		*SPT@10.62 0/0//0/0/0/1 N=1		86.0	11.0		SM		VL				Silty fine to medium SAND, grey. Wet, poorly graded; interbedded with sandy SILT, and SILT with trace sand; silt has low to moderate plasticity, very slow to moderate dilatancy; beds are 25mm to 300mm. 11.0 to 11.09m- no recovery. 11.0
			100	PQDT				85.5	11.5								11.39 to 11.49m- dark grey mottled light grey. 11.5
								85.0	12.0								11.83m - cobble (80mm), subrounded, moderately weathered (covered in clay). 12.0
			5	SPT		*SPT@12.16 2/2//3/2/1/2 N=8		84.5	12.5				L				12.16m- 400mm heave observed prior to SPT. 12.18 to 12.61m- no recovery. 12.5
			100	PQDT				84.0	13.0		ML		F				12.61m- trace gravel; medium to coarse, angular to subangular. SILT with trace sand, grey. Wet, moderate plasticity, very slow to no dilatancy; sand is fine to medium. 13.0
								83.5	13.5								13.5
			100	SPT		*SPT@13.68 0/0//1/1/5/7 N=14		83.0	14.0		SM		MD				13.68m - cobble (100mm), subrounded. Silty fine to medium SAND, grey. Wet, poorly graded. 14.0m- minor gravel; fine to coarse, subangular. 14.0
			100	PQDT				82.5	14.5								14.43m- some gravel; fine to coarse, subangular. 14.5
								82.0	15		ML	M	St				Sandy SILT with minor gravel, grey. Moist, low plasticity, rapid dilatancy; sand is fine to coarse; gravel is fine to coarse, subrounded.

SOIL DESCRIPTION: Soil type, minor components, plasticity or particle size, colour.
ROCK DESCRIPTION: Substance: Rock type, particle size, colour, minor components. Defects: Type, inclination, thickness, roughness, filling.

TONKIN & TAYLOR LTD

BOREHOLE LOG

BH No: BH3
Hole Location: Lower Carpark

PROJECT: QLDC Building | LOCATION: 10 Gorge Road, Queenstown | JOB No: 53614

CO-ORDINATES: 5004614.38 mN, 1258219.53 mE
R.L.: 96.70 m
DATUM:

DRILL TYPE: Roto-Sonic
DRILL METHOD: PQDT/Auto SPT
DRILL FLUID: LP2000

HOLE STARTED: 21/8/14
HOLE FINISHED: 21/8/14
DRILLED BY: McMillan Drilling
LOGGED BY: KPS CHECKED: BMCD

GEOLOGICAL | ENGINEERING DESCRIPTION

GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.	FLUID LOSS	WATER	CORE RECOVERY (%)	METHOD	CASING	TESTS	SAMPLES	R.L. (m)	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	SHEAR STRENGTH (kPa)	COMPRESSIVE STRENGTH (MPa)	DEFECT SPACING (mm)	SOIL DESCRIPTION / ROCK DESCRIPTION
			100	PQDT				81.5			ML	M	St				15.0m- some gravel; fine to coarse, subangular.
			100	SPT		*SPT@15.20 4/16//26/23/18/18 N>50		15.5			GW	W	VD				Silty fine to coarse GRAVEL with some sand, grey. Angular to subangular; wet, well graded; sand is fine to coarse. 15.5
			42	PQDT				81.0									
								16.0									16.0
								80.5									16.27 to 16.72m- no recovery.
								16.5									16.5
			0	SPT		*SPT@16.72 10/40//10 for 2mm N>50 SOLID		80.0			ML	M	H				SILT with trace sand and trace gravel, grey. Moist, low plasticity, slow dilatancy; sand is fine to medium; gravel is fine to medium, subangular. 17.0 16.72 to 17.17m- sample obtained from overcore.
								17.0			SP	W	VD				
			100	PQDT				79.5									Fine to medium SAND with minor silt and trace gravel, grey. Wet, poorly graded; gravel is medium, subrounded.
								17.5									17.5
								79.0			ML		H				Sandy SILT, grey. Wet, low plasticity, rapid dilatancy; sand is fine to medium. 18.0
								18.0									
			100	SPT		*SPT@18.24 8/12//15/20/20/22 N>50		78.5									18.24m- 400mm heave observed prior to SPT.
								18.5									18.5
								78.0									End of borehole@18.69mbgl (refusal).
								19.0									19.0
								77.5									
								19.5									19.5
								77.0									
								20									

