Tonkin & Taylor Christchurch Central City Geological Interpretative Report

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Christchurch City Council

REPORT

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Christchurch Central City Geological Interpretative Report

Tonkin & Taylor

ENVIRONMENTAL AND ENGINEERING CONSULTANTS

REPORT

Christchurch City Council

Christchurch Central City
Geological Interpretative Report

Report prepared for:

CHRISTCHURCH CITY COUNCIL

Report prepared by:

Tonkin & Taylor Ltd

Distribution:

CHRISTCHURCH CITY COUNCIL	**3 copies**
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December 2011 (Version 1.1)

T&T Ref: REP-CCC-INT

Volume 1 of 2

Christchurch Central City
Geological Interpretative Report
Prepared by Tonkin & Taylor Ltd for Christchurch City Council

December 2011(Version 1.1)

Applicability

This report has been prepared by Tonkin & Taylor Ltd on behalf of the Christchurch City Council to assist with future planning and redevelopment of Christchurch Central City following the Canterbury Earthquake Sequence that occurred between 04 September 2010 and 13 June 2011.

The interpretations contained herein may not be used or relied upon in any context or for any purpose other than that for which it was originally intended. It is recognised, however, that this report may assist with the future planning and development within Christchurch Central City.

Tonkin & Taylor Ltd and Christchurch City Council accept no liability to any third party with respect to the contents of this report.

Limitations

All of the interpretations, recommendations and opinions that are contained in this report are made based on observations and records made by Tonkin & Taylor Ltd and other parties at discrete locations. The continuity of the ground conditions between observation points cannot be guaranteed. The investigation techniques adopted occasionally provide incomplete or highly disturbed samples. As such the soil descriptions may be inaccurate or incomplete at some locations.

The nature and continuity of stratigraphy away from the investigation locations are inferred and it must be appreciated that the actual ground conditions could vary from the assumed model. Due care and allowance must be made by the reader to account for this. If subsurface conditions encountered during subsequent investigations vary from those described in this report, additional advice should be sought from suitability qualified engineering geologists or geotechnical engineers.

This report comprises a main body and separate appendices which must be read in their entirety and are intended to be used in conjunction with each other.

The information presented is of a specialist nature and should only be used by suitably qualified and experienced engineering geologists / geotechnical engineers.

Document approved for issue by:

Grant A Lovell
....
Grant A. Lovell
Director, Christchurch Group Manager

Executive Summary

Tonkin & Taylor Ltd (T&T) have been engaged by Christchurch City Council to undertake an extensive ground investigation to evaluate the nature and variability of the geotechnical conditions present within Christchurch Central Business District and the predominantly commercial areas to the south and south-east. This information was used by T&T to develop a database of consistent and high-quality geotechnical information that will be made publicly available to assist with, and expedite, the post-earthquake recovery and rebuilding process.

The information herein has been used to evaluate the extent and severity of the observed land damage that occurred as a result of the major seismic events associated with the Canterbury Earthquake Sequence, and, to assess the potential impact of future large earthquakes. This will assist to inform decisions around land-use planning required for development of the Central City Plan.

The investigation included 48 machine boreholes, 151 cone penetration tests, approximately 45km of geophysical surveys, groundwater level monitoring and laboratory testing of soil samples to identify the nature of the deposits present to depths of up to 30m below ground level.

The investigation confirms the presence of geologically young alluvial deposits that are highly variable both laterally and vertically over short distances. They include soft clays and plastic silts that are sensitive to cyclic softening and loose non-plastic silts, sands and gravels which are susceptible to liquefaction and associated lateral spreading and high groundwater levels. Those deposits identified as susceptible to liquefaction are shown on geological plans and cross sections presented in this report.

The presence of liquefiable deposits has been identified in all areas where significant land damage was observed, and also in many parts of the city where surface manifestation of liquefaction has not been reported. This suggests that liquefaction likely occurred in these areas and should be considered a hazard in future earthquakes.

Preliminary analyses indicate that the extent and severity of liquefaction that occurred following the 22 February 2011 aftershock was not substantially greater than would have been predicted by applying the peak ground accelerations given in NZS 1170.5 (2004). The assessed level of liquefaction to be designed for using the updated hazard factor (Z = 0.30), issued by the Department of Building and Housing (May, 2011), is not significantly greater than the previous requirements for the Ultimate Limit State design case. The mitigation measures designed to address these issues are largely equivalent to designs that would have been adopted for the previous assessed level of liquefaction, when taking into account the inaccuracies inherent in the analytical methods used and inevitable variability of the site characteristics. However, the design of foundation-structure systems will need to take account of the increased risk for the Serviceability Limit State design case.

No areas within the CBD or adjacent commercial areas were identified as having ground conditions that would preclude rebuilding on those sites, although more robust foundation design and/or ground improvement may be required. The risks of lateral spreading adjacent to some sections of the Avon River will require detailed geotechnical assessments, however, the adoption of a minimum 30m set-back required for creation of the Avon River Park will likely preclude the worst affected areas from future development.

The information presented in this report will enable geotechnical specialists to prepare concept designs for foundations / ground improvement options for future development. However, detailed and comprehensive site specific ground investigations and geotechnical assessments, conducted by suitably qualified and experienced geotechnical specialists, will be required on a site specific basis.

Christchurch is not unique in being located on soils susceptible to liquefaction within a seismically active region. There are a number of cities and large urban centres around the world (including Wellington on the North Island), where the level of seismic hazard is similar to or greater than that at Christchurch. Presuming that it is economically feasible to utilise appropriate foundation / ground improvement systems, there are few sites that would be considered unsuitable for development purely on the basis of a liquefaction hazard.

A number of projects have been successfully completed in recent years within Christchurch central city, using a combination of detailed geotechnical investigations and appropriate ground improvement and/or foundation and structure design, to mitigate the identified liquefaction hazard.

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1. Introduction

1.1 Project Background

On 04 September 2010, Christchurch and the wider Canterbury region was rocked by a magnitude 7.1 earthquake, resulting in extensive damage to buildings and infrastructure throughout the region, including severe land damage within the Christchurch eastern suburbs and Kaiapoi. The liquefaction and associated lateral spreading that impacted the eastern suburbs extended along the Avon River into the eastern parts of the central city (within the four avenues) as far west as Manchester Street, although the severity of the land damage west of Fitzgerald Avenue was significantly less than that experienced along the lower reaches of the Avon.

Following the Darfield Earthquake of 04 September 2010, over 7,000 aftershocks have been recorded (known as the Canterbury Earthquake Sequence) associated with the rupture of this previously unknown fault. This has included around 30 events with a magnitude >5, the devastating M6.3 Christchurch Earthquake of 22 February 2011 and the M6.3 aftershock of 13 June 2011.

As well as causing the collapse or partial collapse of a large number of buildings, resulting in 181 fatalities and prolonged closure of much of the central business district (the red zone), the 22 February 2011 aftershock caused further severe land damage in the eastern suburbs and extensive liquefaction and localised lateral spreading throughout the central city and beyond into Merivale, Fendalton, Papanui and the surrounding areas, which was further exacerbated by the 13 June event.

Rebuilding the central city presents significant challenges from a broad spectrum of disciplines. Ensuring the satisfactory performance of land supporting buildings and infrastructure, is but one, albeit very important, consideration.

1.2 Terms of Reference

Tonkin & Taylor Ltd (T&T) have been engaged by Christchurch City Council (CCC) to undertake an extensive ground investigation to reveal the nature and variability of the geotechnical conditions present within Christchurch Central Business District (CBD) and the predominantly commercial areas to the south of Moorhouse Avenue and east of Fitzgerald Avenue, hereafter referred to as the 'central city', and to make this information publicly available to assist with, and expedite, the post-earthquake recovery and rebuilding process.

The extent of the study area is shown on Figure A1 (see Appendix A).

Many areas within the CBD are dominantly residential zones, particularly in the north and north-eastern sections. The investigations completed within the CBD and discussed in this report are concerned primarily with the geotechnical issues that affect the future redevelopment of the business and commercial sectors. Reporting of future development within the residential zones is being completed by the Earthquake Commission. Any decisions on future land use and development control in these areas issued by the Earthquake Commission shall take precedence over the broad advice outlined in this report.

1.3 Purpose

1.3.1 General

The primary purpose for undertaking the extensive ground investigations, as stated above, is to provide a broad overview of nature and variability of the ground conditions within the central city to aid the post-earthquake recovery and rebuilding process. The principal ways in which the publicly available database of high-quality and consistent geotechnical information will achieve this objective are outlined below:

1. By revealing the lateral and vertical distribution of the soil deposits and groundwater levels present will aid understanding of the extent and severity of the observed land damage, to assess the likelihood of liquefaction/cyclic softening having occurred in areas where no land damage was observed and therefore better comprehend which areas of the central city are likely to be subject to land damage from future seismic events.
2. To help identify any areas within the central city where the prevailing ground conditions may limit the feasibility of rebuilding on the land and/or to highlight those locations where significant mitigations measures are likely to be required for different types of development. The investigation data has been used alongside a number of further sources of information has enabled T&T to provide advice to CCC to assist with decisions regarding further land-use options adopted in the draft and Final Central City Plan[1].
3. There currently exists a large body of publicly available data regarding the soil conditions within the central city and beyond (Environment Canterbury well records). This information is however of limited use and applicability for Geotechnical engineering purposes due primarily to a combination of the often limited depths of the investigation holes, the quality of the soil descriptions and/or quantitative or qualitative data regarding the in situ density/stiffness of the materials and definition of the layering present. This value of the information included is however greatly enhanced when set in the context of the investigation data obtained as part of this study.
4. There also exists a significant volume of good quality ground investigation generated from numerous site specific ground investigations for historic developments within the central city, which is not currently publicly accessible due primarily to issues around data ownership and an even greater volume of geotechnical information is being, and will continue to be, generated during the rebuilding process for individual developments and research projects. It is hoped that establishment of a significant database of consistent, high quality geotechnical data will encourage owners of existing and future data will be willing and enthusiastic about adding their own records to the database. In time this would become an invaluable resource to the Christchurch community with substantial economic and hazard reduction benefits.
5. Access to the central city to undertake the scope of the extensive ground investigation works that have been completed, particularly the geophysical surveys (see Section 4), would be largely impractical if large parts of the city were not effectively closed off to traffic and public access, due to disruption and Health and Safety issues. These investigations have therefore been completed in a timely manner to ensure maximum benefit from the current closure of large parts of the central city (particularly the red zone).
6. There are a limited number of contractors with the necessary specialist ground investigation equipment in Christchurch, Canterbury or even the South Island. As a result, a number of contractors have been mobilised to Christchurch to undertake the present investigations, along with even more extensive investigations in the suburbs that are being completed for similar purposes on behalf of the Earthquake Commission (EQC). These operators are also being commissioned to undertake site specific ground investigations for a range of private clients. The high costs associated with mobilising this equipment to Christchurch would be prohibitive for small-scale individual site assessments. There are significant cost savings therefore for undertaking a large number of investigations under a few contracts.

[1] Christchurch City Council. Draft Central City Plan. August 2011.

7. Despite the large number of investigations being completed and the appreciable depths achieved, site specific ground investigations/geotechnical assessments will be required for the detailed design of individual developments. It is envisaged however that the information provided by the city-wide investigations, along with any historical data available through desk study, may be sufficient to permit concept designs to be developed for individual sites in lieu of gaining access to those sites (if in the red zone) and/or investigation contractors available to undertake specific investigations – the demand for which is expected to be very high and likely to exceed capacity for some time into the recovery/rebuilding process. The city-wide investigations should also assist in determining the appropriate scope of the investigations required at specific sites.
8. There are a number of additional health and safety issues to be considered when undertaking investigations within the central city in close proximity to unstable buildings during the ongoing period of enhanced seismic activity. Control of access to the red zone and monitoring of health and safety issues can be more effectively implemented under a single contract.
9. It is also hoped that by undertaking a broad review of the ground conditions across the central city, comparing these with the areas of observed land damage and undertaking liquefaction assessments (using state-of-the-practice methods), a better appreciation of the ability of practitioners to accurately predict the location and severity of land damage resulting from seismic events can be assessed. This may help to identify areas where further study or research projects could be of value and to provide context for those research works that are already underway, particularly by the University of Canterbury (UoC).
10. Part of the current ground investigation works includes for the ongoing monitoring of a large number of standpipes that have been installed within the near-surface deposits in each of the machine boreholes. These will be monitored for a period of at least 12 months and provide a detailed record of the seasonal variability of the groundwater levels across the central city. This is a fundamental requirement for liquefaction analyses and will benefit consideration of issues associated with the design of temporary works and constructability. Monitoring of the seasonal variability of groundwater levels is not an activity that can be routinely completed for individual site assessments due to cost and programme implications and therefore conservative assumptions regarding the maximum groundwater levels are usually assumed. This should provide a basis for justifying a more accurate assessment of the groundwater level variability at specific locations within the central city.
11. By identifying the nature and variability of the deposits across the central city and identifying those materials at depth which represent significant geotechnical issues, such as susceptibility to liquefaction, will help consenting authorities to understand the scope of the ground investigations and level of geotechnical assessments required to accompany building applications for specific sites. The information presented should also assist geotechnical specialists in explaining to landowners / developers the issues that need to be addressed.

1.3.2 Purpose and Layout of the Interpretative Report

The primary purpose for completing the city-wide investigations was to make available, in as short a timeframe as possible, high-quality and consistent factual data concerning the ground and groundwater conditions, which can be used by geotechnical specialists to assist with the initial stages of undertaking detailed site specific assessments. It is envisaged that sufficient information will be available from the recent investigations, along with any historic data available, to allow the principal geotechnical issues that are likely to be present at any specific location within the city to be interpreted.

This preliminary assessment can then be conveyed to the landowner / developer and an appropriate scope of the ground investigations and geotechnical assessment required to address those issues developed. Such preliminary assessments are likely to be limited to a review of the investigation data available for the immediate vicinity of any particular site, with reference to a maximum of perhaps four to six investigation holes and two to four geophysical survey lines.

There is merit, however, in undertaking a broad review of the apparent ground response to seismic shaking across a wide area, to gain a better understanding of the likely response of the ground at any particular location. Such a review is very time-consuming and generally incompatible with the programme and budget constraints for individual developments. A brief summary of the findings of such an assessment, including a comparison of the revealed ground conditions in the areas where significant land damage was observed and the results of liquefaction analyses in these areas, are therefore presented in this report to assist in this respect.

Sections 2 and 3 of this report provide a brief description of the area covered and the principal sources of information used in its preparation. An outline of the scope and factual reporting of the physical ground investigations undertaken as part of this assessment are outlined in Section 4 and Section 5 provides a brief description of the regional geological and geomorphological setting. Section introduces briefly the seismicity of the Canterbury region, with specific reference to the recent Canterbury Earthquake Sequence as recorded at the GNS Science strong motion stations located within the central business district and the likely local ground conditions present at each site.

A brief summary of the nature and variability of the materials encountered across the central city and a generalised geological model is presented in Section 7, with reference to a number of detailed geological plans and cross sections.

Section 8 then presents a summary of the observed land damage (liquefaction and associated lateral spreading) reported within the central city following the three major seismic events of 04 September 2010, 22 February 2011 and 13 June 2011, and compares these with the ground conditions encountered and the results of liquefaction analyses completed. This section also identifies areas where liquefaction is considered likely to have occurred but where little or no land damage was observed. Section 8 also presents a brief review of the levels of liquefaction that may have been predicted across the central city using the pre-September 2010 earthquake design requirements and compares those with the anticipated severity of liquefaction adopting the post-22 February 2011 seismic design requirements issued by the Department of Housing and Building (DBH, May 2011).

Section 9 provides a brief overview of the principal geotechnical issues associated with the ground conditions present and Section 10 provides an indication of the likely scope of ground investigations and geotechnical assessments that will be required to define and mitigate the geotechnical issues present at specific sites and the need for good engineering judgement in support of analyses and peer review for difficult sites.

Section 10 provides an outline of the likely scope of the ground investigations and geotechnical assessments that are likely to be required for sites subject to a high liquefaction hazard.

2. Site Location and Description

2.1 Location

The extent of the area covered by the ground investigations and detailed in this report includes the area bounded by Bealey Avenue to the north, Fitzgerald Avenue to the east, Moorhouse Avenue to the south and Deans Avenue / Harper Avenue to the west. This area of the city is commonly referred to as the 'Four Avenues' and is generally considered to constitute the main Central Business District (CBD). Investigations have not however been completed within Hagley Park, other than around Christchurch Hospital, such that the western limit of the area covered is bounded by Park Terrace, Rolleston Avenue and Harper Avenue.

The extent of the commercial zones covered extends south of Moorhouse Avenue to Harman Street / Disraeli Street at the western end, increasing as far as Brougham Street between Antigua Street / Montreal Street in the west and Wilsons Road to the east, which includes the area surrounding the AMI Stadium. East of Fitzgerald Avenue the commercial area covered extends south from Cashel Street to Moorhouse Avenue and east to Stanmore Road and Nursery Road.

The combined CBD, excluding Hagley Park, and commercial areas to the south and south-east, are herein referred to as the central city.

The approximate boundary of the areas covered is shown on Figure A1 – Site Location Plan, included in Appendix A.

2.2 Description

2.2.1 Area

The area within the 'Four Avenues' covers a distance of approximately 3km from west to east (Deans Avenue to Fitzgerald Avenue) and 2km north to south (Bealey Avenue to Moorhouse Avenue); encompassing an area of around 6.26km^2.

Excluding Hagley Park, the CBD is roughly square in shape, extending 2km from Rolleston Avenue to Fitzgerald Avenue and 2.1km from north to south, covering a total area 4.20km^2.The commercial district to the south and south-east of the CBD covers a total area of 2.32km^2.

2.2.2 Avon River and Topography

The dominant features defining the character of the central city include Hagley Park, which occupies the western side of the 'Four Avenues' and the meandering course of the Avon River, which flows initially from north to south along the edge of Park Terrace, turns west and then east in a large meander through Hagley Park before following a dominantly north-east course across the CBD, flanked by Oxford Terrace to the south and Cambridge Terrace to the north, with a further tight meander immediately west of Fitzgerald Avenue in the north-east part of the CBD.

The CBD and adjacent commercial areas are very level, ranging from a maximum typical elevation of around 8 to 9m above Lyttleton Harbour datum (mean sea level) along the western side of Hagley Park, reducing to 3m adjacent to the Avon River at the north-east corner of the CBD and 4m in the south-east around Ferry Road.

Ground elevations are typically flat or slope very gently towards the Avon River. A number of cross sections (see Section 7) indicate the presence of what may be interpreted to be a former river terrace, varying from between approximately 100 and 200m from the existing channel. The 'terrace' is typically around 0.5 to 1.0m high, although its presence cannot be defined in many areas, which may be the result of urbanisation.

Isolated sections of the river banks are at notably greater elevation on one side when compared to the opposite bank, such as is observed along Oxford Terrace immediately east of Antigua Street, which is at an elevation of around 7mRL, approximately 1.5m higher than Cambridge Terrace on the north side.

The reduced ground elevations in the south-east area of the CBD and commercial areas extend beyond the present study area to Ferrymead. This is thought to be related to a former inlet that extended from the coastline beyond the current estuary.

The observed ground elevations and associated deposits encountered in these areas have had a significant impact on the extent and severity of liquefaction in these areas (see Section 7).

2.2.3 Land Use

For the benefit of readers not familiar with the layout of Christchurch central city, a very broad description of the land-use in different areas of the CBD are shown on Figure A1 and described briefly below. Within these generalised zones are areas and individual properties that fall outside of the categories indicated.

The main business district, where the majority of the multi-storey and high-rise buildings are located, is focused predominantly south of the Avon River around Cathedral Square, extending south to Tuam Street and east to Madras Street, with a few tall buildings located west of the Avon River south of Armagh Street.

The area to the north of the Avon River to Salisbury Street and as far east as Manchester Street, is largely occupied by business and commercial low-rise buildings but with occasional multi-storey developments. Similar land use is present either side of Victoria Street leading north-west to Bealey Avenue and on the eastern side of the Avon River south of Armagh Street.

The majority of the area to the north of Moorhouse Avenue is dominated by one- and two-storey commercial buildings, as are the commercial areas to the south and south-east.

The main residential areas within the ‘Four Avenues’ occupy the north and north-east areas of the CBD either side of the Avon River and along the eastern side of Park Terrace and Rolleston Avenue. The former area is dominated by one- and two-storey dwellings whilst the area opposite Hagley Park includes a number of multi-storey apartment blocks.

3. Sources of Information

3.1 Post-Earthquake Land Damage Mapping and Survey

As principal geotechnical advisors to the Earthquake Commission (EQC) and the Canterbury Earthquake Recovery Authority (CERA), T&T have undertaken extensive mapping and preliminary assessments of land damage within the central city and suburbs following the Darfield Earthquake of 04 September 2010, the Christchurch Earthquake of 22 February 2011 and significant aftershocks, including the Magnitude 6.3 event of 13 June 2011. T&T also have access to and have reviewed, further relevant information obtained from a wide range of sources, including, but not limited to:

- Earthquake Commission (EQC)
- Canterbury Earthquake Recovery Authority (CERA)
- Christchurch City Council (CCC)
- Waimakariri District Council (WDC)
- Environment Canterbury (ECAN)
- University of Canterbury (UOC)
- Institute of Geological & Nuclear Sciences (GNS)
- Land Information New Zealand (LINZ)
- New Zealand Aerial Mapping (NZAM)
- AAM New Zealand (AAM)
- Other local engineering consultancies

This data has been collated and a detailed database established for ease of use and restricted access by a wide variety of stakeholders and is continually updated as further information becomes available.

This information has been used alongside the ground investigation data to aid understanding of the nature and severity of the observed land damage and the risks posed to the central city from future seismic events, as discussed in subsequent sections of this report.

3.2 Published Information

3.2.1 Black Maps

At the time of the initial European settlement of the Canterbury region in the 1850s, a survey of the Christchurch area was undertaken by Captain Joseph Thomas and Thomas Cass (Chief Surveyors). These maps, which are known as the ‘Black Maps’ which refers to the colour code used by the Department of Survey and Land Information for filing purposes, are still available from Archives New Zealand. These indicate the overall pattern of rivers, streams, creeks, vegetation, sand dunes and swamps present at that time.

This information is invaluable for understanding the nature of the near-surface deposits encountered across the central city and for helping to identify some of the geotechnical issues faced in different areas of the city. For instance, the low-lying swamp areas are characterised by soft, compressible ground that are likely to have undergone settlement resulting from artificial drainage and subsequent placement of fill and there were numerous channels and areas of standing water that must also have been infilled.

An extract of the March 1850 map (Sheet 2) covering the CBD area is shown in Figure 3.1.

Figure 3.1: Extract of Black Map dating from March 1850 (Sheet 2).

Further useful information regarding the early history of development within Christchurch, including many descriptions of the 'difficult' ground conditions that had to be overcome, are described in the records of the Christchurch Drainage Board (CDB), including the original drainage scheme report prepared by William Clark in 1878 and a very informative short history of the CDB written by John Wilson (1989).

3.2.2 Geological / Geomorphological Studies and Maps

There have been a number of studies of the geological / geomorphological history of the Christchurch area and wider Canterbury Region and the nature of the deposits present. Most significant of these for the present study are the geological maps and accompanying descriptions for 'Christchurch Urban Area' compiled by Brown & Weeber (1992), produced at a scale of 1:25,000 and for a more regional understanding of the geological setting, 'Geology of the Christchurch Area' by Forsyth et al. (2008). Section 5 and parts of Section 7 of this report draws heavily on the information contained in these documents.

Further informative studies are referenced at the end of this report.

3.2.3 Seismicity

Following the Darfield Earthquake of 04 September 2011 and the subsequent large aftershocks, particularly the Christchurch Earthquake of 22 February 2011, a number of detailed mapping projects, scientific papers and reports have been prepared, particularly by GNS and the UoC.

Following establishment of the Canterbury Earthquakes Royal Commission (CERC) in May 2011, a number of reports have been commissioned concerning the seismicity of the Canterbury Region and ground conditions present within the CBD to assist the inquiry. These reports are published on the CERC website and provide invaluable information as to the nature of the earthquakes that have affected Canterbury since 04 September 2010, the ground response of the deep alluvial materials within the CBD and the hazards posed by future seismicity. The relevant documents are included in the references at the end of this report.

3.2.4 Standards and Guidelines

The design of structures to resist earthquake motions in New Zealand is set out in New Zealand Standard 1170.5 (2004). This includes details of the elastic site spectra, C(T), which are routinely used be geotechnical specialists for input into liquefaction analyses, based on the spectral shape factor (C_h(T)), which depends on the site subsoil class assumed, the hazard factor (Z), return period factor (R) and the near-fault factor (N(T,D)). This information would have been used for the design of new structures prior to the Darfield Earthquake of 04 September 2010. A review of the Z factor was completed by GNS on behalf of the Department of Building & Housing (DBH) and in May 2011, a revised Z factor was issued for use in the Canterbury Region.

