Foundations on Deep Alluvial Soils

Technical Report Prepared for the Canterbury Earthquakes Royal Commission

Misko Cubrinovski
University of Canterbury, Christchurch

Ian McCahon
Geotech Consulting Ltd, Christchurch

August 2011
University of Canterbury, Christchurch

Executive Summary

The series of earthquakes that hit Christchurch in the period between 4 September 2010 and 13 June 2011 caused repeated liquefaction through its suburbs and the Central Business District (CBD). The 22 February earthquake was the most damaging. The liquefaction in the CBD adversely affected the performance of many buildings resulting in residual deformation and damage to buildings. This report presents a general review of the alluvial soils found in the CBD, and identifies the general concepts that should be followed in the design of foundations for buildings on these soils.

The Canterbury Plains are built of complex inter-layered soil formations deposited by eastward-flowing rivers from the Southern Alps towards the Pacific coast. In the top 20 m to 25 m, the CBD soils consist of recent alluvial soils including gravels, sands, silts, peat and their mixtures. The soils are highly variable within relatively short distances, both horizontally and vertically. Considering their composition (sandy soils and non-plastic silts), age (recent deposits, few hundreds to a few thousand years old) and depositional environment (river, swamp and marine sediments), these soils are generally considered susceptible to liquefaction, and in some cases (when deposited in a loose state) they exhibit very low resistance to liquefaction.

The principal zone of liquefaction (due to the 22 February earthquake) stretching west-east along the Avon River affected several high-rise buildings in different ways. Buildings on shallow foundations, supported on loose to medium-dense sands and silty sands that liquefied, suffered differential settlements, residual tilts, and bearing capacity failures (sinking of the building in the soil). Pile supported structures, particularly when the piles reached competent soils at depth, generally showed less differential and residual movements. There is evidence that hybrid building foundations (consisting of shallow and deep foundations or piles of different lengths) performed relatively poorly during the earthquakes. Multi-storey and high-rise buildings supported on shallow foundations sitting on shallow gravels showed mixed performance. The variable thickness of the gravel layer and underlying soil layers contributed to uneven settlements and residual deformations. These adverse effects were particularly pronounced in transition zones where ground conditions and behaviour change substantially over short distances, including zones of marked ground weakness and lateral spreading.

Robust shallow foundations, often accompanied by land improvement measures, and deep foundations reaching competent foundations soils at large depths are appropriate for founding buildings on deep alluvial soils. These types of foundations have shown to provide an improved and acceptable performance during strong earthquakes. Attention to details in the design and due considerations of the soil-foundation-structure system as a whole are essential for ensuring a satisfactory performance during strong earthquakes. The design process has to be supported and based upon results of appropriate field investigations, the extent and nature of which will depend on the particular features of the site and requirements of the building considered. The report indentifies some general concepts that should be followed in the design of foundations for buildings on alluvial soils in relation to the observed performance during the 2010-2011 earthquakes and the current seismic design philosophy.

1 Introduction

In the period between September 2010 and June 2011, the City of Christchurch was shaken by a series of strong earthquakes including the 4 September 2010, 26 December 2010, 22 February 2011 and 13 June 2011 earthquakes. These earthquakes produced very strong ground motions throughout the suburbs of Christchurch and in its Central Business District (CBD) causing substantial damage to buildings, infrastructure and lifelines, and an enormous impact on the community as a whole. The 22 February earthquake was particularly devastating; it caused 181 fatalities, collapse or partial collapse of many unreinforced masonry structures, collapse of two multi-storey reinforced concrete buildings, and widespread liquefaction in the suburbs to the east of the CBD and within the CBD itself. Soil liquefaction in a substantial part of the CBD adversely affected the performance of many multi-story buildings resulting in total and differential settlements, lateral movement of foundations, tilt of buildings, and bearing failures. At the outset, we have to put these unfortunate outcomes in the context of the very strong ground shaking produced by the February earthquake. The ground motions generated by this earthquake in many parts of Christchurch were intense and substantially above the ground motions used to design the buildings in Christchurch.

With this background in mind, this report provides a general review of the alluvial soils found in the Christchurch Central Business District and focuses on their performance and effects to CBD building foundations during the recent strong earthquakes. Typical modes of failure for such soils are discussed, and methods of founding buildings on such soils that would avoid such failures are outlined. A comparison between the liquefaction observed in the recent earthquakes and the anticipated liquefaction during an Alpine Fault event is also presented. The report contains technical information, however, when possible the phenomena and their effects are described in non-technical language for a general audience. We hope that the readers will not be hugely inconvenienced one way or another by the adopted approach, and that the report will offer information to a wide readership, while preserving but not imposing the technical detail.

2 General Characteristics of Seismic Response of Deep Alluvial Soils

Deep alluvial soils influence the performance of land, infrastructure, and buildings during strong earthquakes in two profound ways. As seismic waves propagate through the alluvial soils, from the base rock towards the ground surface, the alluvial soils significantly modify the characteristics of ground shaking. They amplify the shaking and seismic forces for some structures, while for others they reduce or de-amplify the shaking. The composition of alluvial soils, their stratification, thickness and stiffness (resistance to deformation) define the particular features of the modification of the ground motion. In addition, as seismic waves pass through the soils, they deform the soils producing both transient deformations (temporary displacements) and permanent movements and deformations (residual horizontal and vertical displacements, ground distortion, undulation of ground surface, ground cracks and fissures). In cases when the ground deformation is excessive and seriously affecting the performance of land or structures, the soils are considered to have ‘failed’. Thus, soil failure does not necessarily imply a catastrophic collapse, but rather implies excessive deformations that are not tolerable for structures. Soil liquefaction is one form of such failure since it usually results in excessive ground deformation and displacement that severely affects the built environment.