T+T DATATEMPLATE-SPT.GDT kps

Log Scale 1:25

CONE PENETRATION TEST (CPT) REPORT

Client: Tonkin and Taylor Ltd
Location: 10 Gorge Road, Queenstown

Printed: 25/08/2014

CONE PENETRATION TEST

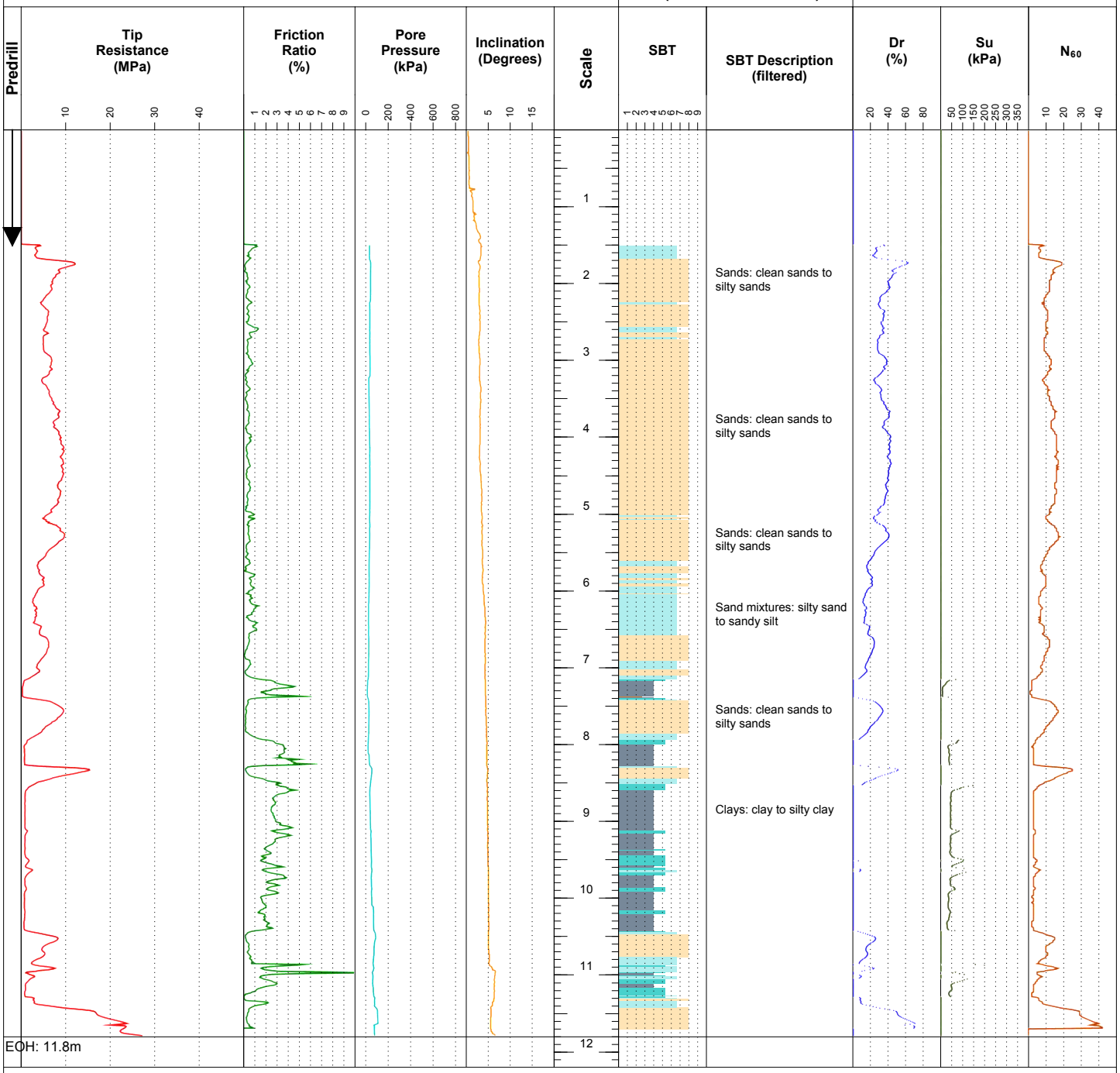
Job:	13839
CPT No.:	CPTu001

Name: QLDC Building, Queenstown
Client: Tonkin and Taylor Ltd
Location: 10 Gorge Road, Queenstown

Grid: NZTM
Datum:
Termination: -

North (m): 5004667.60
East (m): 1258222.23
Elevation (m): -
Hole Depth (m): 11.80

RAW DATA | **SOIL BEHAVIOUR TYPE (NON-NORMALISED)** | **ESTIMATED PARAMETERS**



Operator: P. Buunk
Cone Reference: 081034T
Cone Area Ratio: 0.75
Cone Type: I-CFXYP20-10

Date: 22/08/2014
Predrill: 1.50
Water Level: -
Collapse: 1.50

Tip Resistance (MPa) Initial: -5.6467 **Final:** -5.6036
Local Friction (MPa) Initial: -0.0418 **Final:** -0.0434
Pore Pressure (KPa) Initial: -185.281 **Final:** -178.494

Effective Refusal
Tip:
Gauge: ✓
Inclinometer:
Other:
Target Depth:

Soil Behaviour Type (SBT) - Robertson et al. 1986

0	Undefined	5	Sand mixtures: silty sand to sandy silt
1	Sensitive fine-grained	6	Sands: clean sands to silty sands
2	Clay - organic soil	7	Dense sand to gravelly sand
3	Clays: clay to silty clay	8	Stiff sand to clayey sand
4	Silt mixtures: clayey silt & silty clay	9	Stiff fine-grained

Notes & Limitations

Data shown on this report has been assessed to provide a basic interpretation in terms of Soil Behaviour Type (SBT) and various geotechnical soil and design parameters using methods published in P. K. Robertson and K.L. Cabal (2010), Guide to Cone Penetration Testing for Geotechnical Engineering, 4th Edition. The interpretations are presented only as a guide for geotechnical use, and should be carefully reviewed by the user. Both McMillan Drilling Ltd & Geroc Solutions Ltd do not warranty the correctness or the applicability of any of the geotechnical soil and design parameters shown and does not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used to derive data shown in this report.