The assumed peak ground accelerations (PGAs) determined from NZS 1170.5 (2004) and the revised Z factor have been used, along with the PGAs recorded at the strong motion sites within the CBD for the three major seismic events, have been used to complete liquefaction analyses, the results of which have been compared with the observed liquefaction within the CBD and to assess the future design requirements (see Section 8).

3.3 Historic Ground Investigation Data

3.3.1 Environment Canterbury Well Records

Ever since artesian groundwater was encountered beneath Christchurch in 1858, wells have been sunk, predominantly into the Riccarton Gravel, to tap the abundant supply of potable water. It is estimated that since the early-1860s, more than 10,000 wells have been sunk within the Christchurch Urban Area. Since the 1980s, it has become mandatory for drillers to record details of the wells, including logging of the strata encountered. This information is maintained by ECan and can be accessed via the Council's website.

Within the CBD alone, there are records for no fewer than 450 records and a large number within the surrounding commercial areas and beyond (see Figure A3 – Environment Canterbury Well Locations in Appendix A).

The primary purpose of sinking the wells was to tap into artesian water sources with little or no interest in the materials through which they were drilled. As a result, viewed in isolation, these records are of fairly limited use for geotechnical purposes, for a number of reasons, the most significant of which include:

- The soil descriptions are often Incomplete or inaccurate as the materials encountered were of little or no interest to the drillers and were not logged by suitably qualified and experienced engineering geologists or geotechnical engineers to a standard methodology (it is not uncommon for the logs to offer no description at all or to be simplified to, for example, clay and sand from 0 to 25m)
- the drilling equipment used is not designed to retrieve good quality samples suitable for logging and therefore any descriptions that are included are unlikely to be accurate in many cases
- no in situ testing, such as for the determination of relative density, have been undertaken and the descriptions (where included) are often misleading or incompatible with geotechnical practice (i.e. granular materials are often referred to as soft and clays as dense, and there are instances where recent investigation holes located very close to

former well locations (identified by covers) have described the materials as 'loosely-packed', whereas in fact the equivalent layers were assessed to be dense to very dense on the basis of the penetration testing completed)

- Poor or no information regarding the groundwater levels encountered
- Lack of any laboratory test data
- The actual locations of the wells are often uncertain, in some cases several records are given the same coordinates and yet the records report very different soil descriptions, and ground elevations are in most cases estimated from contour data rather than being accurately surveyed
- Whilst there are a large number of well records, these are sporadic in their distribution.

The information presented on the well records therefore needs to be used with caution and should only be used to supplement high-quality investigation data where this can be reliably justified.

One of the most useful applications of the ECan well data, due to the sharp contrast in soil strata from stiff clays and peats to dense to very dense gravels, is for definition of the depth to the Riccarton Gravels beneath the central city. This information has been used to assist with defining the depth to the Riccarton Gravels, as detailed in Section 7 and Figure B16.

4. Recent Ground Investigations

The most significant component of the work that T&T has undertaken on behalf of CCC has involved the design, procurement, supervision and factual reporting of an extensive ground investigation covering the entire central city.

As detailed in Section 1, the primary focus of the investigations was to compile and make publicly available a comprehensive database of high-quality and consistent geotechnical information that can evolve as further data becomes available. In addition to the large number of exploratory holes and geophysical surveys completed, this information also provides a sound reference for maximising the data presented on the 450+ ECan well records.

This section of the report provides a brief overview of the investigations that have been completed and the works that are ongoing, which will be provided as addenda, and how this information is being reported.

4.1 Fieldwork

The fieldwork component of the investigation was largely completed between June and October 2011. Further fieldwork, including pre-drilling through shallow gravels to allow cone penetration testing of deeper layers, has been completed through October and November 2011. Groundwater level monitoring is planned to continue for 12 months from the initial reading of the final standpipe installed.

4.1.1 Inspection Pits

Due to the high density of buried services located within the central city, it has been necessary to conduct a thorough review of the known service locations and position the intrusive exploratory holes away from these as far as possible. However, to check for unknown buried apparatus, inspection pits have been excavated at each of the machine borehole and piezocone locations. These were typically completed using sucker-trucks (vacuum excavation) to depths of between 1.0 and 1.5m, depending upon the location and proximity of known services. The holes formed for this purpose were then backfilled with loose sand through which subsequent testing could be completed.

The materials encountered in the upper 1 to 1.5m have not been recorded. The nature of these near-surface deposits is of limited significance for the purposes of this study, which is concerned largely with liquefaction hazard, as the groundwater level will typically be at or below this level in most areas, this upper depth is likely to be highly variable and include fill materials at some locations and the foundations for most existing buildings and future structures within the central city will be close to or below this depth.

Site specific investigations are required to investigate the nature of the near-surface materials, particularly where shallow foundations are being considered, to ensure adequate bearing capacity, check for the presence of loose sands which could undergo dry settlement during seismic shaking and to address temporary works design considerations.

4.1.2 Machine Boreholes

A total of 48 machine boreholes, referenced BH-CBD-01 to BH-CBD-48, have been completed across the central city (see Figure A4, Appendix A). These have been advanced using either top-drive rotary or sonic vibration methods to obtain HQ size (96mm outside diameter) cores.

The boreholes were advanced to prove the depth and nature of the upper zones of the Riccarton Gravels, extending to depths ranging from 23m to 31m (BH-CBD-33 and BH-CBD-21, respectively)[2]; covering a total length of over 1.3km. A maximum proven thickness of the Riccarton Gravels was recorded in BH-CBD-11 (10.5m).

Standard Penetration Tests (SPTs) were typically undertaken at 1.5m vertical intervals as the holes were advanced. Upon completion of drilling, standpipes with an approximately 6m long slotted screen were installed between upper and lower bentonite seals to around 8m below ground level and fitted with a flush cover set in concrete. The recovered core was photographed and logged by T&T in accordance with the NZGS Guidelines (2005) and selected samples taken for laboratory testing.

A summary of the boreholes completed, including the eastings and northings, ground elevation and final depth, is provided in Table G1 in Appendix G. Detailed engineering logs and core photographs for each borehole are included in the zone factual reports.

4.1.3 Cone Penetration Testing

Cone Penetration Tests (CPTs) with measurement of pore water pressures (piezocones) have been completed at 151 primary locations across the central city area; referenced CPT-CBD-01 to CPT-CBD-151 (see Figure A5, Appendix A and Table G2 in Appendix G).

Of the 151 test sites, 48 were advanced to depths exceeding 20m, with a maximum of 28.6m achieved at CPT-CBD-148. A further 14 extended to between 15 and 20m and 15 reached depths of between 10 and 15m (total of 77 exceeding 10m depth).

At 12 of the sites, the CPTs had to be abandoned at shallow depths for various reasons and therefore alternative nearby locations (typically within 1m or so) were attempted. These are referenced with the original number with an ‘a’ at the end (i.e. CPT-CBD-029a).

Following completion of the machine boreholes and initial round of CPTs, a broad assessment was made to determine which of the CPTs that had achieved limited depth may be underlain by a significant thickness of materials potentially susceptible to liquefaction that could be reasonably investigated. Pre-drilling using open hole techniques was then completed at 30 locations, the holes formed being backfilled with loose sand through which subsequent testing of the deeper layers could be completed. These are referenced with the original number with a ‘P’ at the end (i.e. CPT-CBD-26P).

A summary of the locations at which pre-drilling was completed, including the depth of the pre-drill and final depth of the piezocone testing, is provided in Table 4.1.

The decision to complete a relatively large number of piezocones) in comparison to the machine boreholes, was based largely on the speed and efficiency of completing each investigation hole, the generally better definition of vertical variability of the stiffness/ density of the interbedded soil layers and the suitability of the data for subsequent use in geotechnical design, particularly liquefaction analyses. To provide a correlation of the ‘interpreted’ soil types predicted by the empirical formula used in the processing of the CPT data, one CPT was positioned close to each of the 48 machine boreholes.

[2] Excluding BH-CDB-33, which was terminated early at a depth of 15.5m?

Table 4.1: Summary of pre-drilled CPTs

Number	Pre-Drill Depth (m)	Final Depth (m)
CPT-CBD-06P	10.00	22.09
CPT-CBD-08P	12.00	18.01
CPT-CBD-14P	10.50	22.43
CPT-CBD-15P	13.50	22.78
CPT-CBD-24P	6.50	22.98
CPT-CBD-26P	6.00	21.05
CPT-CBD-28P	10.0	10.6
CPT-CBD-33P	4.50	TBC
CPT-CBD-34P	9.00	22.61
CPT-CBD-37P	7.50	8.75
CPT-CBD-40P	9.00	20.78
CPT-CBD-41P	9.00	23.5
CPT-CBD-47P	10.5	22.65
CPT-CBD-48P	9.00	23.2
CPT-CBD-49P	9.00	23.17
CPT-CBD-50P	8.50	23.81
CPT-CBD-56P	6.00	7.43
CPT-CBD-57P	10.50	22.43
CPT-CBD-58P	9.00	22.86
CPT-CBD-66P	7.50	22.01
CPT-CBD-75P	9.00	22.01
CPT-CBD-76P	9.00	17.94
CPT-CBD-77P	10.50	26.08
CPT-CBD-78P	12.00	24.12
CPT-CBD-98P	10.5	11.58
CPT-CBD-100P	7.50	22.16
CPT-CBD-101P	12.00	22.64
CPT-CBD-120P	9.00	10.71
CPT-CBD-121P	12.00	23.89
CPT-CBD-150P	10.50	28.38

TBC – data not yet processed

4.1.3.1 Piezocone Calibration Exercise

In modern practice, the most sophisticated liquefaction analyses procedures generally use the data obtained from piezocones, in combination with boreholes, laboratory testing of selected samples and in some cases, geophysical testing to assess the stiffness of the soil. As discussed in Section 8, however, there are a number of uncertainties involved in liquefaction analyses, including the accuracy of the analytical procedures and inherent variability of the ground conditions at any particular site. It is generally assumed however that the ground conditions revealed at any specific location, as recorded by the piezocone, is consistent.

Whilst the quality and calibration of the equipment used for piezocones is tightly controlled by adherence to ASTM D5778 and calibration of the individual cones is required (copies of which are included in the Factual Report appendices), there is some inevitable variability in the results obtained by different testing equipment.

In order to assess the consistency of the data obtained from the different subcontractors, a small-scale calibration exercise was completed at 234 to 240 Armagh Street, at the junction with Madras Street on site where building had been recently demolished (see Figure A7 in Appendix A).

This involved the two rigs completing three test holes within a small site area, with tests from the two machines located within 1m or so of each other.

The results of the calibration exercise and implications for application of this data are discussed in Section 7.1.2.

4.1.4 Geophysical Surveying

Despite the large number of machine boreholes and piezocones completed across the central city, the distances between the intrusive investigation points are still quite large given the anticipated lateral and vertical variability of the ground conditions present. In order to provide some ‘connectivity’ between these isolated investigations, geophysical surveying has been undertaken on a grid pattern along the major roads running east to west and north to south across the central city and along the banks of the Avon River; covering a total distance of approximately 45km.

The overall location of the survey lines in relation to the machine boreholes and piezocones are shown on Figure A6 (Appendix A). Larger scale plans showing the chainage along each survey line, which can be correlated with the velocity versus depth plots provided in the Zone Factual Reports, are shown on Figures A7 to A10. Large scale location maps are also included in each of the Zone Factual Reports, along with the results for each survey line within that area.

Due to access restrictions in and around the ‘Red Zone’, some areas could not be surveyed, as indicated by the gaps between the survey lines. Further surveys are planned to fill in these gaps when suitable access can be arranged. This information will be presented as addenda to this report and the Zone Factual Reports.

4.1.4.1 Multichannel Analysis of Surface Waves

The method adopted for completing the geophysical testing comprised Multichannel Analysis of Surface Waves (MASW). The shear wave velocity tests provide a measure of the small-strain shear modulus (stiffness) of the soil, which is a useful engineering property of the soil that can be used if geotechnical design and liquefaction analyses.

As well as providing a quick and non-intrusive method of revealing the nature of the ground conditions along the survey line, the method is also able to penetrate through shallow gravels that are present over extensive areas of the central city, to investigate the nature of the underlying weaker, potentially-liquefiable deposits.

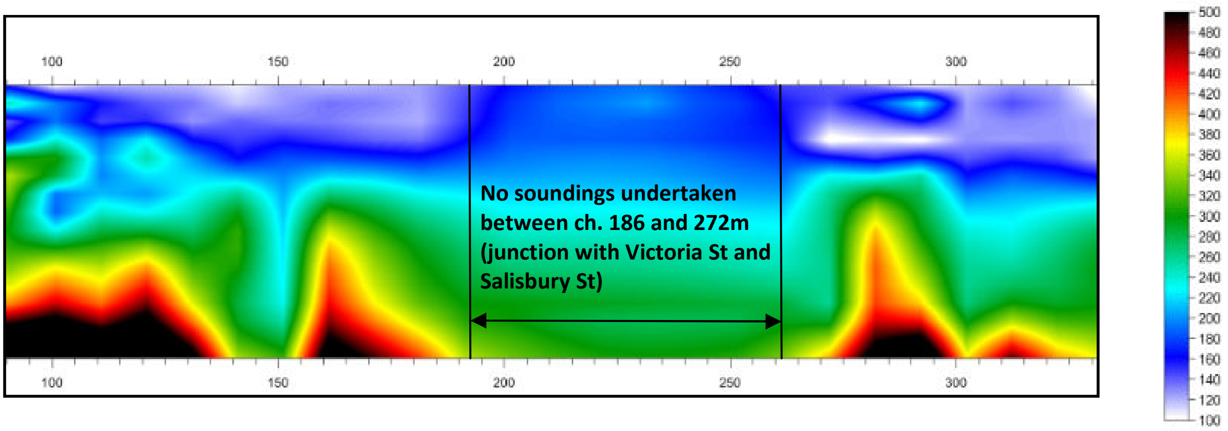
Considerable care is required in interpreting the information and should only be used by specialists familiar with the techniques used. The results provide a measure of the average shear wave velocity with depth and have limited spatial resolution. The information cannot therefore be reliably used to characterise the presence of specific stratigraphical units or the local variability (heterogeneity) of the materials, as discussed further in Section 7.

The plots of shear wave velocity with depth indicate significant variability in stiffness of the ground within the top 30m, which plays a significant role in determining the near-surface ground response. The plots of processed data have generally been terminated at 30m as the quality of the data beyond this level is of variable quality. However, in some areas high quality data is available to depths of up to 42m. Where information to these greater depths is required, the contractor (Southern Geophysical Testing Ltd) should be contacted.

It should be noted that the survey lines are not continuous along the entire length of the road network and typically terminate either side of the major road intersections. The information presented between these survey locations is therefore based on very limited information and should not be relied upon.

An example of where soundings were not conducted across an intersection is shown in Figure 4.1 below. For this survey line soundings were completed at approximate 10m centres along Montreal Street either side of the intersection with Victoria Street and Salisbury Street, but with no soundings completed between chainage 186 and 272m. The loss in detail, particularly in the near surface materials, is apparent. Generally speaking data interpreted between soundings spaced at more than 10 or 15m should not be relied upon. To assist with understanding where these gaps in data occur, the lines shown on Figures A6 to A10 are only continuous where soundings were completed at less than 15m centres.

Figure 4.1: Example of 'gaps' in MASW data (Montreal Street junction with Victoria Street and Salisbury Street)



4.1.5 Surveying

Each of the investigation points, including both the boreholes and piezocones, have been surveyed for location and ground level. These are included on the borehole logs and piezocone summary plots prepared by T&T and tabulated in the Zone Factual Reports.

The location of each shot (typically completed at 10m centres) of the MASW has also been surveyed. The extent of the survey lines shown on Figure 6 and the respective location plans included in the Zone Factual Reports is based on the surveyed results.

4.1.6 Groundwater Monitoring

One of the principal input parameters when undertaking liquefaction analyses is the assumed groundwater level. It is well understood that the water table depth within the central city and eastern suburbs is very high (often within 1 to 2m of the ground surface) and that this varies seasonally.

In order to better understand the depth of the water table and the seasonal variability, standpipes were installed in each of the 48 boreholes and will be monitored on a monthly basis for a period of at least 12 months. It is intended that the results of the ongoing groundwater level monitoring will be issued as addenda to this report at three monthly intervals.

A summary of the groundwater level monitoring results obtained to date is included in Appendix H and discussed in Section 7.

4.2 Laboratory Testing

In order to aid understanding of the nature of the deposits encountered across the central city, including their susceptibility to liquefaction which is dependent upon gradings and plasticity, laboratory tests have been completed on selected samples taken from each of the boreholes. The testing has included particle size distribution, fines content and water content.

A summary of the type and number of tests completed on samples obtained from the 48 boreholes is summarised in Table I1 in Appendix I.

The individual test results are included in the Zone Factual Reports. An overall assessment of the results in relation to the soil deposits present within the central city is included in Section 7.

4.3 Factual Reports

In order to make available the factual investigation data as soon as possible and to present this data in manageable quantities, the central city has been divided into thirteen zones; nine within the CBD and four covering the remaining commercial areas to the south and south-east. The extents of the thirteen zones are shown on Figure A7.

Each of the Zone Factual Reports provides a brief description of the area covered, a summary of the intrusive investigations and geophysical surveys completed, groundwater levels monitored to date, the laboratory test results for the relevant boreholes and a summary of the Environment Canterbury well records relating to that zone. The detailed information is presented in a series of appendices, as follows:

- Appendix A: Investigation Location Plans
- Appendix B: Borehole Logs
- Appendix C: Borehole Core Photographs
- Appendix D: Cone Penetration Testing Results
- Appendix E: MASW Investigation Results
- Appendix F: Standpipe Installation Details
- Appendix G: Laboratory test Results
- Appendix H: ECan Database Well Logs

4.4 University of Canterbury Data

The UoC has undertaken a number of investigations within the CBD. These have been focused on a number of discrete locations or zones, where several intrusive investigations have been completed within a relatively small study area, primarily to reveal the nature of the ground conditions associated with specific sites (see Figure A13).

The investigations have included machine boreholes, in which gel-push sampling has been undertaken to try to retrieve ‘undisturbed’ samples for sophisticated laboratory cyclic triaxial testing, supplemented by a number of piezocones. To date the borehole records and laboratory test results are not available, however T&T has been provided with copies of the piezocone data.

The data from these investigations has been imported into the standard reporting format presented in the factual reports and is included in Appendix L.

It is hoped that when all of the UoC data is available, this can be incorporated into the geotechnical database.

The UoC piezocones were completed by a different contractor to those used for the present investigations. To assist with evaluating the consistency of this data with that obtained by the two contractors used for the present investigations, the calibration test holes were specifically located within one of the UoC sites (Zone 4). UoC piezocones completed close to those undertaken for the calibration exercise are therefore included in the results detailed in Section 7.1.2.

4.5 Geotechnical Database

As detailed in Section 1, the results of the ground investigations are to be made publicly available to assist the preliminary stages of site specific geotechnical assessments. This is to be achieved initially using of the Canterbury Recovery Project Orbit website (www.canterburyrecovery.projectorbit.com).

Each of the Zone Factual Reports and associated appendices can be downloaded, along with the raw data for the piezocones that practitioners may wish to use for geotechnical analyses and design. This report and the associated appendices will also be available for downloading.

It is also intended that the data be made available in a more interactive manner by permitting access to borehole logs, core photographs, piezocone data, geophysical survey plots, geological and liquefaction hazard cross sections and plans and groundwater level monitoring data, by clicking on the relevant icon viewed through a Google Earth platform.

Future management of this database has yet to be determined, but whichever method is adopted, it will be vital for the new data to be included if the database is to achieve maximum benefit to the Christchurch community. It is hoped that the results from site specific ground investigations will be added to the database voluntarily and whoever maintains the database will ensure that the additional data is used to regularly update the geological plans and cross sections and other ‘interpretative’ materials.

4.5.1 Quality

In order to ensure that the high quality and consistency of the database is maintained, it will be important that the information provided is vetted. One of the most significant methods to ensure the quality of the data is maintained is to require all boreholes, trial pits or other investigation methods that rely on the description of materials encountered, are undertaken by suitably qualified engineering geologists, geotechnical engineers or experienced geo-technicians, logged in accordance with the New Zealand Geotechnical Society ‘Guideline for the Field Classification and Description of Soil and Rock for Engineering Purposes’ (NZGS, 2005). The logs should also be reviewed and signed off by an experienced and suitably qualified practitioner.

In the future it is hoped that this information can be provided in a format as recommended in the NZGS ‘Electronic Transfer of Geotechnical and Geoenvironmental Data,’ Version 3.2, NZ v1.0 (NZGS, 2007) or equivalent system.

4.5.2 Location

Associated with the need for high quality and consistent data, is the need to provide detailed coordinates and ground elevations for the various information provided. This should always be included on the individual logs for boreholes, trial pits and piezocones.

5. Regional Setting

In order to provide a context for understanding the composition and distribution of the near-surface deposits that underlie the central city, it is important to first appreciate the broader geological / geomorphological evolution of the Canterbury region. This section provides a brief description of the main geomorphological features and the principal stratigraphic units present.

5.1 Geomorphology

Christchurch is located on the eastern edge of an aggrading gravel outwash plain at the southern end of Pegasus Bay. The central city is underlain by low lying Holocene age coastal margins, abandoned overbank flood channels of the Waimakariri River, bordered to the south by the Port Hills.

The coastal margin of Christchurch, which is situated on the eastern side of the city, is made up of estuaries, lagoons and swamps. This forms part of the eastward advancing (aggrading) coastline which has been formed over approximately the past 6,500 years due to sediment input from Waimakariri River floods and coastal storm/current activity in Pegasus Bay.

The overbank flood channels of the Waimaikariri River, which mainly consist of sands and gravels, are predominately in the western side of the city and slope gently toward the east. These flood channels connect with the present day Heathcote, Avon and Styx rivers. The Avon, Heathcote and Styx rivers meander through the city in a west to east trending direction. The Avon and Heathcote drain into the Estuary. The Styx River drains into the Brook lands Lagoon to the north.

The Port Hills lie to the southeast of the city. These represent part of the northern rim of the extinct Lyttelton volcano and range in age from 5.8 to 12 million years old.

5.2 Geology

The published geology of the Christchurch Urban Area (Brown and Weeber, 1992) indicates Christchurch central city is located on alluvial sand and silt over bank deposits of the Yaldhurst Member of the Springston Formation. These are present overlying fixed and semi-fixed dune and beach deposits of the Christchurch Formation. The Christchurch Formation is in turn underlain by the Riccarton Gravels, which represents the highest of the numerous artesian aquifers present beneath Christchurch.

The Yaldhurst Member sediments represent deposition of fluvial materials, including channel sands and gravels and overbank sand and silts from an ancient braided river system which drained the Christchurch area.

The fixed and semi fixed dune and beach sediments of the Christchurch Formation are inferred to have been deposited by coastal processes. This material is dominated by the sand fraction, but locally includes significant gravel, finer material and shells. Beach sands and gravel deposits are 'semi fixed' due to their proximity to the shoreline and transient depositional process.

The Riccarton Gravels are glacial outwash sediments deposited of Quaternary age. They are present directly below the Christchurch Formation at depths of around 20 to 30m (see Section 7).

6. Seismicity

A number of comprehensive reports have been prepared detailing the historic, recent and potential future seismicity of the Canterbury region. Several reports prepared by GNS Science and the University of Canterbury have been commissioned by the Canterbury Earthquakes Royal Commission (CERC), and are therefore focused on the central city. These are published on the Commission's website (http://canterbury.royalcommission.govt.nz/) and available for downloading.

The reports provide a detailed background to the geological structure of the Canterbury region within the context of New Zealand's active tectonics. The reports provide detailed accounts of the four major earthquakes affecting the CBD (04 September 2010, 26 December 2010, 22 February 2010 and 13 June 2010).

These reports should be referenced for detailed information on the seismicity of the Canterbury region and detailed data regarding the ground motions experienced in the CBD during the Canterbury Earthquake Sequence.

6.1 Canterbury Earthquake Sequence

Figure 6.1 shows the approximate epicentre locations of aftershocks with a magnitude >4.0 recorded following Darfield Earthquake within the main urban areas surrounding the central city, including the Boxing Day 2010 event which is located directly beneath the CBD.