2.1 Soil Liquefaction and Lateral Spreading

Soil liquefaction is a process in which over a very short period of time (several seconds or tens of seconds) during strong ground shaking, the soil transforms from its normal solid state into a heavy liquid mass. As a consequence of liquefaction, the soil essentially loses its strength and bearing capacity (i.e. the capacity to support gravity loads of heavy structures), thus causing sinking of heavy structures into the ground. Conversely, light and buoyant structures (that have smaller mass density than the liquefied soil mass) will be uplifted and float above the surface. Ground deformation associated with liquefaction takes various forms and is often excessive, non-uniform and involves large permanent vertical displacements (settlement) and lateral displacements commonly resulting in large cracks and fissures in the ground, substantial ground distortion and sand/silt/water ejecta covering the ground surface. The large pressures created in the groundwater during liquefaction are in excess of the equilibrium pressures, thus triggering flow of water towards the ground surface. Since these water pressures are very high, the water will carry a significant amount of soil on its way towards the ground surface and eject this on the ground surface. This process inevitably leads to loosening of some parts of the foundation soils and often results in creation of local ‘collapse zones’, sinkholes and ‘vents’ for pore pressure dissipation and flow of pore water.

Lateral spreading is a particular form of land movement associated with liquefaction that produces very large lateral ground displacements from tens of centimetres to several metres, and hence, is very damaging for buildings and infrastructure. Lateral spreading typically occurs in sloping ground or level ground close to water ways (e.g. river banks, streams, in the backfills behind quay walls). Even a very gentle slope in the ground (of several degrees) will create a bias in the cyclic loads acting on the soil mass during earthquakes which will drive the soil to move in the down-slope direction. If the underlying soils liquefy then the liquefied soil mass (‘heavy liquid’) will naturally move down-slope and will continue this movement until equilibrium is re-established (or resisting forces reach the level of driving forces). In areas of Christchurch and Kaiapoi affected by lateral spreading in the 2010 and 2011 earthquakes, the residual slope of the land affected by spreading was often very small (only 2-3 degrees) indicating very low residual strength of the liquefied soils. The process of spreading in backfills behind retaining walls is similar, with large ground shaking first displacing the retaining structure outwards (e.g. towards a waterway), which is then followed by lateral spreading in the backfills.

Liquefaction induces very large strains (i.e. the decrease in the thickness of a soil layer divided by its original thickness, which defines the relative deformation within the soil), typically on the order of several percent. Hence, if for example a 10 m thick layer liquefies, the horizontal displacement of the top of the layer (e.g. at the ground surface) relative to its base (10 m depth) could be in the order of 50-60 cm of cyclic (i.e. back-and-forth) movement during the shaking. A buried structure, including piled foundations through the liquefied layer will be subjected to very large and non-uniform lateral loads from these ground movements and oscillation of the building. There are two particular locations where damage to piles in liquefied soils typically occurs: near the pile top and at the interface between the liquefied soil and underlying unliquefied soil. In some cases, this interface is at large depth, and hence, it imposes serious constraints in firstly identifying if there has been damage caused by the earthquake, and then in repairing or strengthening of the piles, if required. The large ground distortion and highly non-uniform displacements caused by liquefaction often result in stretching of the ground beneath the footprint of the building imposing large loads and damage to shallow foundations if they are not strong enough to resists such forces. Substantial total settlements, differential settlements and tilt of buildings are common consequences of soil liquefaction.

All of the above features and modes of ground deformation are present and very pronounced in the case of lateral spreading. As the ground spreads laterally in one direction, it loads the foundation permanently in this direction in addition to the cyclic transient loadings. Thus, there is a biased push of the foundation in the direction of the spread in addition to the cyclic ground movements. The biased loads associated with spreading are particularly dangerous because they ‘test’ the ductility of structures and their capacity to sustain large deformation without failure or collapse.

The significant softening of the soils due to liquefaction causes filtering out (removal) of the high frequencies of the ground motion, but also amplification of the long-period components of the shaking, resulting in elongated oscillation cycles at liquefied sites. Finally, one should recognize that soil liquefaction is just one form of geotechnical earthquake hazard, in addition to the other more prevalent earthquake hazard, i.e., the ground shaking itself.

2.2 Mechanism Causing Liquefaction

Soil liquefaction occurs in granular soils such as sands, gravels, non-plastic silts and their mixtures. These soils derive their stiffness and strength through grain-to-grain contact stresses. Shallow soils have small grain-to-grain contact stresses, so they are relatively soft and weak. Soils at great depth have large grain-to-grain contact stresses so they are relatively stiff and strong.

When subjected to shaking (straining), granular soils tend to densify or reduce the size of the voids within their granular structure. However, if the soils are fully saturated, i.e. the voids are completely filled with water, then this tendency for densification cannot materialize over a very short period of time (several seconds or tens of seconds of strong shaking) since the water and solid particles are practically incompressible. Instead, this tendency for densification will result in an increase in the pressure in the groundwater (pore water pressure). Liquefaction occurs when the increase in the pore water pressure will reach a level which will effectively cancel out the gravity forces and will essentially separate the particles from each other. The loss of contact between the particles effectively turns the soil into a heavy liquid state, and soil liquefaction results in nearly complete loss of stiffness and strength of soils.