Remarks
Effective Refusal

Hole Depth (m): 11.80

CONE PENETRATION TEST

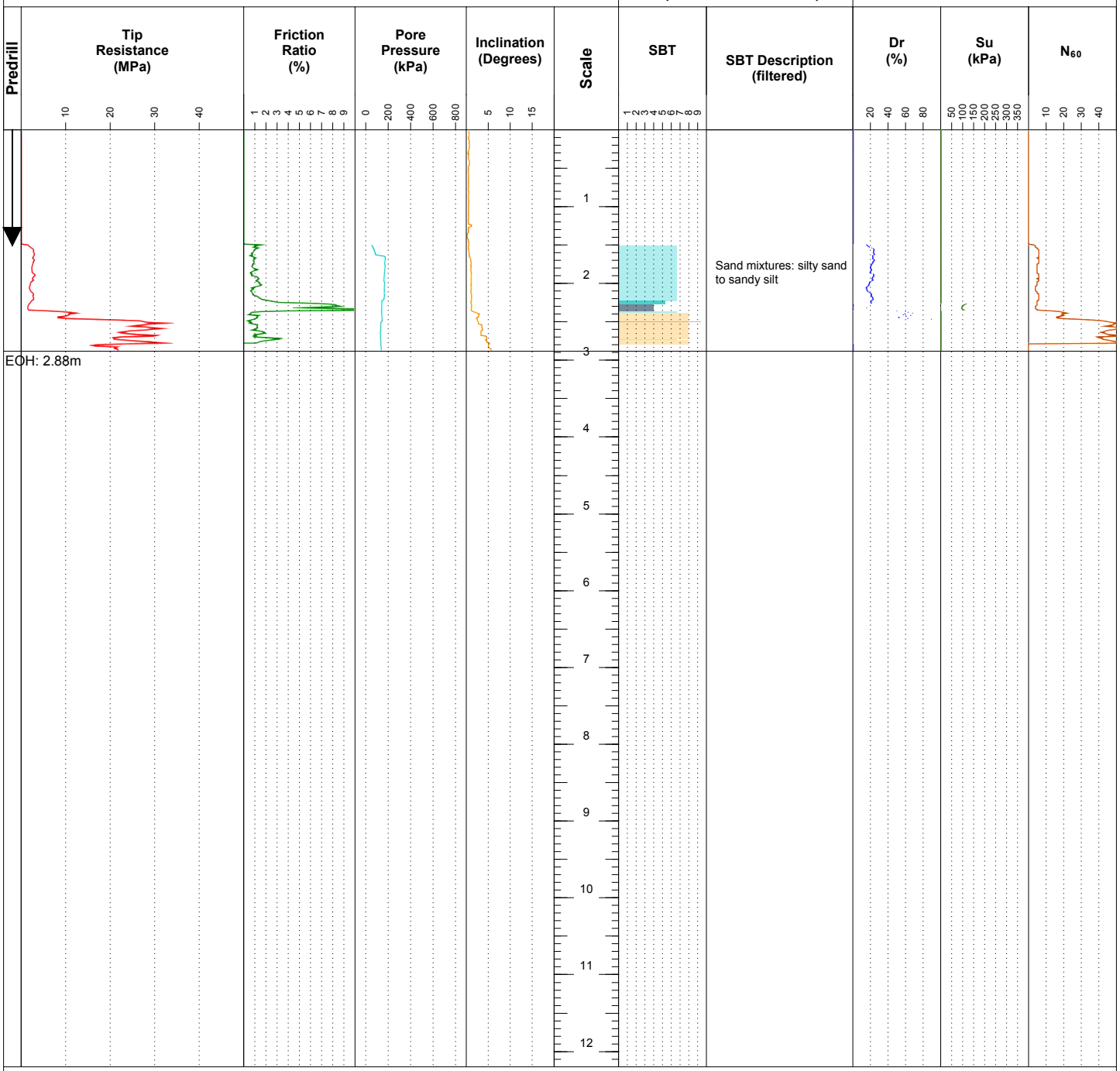
Job:	13839
CPT No.:	CPTu002

Name: QLDC Building, Queenstown
Client: Tonkin and Taylor Ltd
Location: 10 Gorge Road, Queenstown

Grid: NZTM
Datum:
Termination: -

North (m): 5004637.06
East (m): 1258253.85
Elevation (m): -
Hole Depth (m): 2.88

RAW DATA | **SOIL BEHAVIOUR TYPE (NON-NORMALISED)** | **ESTIMATED PARAMETERS**



Operator: P. Buunk
Cone Reference: 081034T
Cone Area Ratio: 0.75
Cone Type: I-CFXYP20-10

Date: 22/08/2014
Predrill: 1.50
Water Level: -
Collapse: 0.50

Tip Resistance (MPa) Initial: -5.615 **Final:** -5.5958
Local Friction (MPa) Initial: -0.0411 **Final:** -0.0437
Pore Pressure (KPa) Initial: -183.315 **Final:** -176.876

Effective Refusal
Tip:
Gauge:
Inclinometer: ✔
Other:
Target Depth:

Soil Behaviour Type (SBT) - Robertson et al. 1986

0	Undefined	5	Sand mixtures: silty sand to sandy silt
1	Sensitive fine-grained	6	Sands: clean sands to silty sands
2	Clay - organic soil	7	Dense sand to gravelly sand
3	Clays: clay to silty clay	8	Stiff sand to clayey sand
4	Silt mixtures: clayey silt & silty clay	9	Stiff fine-grained

Notes & Limitations

Data shown on this report has been assessed to provide a basic interpretation in terms of Soil Behaviour Type (SBT) and various geotechnical soil and design parameters using methods published in P. K. Robertson and K.L. Cabal (2010), Guide to Cone Penetration Testing for Geotechnical Engineering, 4th Edition. The interpretations are presented only as a guide for geotechnical use, and should be carefully reviewed by the user. Both McMillan Drilling Ltd & Geroc Solutions Ltd do not warranty the correctness or the applicability of any of the geotechnical soil and design parameters shown and does not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used to derive data shown in this report.