Figure 6.1 Aftershocks with magnitude >4.0 after the Darfield Earthquake

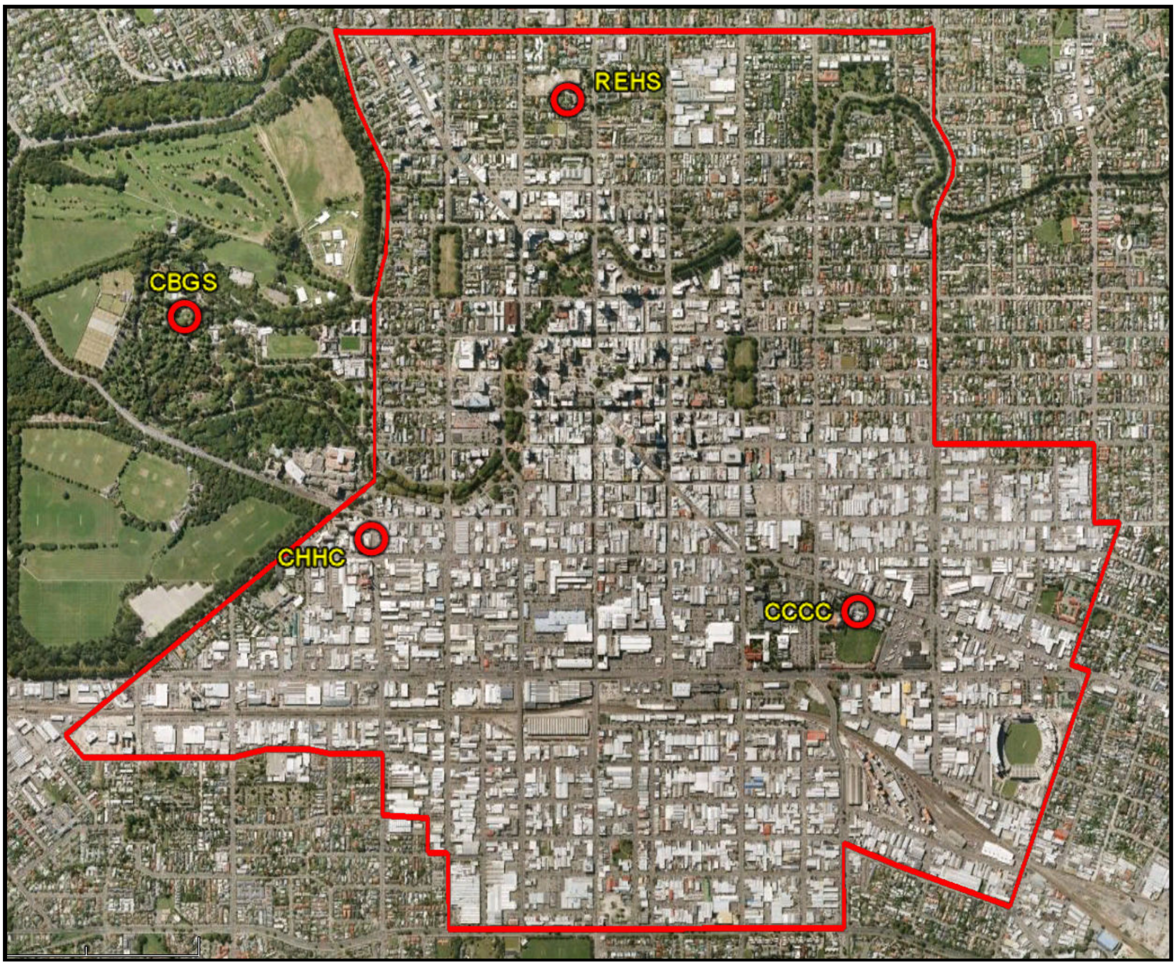
Aerial image sourced from Google (2009). Blue dots indicate aftershocks with magnitude 4.0 to 4.5, green 4.5 to 5.0, yellow 5.0 to 5.5, orange 5.5 to 6.0 and red >6.0 (22 February 2011 and 13 June 2011). Outline of central city is shown.

The following section provides a brief discussion on the nature of the ground conditions thought to be present at each of the strong motion accelerometer stations located within the central city.

6.2 Strong Motion Accelerometers

As a result of the historic seismic activity affecting the Canterbury region and the potential hazard of a large magnitude earthquake resulting from a distant fault (particularly the Alpine Fault and Porters Pass Fault), GNS Science has established and maintains a comprehensive network of strong motion accelerometers in Christchurch. This includes four sites within the central city, as shown in Figure 6.2.

Figure 6.2: Strong motion sites located in Christchurch CBD



Aerial image sourced from Google (2009).

Figure 6.3 below illustrates the acceleration response spectra (5% damped elastic response) for the four strong motion stations located within the central city, along with the design acceleration response spectra taken from NZS 1170.5 (2004) and those based on the updated Zone factor (Z = 0.30) for Christchurch, as specified by DBH (May, 2011).

It is understood that GNS Science is currently undertaking a review of the peak ground accelerations to be used for future seismic design. This is understood to comprise contours of peak ground acceleration for ULS and SLS events and will replace the use of a Z factor.

This information should be incorporated into a revised version of this report and the accompanying analyses and plans / sections.

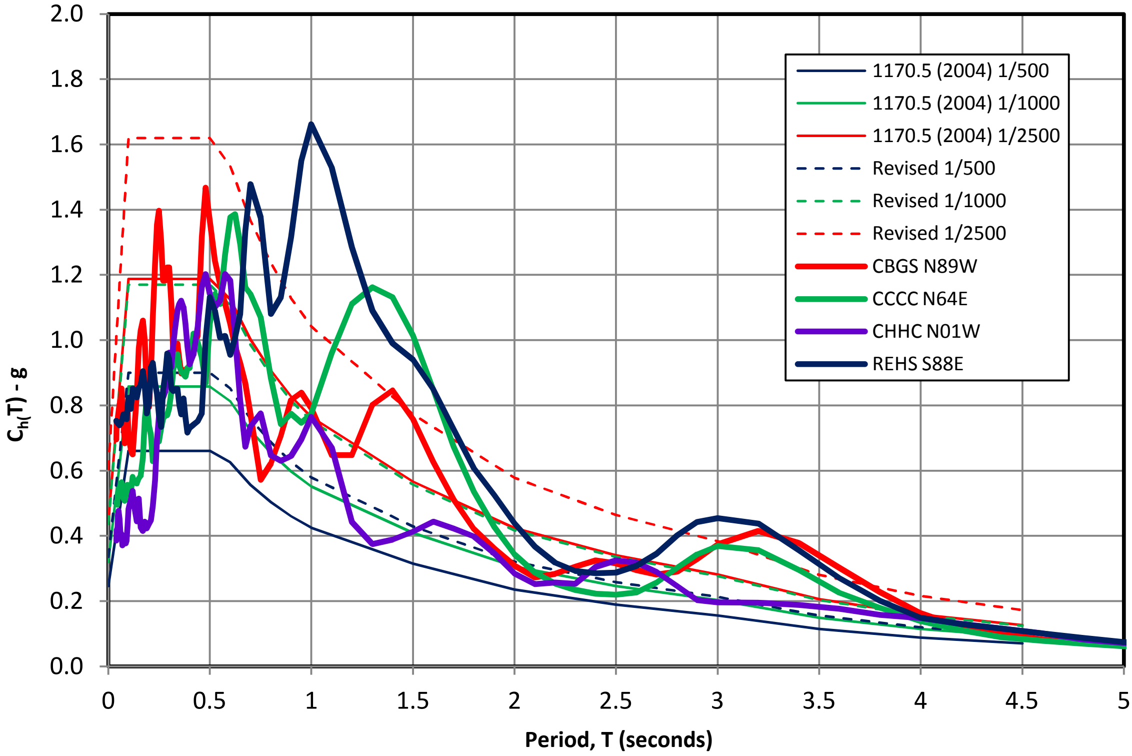
These records provide the basis for the assumed ground motions used for liquefaction analyses, as detailed in Section 8, to back-analyse the predicted level of liquefaction so that these can be compared with observations made following each of the major seismic events between 04 September 20101 and 13 June 2011.

The site response varies quite significantly despite the close proximity of the four stations. This difference in response may be partly due to the different buildings in which the stations are housed (none of the stations are free-field instruments), but the near-surface ground conditions will also have an important role in determining the nature of the motions.

Limited information is currently available regarding the local ground conditions at each of these stations (except REHS - see Section 6.2.1). Whilst no specific investigations have been completed to reveal the ground conditions at each of the stations as part of this study, the investigations that have been conducted provide useful information on the likely nature of the conditions present.

These are described briefly below. In the absence of any subsequent detailed investigations, it is recommended that when considering the ground response at specific locations across the central city, the site conditions are compared with those for the four stations to determine which represents the most relevant data, rather than using the closest station or some interpolation between two or more stations.

Figure 6.3: NZS 1170.5 Elastic Site Response Spectra for Christchurch CBD (Class D Site) and results 22 February 2011 Aftershock

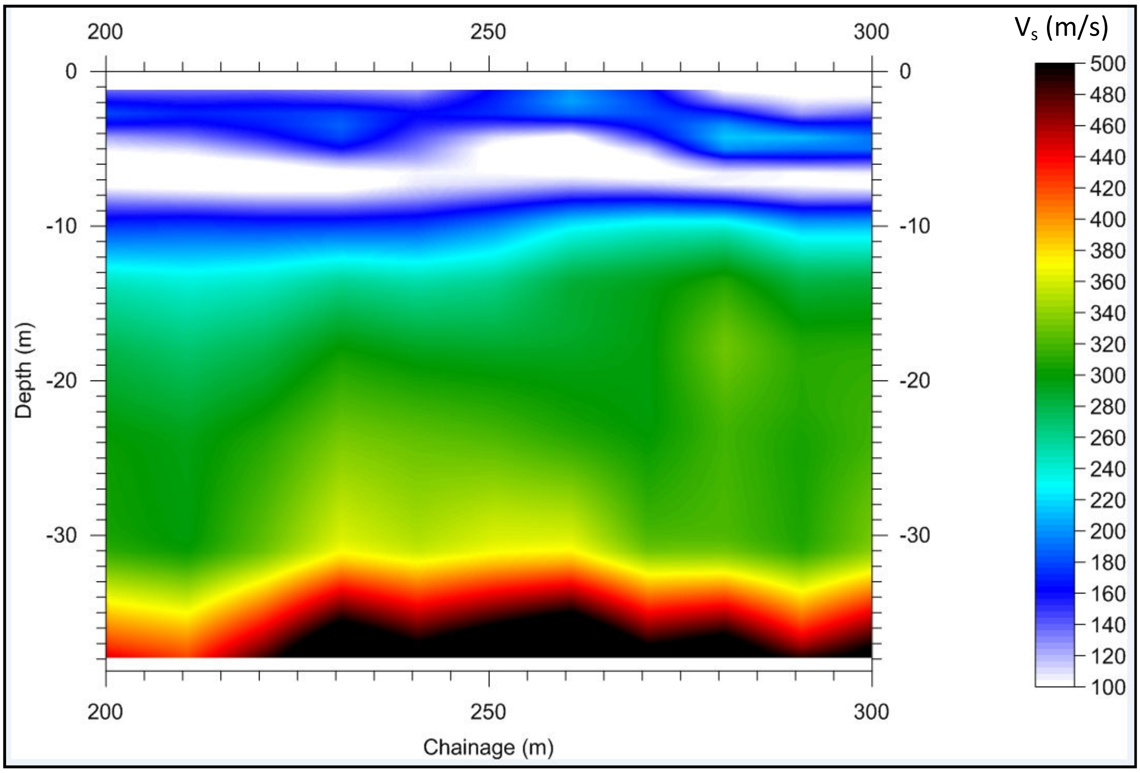


6.2.1 Christchurch Resthaven (REHS)

The closest recent investigations undertaken to the REHS site include CPT-CBD-12 and CPT-CBD-13, located to the west and east, respectively, of the approximate location of the accelerometer. These investigation holes encountered relatively consistent materials, comprising soft to firm (occasionally very soft) plastic silts to depths of around 4.5 to 6.0m, underlain by loose silty sands / sandy silts to between 8 and 9m below ground level. CPT-CBD-13 then indicates the presence of dense to very dense sands to at least 18m below ground level. The upper plastic silts are highly organic with peat layers between 2 and 3m depth (CPT-CBD-12) and from 2 to 4m in CPT-CBD-13.

The MASW data obtained along Colombo Street to the east of the site is shown in Figure 6.4.

Figure 6.4: MASW data for Colombo Street located east of REHS station



Section of MASW data taken along Colombo Street. See Figure A9 for exact location.

This indicates low shear wave velocities to depths of around 8m below ground level (<100m/s), increasing to around 300m/s to depths in excess of 30m.

It is anticipated that a reasonable depth of liquefaction could occur in this area, extending from depths of around 4.5 to 8m (CPT-CBD-12) or 5.5 to 8m in CPT-CBD-13. Relatively little liquefaction was observed at the ground surface in this area, but it is likely that the non-liquefiable crust may have prevented liquefaction flooding / ejecta (see Sections 8 and 9).

As part of the investigations into the collapse of the Pyne Gould Corporation (PGC) building, a ground investigation, comprising a machine borehole and cone penetration test, was undertaken immediately adjacent to the building housing the accelerometer (referenced BH103 and CPT103)[3].

The results of BH103 and CPT103 are largely consistent with the recent investigations, but indicate the highly organic silts and peat layers to be present at depths of between 3 and 5m, overlain by loose silty sands to around 1m below ground level. The top 1m comprises rounded gravels which are likely to represent filled ground.

Below the organic silt / peat layers, further soft silts were encountered to around 7.5m, where they grade into loose becoming medium dense sands, with dense to very sands from 9m to the base of the holes at depths of 15m (BH103) and 20m (CPT103).

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[3] Investigation into the Collapse of the Pyne Gould Corporation Building on 22nd February 2011. Prepared for Department of Building and Housing (DBH). By Beca Carter Hollings & Ferner Ltd. 26 September 2011.

The ground conditions encountered at this site are relatively typical of the area immediately south of Bealey Avenue.

6.2.2 Christchurch Hospital (CHHC)

The CHHC accelerometer is located to the south-east of the main hospital building. Borehole BH-CBD-23 and CPT-CBD-74 are located relatively close to this site. The ground conditions encountered at these two exploratory holes are however somewhat different at depths of between 4 and 8m. Both investigations revealed firm to stiff plastic silts to around 2.5m overlying loose to medium dense sands to around 4m. Below 4m BH-CBD-23 encountered interbedded sandy gravels/gravelly sands and what is interpreted to be relatively loose sands (due to regular core loss). These are present to around 12m below ground level. The piezocone however indicates further loose sands extending to around 8m below ground level underlain by dense gravelly sands to around 12m. Below 12m both investigations revealed dense sands extending to soft to firm silts and peat layers at around 18 to 20m depth with Riccarton Gravels at around 21 to 22m below ground level. There is no nearby MASW data for this site.

A significant thickness of the materials present in the upper 2.5 to 8m are likely to have undergone some degree of liquefaction during the 22 February 2011 aftershock, which is confirmed by observations of liquefaction flooding / ejecta at the ground surface in this area.

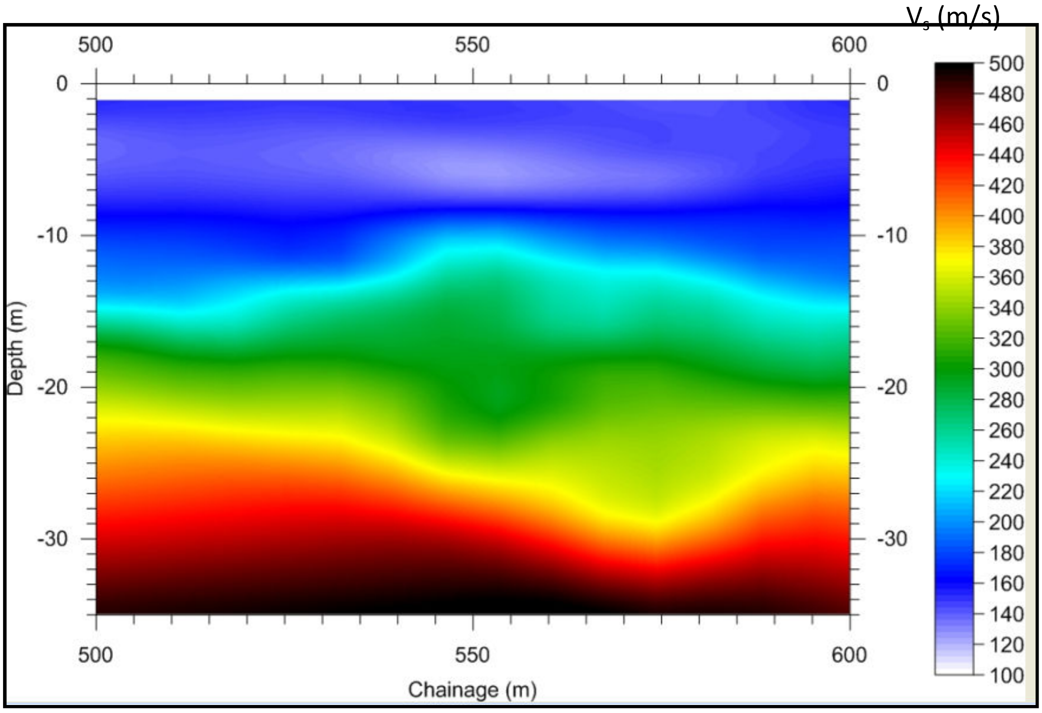
6.2.3 Christchurch Cathedral College (CCCC)

The Christchurch Cathedral College station is located in the topographically low south-east area of the central city. Three piezocones were completed in this area (CPT-CBD-91, -92 and -93), which show relatively consistent ground conditions, as follows:

- 1 to 5m, comprising moderately plastic (largely non-liquefiable) silts, with a loose sand layer between 2 to 3m
- 5 to 9 or 10m, loose to medium dense sands
- 10 to 20m, medium dense to dense sands
- 20 to 25m, soft to firm, becoming stiff, clays and organic silt/peat layers
- >25m, Riccarton Gravels

MASW surveys were completed along both Barbadoes Street and Fitzgerald Avenue close to the site. Figure 6.5 shows a section of survey data taken along Barbadoes Street, which is similar to that recorded along Fitzgerald Avenue. This indicates low shear wave velocities (<150m/s) in the top 10 to 15m, increasing to around 400m/s at the approximate depth of the Riccarton Gravels.

Figure 6.5: MASW data for Barbadoes Street located west of CCCC station



Section of MASW data taken along Colombo Street. See Figure A10 for exact location.

Minor sand boils were observed in the grounds of Christchurch Cathedral College following the 22 February 2011 aftershock, but generally speaking the upper plastic silts appear to have prevented significant liquefaction flooding / ejecta of the loose sand layers within and beneath the silts.

6.2.4 Christchurch Botanical Gardens Station (CBGS)

As shown in Figure 6.1, the CBGS accelerometer is located towards the centre of north Hagley Park. This area was not included within the scope of the central city investigations and we are not aware of any recent detailed investigations having been completed close to this station.

There are however a number of ECan well records in the vicinity of the site, including:

- M35/10619 and M35/1936 located to the east
- M35/7410 (north-west)
- M35/2285 to the south-west

The ground conditions encountered at each of these locations is summarised in Table 6.1.

Excluding M35/2285, each record suggests the site to be underlain by sands to around 2m followed by sandy gravels to depths greater than 10m, with sands to around 20m and a thin cover of clays / silts overlying the Riccarton Gravels at around 20 to 21m.

Only the materials described as loose sand in the upper 2.5m would be considered likely to be highly susceptible to liquefaction (although limited liquefaction within the sandy gravels may be possible). As detailed in Section 7.4, groundwater levels in this area are likely to be below the upper loose sands.

No significant liquefaction was observed in this area following the 04 September 2010, 22 February 2011 or 13 June 2011 events.

It may be reasonably assumed that little or no liquefaction occurred at this site.

Table 6.1: Summary of ground conditions encountered in ECan well records located close to CBGS

Well Record	Dominant Material
M35/10619	Silt/clay to 1.5m Sand and gravel to 11.5m Sands to 16m Soft silty sand to 21m Organic silt/peat to 23.5m Riccarton Gravels below 23.5m
M35/1936	Loose sand to 2.5m Gravel to 14.5m Silty sand to 20m Organic silt/peat to 22m Riccarton Gravels below 22m
M35/7410	Loose sand to 2.5m Gravel to 15m Clay to 21m Riccarton Gravels below 21m
M35/2285	Clay to 4.5m Gravel to 10.5m Silty sand to 23m Riccarton Gravels below 23m

6.2.5 Summary

It is apparent from the ground conditions encountered that some degree of liquefaction is likely to have occurred at all strong motion sites within the central city except CBGS. The ground conditions and degree of liquefaction that is likely to have occurred are relatively similar at the CCCC and CHHC sites, which appear consistent with the observed levels of surface flooding / ejecta observed following the 22 February 2011 event.

Liquefaction is also likely to have occurred at depth at the REHS site, but this has been prevented from being manifest at the ground surface due to the thick cover of plastic silts. The typically soft silts present in the upper 4.5 to 6m at the REHS site, underlain by potentially liquefied loose sands to around 9m, could account for the observed greater motions at this location.

7. Ground and Groundwater Conditions

7.1 Purpose

The purpose of this section is to provide a broad overview of the of the ground and groundwater conditions encountered within the central city. Brief descriptions of the principal deposits and their lateral and vertical distribution are provided and the geological plans and cross sections included in the appendices are introduced.

A brief discussion is included in Section 7.6 regarding the results of the piezocone calibration exercise.

7.2 Geological Plans and Cross Sections

There have been a number of past studies that have attempted to illustrate the nature and variability of the ground and groundwater conditions within Christchurch, most notably those by Elder et al. (1991) and ECan (2004). These have presented maps representing the geology across the entire city based on historic ground investigation data. The geological plans and sections presented here are focused purely on the central city and are based predominantly on the findings of the recent ground investigations, supplemented by ECan well records (where considered reliable), recent investigations completed by T&T, piezocones completed by the UoC and other recently published information (such as that included in the DBH reports).

As detailed in Section 1.3, one of the primary objectives of the present investigations and reporting is to establish a comprehensive publicly-accessible database of geotechnical information that can be added to as new data becomes available and permissions are obtained to include good quality historic investigation results. It is hoped that, as new data becomes available within the central city, together with the suburbs, this will be used to regularly update the ground conditions as presented in the geological plans and cross sections. The plans and sections presented here should therefore be considered a starting point rather than a definitive record.

A series of geological plans are included in Appendix B, illustrating the typical ground conditions that may be anticipated at different depths across the central city based on the deposits encountered in nearby exploratory holes. These are provided at 1m intervals from depths of 1 to 10m, then at 2m intervals to 20m below ground level and finally from 20 to 25m and 25 to 30m depth (see Figures B1 to B16). The adoption of wider intervals with depth reflects both the greater general consistent nature of the deposits at lower elevations and the decreasing number of exploratory holes reaching those depths to justify the level of materials differentiation.

In addition, Figure B17 provides an indication of the variation in depth / elevation to the top of the Riccarton Gravels (discussed in Section 7.3.1.1). No such plan has been prepared that attempts to define the boundary between the Yaldhurst Member and the Christchurch Formation deposits and this division is not shown on the cross sections for the reasons detailed in Section 7.3.1.3.

Geological cross sections have been prepared along each of the major north to south and east to west roads crossing the central city in a rough grid pattern and a section is provided following the approximate course of the Avon River. The location of the 23 cross sections is shown on Figure C1 and the sections are presented as Figures C2 to C24, most of which include two or three sheets due to their length. These generally follow the same rough alignment as the MASW.

The layers included on the sections to depths of around 15 to 20m are based primarily on a mechanical approach to the interpretation of the CPT data using modified published methods. Due to their overall greater depth, the deeper materials are based largely on the borehole records. A legend indicating the colouring of the different materials adopted is provided on Figure C25.

The plans and sections have been developed separately and are able to be used for different purposes. The geological plans are based on the results of the recent investigations (boreholes, piezocones and MASW) supplemented by the ECan well records, UoC piezocones, additional T&T data and other recently published information.

The plans are intended to aid understanding of the geological evolution of the central city and therefore provide an indication of the range of materials that can be reasonably expected to occur at any particular depth at specific locations. The regions are not intended to represent sharp boundaries between material types, but illustrate the 'dominant' deposits encountered at these locations from existing investigations. As further information becomes available, it is expected that these areas will be modified and better defined.

Following completion of these plans we have attempted to 'test' the accuracy and benefit of the information for planning site-specific investigations and comparing the results with the actual ground conditions encountered. The information has proved useful and generally consistent with the materials anticipated, although some notable departures have been identified which serve to highlight the benefit of updating this information as new data becomes available and the need for detailed site-specific assessments.

Definition of material types represents a combination of geological descriptions (in terms of the depositional environment) and engineering properties (particularly density). Generally speaking the loose to medium dense silty sands / sandy silts and interbedded sands and silts, are considered to have a low resistance to liquefaction, whereas materials described as silts/clays are typically of moderate plasticity (based on description, not testing) and therefore are more 'clay-like' and less susceptible to liquefaction.

Where deposits include a reasonable proportion of organics, they have generally been included in the Organic Silts/Peats zones, where low bearing capacity and consolidation settlements under static loading and cyclic softening during earthquakes are likely to be the dominant engineering concerns.