The additional pressures generated in the groundwater (termed excess pore water pressures), increase with depth, and are in excess of the equilibrium pressures under gravity loads. Hence, redistribution of pressures and flow of groundwater is triggered immediately at the onset of liquefaction, resulting in upward flow of water from the high excess pressures at larger depths towards the zero pore pressures at the ground surface. This is why soon after the triggering of liquefaction, water and soil mixture start spurting and littering the ground surface.

Loose soils have more voids in their inherent structure (since they were not well compacted when deposited). Hence, when shaken, they show large tendency for densification (contraction) which in turn leads to rapid pore water pressure build-up and eventual liquefaction in only few cycles of strong shaking. Since these soils are loosely packed and are highly deformable (compressible), liquefaction will be severely manifested and will result in very large ground movements and nearly complete loss of load carrying capacity. This is why loose soils are particularly prone to liquefaction and show very severe consequences of liquefaction. Conversely, very dense soils show very limited tendency for densification and hence produce low excess pore water pressures, and therefore they have much higher liquefaction resistance.

Clays, clayey soils and plastic soils in general, derive stiffness and strength from an additional mechanism (cohesion) and hence are considered non-liquefiable. Softening of these soils and large deformation especially of soft clays and peat can produce severe ground deformation and impacts on buildings and infrastructure, but their response mechanism is different from the soil liquefaction outlined above.

2.3 Liquefaction Assessment

The conventional method (state-of-the-practice) for liquefaction assessment involves the following evaluation steps.

(1) *Liquefaction susceptibility*: In this step, based on the grain-size composition and plasticity of soils, it is determined whether the soils at the site in questions are liquefiable or not. If the soils are deemed non-liquefiable, then further liquefaction evaluation is not required (Bray and Sancio, 2006; Idriss and Boulanger, 2008; NZGS, 2010).

(2) *Liquefaction triggering*: If the soils (or some of the layers) are liquefiable, then a triggering analysis is conducted to determine whether (and which) soil layers are going to liquefy when shaken by a particular ground motion (the design earthquake) specified in terms of peak ground acceleration (PGA) and earthquake magnitude (M_w). In this analysis step, a factor of safety against triggering of liquefaction is calculated as a ratio of the liquefaction resistance (cyclic strength of the soil, or resistance capacity) and cyclic stresses in the soil induced by the design earthquake (seismic load/demand) (Youd et al., 2001; Seed and Idriss, 1982; Idriss and Boulanger, 2008). In the simplified procedure, the peak ground acceleration (PGA) is used as a measure for the amplitude of

ground shaking while the earthquake (moment) magnitude (M_w) is used as a proxy for the duration of shaking (i.e. number of significant stress cycles).

(3) *Liquefaction-induced ground deformation*: In this step, consequences of liquefaction in terms of ground displacements/deformation are estimated for a free field (land not affected by structures or built environment) level ground or sloping ground conditions. Using the computed factor of safety and estimated thickness of the liquefied soils in the triggering analysis, liquefaction-induced settlements and lateral ground displacements are calculated using empirical methods (e.g. Ishihara and Yoshimine, 1992; Tokimatsu and Seed, 1987; Tokimatsu and Asaka, 1998). Similar approaches are used for estimating lateral ground displacements due to spreading (e.g., Youd et al., 2002).

(4) Impacts of liquefaction on building foundations: Using the ground displacements and loads estimated in the previous step, the impacts of liquefaction on building foundations are then analyzed. This includes calculation of loads acting on the foundations, displacement and deformation of the foundations, as well as estimating the resulting damage to the foundation.

(5) Countermeasures against liquefaction: In the final step of the assessment, countermeasures against liquefaction are considered either to prevent the occurrence of liquefaction or to reduce its impacts on ground deformation and foundations, and bring their seismic performance within tolerable limits. Ground improvement and foundation strengthening are the two principal mechanisms used as countermeasures against liquefaction.

More details of the liquefaction evaluation procedure and further references are given in NZGS (2010).

3 Christchurch CBD Soils

3.1 Regional and Local Geology

The City of Christchurch is located on deep alluvial soils of the Canterbury Plains, except for its southern edge, which is located on the slopes of the Port Hills of Banks Peninsula, the eroded remnant of the extinct Lyttelton Volcano. The river floodplain, Pacific coastline, and the Port Hills are the dominant geomorphic features of the Christchurch urban area (Figure 1).

The Canterbury Plains are built of complex inter-layered soils deposited by eastward-flowing rivers from the Southern Alps into the Pegasus Bay and Canterbury Bight on the Pacific coast. The plains cover an area approximately 50 km wide by 160 km long, and consist of very thick soil deposits. At Christchurch, surface postglacial sediments have a thickness between 15 m and 40 m and overlie 300 m to 500 m thick sequence of gravel formations interbedded with sand, silt, clay and peat layers. As illustrated in Figure 2 (Brown and Webber, 1992), these inter-layered formations of gravels and fine-grained soils form a system of gravel aquifers, with artesian water pressures (elevated groundwater pressures). Both the deep alluvial deposits and the presence of aquifers are important features influencing the ground shaking during earthquakes and foundations of CBD buildings.