Remarks
Effective Refusal

Hole Depth (m): 2.88

CONE PENETRATION TEST

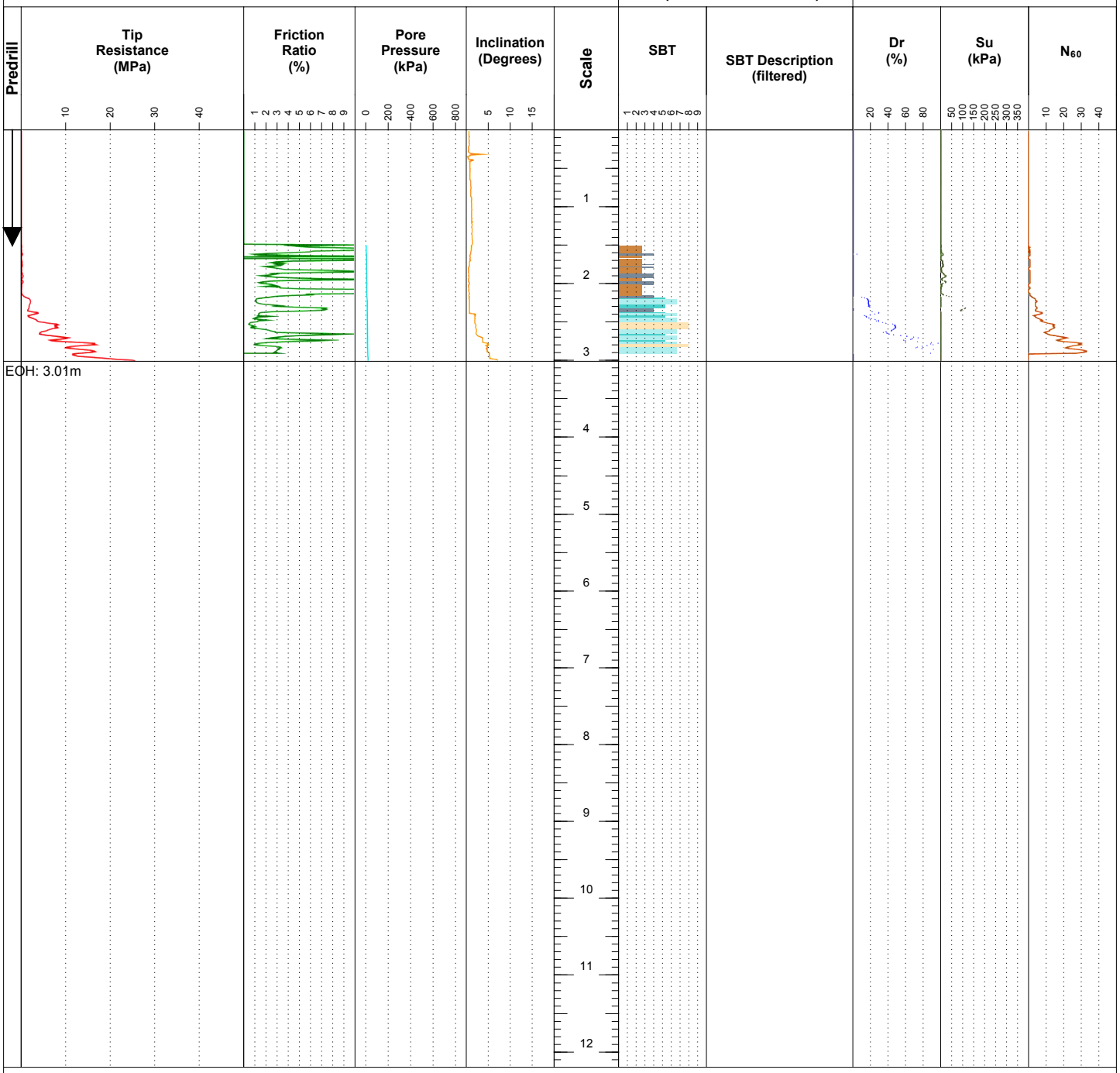
Job:	13839
CPT No.:	CPTu002-R1

Name: QLDC Building, Queenstown
Client: Tonkin and Taylor Ltd
Location: 10 Gorge Road, Queenstown

Grid: NZTM
Datum:
Termination: -

North (m): 5004637.06
East (m): 1258253.85
Elevation (m): -
Hole Depth (m): 3.01

RAW DATA | **SOIL BEHAVIOUR TYPE (NON-NORMALISED)** | **ESTIMATED PARAMETERS**



Operator: P. Buunk
Cone Reference: 081034T
Cone Area Ratio: 0.75
Cone Type: I-CFXYP20-10

Date: 22/08/2014
Predrill: 1.50
Water Level: -
Collapse: 1.10

Tip Resistance (MPa) Initial: -5.6157 **Final:** -5.6318
Local Friction (MPa) Initial: -0.0425 **Final:** -0.0436
Pore Pressure (KPa) Initial: -187.659 **Final:** -186.485

Effective Refusal
Tip:
Gauge:
Inclinometer: ✔
Other:
Target Depth:

Soil Behaviour Type (SBT) - Robertson et al. 1986

0	Undefined	5	Sand mixtures: silty sand to sandy silt
1	Sensitive fine-grained	6	Sands: clean sands to silty sands
2	Clay - organic soil	7	Dense sand to gravelly sand
3	Clays: clay to silty clay	8	Stiff sand to clayey sand
4	Silt mixtures: clayey silt & silty clay	9	Stiff fine-grained

Notes & Limitations

Data shown on this report has been assessed to provide a basic interpretation in terms of Soil Behaviour Type (SBT) and various geotechnical soil and design parameters using methods published in P. K. Robertson and K.L. Cabal (2010), Guide to Cone Penetration Testing for Geotechnical Engineering, 4th Edition. The interpretations are presented only as a guide for geotechnical use, and should be carefully reviewed by the user. Both McMillan Drilling Ltd & Geroc Solutions Ltd do not warranty the correctness or the applicability of any of the geotechnical soil and design parameters shown and does not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used to derive data shown in this report.