Due to their linear form and reliance on detailed deep investigation holes, the geological cross sections are based almost exclusively on the recent ground investigations with little reference to the ECan boreholes or additional data sources located away from the main north to south and east to west highways. This results in quite long linear interpolations of the data in many places. It is not intended to suggest that the strata will actually be this consistent between the exploratory holes. It is envisaged that future investigations located close to the cross section alignments will allow these to be updated and refined.

Due to the scale of the sections it has not been possible to include on the figures the actual in situ test data, such as the SPT results, groundwater strikes/levels or laboratory test results. It is hoped, however, that this information can be included on electronic versions of the sections as layers that can be turned on/off when viewed at different scales.

7.3 Ground Conditions

One of the main purposes of this study was to provide data that will help to better understand the broad geological environment in which the materials present beneath the central city have been deposited and to develop from this a conceptual three dimensional geological model. This depositional evolution is better understood from starting with the oldest (deepest) materials continuing with decreasing age. The following sections therefore describe the Riccarton Gravels encountered followed by the predominantly marine and estuarine deposits of the Christchurch Formation and finally the more recent predominantly fluvial deposits of the Yaldhurst Member.

7.3.1 Materials Encountered and General Distribution

7.3.1.1 Riccarton Gravels

The depth to the top of the Riccarton Gravels is of interest for geological, seismic and engineering aspects.

The overall depth/elevation of the Riccarton Gravels within and beyond[4] the central city provides an indication of the geological environment present towards the end of the last glacial maximum before subsequent sea level rise and deposition of the overlying Christchurch Formation materials commenced.

The ground response to seismic shaking is affected significantly by the nature of near-surface materials, particularly the top 30m or so. The Riccarton Gravels forms a relatively convenient boundary for defining the ground response of the materials above and below this level.

From an engineering standpoint, many larger structures, particularly those located in areas likely to be subject to deep liquefaction, may consider piling to the Riccarton Gravels as a foundation option.

7.3.1.1.1 Depth/Elevation

The depth to the upper surface of the Riccarton Gravels was proven in 47 of the 48 recent machine boreholes (excluding BH-CBD-33) and may be interpreted from around 50 of the 151 piezocone tests (including the pre-drilled holes). These indicate a depth range across the central city varying from 18.6 to 29.0m below ground level (average 23.3m), with equivalent elevations ranging from -11.8 to -24.6mRL (average -17.6mRL).

The depth to the Riccarton Gravels has also been inferred from over 160 of the ECan well records located within the CBD (where this information is considered reliable). These suggest a similar depth range, varying from 19.2 to 30.0m below ground level (average 24.0m), with elevations ranging from -10.9 to -25.2mRL (average (18.0mRL).

The depths/elevations of the Riccarton Gravels at these locations have been plotted onto a plan of the central city. An overall pattern is clear, indicating an increase in depth / decreasing elevation from west to east as would be expected. However, there is a great deal of local variability, such that it is not meaningful to provide a plot of depth or elevation contours across the central city. Because the overall surface gradient of the Riccarton Gravels is very gentle (around 1°), any data points that intercept local braided channels, which may be several metres deep, would make a contour plan look very complicated and unhelpful as it would not accurately depict all local variations present.

In a number of locations, adjacent data points 100 to 200m apart indicate depth variations of 3 or 4m and given the braided nature of the ancient river bed, it is expected that vertical differences of at least 2 to 3m could occur over much shorter distances and feasibly within a single development site.

The data does however indicate a general deepening of the Riccarton Gravels in a zone spreading east and south-east from around Cathedral Square, where the depth typically increases from

[4] Data being obtained from the suburb wide ground investigations being completed on behalf of the EQC can be combined with the central city information to provide a broader picture of the depth/elevation of the Riccarton Gravels across the main Christchurch urban area.

around 25m to 30m below ground level around Nursery Road / Wilsons Road. This is shown by the very approximate depth / elevation contours provided on Figure B16 and the Geology Plan (Figure B16). These indicate the dominant materials present between 25 and 30m depth, where the Riccarton Gravels are largely absent.

Inspection of the MASW data confirms both the local variability of the approximate depth to the Riccarton Gravels across the central city and the increasing depth from west to east as well as within the area to the east and south-east of Cathedral Square.

The MASW data located towards the western side of the CBD, particularly along Rolleston Avenue and Montreal Street indicates a number of possible over-deepened channels, where the surface of the Riccarton Gravels is encountered at depths in excess of 25m where a depth of around 20m is more typical. These may represent the braided channels of the river that was responding to a new base level during a period of rapid lowering of sea level during the last glacial maximum prior to the marine transgression that resulted in deposition of the Christchurch Formation marine deposits.

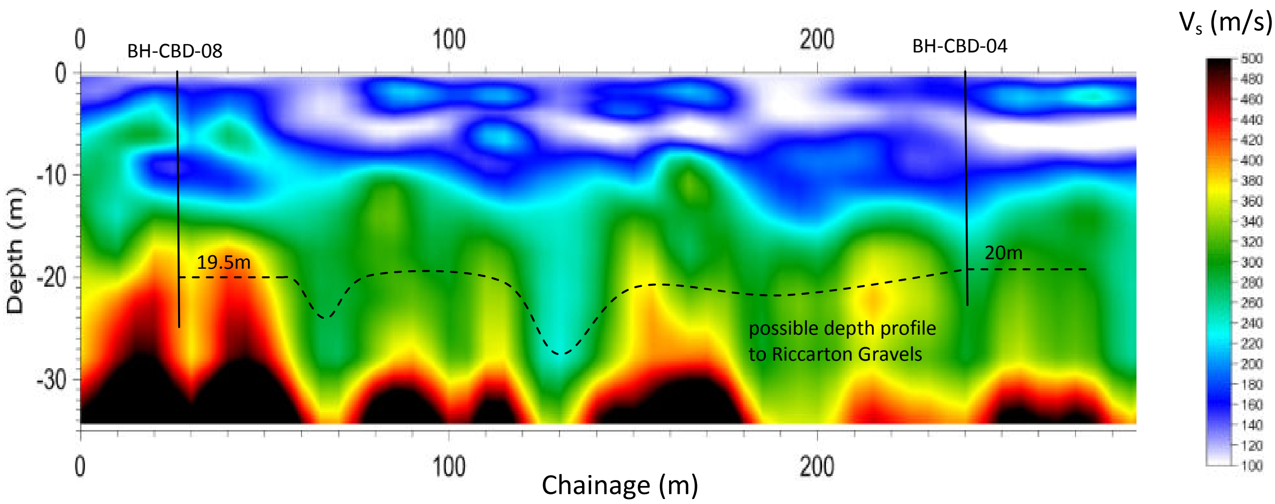
The deepest recorded depth of the Riccarton Gravels (>30m) occurred at ECan well record M35_2149, located near the junction of Manchester Street and Salisbury Street. This does not fit with the general pattern described above and may also represent a localised buried channel, although we cannot confirm the accuracy of this well record.

It should be noted that there are relatively few data points (either from recent investigations or reliable ECan records) covering the north-east part of the central city. Further information in this area would be beneficial to confirm or otherwise typical depths north of the Avon loop.

The Riccarton Gravels were proven to a maximum thickness of 10.5m in BH-CBD-11 during the recent investigations.

As detailed in Section 4.1.4, the MASW data provides an indication of the shear wave velocity of the materials present and does not indicate different lithologies. Whilst it is apparent that the shear wave velocities associated with the Riccarton Gravels are generally high, these cannot be reliably used to indicate the depth of the upper surface of the Riccarton Gravels. Figure 7.1 below provides a simple illustration of this where the MASW plot obtained for Park Terrace is correlated with the results of two machine boreholes, where the depth to the Riccarton Gravels is known with a high degree of accuracy.

Figure 7.1: Depth to Riccarton Gravels indicated by machine boreholes and MASW



The Riccarton Gravels were encountered in BH0CBD-04 at a depth of around 20m which corresponds with a shear wave velocity of around 240m/s, whereas the surface of the Riccarton Gravels encountered in BH-CBD-08 at a very similar depth (19.5m), is associated with Vs of around 400m/s. This is due to the 'averaging' of the MASW data.

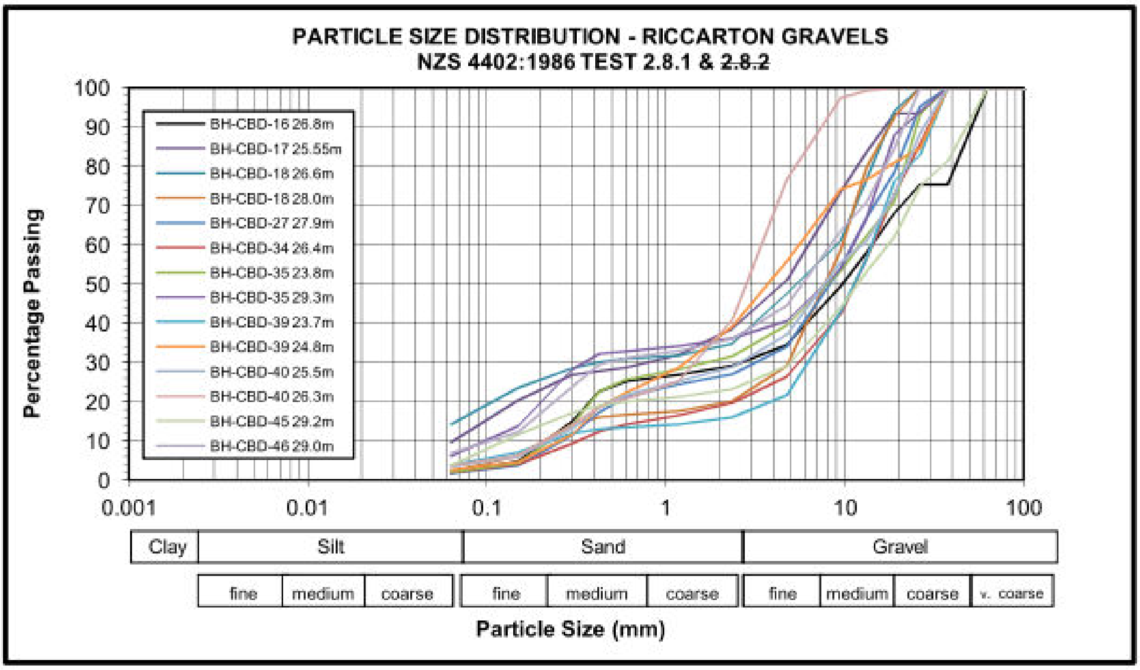
7.3.1.1.2 Density

Upon encountering the Riccarton Gravels, typical SPT N blow counts increased to 30 and above, and often 50 blow counts were recorded for limited penetration. The Riccarton Gravels may therefore generally be considered to be dense to very dense, although the presence of very coarse gravels and cobbles may result in an over-estimate of the actual density of the materials when tested using the SPT.

7.3.1.1.3 Particle Size Distribution

A total of 14 particle size distribution tests (PSDs) have been completed on samples obtained from ten of the machine boreholes spread across the central city. These results are summarised in Figure 7.2, which indicate them to comprise typically sandy to sandy fine- to coarse-grained gravels. They are relatively well-graded with a uniformity coefficient (C) of around 50 to 100, but with somewhat of a gap grading in the coarse sand to fine gravel sizes. This is of limited significance for the present purposes of this report where the sandy gravels of the Yaldhurst Member are typically separated from the underlying Riccarton Gravels by the Christchurch Formation deposits, but may help to define this boundary further to the west beyond the extent of the Christchurch Formation deposits where the Yaldhurst Member grades directly into the Riccarton Gravels.

Figure 7.2: Summary of grading analyses for Christchurch Formation deposits



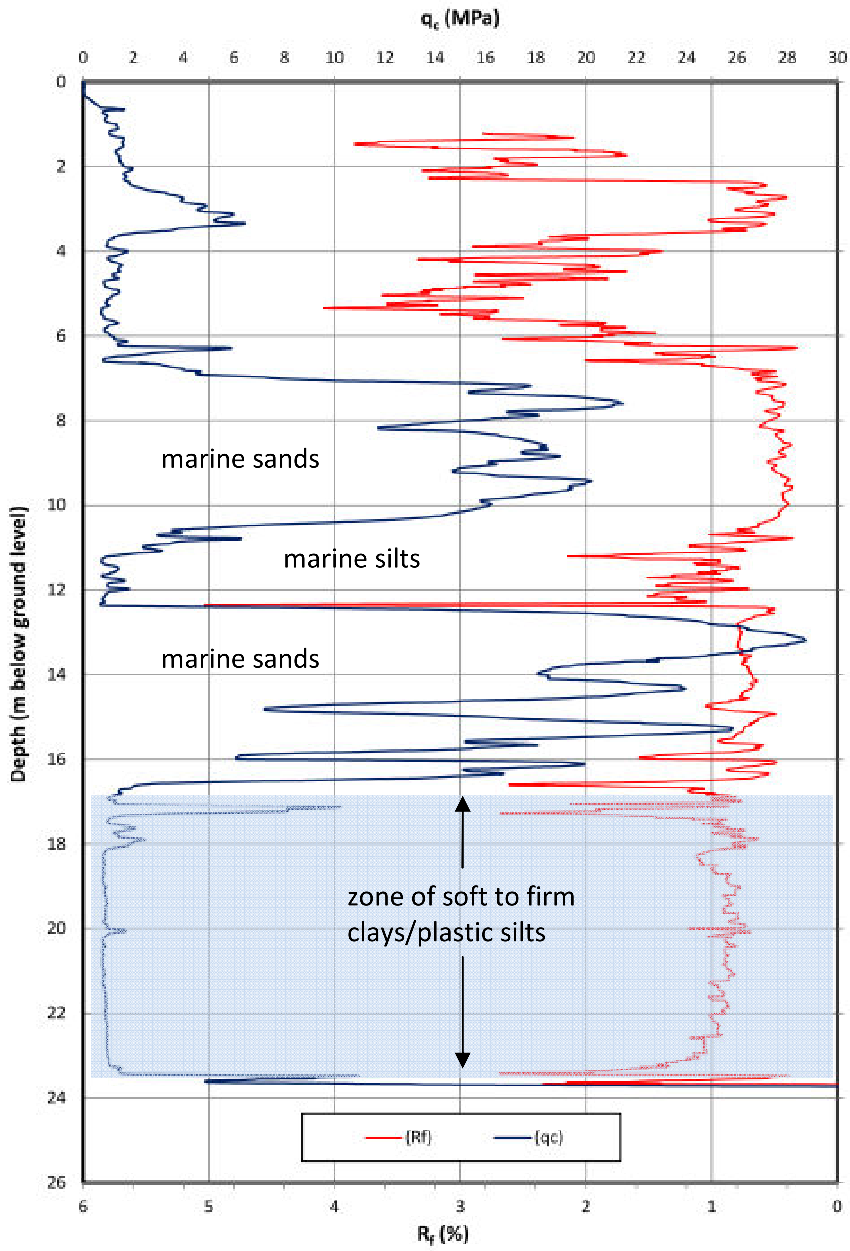
7.3.1.2 Christchurch Formation

The Christchurch Formation encompasses all materials deposited immediately during the post-glacial marine transgression, which is thought to have advanced as far as the central city around seven to eight thousand years ago. It includes beach, estuarine, lagoonal, dune and coastal swamp deposits of gravel, sand, silt, clay, shell and peat (Brown & Weeber, 1992).

7.3.1.2.1 Plastic Clays / Silts (including organic silts and peats)

As can be seen from Figures B16 to B13, the oldest and deepest Christchurch Formation deposits, directly overlying the Riccarton Gravels, comprise clays and plastic silts, and often include layers of very organic silts and peats, with loose to medium dense non-plastic silts and fine sand layers at the higher elevations (see typical piezocone plot in Figure 7.2). These are often described in the boreholes as being soft to firm (occasionally very soft), and the tip resistance recorded in the piezocones that intercept these deep layers are typically around 1MPa; equivalent to undrained shear strengths predominantly in the soft to firm range.

Figure 7.2: Piezocone CPT-CBD-113 data



As shown in Figure B15, these deposits are present to a reasonable thickness (>2.5m) overlying the Riccarton Gravels over the entire south-east portion of the central city – the majority of the area south of the Avon River. Similar deposits are present overlying the Riccarton Gravels at most locations further north and west also, but become thinner north from the river.

A distinct peat layer is often present overlying the Riccarton Gravels. These would likely have formed under the swampy conditions that occurred following commencement of the post-glacial marine transgression, but before the coastline reached the central city. The rise in sea level would have resulted in a waterlogged environment with a relatively rich nutrient supply, conditions suitable for the growth of peat.

As sea levels continued to rise, organic silts and clays eventually interbedded silts and fine sands were deposited over the peats. These would have accumulated in a very low-energy estuarine or lagoonal environment prior to inundation from the sea. The peats and overlying non-marine facies are therefore likely to have been deposited around 10,000 to 7,000 years ago. Radiocarbon dating of the organic materials present within these layers would be beneficial to confirm this timing.

7.3.1.2.2 Marine-Dominated Sands

As indicated on Figure B14 ad B13, at depths of 16 to 20m below ground level, the soft to firm clays and silts are progressively replaced by medium dense to dense sands and these become the dominant materials present across the entire central city to depths of around 9 to 10m below ground level (as depicted on Figures B12 to B9).

These represent the beach and dune deposits of the post-glacial marine transgression that is believed to have reached the central city around 7,000 years ago. They typically comprise fine to medium sands, with occasional silty sand and silt layers (or closely interbedded sands and silts). These finer grained deposits are taken to be representative of periods of high sediment input to the shoreline during periods of flood or potentially short-term deeper water conditions. These layers can be seen in piezocone CPT-CBD-113 in Figure 7.2 extending from around 7 to 18m below ground level.

The marine-dominated sands include occasional shells and rare shell beds. Shells were reported in 32 of the 48 boreholes at depths ranging from around 8m (BH-CBD-12) to 22.5m (BH-CBD-03) below ground level, with equivalent elevations of -3 to -16.9mRL, as summarised in Table 7.1. The absence of reporting of shells in the remaining boreholes should not be taken as indicating that shells are not present at these locations, it may simply be that these were lost in sections of poor-recovery or simply not identified during the logging.

The presence of shells confirms the marine transgression extended across the entire central city. These shells offer a potential to further constrain the age of the Christchurch Formation deposits present beneath the central city through radiocarbon dating and other methods.

Table 7.1: Summary of depth / elevation of maximum recorded elevation of shell bands present beneath the central city

Borehole	Ground Level (mRL)	Depth (m bgl)	Elevation (mRL)	Borehole	Ground Level (mRL)	Depth (m bgl)	Elevation (mRL)
BH-CBD-01	7.65	-	-	BH-CBD-25	5.54	16.8	-11.3
BH-CBD-02	5.91	-	-	BH-CBD-26	4.62	15.5	-10.9
BH-CBD-03	5.62	22.5	-16.9	BH-CBD-27	3.65	15.0	-11.3
BH-CBD-04	6.45	16.6	-10.1	BH-CBD-28	8.51	18.2	-9.7
BH-CBD-05	6.53	21.8	-15.3	BH-CBD-29	6.79	18.1	-11.3
BH-CBD-06	2.94	-	-	BH-CBD-30	6.34	20.5	-14.1
BH-CBD-07	4.21	10.5	-6.3	BH-CBD-31	6.17	17.1	-10.9
BH-CBD-08	6.59	-	-	BH-CBD-32	4.44	12.1	-7.6
BH-CBD-09	6.61	-	-	BH-CBD-33	10.44	-	-
BH-CBD-10	5.28	-	-	BH-CBD-34	9.59	-	-
BH-CBD-11	3.49	14.5	-11.0	BH-CBD-35	8.67	-	-
BH-CBD-12	4.96	8.0	-3.0	BH-CBD-36	7.68	-	-
BH-CBD-13	6.87	19.5	-12.6	BH-CBD-37	4.74	15.0	-10.3
BH-CBD-14	6.60	16.8	-10.2	BH-CBD-38	8.21	-	-
BH-CBD-15	5.16	16.0	-10.8	BH-CBD-39	6.15	15.0	-8.9
BH-CBD-16	5.09	18.8	-13.7	BH-CBD-40	4.76	17.0	-12.2
BH-CBD-17	5.93	-	-	BH-CBD-41	6.66	-	-
BH-CBD-18	5.95	13.1	-7.1	BH-CBD-42	5.58	-	-
BH-CBD-19	5.60	11.6	-6.0	BH-CBD-43	7.21	12.5	-5.2
BH-CBD-20	5.27	11.6	-6.3	BH-CBD-44	3.12	17.0	-13.9
BH-CBD-21	6.36	13.0	-6.6	BH-CBD-45	4.51	17.0	-12.5
BH-CBD-22	5.24	15.9	-10.6	BH-CBD-46	4.39	17.5	-13.1
BH-CBD-23	6.71	17.6	-10.8	BH-CBD-47	7.21	-	-
BH-CBD-24	6.60	-	-	BH-CBD-48	5.21	12.5	-7.3

7.3.1.2.3 Density

In situ testing (SPTs and piezocone tip resistances), generally indicate an increase in density with depth, as would be expected, but the presence of silty layers interrupts this overall pattern. Generally speaking the materials encountered at the base (the oldest deposits) are dense to very dense, becoming medium dense to dense at mid-level and are typically medium dense in the higher elevations. It is likely that some of the layers which appear to be loose to medium dense at depth in the piezocone traces and SPT N results are actually very silty sands or sandy silts. Shear wave velocities associated with these materials at depths of around 10 to 20m are typically in the 200 to 400m/s range.

It should be noted however that some areas are dominated throughout the depth range represented by marine sands as being loose to medium dense. This is true of almost the entire depth of the materials encountered in the south-east portion of the central city around Wilsons Road and the AMI Stadium, where some of the most severe surface manifestation of liquefaction was observed. Shear wave velocities associated with these looser deposits at depths from around 10 to 20m are typically in the 180 to 300m/s range.

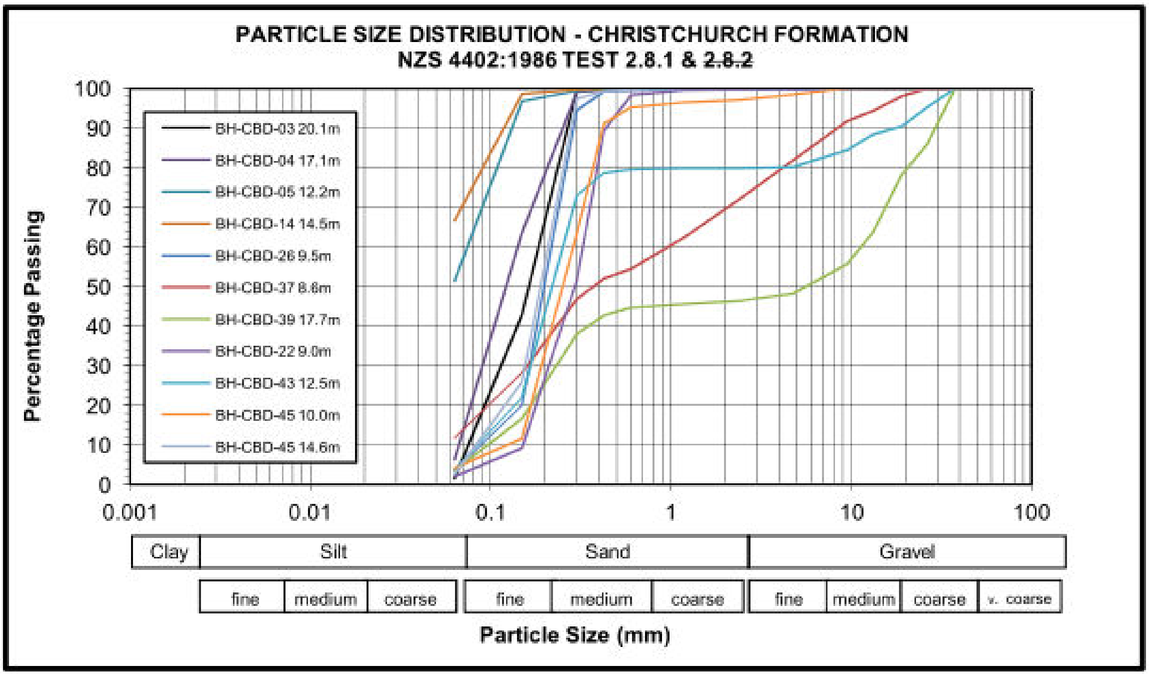
Other significant areas of predominantly loose sands were also encountered north of Ferry Road to the west of Fitzgerald Avenue and around the southern end of Antigua Street / Montreal Street where these cross Moorhouse Avenue; further areas where significant liquefaction occurred.

7.3.1.2.4 Particle Size Distribution

A total of 11 PSDs have been completed on samples obtained from ten boreholes spread across the central city. The results are summarised in Figure 7.3. These indicate materials with a wide range of gradings, from the very uniformly graded fine to medium sands (C = <5) to lower energy deposits with fines contents in excess of 50%, representing some of the silt layers encountered, and occasional very sandy gravels / gravelly sands.