The shallow soils, in particular the top 20 m of the deposit, are the most important for foundations of multi-storey buildings and liquefaction evaluation. In Christchurch, these surface sediments comprise alluvial gravels, sands and silts (so-called Springston formation, which is dominant in the western part of Christchurch) or estuarine, lagoon, beach, dune, and coastal swamp deposits of sand, silt, clay, and peat (Christchurch formation, predominant in the eastern suburbs). These surface soils overlie the Riccarton Gravel, shown in Figure 2, which is the uppermost gravel of an older age (14,000 – 70,000 years old) and also the topmost aquifer with artesian pressures. The

Figure 1. The City of Christchurch (Google Earth, 2011)

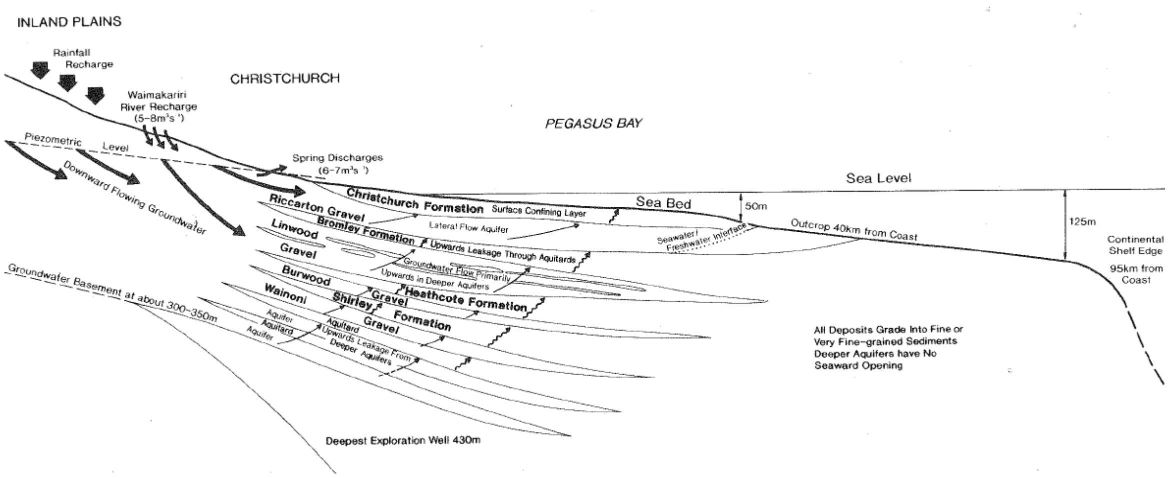


Figure 2. Section through Canterbury Plains showing a sequence of gravel formations inter-layered with clay, silt, sand and peat soils; the complex inter-layering forms a system of aquifers with artesian pore pressures and offshore discharge (Brown and Weeber, 1992)

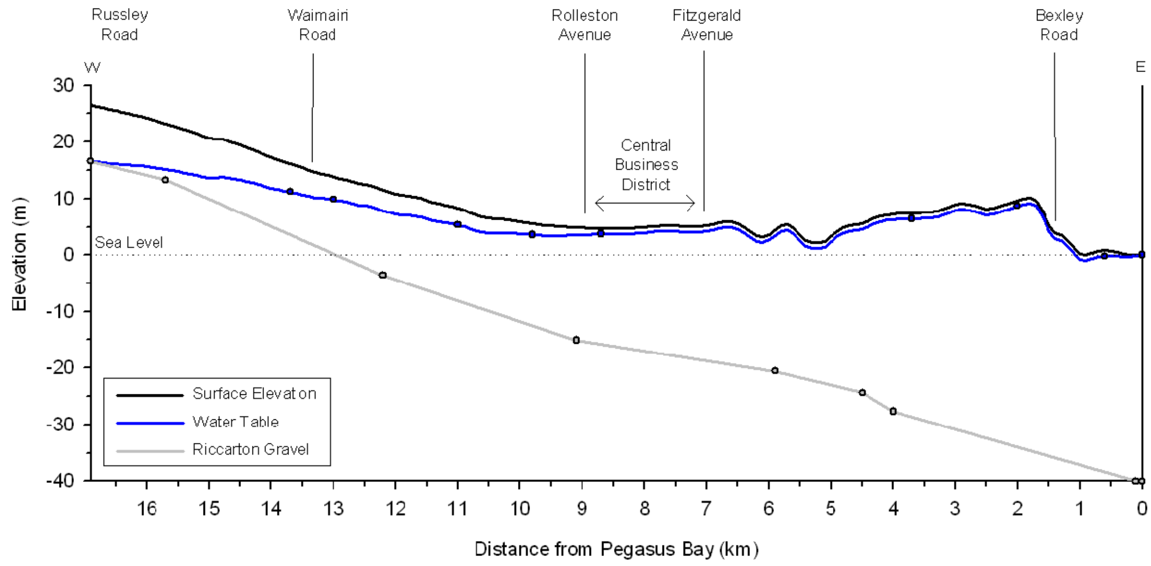
thickness of the surface soils or depth to the Riccarton Gravel is indicated in Figure 3 along two east-west cross sections aligning with the Bealey Avenue and Moorhouse Avenue respectively. The thickness of the surface alluvial soils is smallest at the west edge of the city (approximately 10 m thick) and increases towards the coast where the thickness of the Christchurch formation reaches about 40 m. Within the CBD, the thickness of the alluvial soils is approximately 20 m to 25 m.