Remarks
Effective Refusal

Hole Depth (m): 3.01

CONE PENETRATION TEST

McMILLAN Drilling GROUP

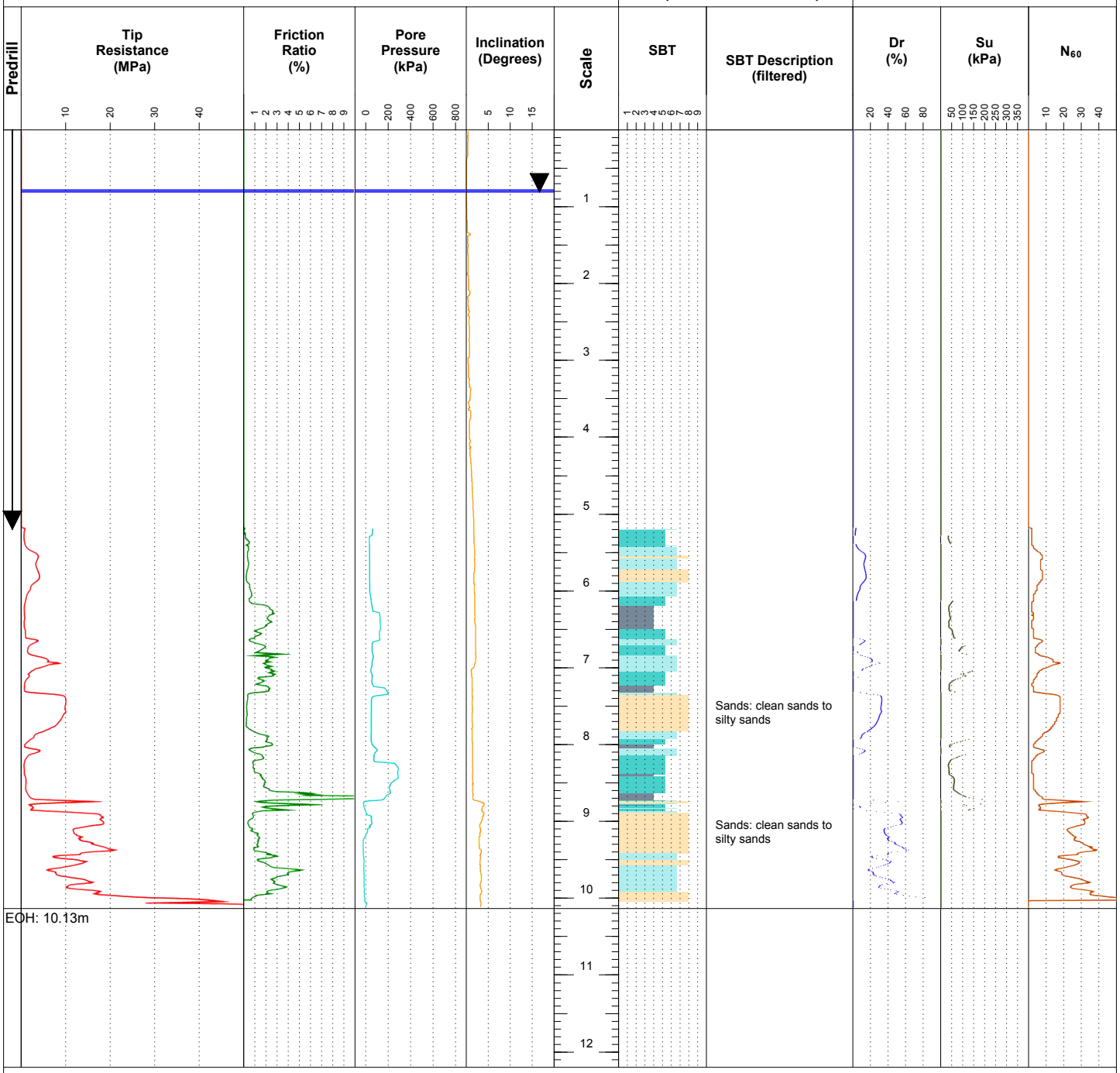
Job:	13839
CPT No.:	CPTu002-R2

Name: QLDC Building, Queenstown
Client: Tonkin and Taylor Ltd
Location: 10 Gorge Road, Queenstown

Grid: NZTM
Datum:
Termination: -

North (m): 5004636.85
East (m): 1258255.26
Elevation (m): -
Hole Depth (m): 10.13

RAW DATA | **SOIL BEHAVIOUR TYPE (NON-NORMALISED)** | **ESTIMATED PARAMETERS**



Operator: P. Buunk
Cone Reference: 081034T
Cone Area Ratio: 0.75
Cone Type: I-CFXYP20-10

Date: 23/08/2014
Predrill: 5.19
Water Level: 0.80
Collapse:

Tip Resistance (MPa) Initial: -5.7569 **Final:** -5.662
Local Friction (MPa) Initial: -0.0425 **Final:** -0.0432
Pore Pressure (KPa) Initial: -184.885 **Final:** -185.359

Effective Refusal
Tip: ✓
Gauge:
Inclinometer:
Other:
Target Depth:

Soil Behaviour Type (SBT) - Robertson et al. 1986

0	Undefined	5	Sand mixtures: silty sand to sandy silt
1	Sensitive fine-grained	6	Sands: clean sands to silty sands
2	Clay - organic soil	7	Dense sand to gravelly sand
3	Clays: clay to silty clay	8	Stiff sand to clayey sand
4	Silt mixtures: clayey silt & silty clay	9	Stiff fine-grained

Notes & Limitations

Data shown on this report has been assessed to provide a basic interpretation in terms of Soil Behaviour Type (SBT) and various geotechnical soil and design parameters using methods published in P. K. Robertson and K.L. Cabal (2010), Guide to Cone Penetration Testing for Geotechnical Engineering, 4th Edition. The interpretations are presented only as a guide for geotechnical use, and should be carefully reviewed by the user. Both McMillan Drilling Ltd & Geroc Solutions Ltd do not warranty the correctness or the applicability of any of the geotechnical soil and design parameters shown and does not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used to derive data shown in this report.

Remarks
Effective Refusal

Hole Depth (m): 10.13

McMILLAN Drilling GROUP

CONE PENETRATION TEST

Job: 13839

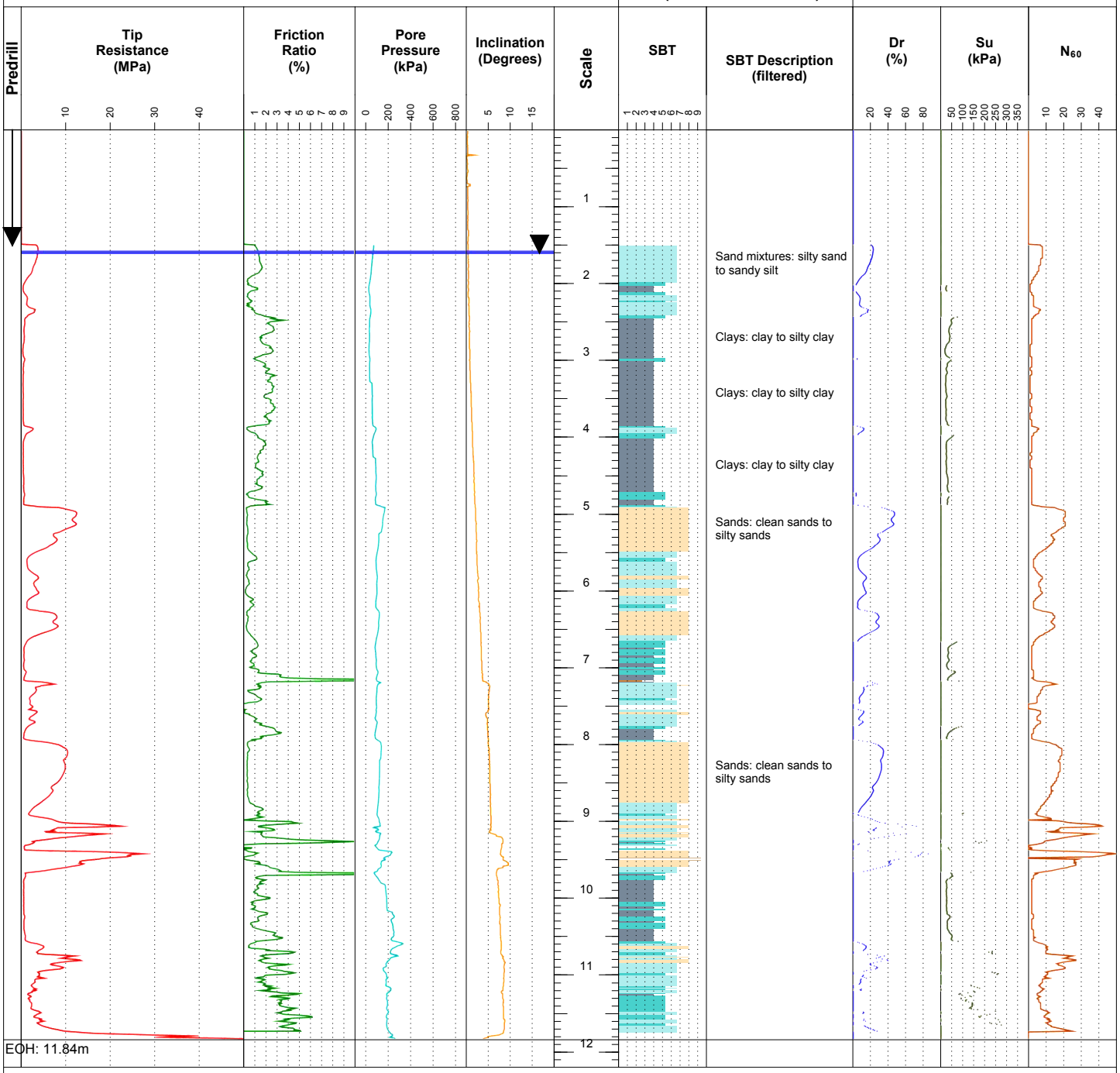
CPT No.: CPTu003

Name: QLDC Building, Queenstown
Client: Tonkin and Taylor Ltd
Location: 10 Gorge Road, Queenstown

Grid: NZTM
Datum:
Termination: -

North (m): 5004615.16
East (m): 1258217.00
Elevation (m): -
Hole Depth (m): 11.84

RAW DATA | **SOIL BEHAVIOUR TYPE (NON-NORMALISED)** | **ESTIMATED PARAMETERS**



Operator: P. Buunk
Cone Reference: 081034T
Cone Area Ratio: 0.75
Cone Type: I-CFXYP20-10