This range in gradings represent the varying depositional environments expected to be found within the Christchurch Formation, ranging from the very fine estuarine / lagoonal deposits (clays and silts), the uniformly graded beach / dune sands that dominated during the main post-glacial marine transgression and more well-graded and gravelly materials which represent significant mixing of fluvial materials into the marine environment, which are transitional with the overlying Yaldhurst Member.

Figure 7.3: Summary of gradings analyses for Christchurch Formation deposits



7.3.1.3 Yaldhurst Member (Springston Formation)

7.3.1.3.1 Transition from Marine-dominated to Fluvial-dominated Deposition

It is believed that around 6,500 to 6,000 years ago, the post-glacial marine transgression reached its maximum extent and sea levels stabilised at a level similar to the present-day sea level.

Between depths of around 7 to 10m below ground level, the dominant marine sands deposited during the marine transgression are progressively replaced by fluvial deposits as the prograding alluvial fan advanced seawards. Because of this gradual increase in fluvially-derived materials, the depth and distribution of the interface between the top of the Christchurch Formation and base of the Yaldhurst Member is not represented by a distinct change in lithologies, such as boundary between the Christchurch Formation and underlying Riccarton Gravels.

7.3.1.3.2 Sandy Gravels

As can be seen from Figures B9 to B2 (covering depths 10m to 2m below ground level), sandy gravels dominate much of the western half of the CBD, and extend locally to the north of the Avon River and south beyond Moorhouse Avenue. These fluvial channel sediments are thought to have been deposited from a subsidiary channel of the Waimakiriri River. The sandy gravels are interbedded with lower energy sand layers, many of which are inferred from the poor recoveries when drilling through the gravel layers.

The ‘tongue’ of the channel deposits extends most of the way along the course of the existing Avon River, with input appearing to be derived from the west through Hagley Park and south-west from the Heathcote Valley, suggesting at least two former channels (or a main channel and flood channel) existed in this part of the city.

The extent and particle size of the gravels typically increases with elevation, reaching a maximum extent across the central city at depths of around 4 to 6m below ground level (see Figures B4 and B5). The extent of the gravels then reduces to become a relatively narrow ‘tongue’ of material along the present alignment of the Avon River, extending from Hagley Park to around Barbadoes Street, between depths of 4 to 2m below ground level (see Figures B3 and B2).

These upper river gravels are typically indicated to be medium dense to dense (and very dense), based on the in situ testing. However, the results of the SPTs and particularly the piezocones, may be over-estimating the density of these layers due to the presence of poorly graded medium to coarse gravels, which tend to result in high SPT blow counts and tip piezocone tip resistance. Given the age of these deposits (typically around 6,000 to 3,000 years old) and the presence of relatively loose interbedded sands (many of which could not be successfully sampled), it would be prudent to assume these to be medium dense unless more detailed testing is completed to confirm otherwise. This aspect is particularly important when assessing the liquefaction hazard associated with the gravels, as discussed in Sections 8 and 9.

The presence of the near-surface gravels resulted in a number of the piezocones having to be terminated at shallow depth. To ensure that the underlying predominantly marine sands were investigated using the piezocone equipment, which is generally considered more applicable for liquefaction analyses than SPT results due to the continuous profile achieved, approximately 30 holes were pre-drilled. The pre-drills were typically advanced to around 0.5 to 1.0m below the base of the gravels to ensure that the holes were not terminated within a sand layer with further gravels beneath (although this did occur at CPT-CBD-98P, which was only advanced a short distance before encountering further sandy gravels at a depth of around 10.5m).

7.3.1.3.3 *Clays and Silts and Organic Silts / Peats*

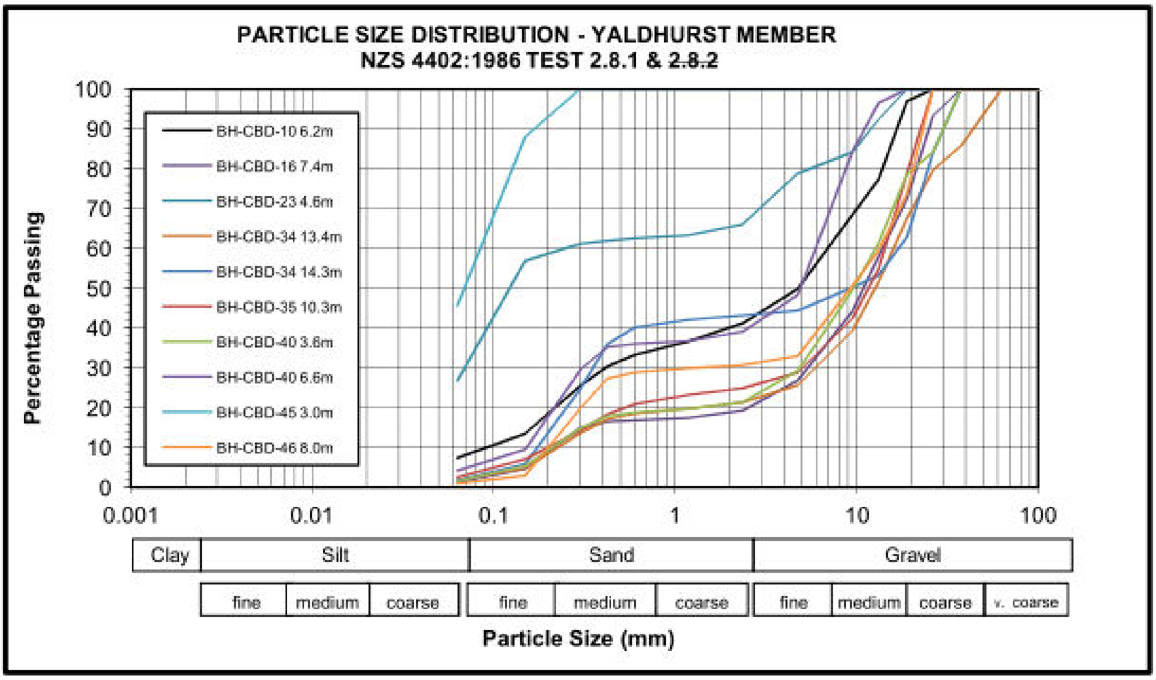
As can be seen from Figures B9 to B2 (covering depths 10m to 2m below ground level), the sandy gravels deposited in the fluvial channel are flanked by overbank deposits including clays / plastic silts and interbedded loose silty sands / sandy silts, with occasional organic silt and peat layers. These are largely confined between the 'tongues' of gravels to the south and north of the Avon River, in a zone to the east of Cathedral Square and in the south-east part of the central city, where yet a further 'tongue' of gravel is present.

The proportion of organic silt and peat layers is much greater in the area to the north of the Avon River and east of Cathedral Square. These are more representative of a predominantly swampy environment with occasional incursions of flood deposits. Relatively few narrow organic layers occur in the southern half of the central city. Both areas however contain numerous layers/pockets of interbedded non-plastic silts/loose sands, representative of higher energy overbank flood deposits.

7.3.1.3.4 *Particle Size Distribution*

A total of 10 PSDs have been completed on samples obtained from eight of the machine boreholes spread across the central city. These are presented on Figure 7.4. The majority of the samples tested comprise the relatively well-graded sandy to very sandy fine- to coarse-grained gravels. They are relatively well-graded with a uniformity coefficient (C) of around 50 to 100, but with somewhat of a gap grading in the coarse sand to fine gravel sizes. These show a similar grading to the Riccarton Gravels, but many samples include a higher proportion of medium to coarse sand particles.

Figure 7.4: Summary of gradings analyses for Yaldhurst Member deposits



7.3.2 Buried and Infilled Channels

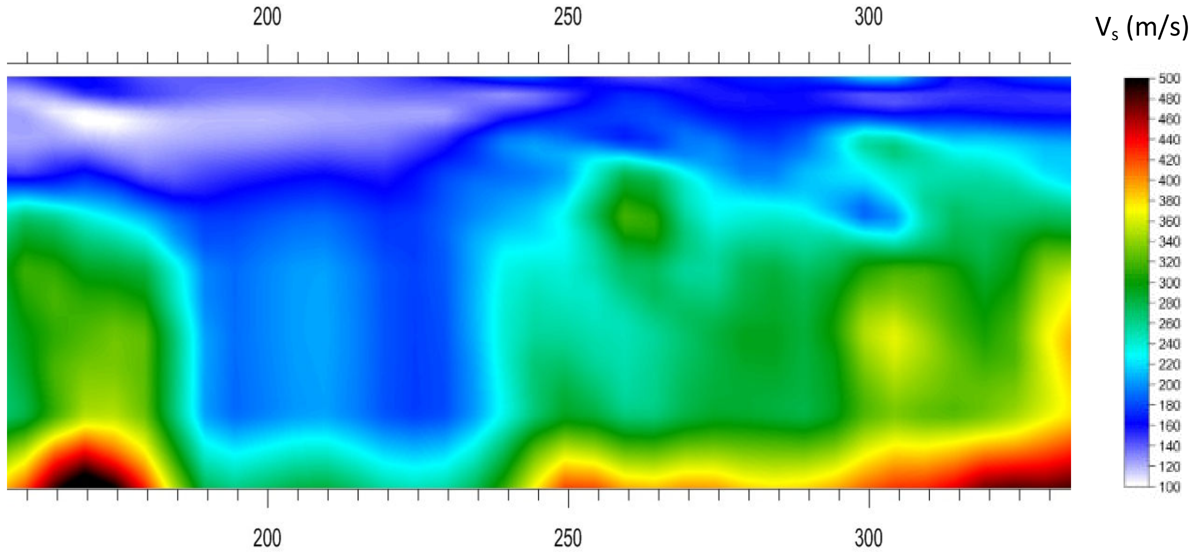
The MASW data provides a good indication of locations where deep buried and shallow infilled channels may be located across the central.

Figure 7.5 illustrates a probable buried channel extending down to a depth of around 30m, well into the surrounding Riccarton Gravel. This data is taken along Montreal Street between Cashel Street and Moorhouse Avenue. A piezocone was located along Montreal Street adjacent to this feature. The piezocone was however terminated on what appears to be a dense sand or sandy gravel layer, which is not picked up by the shear wave velocities. It is possible therefore that a sandy gravel layer is present overlying the soft/loose materials present in the buried channel.

This finding highlights the need not to limit investigations to depths where it is assumed that dense to very dense sands or gravelly sands have been encountered. There is a risk that these layers could be underlain by weaker materials that may not be suitable for certain foundation / ground treatment options.

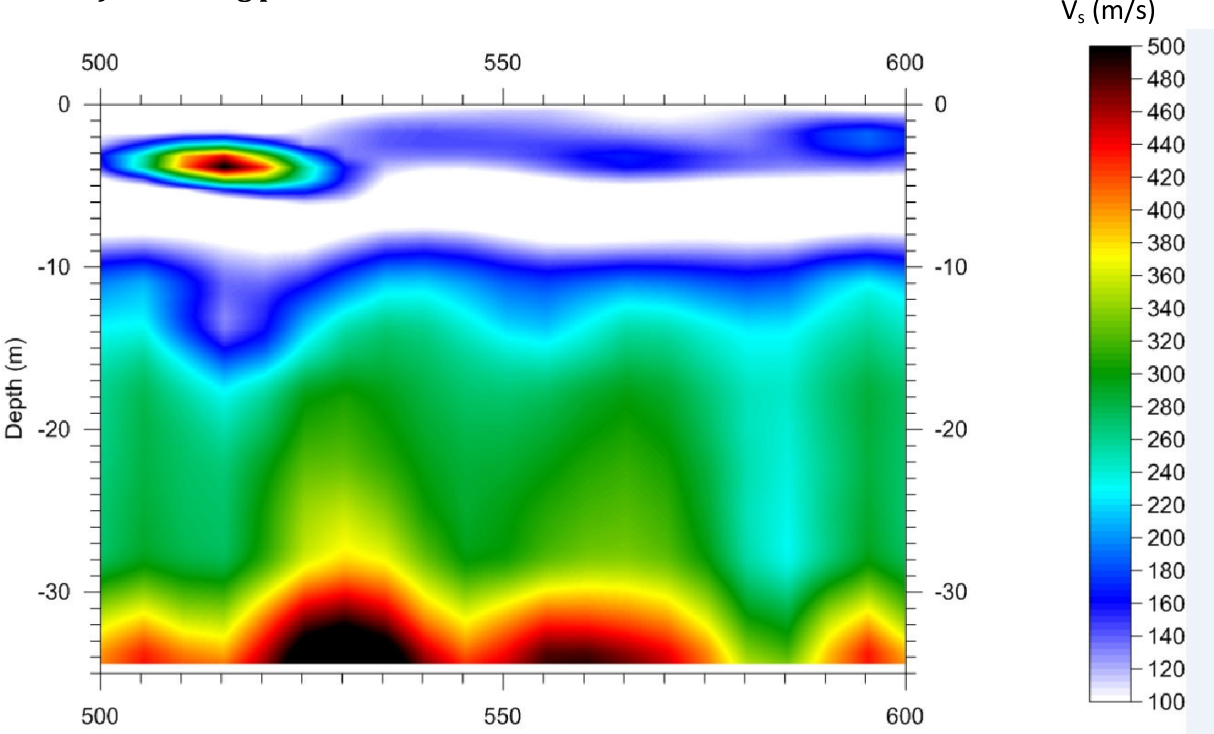
Similar deep buried channels are indicated along many of the MASW sections, particularly in the south-west part of the central city and through the central eastern area. This would suggest that a former large braided channel was present extending from the south-west through the central part of the CBD and out to the east. This would seem consistent with the later deposition of fluvial sandy gravels (Yaldhurst Member) in the top 9 to 10m.

Figure 7.5: MASW data taken along Montreal Street (between Cashel Street and Moorhouse Avenue) indicating presence of a buried channel



The MASW data also appears to identify the presence of more recently infilled channels at shallow depth, as shown in Figure 7.6. These may be natural channels infilled with flood gravels or possibly backfilled open channels, as shown on the 'Black Maps'. Care is required during geotechnical investigations not to mistake such features with shallow gravels suitable for founding structures.

Figure 7.6: MASW data taken along Park Terrace (between Kilmore Street and Bealey Avenue) indicating presence of an infilled channel



7.4 Groundwater Conditions

7.4.1 Groundwater Monitoring

Upon completion of drilling at each machine borehole location, a standpipe or level logger was installed to monitor groundwater levels within the near-surface deposits (i.e. top 8 to 10m below ground level)[5]. The standpipes are to be monitored and the level logger data downloaded monthly for at least one year. The results will be presented as addenda to this report at three monthly intervals.

7.4.2 Groundwater Levels

To date groundwater levels have been recorded at 40 of the 41 standpipes and several months of data has been retrieved from the four level loggers, the results for which are presented in tabulated and graphical form in Appendix H. A summary of the maximum groundwater levels reported in each standpipe and the maximum level recorded in the level loggers during this period are presented in Table 7.2.

[5] Of the 48 boreholes completed, standpipes were installed at 42 locations, one of which had to be removed and backfilled due to flowing artesian groundwater, and level loggers installed at a further four locations.

Table 7.2: Summary of Groundwater Monitoring Results

Borehole	Installation Type	Maximum Recorded Groundwater Level		Borehole	Installation Type	Maximum Recorded Groundwater Level	
		Depth (m bgl)	Elevation (mRL)			Depth (m bgl)	Elevation (mRL)
BH-CBD-01	Standpipe	0.5	5.2	BH-CBD-25	Standpipe	3.4	2.2
BH-CBD-02	Standpipe	1.6	4.3	BH-CBD-26	Standpipe	1.6	3.0
BH-CBD-03	Level Logger	0.2	5.4	BH-CBD-27	Standpipe	2.7	1.0
BH-CBD-04	Standpipe	1.7	4.8	BH-CBD-28	Standpipe	1.0	7.5
BH-CBD-05	Standpipe	1.4	5.1	BH-CBD-29	Standpipe	0.4	6.4
BH-CBD-06	Standpipe	1.0	2.0	BH-CBD-30	Level Logger	1.3	5.1
BH-CBD-07	Standpipe	2.6	1.6	BH-CBD-31	Standpipe	0.7	5.5
BH-CBD-08	Standpipe	2.1	4.5	BH-CBD-32	Standpipe[1]	-	-
BH-CBD-09	Standpipe	1.4	5.3	BH-CBD-33	Standpipe[1]	-	-
BH-CBD-10	Standpipe	2.0	3.3	BH-CBD-34	Standpipe	1.5	8.1
BH-CBD-11	Standpipe	0.3	3.2	BH-CBD-35	Standpipe[1]	-	-
BH-CBD-12	Standpipe	0.6	4.4	BH-CBD-36	Standpipe	2.6	5.1
BH-CBD-13	Standpipe	2.5	4.3	BH-CBD-37	Standpipe	0.3	4.5
BH-CBD-14	Standpipe	2.6	4.0	BH-CBD-38	Standpipe	1.1	7.1
BH-CBD-15	Standpipe	2.1	3.1	BH-CBD-39	Standpipe	1.2	5.0
BH-CBD-16	Standpipe	2.6	2.5	BH-CBD-40	Standpipe	1.1	3.7
BH-CBD-17	Standpipe	-	-	BH-CBD-41	Standpipe	1.2	5.5
BH-CBD-18	Standpipe	3.4	2.5	BH-CBD-42	Standpipe	1.7	3.8
BH-CBD-19	Level Logger	0.7	4.9	BH-CBD-43	Standpipe	1.6	2.6
BH-CBD-20	Standpipe	3.2	2.1	BH-CBD-44	Level Logger	0.6	2.5
BH-CBD-21	Standpipe	2.7	3.7	BH-CBD-45	Standpipe	2.7	1.8
BH-CBD-22	Standpipe	-	-	BH-CBD-46	Standpipe	2.9	1.5
BH-CBD-23	Standpipe	3.1	3.6	BH-CBD-47	Standpipe	1.6	5.6
BH-CBD-24	Standpipe	3.4	3.3	BH-CBD-48	Standpipe	1.6	3.6

[1] Standpipes not installed, lost or removed.

7.4.2.1 Depth to Groundwater

As can be seen, there is reasonable variability in the depth to groundwater level across the central city, ranging from 0.2m below ground level at BH-CBD-03, located at the junction of Bealey Avenue and Barbadoes Street, to a maximum recorded depth of 3.7m in BH-CBD-25, positioned on St Asaph Street near the junction with Madras Street and Ferry Road; with an average depth across the central city of around 1.9m below ground level.

There are some areas within the central city where relatively elevated groundwater levels occur, such as around the lower Avon River in the north-east part of the CBD (i.e. BH-CBD-06, -11 and -

12 all recorded depths less than 1m), in the south-east (BH-CBD-37, -40, -42 and -44) and along the western half of Moorhouse Avenue (BH-CBD-28, -29 and -31).

Relatively deep groundwater levels (>2.5m below ground level) also occur in a few specific areas, such as along Rolleston Avenue from around Armagh Street to the Christchurch Hospital (BH-CBD-13, -14, -21, -23, -47 and -48), and the central eastern part of the CBD between Armagh Street to the north and St Asaph Street to the south (BH-CBD-18, -20, -24, -25, -27, -45 and -46).

It should be noted however that groundwater levels vary spatially over short distances and therefore site-specific investigations should not rely exclusively on the results of the monitoring that is currently being undertaken. For instance, BH-CBD-18 located at Latimer Square, recorded a maximum groundwater level of 3.4m below ground level. Approximately 300m to the east along Worcester Street, BH-CBD-19 recorded a groundwater depth of 1.2m on the same day, and a further 350m east at the junction of Worcester Street and Fitzgerald Avenue, the groundwater depth was recorded at 3.2m below ground level in BH-CBD-20.

7.4.2.2 Groundwater Elevation

Groundwater elevations typically reduce from a maximum of around 8.0mRL in the more elevated south-west area of the central city (BH-CBD-34), to around 1.5 to 2.0mRL along the eastern side (BH-CBD-07, -20, -44, -45 and -46), with a low point recorded at BH-CBD-27 at the junction of St Asaph Street and Fitzgerald Avenue (1.0mRL), with an average elevation across the central city of 4.0mRL.

Groundwater is also elevated to the north of the Avon River, typically reducing south from Bealey Avenue, as would be expected. Groundwater levels recorded on the north bank of the Avon River (BH-CBD-10, -11, -15) are typically higher than those located to the south of the river. This suggests that the near-surface regional groundwater flow is largely unaffected by the Avon River (which is expected to be effluent through the central city). Flow appears to occur from the east and north-east from the southern side and south-east from the northern area of the central city, with a low spot focused south-east of Cathedral Square along the Ferry Road direction to Waltham / Phillipstown.

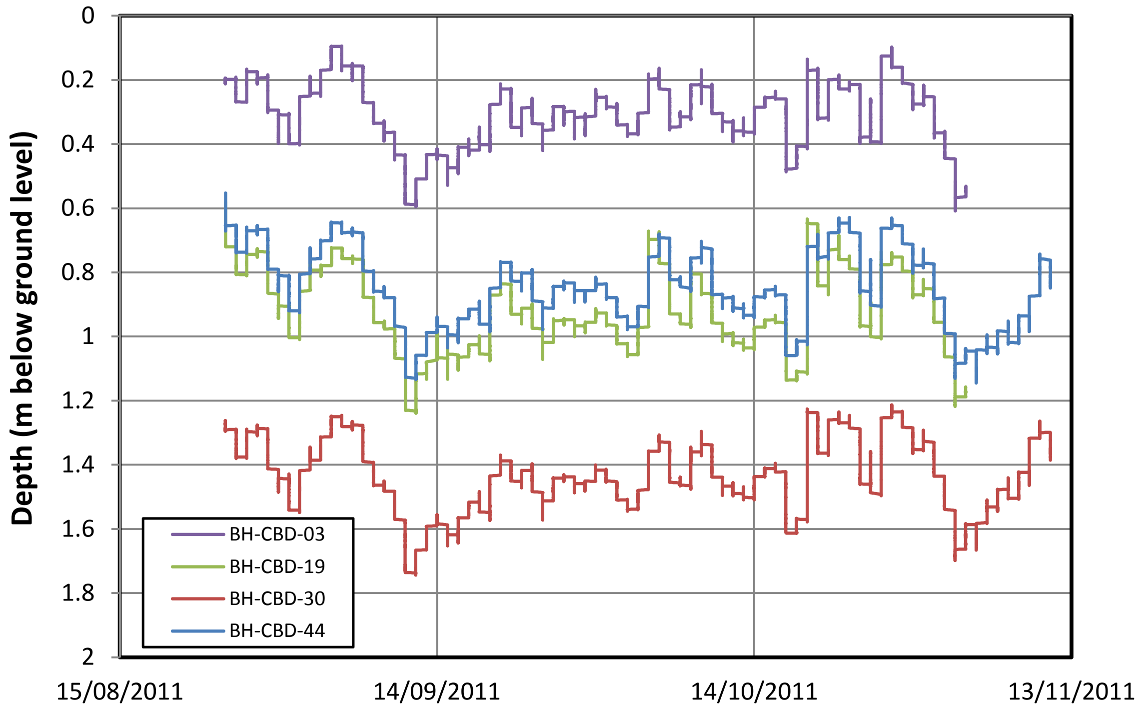
7.4.2.3 Groundwater Fluctuations

As indicated, it is planned for the groundwater monitoring to continue for at least 12 months to determine the likely seasonal variability on an annual basis. To date we have too few readings from the standpipes to assist this understanding. The early results obtained for the four level loggers however provide some useful information.

Figure 7.5 shows the variation in depth to groundwater over the August to November 2011 (this is shown in greater detail in Figure H3 in Appendix H).

As can be seen, the magnitude and timing of the fluctuations are very consistent across the four level loggers and show limited variation of the time period covered (<0.25m).

Figure 7.6: Groundwater depth variation recorded in four level loggers (August to November 2011)



7.4.3 Artesian Groundwater

The Riccarton Gravels are the highest of several artesian aquifers present beneath the Canterbury Plains. Of the 47 boreholes which terminated within the Riccarton Gravels, the vast majority recorded artesian groundwater flows and the remaining were sub-artesian, but generally within t a few metres of the ground surface.

No monitoring of the groundwater pressures within the Riccarton Gravels has been undertaken for this study, but this information will be required where consideration is given to piling into the gravels, as discussed in Sections 9 and 10.

7.5 Conceptual Geological Model

Inspection of the geological plans and cross sections presented in Appendices B and C allows a quick, basic understanding to the geological evolution of the central city.

The Riccarton gravels, which are typically encountered at depths of between 18 and 30m below ground level, increasing approximately from west to east, represent glacial outwash sediments deposited during and up until the last glacial maximum.

The Riccarton Gravels were then overlain by swampy materials, including relatively thick peats and organic silts, during the early stages of the current interglacial before sea level rise had resulted in drowning of the central city. As sea levels approached the eastern side of the what is now the central city between approximately 10,000 and 7,000 years ago, lagoonal and estuarine deposition commenced resulting in the accumulation of thick clays and silts with interbedded silty sands / sandy silts, particularly in the lower south and eastern areas, which thin towards the north and are largely absent in the far north and western areas.