Brown and Webber (1992) describe the original site conditions and development of Christchurch as follows: *"Originally the site of Christchurch was mainly swamp lying behind beach dune sand; estuaries and lagoons, and gravel, sand and silt of river channel and flood deposits of the coastal Waimakariri River floodplain. The Waimakariri River regularly flooded Christchurch prior to stopbank construction and river realignment. Since European settlement in the 1850s, extensive drainage and infilling of swamps has been undertaken."*

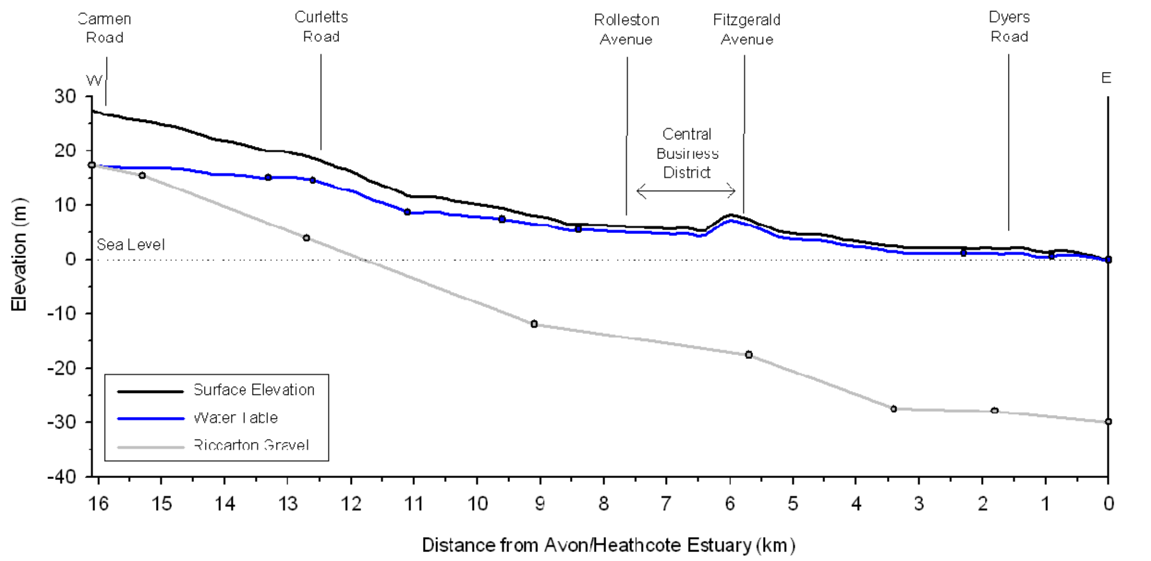
The deeper gravel strata and interlaying with silts, sands and some peats resulted from complex depositional processes during episodic glacial (colder) and interglacial (warmer) periods in which adjustment of river profiles, changes of coastline, coastal re-deposition, and inland reworking took place. Relatively recent but numerous episodes of flooding by the Waimakariri River, and reworking of soils by the spring fed waters of Avon River and Heathcote River until they were channelized, particularly influenced and characterized the present day surficial soils.

3.2 Groundwater

Canterbury has an abundant water supply through rivers, streams and very active groundwater regime including rich aquifers. It is estimated that over 10,000 wells have been sunk within the Christchurch urban area since 1860s (Brown and Weeber, 1992). The dominant features of present day Christchurch are the Avon and Heathcote rivers that originate from springs in western Christchurch, meander through the city, and feed the estuary at the southeast end of the city. Figure 4 shows the Avon River, streams and gullies within the CBD as depicted in the Christchurch maps from 1850s (ANZ, 2011).



(a) East-West cross section aligned with Bealey Avenue



(b) East-West cross section aligned with Moorhouse Avenue

Figure 3. East-West cross sections indicating the thickness of surface soils or depth to the Riccarton Gravel along the directions of: (a) Bealey Avenue, and (b) Moorhouse Avenue; ground elevation derived from a Landcare Research 15m-resolution Digital Elevation Model (Landcare Research, 2011); depth to water table and Riccarton Gravel interpolated from contour data (circles) from Brown and Weeber (1992)