Date: 22/08/2014
Predrill: 1.50
Water Level: 1.60
Collapse: 3.70

Tip Resistance (MPa) Initial: -5.6493 **Final:** -5.737
Local Friction (MPa) Initial: -0.0431 **Final:** -0.0437
Pore Pressure (KPa) Initial: -188.041 **Final:** -181.554

Effective Refusal
Tip: ✓
Gauge:
Inclinometer:
Other:
Target Depth:

Soil Behaviour Type (SBT) - Robertson et al. 1986

0	Undefined	5	Sand mixtures: silty sand to sandy silt
1	Sensitive fine-grained	6	Sands: clean sands to silty sands
2	Clay - organic soil	7	Dense sand to gravelly sand
3	Clays: clay to silty clay	8	Stiff sand to clayey sand
4	Silt mixtures: clayey silt & silty clay	9	Stiff fine-grained

Notes & Limitations

Data shown on this report has been assessed to provide a basic interpretation in terms of Soil Behaviour Type (SBT) and various geotechnical soil and design parameters using methods published in P. K. Robertson and K.L. Cabal (2010), Guide to Cone Penetration Testing for Geotechnical Engineering, 4th Edition. The interpretations are presented only as a guide for geotechnical use, and should be carefully reviewed by the user. Both McMillan Drilling Ltd & Geroc Solutions Ltd do not warranty the correctness or the applicability of any of the geotechnical soil and design parameters shown and does not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used to derive data shown in this report.

Remarks
Effective Refusal

Hole Depth (m): 11.84

TEST DETAIL

PointID: CPTu001

Sounding: 3

Operator: P. Buunk
Cone Reference: 081034T
Cone Area Ratio: 0.75
Cone Type: I-CFXYP20-10

Date: 22/08/2014
Predrill: 1.50
Water Level: -
Collapse: 1.50

Effective Refusal
Tip:
Gauge: ✓
Inclinometer:
Other:

Tip Resistance (MPa) Initial: -5.6467 **Final:** -5.6036
Local Friction (MPa) Initial: -0.0418 **Final:** -0.0434
Pore Pressure (kPa) Initial: -185.281 **Final:** -178.494

Target Depth:

PointID: CPTu002

Sounding: 2

Operator: P. Buunk
Cone Reference: 081034T
Cone Area Ratio: 0.75
Cone Type: I-CFXYP20-10

Date: 22/08/2014
Predrill: 1.50
Water Level: -
Collapse: 0.50

Effective Refusal
Tip:
Gauge:
Inclinometer: ✓
Other:

Tip Resistance (MPa) Initial: -5.615 **Final:** -5.5958
Local Friction (MPa) Initial: -0.0411 **Final:** -0.0437
Pore Pressure (kPa) Initial: -183.315 **Final:** -176.876

Target Depth:

PointID: CPTu002-R1

Sounding: 22

Operator: P. Buunk
Cone Reference: 081034T
Cone Area Ratio: 0.75
Cone Type: I-CFXYP20-10

Date: 22/08/2014
Predrill: 1.50
Water Level: -
Collapse: 1.10

Effective Refusal
Tip:
Gauge:
Inclinometer: ✓
Other:

Tip Resistance (MPa) Initial: -5.6157 **Final:** -5.6318
Local Friction (MPa) Initial: -0.0425 **Final:** -0.0436
Pore Pressure (kPa) Initial: -187.659 **Final:** -186.485

Target Depth:

PointID: CPTu002-R2

Sounding: 222

Operator: P. Buunk
Cone Reference: 081034T
Cone Area Ratio: 0.75
Cone Type: I-CFXYP20-10

Date: 23/08/2014
Predrill: 5.19
Water Level: 0.80
Collapse:

Effective Refusal
Tip: ✓
Gauge:
Inclinometer:
Other:

Tip Resistance (MPa) Initial: -5.7569 **Final:** -5.662
Local Friction (MPa) Initial: -0.0425 **Final:** -0.0432
Pore Pressure (kPa) Initial: -184.885 **Final:** -185.359

Target Depth:

PointID: CPTu003

Sounding: 1

Operator: P. Buunk
Cone Reference: 081034T
Cone Area Ratio: 0.75
Cone Type: I-CFXYP20-10

Date: 22/08/2014
Predrill: 1.50
Water Level: 1.60
Collapse: 3.70

Effective Refusal
Tip: ✓
Gauge:
Inclinometer:
Other:

Tip Resistance (MPa) Initial: -5.6493 **Final:** -5.737
Local Friction (MPa) Initial: -0.0431 **Final:** -0.0437
Pore Pressure (kPa) Initial: -188.041 **Final:** -181.554

Target Depth:

CPT CALIBRATION AND TECHNICAL NOTES

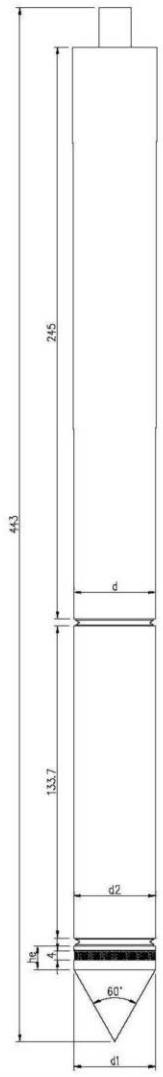
These notes describe the technical specifications and associated calibration references pertaining to the following cone types:

- ELCI-10CFXY measuring cone resistance, sleeve friction and inclination (standard cone);
- ELCI-CFXYP20-10 measuring cone resistance, sleeve friction, inclination and pore pressure (piezocone).