Continued post-glacial sea level rise continued with the entire central city becoming drowned around 6,500 to 6,000 years ago, at which time sea level rise abated and has remained at a similar level to the present day. Beach and dune sands were the dominant sediments accumulating at this time, and include silt layers and occasional shelly beds.

Between approximately 6,000 and 3,000 years ago, fluvial channel and overbank deposits have resulted in gradual progradation from west to east across the central city and beyond to the eastern suburbs. River channels deposited sandy gravels during this time, which cover large parts of the western side of the central city and extend along the current alignment of the Avon River and at the same time, clays and plastic silts accumulated, particularly in the southern half of the central city, whilst a combination of swampy materials, including thick peat and organic silts, accumulated in the northern and central areas.

7.6 Piezocone Calibration Exercise Results

As detailed in Section 4.1.3.1, in order to assess the consistency of the piezocone data obtained by the two principal testing contractors, a calibration exercise was completed to compare the results of a number of tests completed adjacent to one another. It was further decided that the test would be completed close to one of the UoC test zones so that the results of those piezocones, which were completed by a third contractor, could also be included.

The calibration exercise was completed at 234 to 240 Armagh Street, located at the junction with Madras Street, on a site where the building, which we understand was affected by severe differential settlements following the 22 February 2011 earthquake, had been recently demolished.

Three tests were completed by both Opus International Consultants Ltd (Opus) and Perry Drilling Ltd (Perry Drilling), positioned to within 1m of one another. These are referenced CPT-CBD-OC01 to OC03 and CPT-CBD-PC01 to 03, the positions of which are shown on Figure A8, along with the nearby UoC piezocones. The results of the individual piezocone tests completed by Opus and Perry Drilling are included in Appendix J and the UoC tests are presented in Appendix L.

The results of the calibration exercise demonstrate three interesting aspects. As follows:

1. The data obtained from the different contractors equipment is reasonably consistent
2. The value of the data obtained when testing within the ground conditions present in Christchurch central city is significantly improved by allowing a relatively small increase in the maximum force permitted by the equipment
3. The highly variable nature of the ground conditions even on a local scale

These aspects are discussed further in the following sections with reference to the test results.

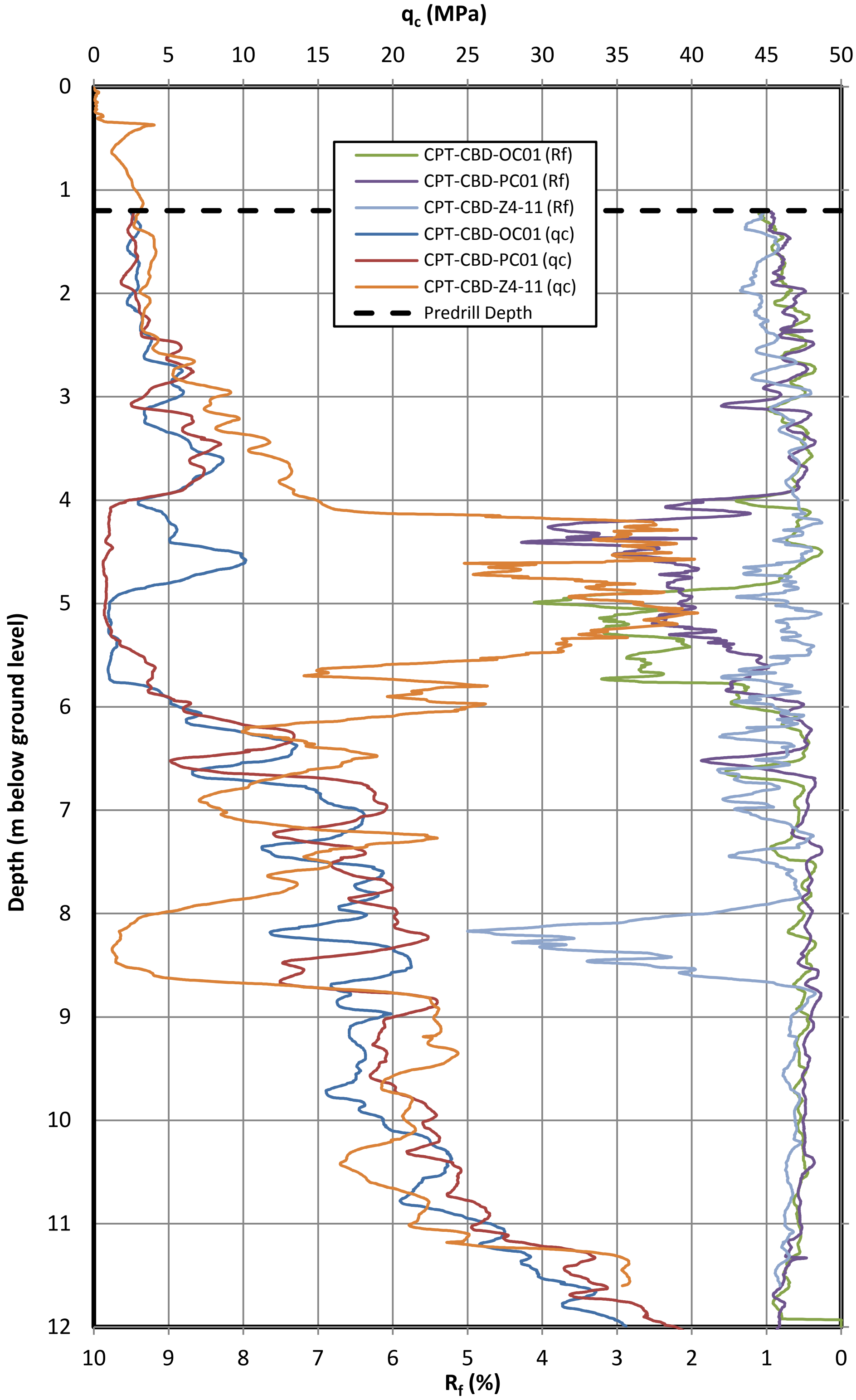
7.6.1 Consistency of data

The results of all three tests completed by Opus and Perry Drilling are included in Appendix J. The results for the testing completed at position 1 are shown in Figure 7.5 below.

It can be seen from Figure 7.5 that where the piezocone is intercepting similar materials, such as between 1 and 4m in the case of CPT-CBD-OC01 and CPT-CBD-PC01, the tip resistance recorded by both sets of equipment is relatively consistent and would be interpreted in the same manner for use in geotechnical design.

The friction ratio for the UoC test is however significantly higher than the Opus and Perry Drilling tests in the top 2 to 3m. It is well understood in geotechnical practice that results for sleeve friction are typically much less accurate than tip resistance and are therefore rarely used in design situations.

Figure 7.5: Piezocone calibration plots with University of Canterbury data (CPT-CBD-OC01, CPT-CBD-PC01 and UoC Z4-11)

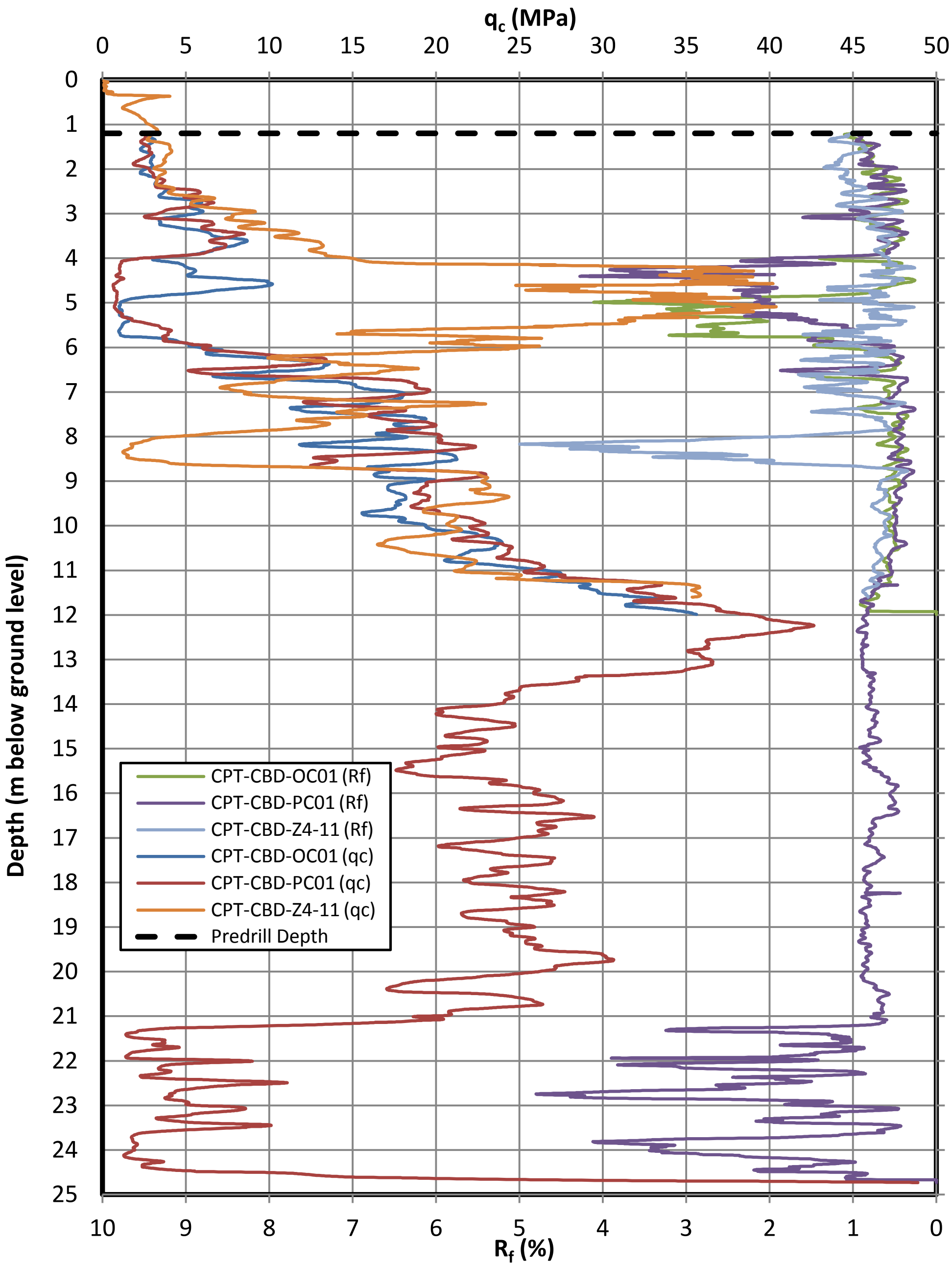


Within the medium dense to dense sands encountered below a depth of approximately 7m, the tip resistance for all three machines are comparable within the required degree of accuracy for geotechnical design.

7.6.2 Maximum Permitted Force

Figure 7.6 presents the full set of data obtained by the three sets of testing equipment at position 1. It can be seen that both CPT-CBD-OC01 and CPT-CBD-Z4-11were terminated at a depth of approximately 12m below ground level, at which point the tip resistance reached around 35MPa. However, as can be seen from the CPT-CBD-OP-01 plot, a small additional push permitted the hole to continue down to 25m at which depth the Riccarton Gravels were encountered.

Figure 7.6: Piezocone calibration plots with University of Canterbury data (CPT-CBD-OC01, CPT-CBD-PC01 and UoC Z4-11



The shallow tests would leave a degree of uncertainty as to the nature of the deposits below 12m, particularly with respect to liquefaction hazard where looser sands may be present and indeed the depth of the Riccarton Gravels. Inspection of a number of deep piezocone tests completed for this project indicates that the dense marine sands rarely offer a resistance significantly above 40MPa. Use of equipment able to penetrate through the dense Christchurch Formation sediments (i.e. >40MPa) may therefore provide data to the full depth required for most geotechnical design purposes; except where piling into the Riccarton Gravels is required.

7.6.3 Local Variability

This plot also serves to provide a very useful indication of the local variability of the ground conditions that may be expected within some areas of the central city. The data from the three plots shown on Figure 7.5 is reasonably consistent to a depth of 4m and below 6m (excluding a narrow clay/silt band encountered in Z4-11 at a depth of 8m).

The materials encountered between 4 and 6m in the three test locations, all of which are within approximately 8m of one another, indicate relatively loose sands which would be expected to liquefy under a significant seismic event (CPT-CBD-OC01), non-liquefiable clayey silts (CPT-CBD-PC01) and medium dense to dense gravelly sands / sandy gravels (UoC Z4-11).

These three materials would be expected to behave very differently during significant seismic shaking and would therefore be considered separately for geotechnical design purposes. A design approach that would seem appropriate for one part of the site may not be suitable given the conditions encountered a short distance away.

This variability serves to highlight two further aspects:

- Detailed site specific investigations are clearly required for sites within the central city and a high density of investigation points is warranted both for design and subsequent construction control and monitoring
- Great care is required when assessing the ground conditions encountered and sound engineering judgement should be applied to any geotechnical analyses as variations to the assumed ground conditions can have very significant impact on unrealistically precise calculations.

8. Liquefaction Hazard

8.1 Overview

It has long been appreciated by experienced geotechnical engineers and engineering geologists that large areas of Christchurch city are underlain by deposits that are susceptible to liquefaction (Elder et al. 1990, Brown & Weeber, 1992). A number of studies have been completed to assess the likely distribution and severity of liquefaction across the city that may be triggered by various seismic events (ECan, 2004).

These assessments primarily assume a distant seismic source associated with a large magnitude earthquake (i.e. movement of the Alpine Fault or Porters Pass Fault). The presence and potential significance of local blind faults has not been specifically addressed, but the overall impacts on the city are, in general terms, similar to those predicted to result from a larger, more distant source.

In order to undertake liquefaction hazard modelling of large areas, a good understanding of the geological materials present, the depositional environment under which they were laid down and groundwater levels is fundamental to identifying those areas at greatest risk. With the above information, it is possible to broadly map the distribution and likely severity of the liquefaction hazard; such has been undertaken in the State of California[6].

Detailed ground investigations aimed at identifying the local variability of the soil deposits, their geotechnical properties (i.e. plasticity, grading and density) and the local groundwater regime, help to refine the extent and severity of the liquefaction hazard. With appropriate data regarding the geotechnical properties of the soil and local groundwater conditions, liquefaction analyses can then be completed, taking account of the predicted ground motions, to further define the likelihood and severity of liquefaction that could occur under certain scenarios.

This section of the report presents a summary of the preliminary liquefaction analyses that have been completed by T&T for the central city utilising the recently gathered ground investigation data. The results are briefly described in the context of the ground model outlined in Section 7.3 coupled with liquefaction-induced land damage mapping completed following the major earthquakes of 04 September 2010, 22 February 2011 and 13 June 2011. A summary of this work and the data utilised is presented in Section 8.2.

8.2 Land Damage Mapping

Shortly after each of the main earthquake events, detailed mapping of the extent and severity of land damage was completed by T&T for the CBD and adjacent commercial areas. This mapping was based largely on observed surface manifestation of liquefaction and including lateral spreading, the presence of ejected material (groundwater, sand and silt), ground cracking and general deformation of the ground surface. A simplified plan indicating the extent and level of the observed land damage reported following the 22 February 2011 aftershock is presented as Figure D1 in Appendix D.

It is important to appreciate that, whilst the presence of sand boils is a confirmation that liquefaction has occurred, the absence of sand boils or other ground disturbance does not mean that liquefaction has not occurred beneath the surface. The extensive coverage of land within the central city by large footprint buildings and thick pavements may have prevented significant formation of sand boils. Additionally, there are many locations within the central city where a

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[6] State of California Department of Conservation Seismic Hazards Zonation Program.

relatively thick crust of non-liquefiable materials may have prevented surface expression of liquefaction.

The change in ground elevation since 04 September 2010 (inferred from the LiDAR data and taking account of likely regional tectonic uplift/subsidence), suggests that ground deformation has occurred in areas where little or no land damage was observed.

The results of the analyses detailed below also indicate that liquefaction is likely to have occurred in many areas where little or no surface manifestation of liquefaction was observed.

It should also be understood that the mapping of observed liquefaction-induced land damage relates to a specific seismic event. In the case of the 22 February 2011 aftershock, a relatively small magnitude (short duration) earthquake generated high vertical and horizontal ground accelerations in the CBD due to its close proximity. Ground motions generated by other seismic events could result in different patterns and severity of land damage. This is illustrated by the contrasting distribution and severity of liquefaction resulting from the 04 September 2010 earthquake relative to the aftershocks of 22 February and 13 June 2011.

Figure D1 does not therefore provide a definitive plan of the extent and severity of liquefaction and associated lateral spreading that resulted from the 22 February 2011 aftershock or which could result from future earthquakes and should not be used for such purposes. It should be noted, however, that observations made following a number of significant seismic events around the world indicate that those areas that have suffered liquefaction during one event will often liquefy during subsequent earthquakes, even when the source and nature of the resulting ground motions vary. It would be unwise, therefore, not to take proper account of the land damage mapping that has been completed as providing a good guide to identifying those areas which are likely to be affected by future liquefaction events, irrespective of what specific liquefaction assessments may suggest, as discussed further in Section 8.4.

8.2.1 Overview of Observed Land Damage

As can be seen from Figure D1, the Darfield Earthquake of 04 September 2010 resulted in little or no observed land damage across much of the central city. Moderate to severe levels of liquefaction were however observed in parts of the north-east area of the CBD. The low-lying area to the north of the Avon River, between Colombo Street and Barbadoes Street, was affected by moderate to severe levels of liquefaction. Similar levels of land damage, but including localised lateral spreading, were observed on the point bar deposits within the Avon River meander immediately west of Fitzgerald Avenue. These affect the low-lying, largely residential area, north of Chester Street.

Mapping conducted following the 22 February 2011 aftershock recorded little or no observable land damage across approximately 40% of the CBD, including the city core around Cathedral Square. Moderate levels of liquefaction were reported across a further 40% of the CBD and the majority of the adjacent commercial areas to the south and south-east. Around 10% of the CBD was affected by severe liquefaction, including the area north-east of the Avon River / Colombo Street intersection; areas north-west of the Moorhouse Avenue / Colombo Street intersection; a section of the CBD located north of Ferry Road between Barbadoes Street and Fitzgerald Avenue and several other more localised pockets. Severe liquefaction was also observed south of Moorhouse Avenue around Montreal Street and in the vicinity of the AMI Stadium.

Severe liquefaction accompanied with localised lateral spreading was observed at discrete locations along the banks of the Avon River through the CBD. This was most severe at the downstream end, affecting the mainly residential areas within the meander immediately west of Fitzgerald Avenue (north of Chester Street) and between Peterborough and Salisbury streets east of Manchester Street. These latter areas correlate strongly with the zones affected by moderate to severe liquefaction following the Darfield Earthquake. Lateral spreading was also observed either side of the Avon River between Colombo and Manchester streets, on the south side of the river immediately west of Colombo Street and a short section adjacent to Oxford Terrace immediately east of Antigua Street.

To supplement the land damage mapping and better understand the distribution and severity of liquefaction that occurred across the central city, T&T have reviewed the detailed aerial photography that was obtained shortly after the 04 September 2010, 22 February 2011 and 13 June 2011 seismic events. The extent of sediment laden groundwater ejected to the surface can be readily seen on these images and supports the findings of the land damage mapping that was completed by T&T staff.

We have also compared the land damage mapping with the change in ground elevation following the earthquakes. This has been achieved by comparing Digital Elevation Models developed from the LiDAR data sets obtained prior to the Darfield Earthquake (by AAM Brisbane, 2003 and 2005) and following each of the major earthquakes (by New Zealand Aerial Mapping (NZAM), 2010 and 2011).

Whilst the accuracy of the data is not sufficient to determine small changes in ground elevation resulting from minor to moderate liquefaction, there is clear correlation with large changes in elevation within the zones affected by severe liquefaction and lateral spreading, in particular where settlements in excess of 300mm have occurred.

8.3 Liquefaction Assessment

The data obtained from the 150+ piezocones spread across the central city has been used to undertake preliminary liquefaction analyses. This has been completed for the following principal purposes:

- to permit a better understanding of the distribution and severity of the observed liquefaction-induced land damage across the CBD and commercial areas
- to assess which areas may have been subject to liquefaction where little or no land damage has been observed
- to provide an indication of the level of ground accelerations required to trigger liquefaction in the susceptible layers to highlight the potential extent of the liquefaction hazard across the central city from future seismic events
- to provide some insights into the applicability of the liquefaction analyses methodology currently adopted as state-of-the-practice for such assessments for the soils encountered in Christchurch central city.

The analyses provide a broad overview of the liquefaction hazard within the central city.The results should be considered as providing a preliminary indication of the liquefaction hazard only and should not be used for detailed design.

Detailed site assessments will be required to estimate the level of liquefaction hazard at specific locations, based on the findings of comprehensive geotechnical investigations. This should include more rigorous liquefaction analyses than has been completed for this report, taking into consideration the particle size distribution[7] and shear wave velocity[8] of the perceived liquefiable

[7] As a result of the significant lateral and vertical variability of the soils encountered across the central city combined with the limited number of borehole investigations and laboratory tests (for the size of the study area), it was not deemed representative to include this information in these preliminary analyses.

[8] This information was not available for all piezocone locations at the time of completing the analyses and therefore a consistent approach was adopted assuming an average shear wave velocity for potentially liquefiable materials of V_s = 175m/s.

materials, and the application of engineering judgement when considering, for example, the 'actual' relative density of non-cohesive layers where these are interbedded with thin clay/plastic silt materials[9].

8.3.1 Methodology

The liquefaction analyses have been carried out by T&T using the simplified method of Seed et al. (2003), with estimates of the potential ground deformations (settlement) based on the procedure presented by Ishihara & Yoshimine (1992). The results of the individual analyses are presented in Appendix G and discussed in the following sections.

As indicated in Section 4, each of the piezocone holes were pre-drilled to a depth of around 1.2m below existing ground level to check for the presence of buried services before being backfilled with loose sand prior to commencing the CPT. Data for the upper 1.2m has therefore not been included in the assessment. This depth would typically be within the unsaturated zone for most locations and therefore would not be expected to liquefy, even if the materials are susceptible to liquefaction.

Of the 151 piezocones completed, approximately 60 were advanced to depths in excess of 15m below ground level. However, a number of the piezocones were terminated at relatively shallow depths where they encountered dense gravelly sand / sandy gravel layers or coarse gravels / cobbles. Where it is considered likely that deeper materials present beneath these near-surface layers may be susceptible to liquefaction, as judged from adjacent borehole investigations and a general understanding of the geology in these areas, pre-drilling was completed to penetrate through the upper dense layers to allow testing of the deeper, looser deposits[10].

For completeness, liquefaction analyses were completed using the results of all piezocones, even where these terminated at shallow depths. Where tests were terminated at depths less than approximately 15m below ground level, it should not be assumed that no deeper liquefiable materials are present and should be investigated by deep boreholes for site specific assessments.

8.3.2 Ground Accelerations

The triggering of liquefaction in susceptible materials is dependent upon the ground motions to which the soil is subjected. For the procedure used in this report, the ground motion is specified in terms of the amplitude (peak ground acceleration – PGA) and duration of the shaking, for which the moment magnitude (M_w) is adopted as a proxy.

For the purposes of comparing the results of the analyses with the observed liquefaction-induced land damage resulting from each of the major earthquakes, the analyses have been completed adopting the geometric mean of the of the highest recorded ground accelerations of the four strong motion stations located within the CBD, rather than the closest station to each piezocone location[11]. In the case of the 22 February 2011 event, this corresponds to a PGA of 0.52g recorded at the Resthaven station (Cubrinovski & McCahon, 2011), along with M_w = 6.2 (GNS, 2011).

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[9] It is well understood that the results of both SPTs and CPTs are influenced by the presence of softer or less dense zones present in the vicinity of the section being tested. It is clear from the results of the CPT liquefaction analyses that many of the deeper narrow bands of 'liquefiable' materials present within otherwise dense to very dense non-liquefiable layers are due to the presence of thin clay/plastic silt bands or 'pockets' of loose sands/non-plastic silts, which result in lower than expected q_c values immediately above and below. In reality, these zones are unlikely to liquefy to a significant extent.

[10] This information was not available at the time of preparation of this report and therefore is not included in the results presented below.

[11] Use of the closest seismic station records may not be the most representative data as this is likely to be influenced more by the near-surface ground conditions at the recording station site rather than proximity.

In order to compare the levels of liquefaction assessed for the recent earthquakes with current design code requirements, we have also run the analyses using the serviceability limit state (SLS) and ultimate limit state (ULS) peak ground level horizontal accelerations (PGHAs) recommended in NZS 1170.5 (2004) and the updated Zone factor (Z) issued by the Department of Building & Housing New Zealand (DBH, 19 May 2011). A summary of the six 'scenarios' considered are summarised in Table 8.1.