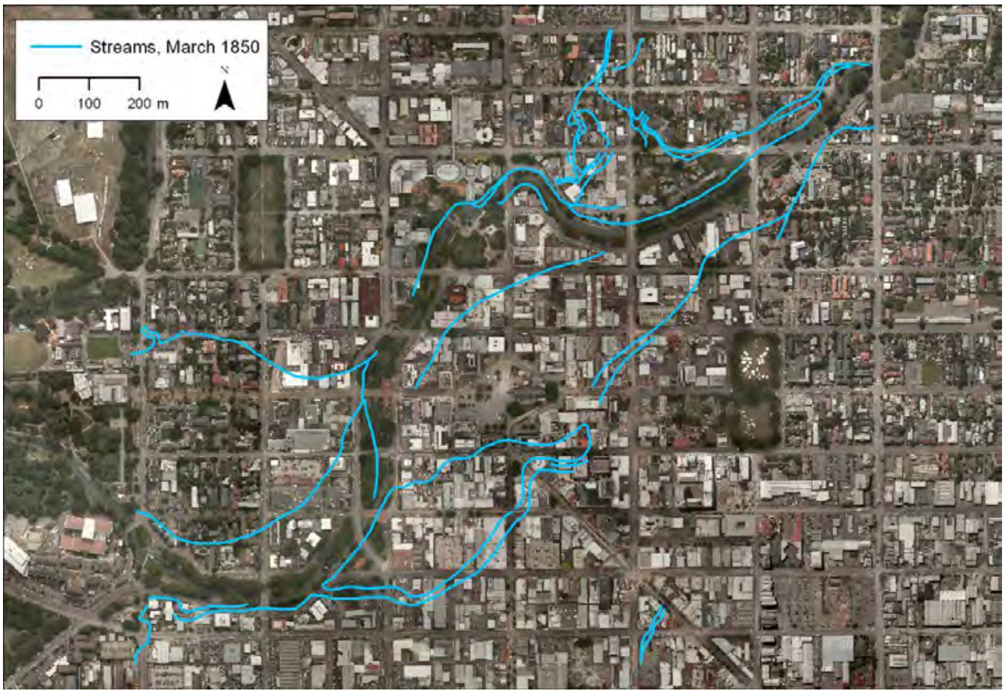


Figure 4. Streams in central Christchurch as mapped in March 1850, superposed on aerial photography captured on 24 February 2011. Streams digitised from the Black Map of Christchurch (March 1850), downloaded from Archives New Zealand (ANZ, 2011) (http://archives.govt.nz/gallery/v/Online+Regional+Exhibitions/Chregionalofficegallery/sss/Black+Map+of+Christchurch/)

As a consequence of this abundant water supply through open channels, aquifers and low-lying land near the coastline, the groundwater level is relatively high across the city. The water table is about 5 m deep in the western suburbs, becoming progressively shallower eastwards, and approaching the ground surface near the coastline, as indicated in Figure 3. To the east of CBD, generally the water table is within 1.0 m to 1.5 m of the ground surface. Seasonal fluctuations of the groundwater level are relatively small, within 0.5 m to 1.0 m.

3.3 Characteristics of CBD Soils

The shallow alluvial soils vary substantially within short distances, both horizontally and vertically. This variation is depicted in Figure 5 where a simplified stratification up to 30 m depth is shown for a cross section through the CBD soils along Hereford Street (Elder and McCahon, 1990). Starting from west at the Rolleston Avenue, the profile is characterised by a shallow sand and silty sand layer overlying alluvial sandy gravels. The gravels, which are about 10m thick at Rolleston Avenue, get thinner towards east and eventually disappear near the city centre. The eastern part of the section is then dominated by silts, silty sands and peat at shallow depths, from the ground surface up to about 5 m to 8 m depth. The alluvial sandy gravels reappear near the eastern edge of this section. Loose to medium-dense sand and dense to very-dense sand layers comprise the soils between 10 m and 20 m depth. These layers are underlain by silt, sandy silt and peat mixtures which sit on the Riccarton Gravel. It is important to emphasize that the presented soil profile is a gross simplification of the reality in order to depict the general features in the stratification and predominant soil layers. The actual soil conditions are much more variable or less uniform, both in geometry and soil properties.

To further illustrate the spatial variability of foundation soils, Figure 6 delineates several zones indicating the predominant soils in the top 7 to 8 m of the CBD deposits. In the south-west part of the CBD, alluvial gravels are encountered at shallow depths of 2.5 m to 3.5 m, while soft silts and peat comprise the top soils in the south-east part of the CBD. Relatively clean and deep sands dominate the stretch along Avon River; this was the area most severely affected by liquefaction in the 22 February earthquake. Further to the north of this zone towards Bealey Avenue, soft silty soils and peat are encountered in the top 7 to 8 m of the deposit.

From a geotechnical viewpoint, the alluvial gravels reaching shallow depths, the dense to very dense sand layer at about 15 m depth, and the deeper Riccarton Gravel are the most competent for building foundations. However, since Riccarton Gravel is an aquifer with artesian water pressures, it has been avoided wherever possible for foundations of buildings since it does impose some complex issues around constructability, higher costs of foundations resting in or passing through this layer, upward flow of water along piles, and potential contamination of the groundwater supply to Christchurch.

3.4 Typical Soil Profiles

Typical soil profiles in the north-west part of the CBD are shown in Figure 7a where interbedded silty sand and sand 3 m to 4 m thick overlies a soft to very soft layer of peat and peaty silt about 1 m to 1.5 m thick. This in turn overlies a soft clayey silt with traces of organics to about 8–9 m depth where there is a reasonably dense gravel layer, typically 2 – 3 m thick overlying dense to very dense sand. The sand becomes a little looser and siltier below about 20 m with the Riccarton gravels at 21 m to 23 m depth. In this area, older buildings are on shallow foundations, but virtually all commercial buildings within the last twenty years have been piled to about 10 m depth, because of the soft ground above about 8 m depth.