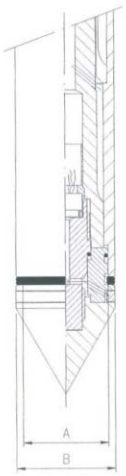
Dimensions

Dimensional specifications for both cone types are detailed below. All tolerances are routinely checked prior to testing and measurements taken are manually recorded on CPT field sheets. All field sheets are kept on file and available on request.

A.P. van den Berg Machinefabriek tel.: +31 (0)513-631355 info@apvandenberg.com	**DEVIATION of Straightness + MINIMUM Dimensions tip, friction jacket, cone adapter**	Standards: EN ISO 22476-1 APB-standard
Type of cone:	Icone 10 cm^2	
ALLOWABLE SIZE VARIATION		
Diameter of tip:	$35,3 \leq d1 \leq 36,0$	
Diameter of centering ring CFP	$35,3 \leq d1 \leq 36,0$	
Diameter of friction jacket:	$d_1 \leq d_2 < d_1 + 0,35$	
Height dimension of tip edge:	$7 \leq h_e \leq 10$	
PRODUCTION DIMENSIONS		
Tip:	$d_1 = 35,7\ ^{+0,2}_{0}$	
Jacket (C-cone):	$d_2 = 35,7\ ^{+0,2}_{0}$	
Friction jacket (CF-cone):	$d_2 = 35,9\ ^{+0,1}_{0}$	
Tip for used cone:	$d_1 = 35,5\ ^{+0,1}_{0}$	
MINIMUM DIMENSIONS		
Minimum diameter jacket (C-cone):	$d_2 = 35,2$ (APB standard)	
Minimum diameter friction jacket (CF-cone):	$d_2 = 35,3$	
Use "used cone"-tip when friction jacket diameter:	$d_2 \leq 35,65$	
Minimum diameter of cone adaptor:	$d = 35,3$	
Maximum deviation of straightness:	1 mm on a length of 1000 mm (max. oscillation 1,0 mm.)	



Cone surface ratio



$A = 0,25 \times 3,14 \times 30,9 \times 30,9 = 750\ MM^2$
$B = 0,25 \times 3,14 \times 35,7 \times 35,7 = 1000\ MM^2$

$\alpha = A/B \qquad \beta = 1 - A/B$

$\alpha = 750/1000 = 0,75$
$\beta = 1 - 0,75 = 0,25$

CPT CALIBRATION AND TECHNICAL NOTES (cont.)

Calibration

Each cone has a unique identification number that is electronically recorded and reported for each CPT test. The identification number enables the operator to compare 'zero-load offsets' to manufacturer calibrated zero-load offsets.

The recommended maximum zero-load offset for each sensor is determined as ± 5% of the nominal measuring range.

In addition to maximum zero-load offsets, McMillan Drilling Services also limits the difference in zero load offset before and after the test as ± 2% of the maximum measuring range. See table below:

	Tip (MPa)	Friction (MPa)	Pore Pressure (MPa)
Maximum Measuring Range:	150	1.50	3.00
Nominal Measuring Range:	75	1.00	2.00
Max. 'zero-load offset':	7.5	0.10	0.20
Max 'before and after test':	3	0.03	0.06

Note: The zero offsets are electronically recorded and reported for each test in the same units as that of each sensor.

TEST CERTIFICATE
Icone (all versions)

Supplier:	A.P. van den Berg Machinefabriek, Heerenveen The Netherlands
Production-order:	63929.004
Client:	Mc Millan
Cone-type:	ELCI 10 CFXYP20
Cone-number:	081034

To test / To check item	Required value	Checked value
Check Quad-ring groove behind friction sleeve with check ring ; Place ring behind friction sleeve without O-rings and Q-rings mounted Place friction sleeve, prepressure ring, centering ring and tighten tip	Sleeve fixed	O.K.
Isolation-resistance	>0.5 GΩ	O.K. GΩ
Straightness total Icone (10 cm2). (Base part Icone: S <= 0,2 mm) **For dimensions 15cm Icones: see standard/table.**	S<= 2,2 mm	O.K. mm
All Measurements: Test with both GOnsite! v2.xx and v3.12.		O.K.
Check alarm-settings Icone. Alarm values are set. (Kill Shutdown)	O.K.	O.K.
Software version? Check at opening screen. **NOTE DOWN version nr.:**	Version:	1.8
Calibration date of Icone; check cone data [F1]..[F1]	O.K.	O.K.
Zero-Value Tip	Good	-5,80 MPa
Zero-Value Local Friction	Good	-0,047 MPa
Zero-Value Pore Pressure	Good	-194 kPa
Zero-Value Inclination X Zero-Value Inclination Y	-2° < X <+2° -2° < Y <+2°	0,0 ° 0,2 °
Measurements Tip resistance OK?	Yes	0-75MPa
Influence of Tip on Local Friction? Tip: **Max Load**; Mantle free? 10cm²: **150 kN.** // 15 cm²: **150 kN.**	150 kN	O.K.
Measurements Local Friction OK?	Yes	0-1MPa
Local Friction: **Max Load**	O.K.	O.K.
Measurements Pore Pressure OK?	Yes	0-200kPa
Measurements Inclination OK?	Yes	-24-0-24°
Cone recognition on disconnecting and connecting Icone again?	Yes	O.K.

Remarks: /

Calibrated by:	C.J. Ouwejan	Date: 28-3-'14	Sign.:
Final check:	J.E. Tenhage	Date: 31-3-'14	Sign.:

Work instructions: H:\Electro\Productie\Icone\beschrijving beproeving Icones.doc.

McMILLAN Drilling GROUP

www.tonkin.co.nz

ENVIRONMENTAL AND ENGINEERING CONSULTANTS