Table 8.1: Summary of liquefaction analyses ground motion parameters

Scenario	Zone Factor (Z)	Return Period Factor (R_s / R_u)	Moment Magnitude (M_w)	Peak Ground Acceleration (g)
(1) 1170.5 (2004) - ULS[1]	0.22	1.0[2]	7.5	0.25
(2) DBH Update – SLS[1]	0.30	0.25[2]	7.5	0.11
(3) DBH Update – ULS[1]	0.30	1.0[2]	7.5	0.34
(4) 22 February 2011	-	-	6.2	0.52
(5) 04 September 2010	-	-	7.1	0.25
(6) 13 June 2011	-	-	6.0	0.26

1. Assumes a Site subsoil class – D (deep or soft soil) for calculation of the spectral shape factor ($C_h(0)$.

2. Corresponding to an annual probability of exceedance of 1/25 for SLS and 1/500 for ULS.

8.3.3 Groundwater Level

For the analyses presented we have assumed a blanket groundwater level of 1.2m below existing ground level, to match the pre-drill depth detailed in Section 8.3.1. This is sufficient for the present purposes but site specific liquefaction analyses should take account of the local groundwater regime through adoption of groundwater level monitoring at the time of investigations, with some allowance for seasonal fluctuations, supported by monitoring data that will be obtained from the current work or other suitable sources (or a conservative assumption made).

8.4 Summary of Results

8.4.1 Presentation

The results of the individual liquefaction analyses are included in Appendix G. Two sheets are provided for each piezocone test which covers the six levels of seismic shaking that has been analysed. These results have been used in conjunction with the ground model detailed in Section 7.3, to identify the layers that are considered likely to liquefy during ground motions for Scenario 3 (i.e. the ULS for the current design Z factor of 0.30).

These have been simplified to show the main zones of liquefaction on Figures E2 to E24 in Appendix E, ignoring thin layers of non-liquefied material that are present within a thick liquefied layer and vice versa for partially liquefied deposits. Again, this is considered appropriate for the purposes of this report, but detailed site specific assessments would need to consider the impact of marginally liquefied layers for site specific ground response analyses.

This information is also presented in plan form to provide a broad illustration of the distribution of the materials assessed to be at risk of liquefying under Scenario 3 (ULS event for current design factor Z = 0.30). Four plans are provided indicating where liquefaction is anticipated to occur at different depth intervals, as follows:

- 1.2 to 3m
- 3 to 5m
- 5 to 10m
- 10 to 20m

The cross sections and plans are based on a very limited number of investigation locations (for the size of the area covered) and therefore interpolation between the points should not be relied upon for anything other than an initial screening of the potential for liquefaction hazard at any specific site. This information is provided largely to guide the extent and depth of site specific ground investigations and geotechnical assessments that are likely to be required at different locations across the city and to provide a context for interpreting the findings at specific sites.

8.4.2 General Observations

The analyses indicate that a liquefaction hazard is present across virtually the whole of the CBD and adjoining commercial areas, and is not limited to those locations where liquefaction-induced land damage has been observed (i.e. suggesting that area wide deep liquefaction is likely to have occurred). This observation is considered generally consistent with the LiDAR data which suggests settlement may have occurred in some areas where no land damage has been mapped at the ground surface.

Of the 150 or so piezocones analysed, approximately 75% indicated what may be considered a reasonable level of liquefaction (taken, arbitrarily, as a minimum of 1m of liquefied thickness), for both the previous and current ULS design cases and each of the recent seismic events (scenarios 4 to 6).

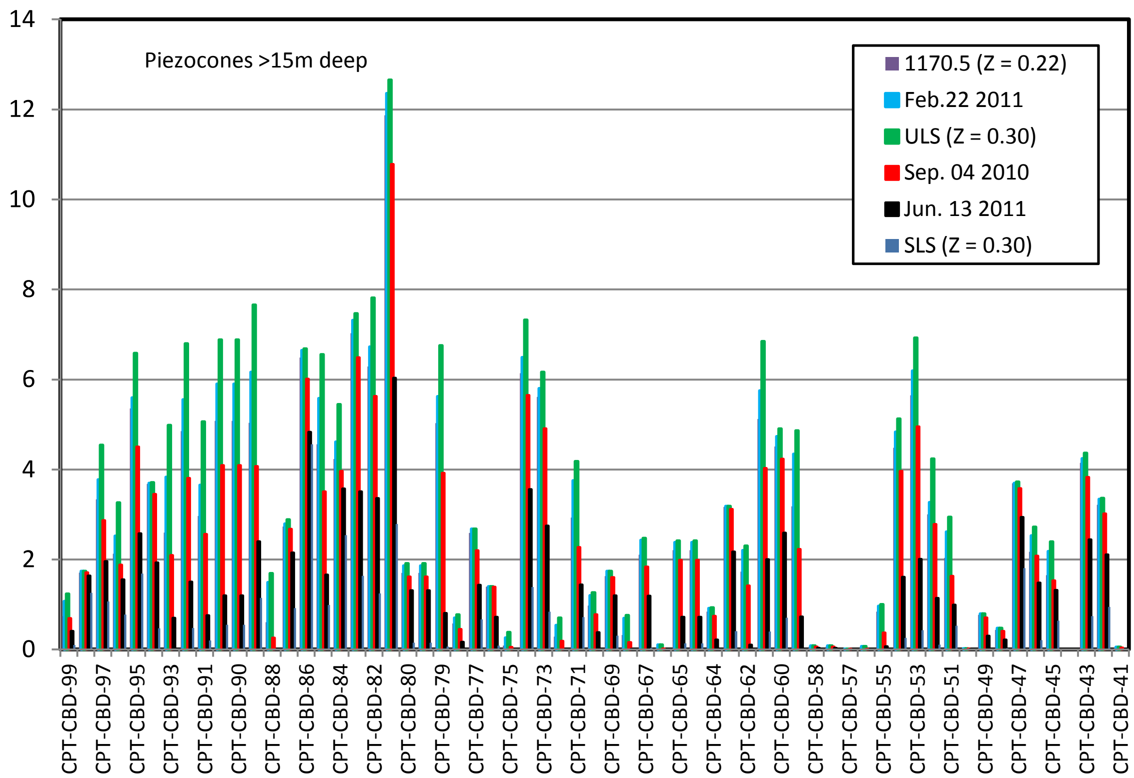
This assessment includes the results from the piezocones that penetrated a depth of less than 5m below ground level before terminating on a dense layer. When excluding these shallow piezocones, which may not have encountered all liquefiable zones at depth, virtually all of the analyses indicate liquefied thicknesses greater than 1m. A similar minimum thickness of liquefied material is estimated in approximately 35% of the piezocones for the current SLS design case (Scenario 1, with Z=0.30 for a 1/25 annual probability of exceedance); the vast majority resulting from liquefaction of layers present at depths ranging from 5 to 10m below ground level.

The maximum liquefiable thickness was recorded for CPT-CBD-32, with 13.1m of liquefied layers in the upper 20m. This was located at the junction of Manchester Street and Kilmore Street in an area affected by severe liquefaction. The ground here comprises loose silty sands to around 3m below ground level, overlying 2 to 3m of medium dense to dense sandy gravels, which in turn overlie loose to medium dense sands to a depth of around 20m. Piezocones undertaken by the University of Canterbury (UoC) a short distance to the west of CPT-CBD-32 (referenced Zone 1), encountered similar materials but without the medium dense to dense gravels between 3 to 5 or 6m below ground level. Analysis of the UoC data indicates a similar thickness of liquefied materials. The level of surface ejecta and settlement (as suggested by the LiDAR data) at the UoC site was significantly greater than at CPT-CBD-32, suggesting that the gravel layer at this location may have reduced the severity of the effects of liquefaction at the ground surface and/or the volume of pressurised groundwater reaching the surface.

One aspect of these analyses which is highly relevant to rebuilding of the CBD and adjacent commercial areas is that, in general terms, the severity of liquefaction assessed for the different scenarios are relatively comparable. The new proposed Z factor of 0.30 paired with a M_w = 7.5, predicts marginally higher levels of liquefaction than the 22 February 2011 event. The NZS 1170.5 (2004) Z factor of 0.22 gives rise to a marginally lower level of liquefaction than the 22 February event, but significantly higher than both the 04 September 2010 and 13 June 2011 aftershocks.

This variation in liquefied thickness for the various scenarios is illustrated in Figure 8.1 for each piezocone that penetrated a minimum depth of 15m below ground level.

Figure 8.1: Estimate of liquefied layer thicknesses for different seismic events



Note: For clarity, only piezocones that penetrated depths >15m are included and only a limited number of piezocones are listed along the bottom axes. Refer to Figure A5 (Appendix A) for locations of piezocones included.

The amount of liquefaction predicted from the Darfield Earthquake is typically higher than that recorded for the 13 June 2011 aftershock, and yet the level of observed liquefaction within the CBD and adjoining areas was generally more widespread and severe than that resulting from the 04 September earthquake. It should be noted however that the liquefaction analyses for the 13 June 2011 event takes no account of the M_w 5.3 aftershock that occurred shortly before the main M_w 6.0 event, which may have resulted in elevated pore water pressures and hence made the impact of the subsequent earthquake M_w 6.0 greater than that suggested by the present analyses.

It is also possible however that the emphasis placed on the duration of shaking, as expressed by the earthquake magnitude in the liquefaction analyses procedure, may result in an over-estimate of the liquefaction severity. If this is the case then the degree of liquefaction predicted using a magnitude weighting approach may result in an over-estimate of the liquefaction hazard from future seismic events.

It is also worth noting that, in the vast majority of the cases analysed, the predicted liquefaction-induced ground settlements are greater than the ground deformations approximated by the LiDAR data within the CBD. This may be accounted for, in part, by the fact that the analyses have adopted the upper limit of PGAs recorded across the entire CBD (from the Resthaven strong motion station), whereas the actual ground accelerations at specific locations may be at lower levels. However, a reduction in the ground acceleration does not account for the total difference in assessed and approximated settlements. More rigorous analyses of the data, taking better account of the grain size and shear wave velocity of the liquefiable layers, may reduce the differences observed.

Locations where more precise actual settlement records are available may be useful for comparison with the predicted settlements and allow fine-tuning of the method to suit the Christchurch soils.

It has been noted that particularly high vertical accelerations were recorded at each of the CBD strong motion recorders during the 22 February 2011 event although the very high vertical accelerations were more significant in the hill sites. It is possible that the high vertical accelerations could have resulted in very rapid build-up of excess pore water pressures due to a slap-down effect.

Further more detailed analyses are required to better understand these observations. The information collated for this study is likely to provide valuable data for future research efforts in this area.

Figures 8.2 and 8.3 below depict the variation in estimated liquefaction for different Z factors for a number of the piezocones where the greatest thickness of liquefied materials was assessed (in most cases extending to depths of 15m or more), with the location of the NZS 1170.5 (2004) and current Z factor highlighted (for a site subsoil class D).

It can be seen from Figure 8.2 that for an increase in the PGA from 0.25 to 0.34 (equivalent to a Z factor of 0.22 and 0.30, respectively), the increased thickness of the liquefiable layers is very limited (<10%) for the records with the greatest thickness of liquefied materials (CPT-CBD-81 and CPT-CBD-141), whilst those with the lower overall liquefied thickness increased by around 35%. As shown in Figure 8.3, the equivalent predicted increase in settlements are however less (<5% and <30%, respectively) than the liquefied thickness, as would be expected.

Figure 8.2: Change in estimated thickness of liquefiable layers with increase in Z factor from Z = 0.22 to Z = 0.30 for four example piezocones

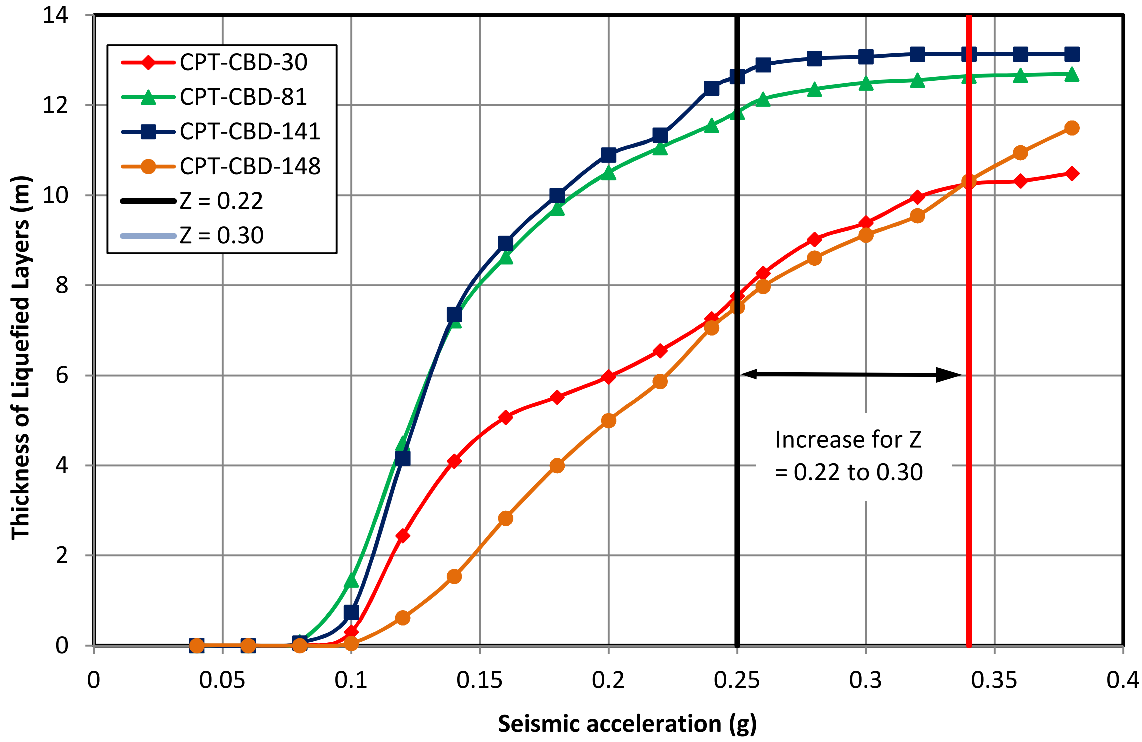
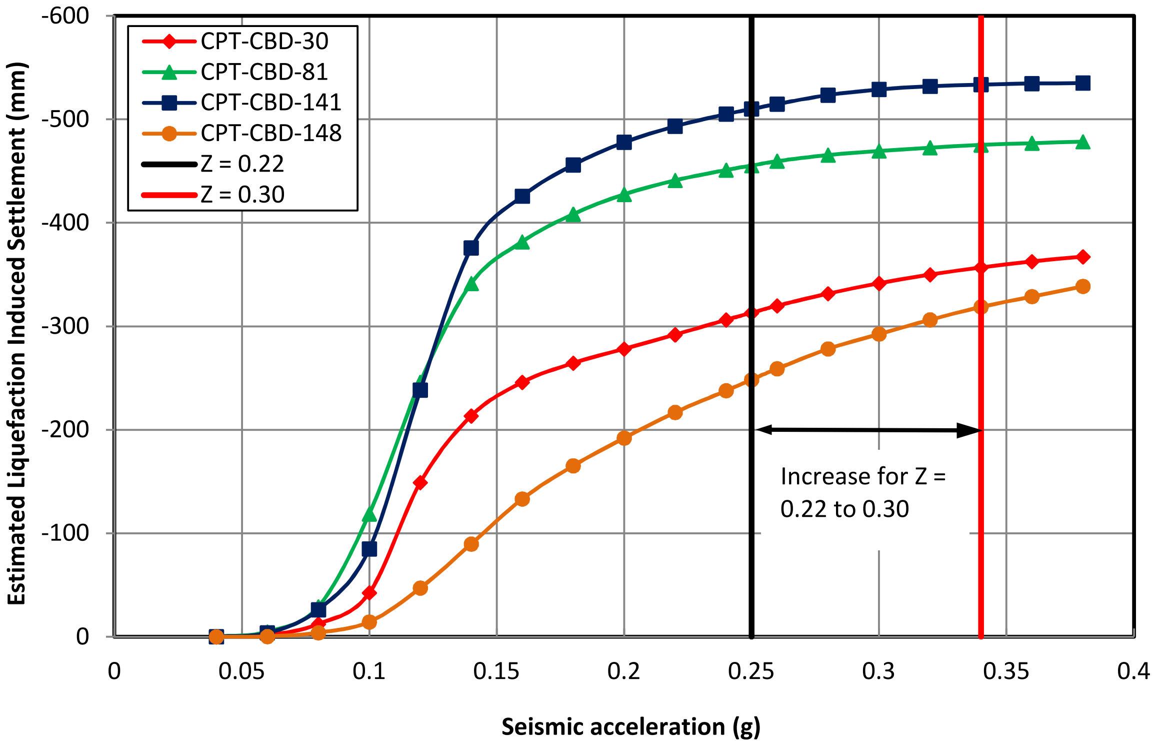


Figure 8.3: Change in estimated liquefaction induced settlements with increase in Z factor from Z = 0.22 to Z = 0.30 for four example piezocones



It is apparent however from inspection of the individual liquefaction analyses plots included in Appendix K, that the zones affected by liquefaction occur at the same depths, with simple widening of the liquefied bands to include more of the medium dense sand layers.

Given the severity of the liquefaction assessed at these locations and the inevitable variability of the ground conditions present and the inaccuracies involved in the analytical methods used in liquefaction prediction (particularly with respect to differential settlements, where a factor of 2 to computed settlements would typically be applied), it is unlikely that the increase in liquefaction hazard resulting from the updated Z factor would modify significantly the mitigation measures adopted for a development at these sites.

It is worth noting, however, that the predicted level of liquefaction for the SLS using the 1170.5 (0.06g) would increase from virtually no liquefaction to as much as 2m thickness of liquefied material at 0.11g (for the revised Z = 0.30 case).

8.4.3 Additional Observations on Specific Ground Conditions

8.4.3.1 Effect of non-liquefied crust

The presence of a thick non-liquefied crust often prevents any surface manifestation of liquefaction. The liquefaction assessments undertaken suggest that some areas of the central city where no liquefaction flooding was reported are likely to have suffered liquefaction of some deep layers. This would appear consistent with the work of Ishihara (1985) and Youd & Garris (1995), which provides a general correlation for predicting whether surface manifestation of liquefaction is likely based on the depth of the non-liquefied crust to the deep liquefied layer thickness.

Generally speaking the magnitude of total and differential settlements in areas where liquefaction was observed to have reached the ground surface, were considerably worse than those areas where no such flooding occurred.

It is worth noting however, that, the presence of a relatively thick non-liquefied 'crust' overlying a liquefied layer at depth may not always have prevented pressurised groundwater and soil being ejected and/or significant deformations of the ground surface. There are a number of locations within the central city where a non-liquefiable plastic silt/clay layer is present overlying liquefiable sands in which significant volumes of material have been ejected to the ground surface. For instance, CPT-CBD-85 located at the junction of Antigua Street / Balfour Terrace, where large volumes of surface ejecta occurred, shows a 4m thick non-liquefied layer overlying deep liquefiable sands. These may however be associated with penetrations of the non-liquefiable crust by services or other loosely backfilled zones.

8.4.3.2 Shallow gravels

It is also worth noting that many places within the CBD and particularly along the western end of Moorhouse Avenue and the commercial areas to the south, significant surface ejecta was observed despite the presence of a shallow medium dense to dense gravel layer. In many places these gravels are overlain by a relatively shallow depth (1 to 2m) of loose saturated sands and therefore it may be assumed that it is merely the sands that are liquefying.

The large volumes of sediment laden groundwater rising to the surface would suggest, however, that either liquefaction of the surface sands is very severe or indeed some liquefaction of the gravel layers is also occurring. It may also be possible that the large volumes of pressurised water emanating from the gravel layers are rising very rapidly due to the high permeability of these materials resulting in strong seepage forces compared to areas underlain by less permeable fine sands.

As a corollary to this, there are also locations where loose saturated sands are present overlying gravels where little or no surface manifestation of liquefaction is apparent. To add to the complexity, there are areas that are underlain almost directly by what appear to be medium dense to dense gravels (i.e. at depths close to the groundwater level), where relatively large volumes of surface ejecta have been observed. At these locations it may be the presence of coarse gravels /cobbles that prevent penetration by the piezocone tip and result in relatively high SPT N blow counts, where in fact the materials may exist in a relatively loose to medium dense state and therefore susceptible to liquefaction.

8.4.4 Lateral Spreading

As detailed in Section 8.2.1, severe liquefaction accompanied with localised lateral spreading was observed at a number of discrete locations along the banks of the Avon River following the 22 February 2011 significant seismic event. The most extensive spreading was located in the mainly residential areas within the river meander immediately west of Fitzgerald Avenue (north of Chester Street) and around Peterborough and Salisbury streets east of Manchester Street. These areas were also badly affected by liquefaction (but with little or no lateral spreading) following the 2010 Darfield Earthquake. Much of the land within the Avon River meander is currently located within the Orange Zone[12]. The future requirements for land in these areas is currently being assessed and will be reported on by Earthquake Commission and is therefore not considered in further detail here.

[12] The area around Peterborough Street and Manchester Street has recently been re-zoned green.

Further upstream, lateral spreading was observed on either side of the river between Colombo and Manchester streets and on the south side of the river immediately west of Colombo Street. We also understand that localised lateral spreading has been identified on the north side of the River Avon immediately west of Colombo Street, following detailed inspections at a specific site. Lateral spreading was also identified along a short section on the south side of the river immediately east of Antigua Street in the western part of the CBD.

It should be noted that the extent of the lateral spreading observed within the CBD is not comparable to that which has occurred along sections of the lower Avon River in the eastern suburbs and Kaiapoi.

A number of survey transects have been completed through various sections of the Avon River by staff from the University of Canterbury. These have suggested lateral displacements of up to 0.7m at the lower reaches of the river within the CBD, with the majority of the movement being confined to the first 20m or so from the edge of the river, with typically less than 100mm cumulative displacement occurring at greater distances from the river. A lateral movement of approximately 40mm was recorded at a distance of nearly 200m at one location, although this may be related to a more localised feature.

The ground conditions at each of these locations are relatively similar, comprising a few metres depth of relatively loose silty sands overlying typically medium dense to dense sandy gravels, which are in turn underlain by typically medium dense to dense gravelly sands and sands. The majority of the lateral movement is therefore anticipated to be confined to the upper few metres. Engineering works to mitigate this spreading hazard are considered achievable.

Clearly any proposed new developments at sites located close to the Avon River will need to undertake a detailed assessment of lateral spreading hazard. In many cases any identified lateral spreading hazard may be mitigated by the appropriate design of foundation systems, possibly incorporating some form of ground treatment. These works can generally be completed within the individual site boundary. However, where there is considered to be a significant risk of lateral spreading over an extended length of the river, coordinated ground treatment options along the river bank may be considered appropriate. Such works offer many advantages and should be explored where possible.

CCC has proposed in the Draft City Plan to include a 30m set-back from the river to create the Avon River Park. This will ensure that no development can take place within the zone likely to be most severely damaged by any future lateral spreading, but also ensures reserve land is available that could potentially be used for ground improvement works to support adjacent land. Council is encouraged to ensure that this land can be made available to potential developers for such purposes and may wish to consider the joint benefits of providing mitigation measures at critical locations to protect council infrastructure.

8.4.5 Impact on Central City

It should be recognised that, apart from a few localised areas, the overall impact of liquefaction and lateral spreading on the central city resulting from the recent seismic events, has not been as severe as that which has occurred in many of the eastern suburbs and Kaiapoi. This is considered to be due to a combination of the generally better ground conditions present, greater land coverage from buildings and heavy pavements, lower groundwater levels and more substantial foundations.

8.5 Future Design Requirements

As indicated in Section 6, it is understood that GNS Science are currently developing contours specifying the required design peak ground accelerations to be used in design within

Christchurch, and this will supersede the updated Z factor detailed in the DBH guidance of May 2011.

This new design requirement could have a significant impact on the extent and distribution of the assumed liquefaction hazard presented in this report and the accompanying plans and sections. Consideration should be given to reviewing the applicability of this report and accompanying plans following release of the GNS Science advice.

9. Principal Geotechnical Considerations

9.1 Purpose

The purpose of this section is to briefly describe the main geotechnical issues that will need to be addressed for the different ground conditions encountered across the central city. These have been simplified into four basic terrains, as follows:

- Sites underlain by significant depth of soft, potentially compressible soils
- sites underlain by materials susceptible to liquefaction to relatively shallow depths underlain by none liquefiable materials
- shallow gravels which may or may not be underlain by deeper liquefiable materials
- Sites where liquefiable materials extend to considerable depth.

At some sites more than one of these ground conditions may exist.

Section 9.6 discusses the nature of the deposits present across the central city in terms of site subsoil classes[13].