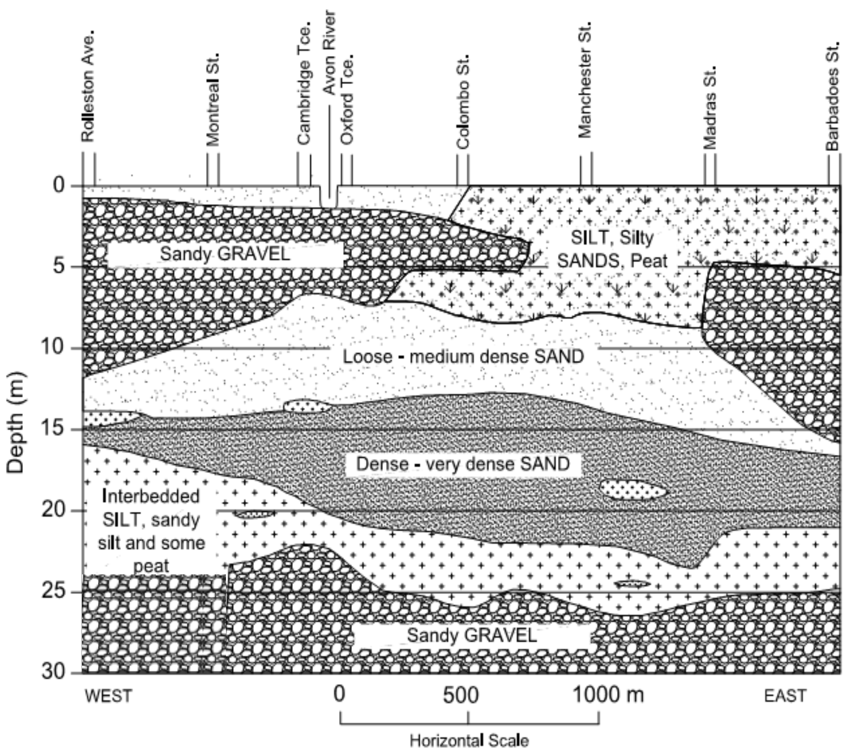


Figure 5. Subsurface cross section of Christchurch CBD along Hereford Street (reproduced and modified from Elder and McCahon, 1990)

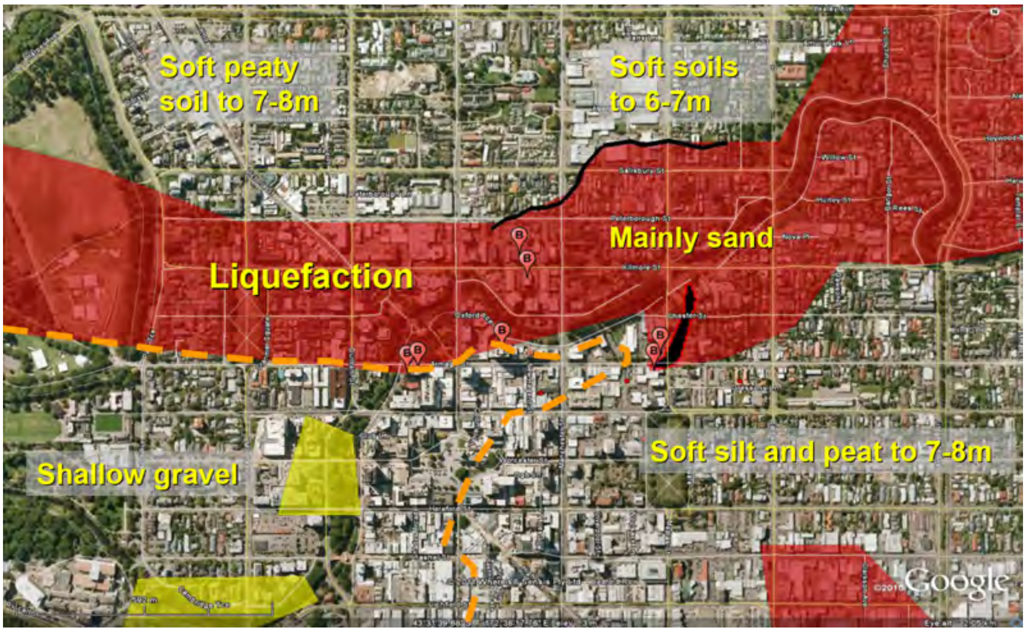


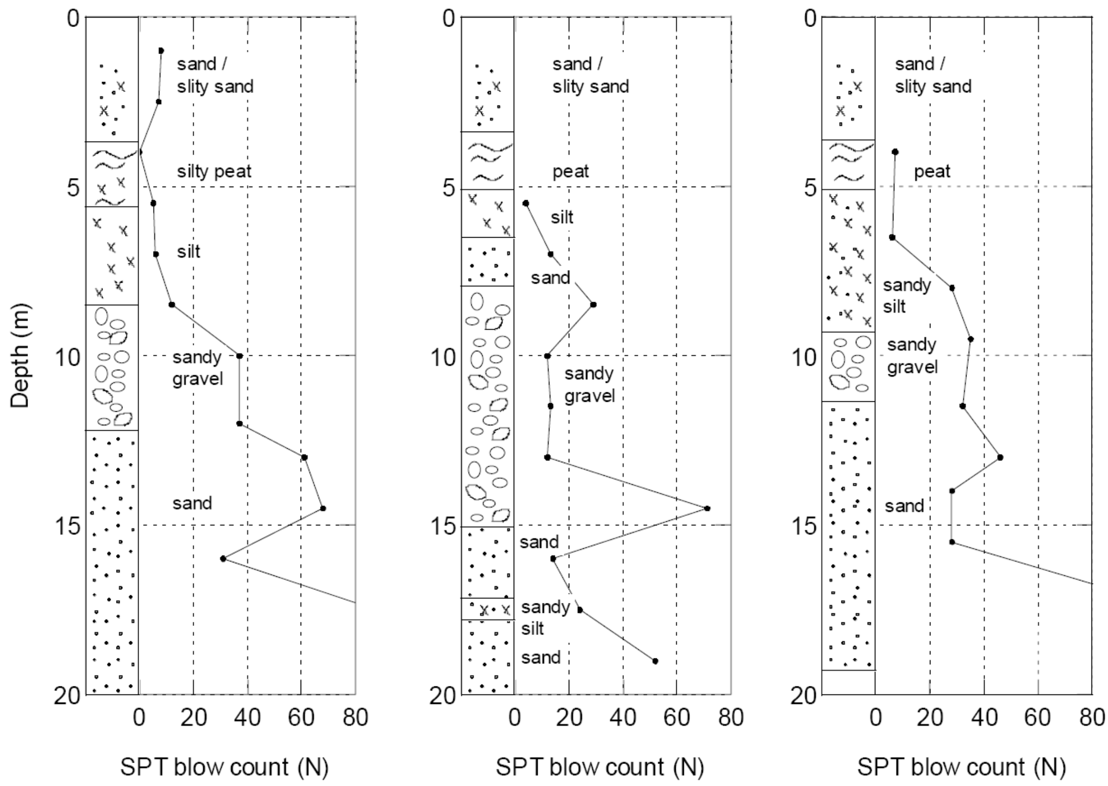
Figure 6. Preliminary liquefaction map indicating zones (in general terms, not on property basis) within the CBD affected by liquefaction in the 22 February earthquake (Cubrinovski and Taylor, 2011); predominant soils in the top part of the deposits are also indicated