9.2 Soft Ground

As indicated in Section 7 and shown on the geological plans and sections, there are a number of quite extensive areas of the central city that are underlain by soft clays and plastic silts, including some very soft organic silt and peat layers. The geotechnical issues to be considered for building in these areas are largely unrelated to seismic effects, with particular issues around low bearing capacity, high compressibility and are often associated with high groundwater levels.

These deposits do, however, often include layers of loose non-plastic silts and fine sand layers, and occasionally, there is a surface layer of liquefiable materials present. Surface manifestation of liquefaction was observed at several locations in these areas, even where a crust of non-liquefiable materials are present.

In addition to the risks of liquefaction of the interbedded materials, the soft, largely normally-consolidated to slightly over-consolidated saturated clays and plastic silts have high natural water contents and liquidity index. This is likely to make them sensitive to cyclic softening, with the resulting loss of strength and large deformations during application of seismic shaking and post-shaking reconsolidation settlements.

In most cases, these deposits rest on medium dense to dense sands, which are not particularly susceptible to liquefaction (or the resulting settlements would not be expected to be large). However, there are some areas where loose silty sands to medium dense sands were encountered beneath the clays and plastic silts. Where these are susceptible to liquefaction, then the combined thickness could extend beyond 10m. This may dictate that the site is classified as being Class E – Very soft soil site, in accordance with the definition provided in NZS 1170.5 (2004) – see Section 9.6.

Where the soft soils are directly underlain by dense to very dense sands, the rapid change in stiffness can cause issues associated with resonance of the overlying structures, which will need to be considered.

It is understood that many of the buildings in these areas are constructed on piles driven to the dense sand layer. Lateral movements occurring during seismic shaking could result in significant damage to the piles, the pile/building connections and possible creation of a void around the pile

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[13] In accordance with 1170.5 (2004).

shaft due to the differential movement of the pile relative to the surrounding ground. This could result in enhanced shaking of the supported structure during subsequent aftershocks.

As detailed in Section 6, the ground conditions in the vicinity of the Resthaven strong motion accelerometer include soft soils to depths of around 6m below ground level, underlain by potentially liquefiable loose sands to around 9m below ground level. The peak ground accelerations recorded at this site were greater than each of the remaining stations located in the CBD during each of the three major seismic events and the Boxing Day aftershock. This is likely to have been a direct consequence of the deep soft ground conditions present and requires that the assumed future design ground motions take account of these conditions.

9.3 Shallow Liquefiable Materials

There are a number of areas within the central city where loose/weak, low-plasticity silts, sands or sandy gravels, which are susceptible to liquefaction, are underlain by a relatively thick layer of non-liquefiable dense to very dense sandy gravels / gravelly sands.

Other areas occur within the city where shallow liquefiable materials are underlain by non-liquefiable clays or plastic silts which in turn rest on relatively dense sands / sandy gravels.

One of the principal geotechnical issues to be considered at these sites will be to limit the impact of any resulting ground deformations, particularly differential settlements that could impact on shallow foundations.

These may be addressed by digging out and replacing the liquefiable materials (may not be feasible in high groundwater areas), constructing a geotextile-reinforced gravel raft with a stiff ground slab, piled foundations or other ground improvement techniques.

Investigations in these areas will need to confirm not only the depth of the near-surface liquefiable layers, but also ensure that the non-liquefiable layer is not underlain by further loose non-plastic silts or sands that could liquefy or soft cohesive materials which could undergo cyclic softening, either of which could result in a punching type failure.

9.4 Shallow Gravels

Where gravels are encountered close to the ground surface, as present at several locations within the central city, the geotechnical issues to be considered are generally less complex. These sites are likely to have reasonably high bearing capacity, low compressibility and typically low risk of liquefaction.

Shallow spread foundations will often be suitable in these areas, even for quite large and heavy structures. The most significant issue for some developments will be temporary works design when basements are required to extend below the water table.

However, some areas where these ground conditions have been encountered were affected by large volumes of liquefaction flooding and ejecta following the 22 February and 13 June 2011 aftershocks. In these areas it is likely that the underlying sandy gravels / gravelly sands are not as dense as suggested by the penetration resistance recorded by piezocones and SPTs, resulting from the presence of coarse particles.

Whilst it is generally considered that stiff foundations resting on the sandy gravels would unlikely be severely affected by the near-surface liquefaction and associated land deformation, the impact on site infrastructure, including access roads, car parks and buried services, could cause significant disruption to the operation of these buildings.

Where the sandy gravels are underlain by loose to medium dense sands or silts that could liquefy, there is a high risk of differential settlement of the overlying gravels and supported structures founded within these materials.

In the majority of cases the thickness of the gravels is likely to be sufficient such that a stiff foundation can be designed to accommodate localised differential settlements. There will however be areas towards the edge of the sandy gravel deposits where this is not the case and deeper foundations and/or ground improvement will be required.

Ground investigations in these areas will need to be carefully planned and monitored to ensure that an accurate assessment of the density of the sandy gravels is obtained. This may not be possible using either CPTs or SPTs, particularly if the coarse gravels or cobbles are abundant.

Penetrating through the gravel layer to investigate the materials at depth can only be realistically achieved with boreholes and pre-drilling for CPTs. However, the gravels often contain layers of relatively loose sand which is likely to be susceptible to liquefaction. These sand layers within the coarse gravels can be very difficult to sample, as experienced during these investigations.

Careful monitoring of the drilling is required to identify where the drill bit is encountering weaker materials and undertake SPTs at these locations, rather than maintaining a standard vertical spacing. Where good results cannot be obtained then it may be prudent to assume that zones of core loss are likely to represent loose potentially liquefiable sands.

9.5 Deep Liquefiable Materials

There are a few locations within the central city that are underlain by loose silty sands and sands extending to considerable depths, which are particularly susceptible to liquefaction. These areas include the land between Kilmore Street and Peterborough Street and in the south-east area of the central city, where very large volumes of groundwater, sand and silt were released to the surface and accompanied by large ground deformations (settlement).

Significant ground improvement works will be required in these areas or deep piled foundations considered. These will not significantly reduce the amount of liquefaction or resulting settlements, and therefore buildings could be left standing above the settled ground following a large earthquake. Such designs will also need to take account of negative skin friction forces acting on pile shafts.

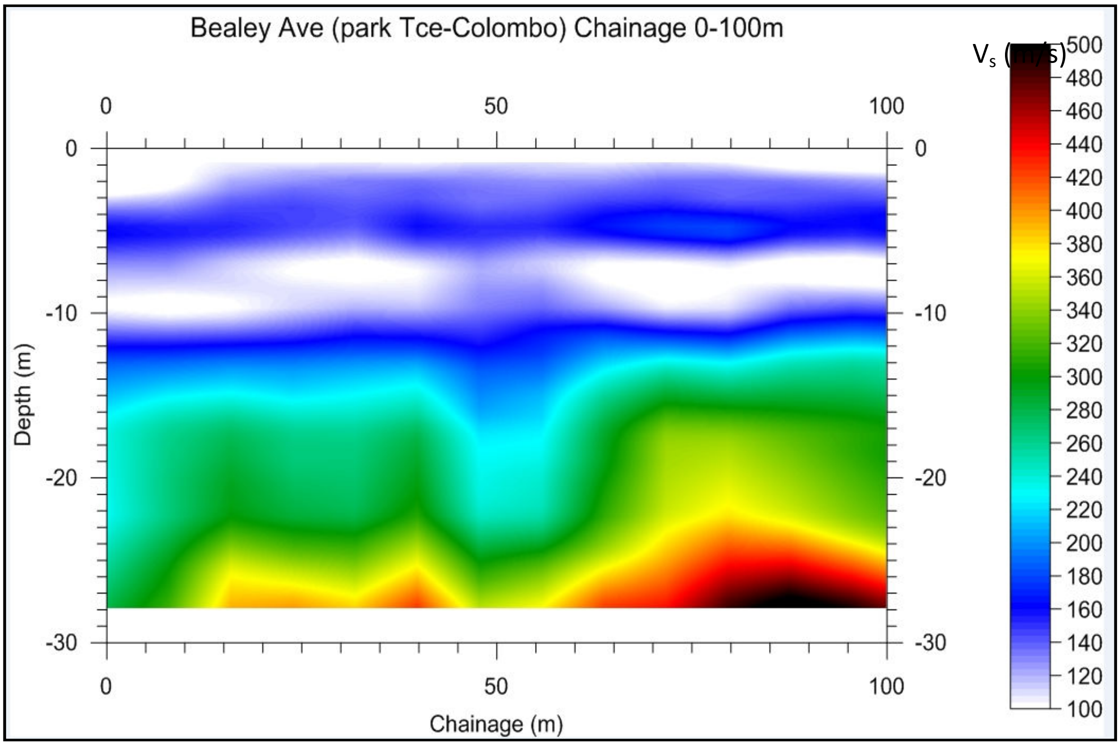
9.6 Site Subsoil Class

The investigations completed, including the borehole SPTs piezocone tip resistance and shear wave velocities from the geophysical surveys, indicate that most sites within the central city are likely to be assigned a site subsoil Class D – Deep or soft soil site, in accordance with the requirements of Section 3 of NZS 1170.5 (2004). This will need to be confirmed by site specific assessments.

As detailed in Sections 9.2 and 9.5, however, there is a possibility that some sites may be more appropriately assigned as Class E – very soft soil sites. This may be true where the near-surface materials comprise soft clays and high-plasticity silts, and where the depth of liquefaction is significant (>10m), or a combination of the two.

By way of an example, a plot of the shear wave velocity with depth at the junction of Bealey Avenue and Harper Avenue / Park Terrace is shown below (increasing chainage is moving east along Bealey Avenue). This indicates materials to depths in excess of 10m with shear wave velocities less than 150m/s.

Figure 9.1: MASW plot east along Bealey Avenue from Park Terrace indicating low shear wave velocities to depths in excess of 10m



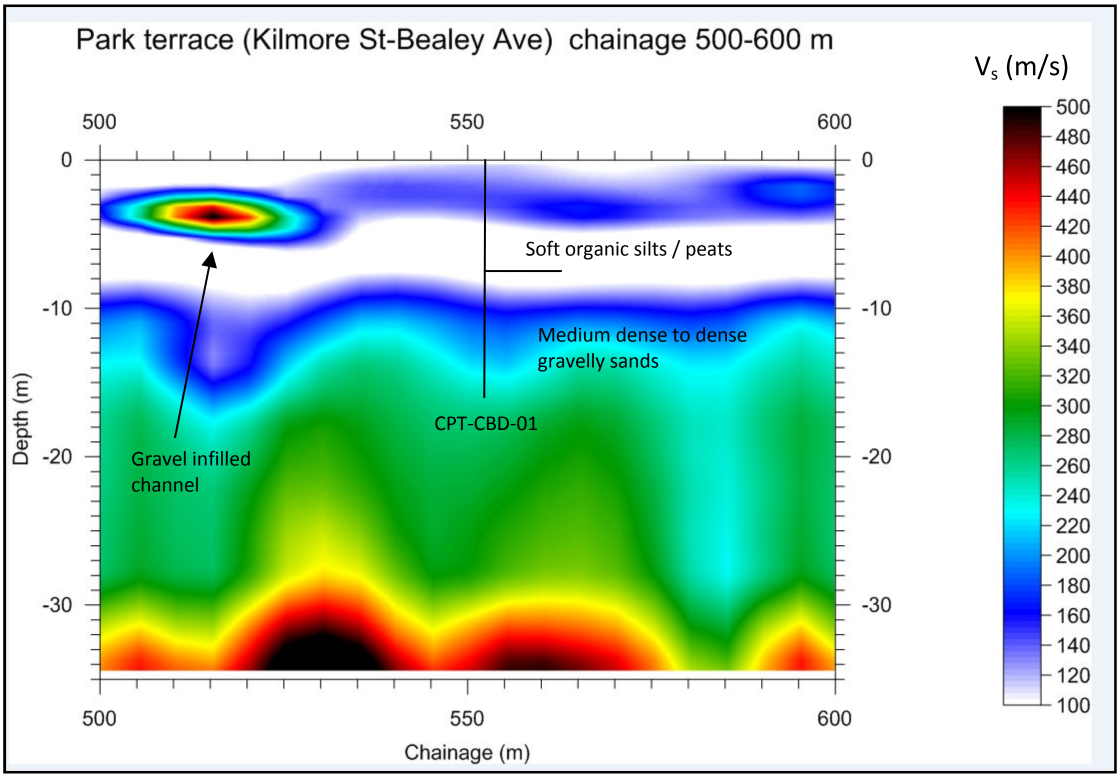
The nearest exploratory holes to this location include BH-CBD-01/CPT-CBD-01, located a short distance along Park Terrace. Both exploratory holes encountered medium dense to dense sands below the organic silt and peat layers at depths of around 7.5m.

Figure 9.2 shows the approximate location of the boreholes relative to the MASW survey and the depths of this interface. Comparison of these records indicates that the change from very low shear wave velocities associated with the soft organic silts and very loose / weak interbedded sands and silts and the higher velocities associated with the underlying medium dense to dense sands is relatively well defined, but the depth of the low shear wave velocities may be slightly over-estimated (by 1 or 2m) as a result of the ‘averaging’ of the data.

Note that what appears to be a gravel filled channel is also shown in Figure 9.2. Care would be needed to not mistake such a feature for the surface of deep gravels suitable for foundations, which could easily be mis-interpreted from a limited number of shallow investigations.

In these cases, the detrimental effect of the soft soils on the response of structures will need to be considered, but it should be recognised that this is a complex issue to resolve and will require close interaction of geotechnical specialists and structural engineers.

Figure 9.2: MASW plot north along Park Terrace indicating low shear wave velocities and results for CPT-CBD-01



9.7 Fault Surface Rupture

Information regarding the presence of active faults directly beneath the central city is very limited at this time. We understand that GNS Science may be undertaking further deep seismic surveys to assess this risk in the near future. Clearly the impact of a surface rupture occurring, of a similar extent / magnitude as occurred along sections of the Darfield fault during the 04 September 2010 earthquake could have very severe consequences on structures built in the central city.

10. Requirements for Site Specific Ground Investigations and Geotechnical Assessments

10.1 General

The factual ground investigation data presented in the zone reports and the interpretative information presented here provides a broad overview of the nature and variability of the ground and groundwater conditions and associated geotechnical hazards present within the central city. This information may be used by geotechnical specialists to assist with the preliminary stages of undertaking site specific assessments for individual developments. This may include developing concept foundation / ground improvement designs and to help determine the appropriate scope of the ground investigations and geotechnical assessment required to advance the development through to detailed design.

It is not intended that this information be used for detailed geotechnical analyses or for justification of proposed foundation / ground improvement designs and should only be referenced in support of site specific ground investigations / geotechnical assessments.

Experienced and suitably qualified engineering geologist and geotechnical engineers are well aware of the benefits of undertaking comprehensive ground investigations and geotechnical assessments. Detailed investigations reduce the risks of unforeseen ground conditions, which are amongst the most common cause for claims during construction. By eliminating some of the uncertainties inherent with limited investigations, significant cost savings may be possible by allowing less conservative design assumptions.

The following sections provide an indication of the scope of ground investigations and geotechnical assessments that are may be required to adequately address the geotechnical issues for sites with a high liquefaction / lateral spreading hazard. This is provided primarily to help inform landowners and other non-geotechnical specialists of what is likely to be an appropriate level of investigation and assessment and to provide guidance to CCC on the scope of the geotechnical information to be submitted in support of building consent applications for difficult sites.

Whilst large areas of the central city are located on deposits susceptible to liquefaction during a ULS event, as shown on the liquefaction hazard sections and plans, the impact of liquefaction occurring within some of these layers, particularly at depth, may be very limited. Careful design of foundations for static conditions alone may be sufficient to provide an adequate level of protection during a ULS event.

For relatively straightforward sites where there is not considered to be a high risk of significant deep liquefaction occurring then a reduced level of investigation than outlined below may be appropriate. This assessment should be based on sound engineering judgement made by a suitably qualified geotechnical specialist.

10.2 Scope of the Geotechnical Assessment

The scope of the ground investigation and geotechnical assessment required for a specific development is a function of both the proposed structure and the anticipated site conditions.

A review of the site conditions at any location within the central city will need to incorporate not only the natural deposits and groundwater conditions, but also take account of the historic uses of the site. Particular reference is required to the potential for existing fill, former building foundations and/or ground treatment, and the level of land damage that occurred during the Canterbury Earthquake Sequence.

The general approach to the geotechnical assessment for high risk sites will therefore include a number of stages, which may include some, or all, of the following:

1. Desk Study - to collate all readily available existing information pertaining to the site, including historic uses (from maps, aerial photographs), ground investigation data / geotechnical reports, foundation plans, observed land damage reported following the various large earthquakes and information regarding the foundation layout and loads for the proposed development.
2. Site Inspection – undertaken by a suitably experienced geotechnical specialist, to assess the general layout of the site and presence of existing structures / hazards, the level of land damage (where still evident), and to identify locations for subsequent ground investigation works.
3. Design, procurement and management of a detailed ground investigation. This will include:
    a. obtaining of consents to complete the intrusive works
    b. identification of buried and overhead services
    c. preparation of traffic management plans (TMPs), where required
    d. engaging ground investigation contractors
    e. preparation and review of health & safety plans to undertake the investigations
    f. site identification of buried services and excavation of inspection pits, where necessary
    g. site supervision of the physical investigations
    h. groundwater level monitoring
    i. logging of borehole cores / trial pits by suitably qualified engineering geologist, geotechnical engineers or experienced geo-technicians
    j. laboratory testing of selected samples
    k. factual reporting of the field investigations and laboratory test results
4. Review of the site conditions revealed from the desk study and investigation phases.
5. Geotechnical analyses of the data.
6. Preparation of a Geotechnical Assessment Report.
7. Detailed Design Report.

This level of investigation and assessment would not be necessary for small developments or on sites unlikely to be affected by severe liquefaction.

10.2.1 Ground Investigations

The ground conditions within Christchurch central city are known to be highly variable on a local, as well as a regional scale, as a result of the geomorphological / geological environment. There exists a risk that the ground conditions at any specific site may show considerable lateral and/or vertical variability, for all but the smallest of sites.

To limit the number of exploratory holes required to provide a reasonable assessment of the potential variability present within the site, geophysical survey techniques, such as MASW, are likely to prove an efficient technique. There are benefits to completing the geophysical surveys early on in the investigation programme, as these may indicate significant variability or anomalous features. Subsequent exploratory holes can then be positioned to investigate the variable conditions indicated.

Each site will be different and the precise scope of the ground investigation will vary. However, to provide an indication of the quantum of investigation works that would be required, we have assumed two specific scenarios, one comprising a proposed three- to five-storey office building, and a single level tilt slab warehouse. It is assumed that both sites are underlain by a significant depth of potentially liquefiable materials to the Riccarton Gravels.

The proposed scope of the ground investigations detailed should be sufficient to adequately identify the nature and potential variability of the ground conditions across the site. This level of investigation should be adequate to advance the detailed design of the foundations (and ground treatment, if necessary) for the proposed development with little or no requirement for further investigations, other than that which may be necessary during construction. However, if highly variable conditions are encountered during this phase of investigations, subsequent stages comprising further investigation points and/or other techniques may be deemed necessary or beneficial.

➢ **Five Storey Office Building** (footprint approximately 1,000m^2)

- 2 or 3 machine boreholes advanced to depths of between 20 and 30m (with SPTs at 1.0m vertical centres reducing to 1.5m below 10m depth)
- 8 to 10 piezocones to depths of up to approximately 20m (pre-drilling to permit testing of the soils beneath an upper dense gravel layer may be required if the boreholes and/or geophysical surveys indicate potentially liquefiable materials at depth)
- 3 to 5 machine excavated trial pits to depths of around 2 to 4m[14]
- Geophysical surveying (such as MASW) to estimate the shear wave velocity to a depth of approximately 20 to 30m, and identify variability between intrusive investigation points
- Inclusion of at least one standpipe installed to a depth of around 10m with a sand backfill and lockable flush cover, with a number of return visits to record the groundwater level variability
- A single standpipe piezometer installed within the artesian / sub-artesian gravel aquifer (Riccarton Gravels).This would only be necessary where it was considered likely that deep piled foundations and/or ground treatment to these depths was envisaged
- Laboratory testing on selected samples collected from the boreholes and/or trial pits, including plasticity limits and gradings. Where shallow foundations are being considered in areas underlain by compressible materials (such as organic silts and peat, which are common in many areas of the central city), natural water content and consolidation testing may also be required.

➢ **Single level tilt slab warehouse** (footprint approximately 5,000m^2)

- 3 machine boreholes advanced to depths of between 20 and 30m (with SPTs at 1.0m vertical centres reducing to 1.5m below 10m depth)
- 15 to 20 piezocones to depths of up to approximately 20m (pre-drilling to permit testing of the soils beneath an upper dense gravel layer may be required if the boreholes and/or geophysical surveys indicate potentially liquefiable materials at depth)
- 8 to 10 machine excavated trial pits to depths of around 2 to 4m
- Geophysical surveying (such as MASW) to estimate the shear wave velocity to a depth of approximately 20 to 30m, and identify variability between intrusive investigation points

[14] Trial pits are primarily required to provide an indication of the near-surface ground conditions, particularly where fill materials are suspected, to aid the design of ground slabs / shallow foundations and for evaluating potential construction issues and temporary works design requirements. These may not be required at all sites.

- Inclusion of at least one standpipe installed to a depth of around 10m with a sand backfill and lockable flush cover with a number of return visits to record the groundwater level variability
- A single standpipe piezometer installed within the artesian / sub-artesian gravel aquifer (Riccarton Gravels).This would only be necessary where it was considered likely that deep piled foundations and/or ground treatment to these depths was envisaged
- Laboratory testing on selected samples collected from the boreholes and/or trial pits, including plasticity limits and gradings. Where shallow foundations are being considered in areas underlain by compressible materials (such as organic silts and peat, which are common in many areas of the central city), natural water content and consolidation testing may also be required

On sites where there have been a number of previous buildings or structures, buried foundations or poorly backfilled excavations may be present. Backfilled channels may be present (as suggested by the 'Black Maps' or other sources) or buried tree stumps in former swampy areas. Ground penetrating radar may be beneficial on such sites.

10.2.2 Analyses and Reporting

10.2.2.1 Desk Study

Depending upon the size and complexity of the site and the associated ground conditions, preparation of a formal Geotechnical Desk Study Report may be advantageous. For many sites, however, it will be sufficient to utilise the desk study information to design the ground investigation and the present the results in the subsequent Geotechnical Interpretative Report.

10.2.2.2 Factual Data

The results of the ground investigation should ideally be presented in a standalone factual report, or for smaller sites, may be included in the appendices to the Geotechnical Assessment Report. Preparation of a separate factual report will be preferable if the information is to be subsequently included on the geotechnical database discussed in Section 4.5.

10.2.2.3 Analyses

Following the desk study and ground investigation, a conceptual ground model should be developed and geotechnical analyses undertaken, which is likely to include, as a minimum:

- geotechnical material parameters to be adopted for design
- liquefaction and, where necessary, lateral spreading hazard assessments
- assessment of the potential for cyclic softening of 'clay-like' soils and consolidation settlements under static loads, where soft/compressible materials are encountered
- Preliminary foundation and/or ground treatment design options, considering the soil-foundation-structure interaction under static and seismic loading.

10.2.2.4 Geotechnical Assessment Report

This will lead to the preparation of the Geotechnical Assessment Report, presenting a summary of the works undertaken, an interpretation of the site conditions encountered (conceptual geological model). This will include a discussion on the principal geotechnical issues identified, including:

- options/conclusions regarding the future development of the site
- requirements for further investigations/assessment (if necessary)

- preliminary foundation and/or ground treatment options
- construction issues / temporary works design.

10.2.2.5 Design Report

If the development then proceeds to detailed design stage, a detailed Design Report should be prepared for the preferred design option and this should be submitted, along with the desk study, factual and interpretative reports, in support of a building consent application. The Design Report should reference, and be consistent with, the conclusions of the Geotechnical Assessment Report and Factual Report. This should also provide an outline of the site inspections and testing that will be required during construction to confirm the design assumptions, where applicable.

All reports should be compiled by, reviewed and authorised by suitably qualified geotechnical specialists.

10.2.2.6 Peer Review

The deep alluvial deposits present within Christchurch central city combined with the seismic hazard dictate that the issues surrounding geotechnical design and the combination of ground-foundation-structure interaction are very complex. It is therefore considered prudent for peer reviews to be included for significant or complex sites.

Early involvement of peer reviewers is generally considered more effective and efficient than the peer review being completed once the design and reports have been finalised and submitted for building consent.

Clients and their appointed advisors are therefore to be encouraged to be proactive at identifying where a peer review is beneficial or necessary, and to commence this process at an early stage. This may start as early as agreeing the appropriate scope of the ground investigation works required, as the peer reviewers knowledge of the local site conditions may be of assistance even at this early stage.

CCC are also encouraged to advise landowners / developers as early as possible where they consider that a peer review will be required.

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