Typical soil profiles to the east of the Cathedral Square comprise surface silt and silty sand soils overlying soft to very soft clayey silt with some peaty horizons from about 2 m to 7 m depth. Below 7 m there is a dense to very dense sand; a silt layer below about 20 m and then the Riccarton gravels at about 23 m depth. Again, while older buildings may be on shallow foundations, virtually all buildings constructed within the last twenty years have been piled to 8–10 m depth, because of the soft ground above about 7 m depth.

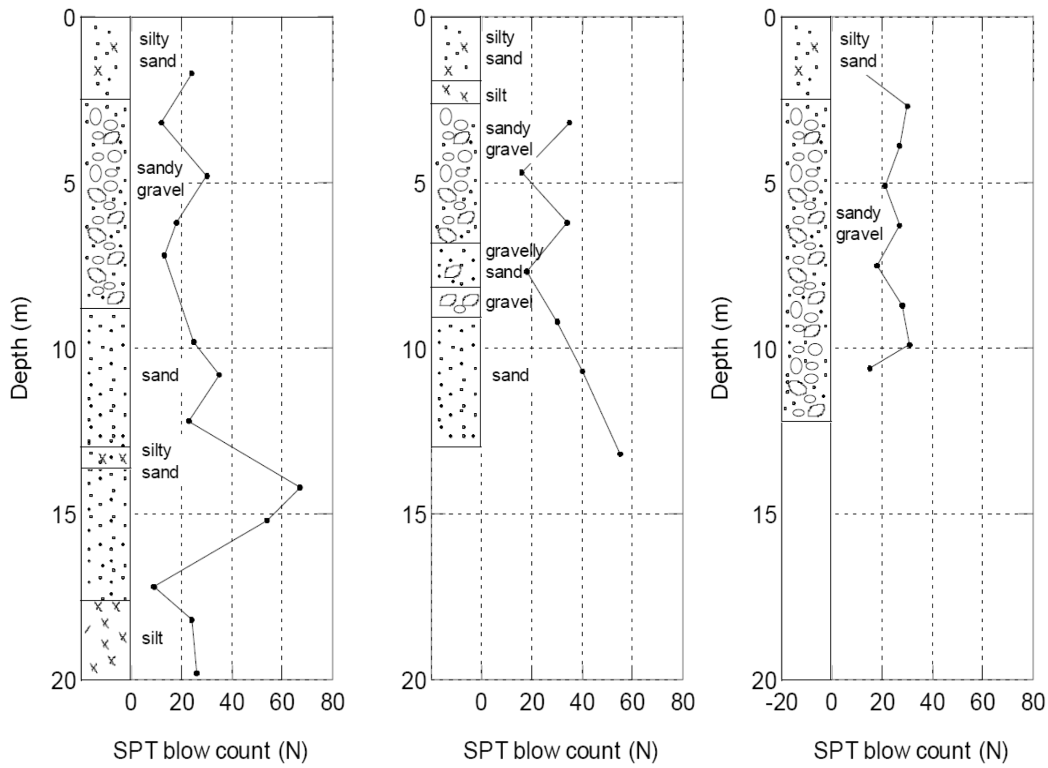
The south-west part of the CBD is dominated by shallow gravels 5 m to 8 m thick, up to 8-9 m depth (Figure 7b). The gravels reach shallow depths of about 2-3m and are covered by silty sands near the surface. Medium dense to dense sands underlie the gravels up to about 15-16 m interbedded with some silty layers and thin layers of organic soils. Silt layers comprise the deeper layers until the Riccarton Gravel is reached at about 22 m depth. Most of the building foundations in this area rest on the shallow gravels.

Within the CBD, the water table is generally within 1.5 m to 2.0 m of the ground surface. Thus, the soils below 1.5-2.0 m depth are fully saturated and all the voids between soil particles are filled with water.

Another factor of importance for the seismic performance of alluvial soils, and particularly for liquefaction, is the age of the deposit. Old soil deposits are stronger, less deformable (have higher stiffness) and gain liquefaction resistance through several complex aging mechanisms. Young or very recent sediments are the most vulnerable to liquefaction (Youd et al., 2003). Data on age of the soils based on radiocarbon dating of samples from the Christchurch area presented by Brown and Weeber (1992) is plotted in Figure 8 correlating the depth of the soils beneath the ground surface and their age. The shallow soils within the top 10 metres are less than 4000 years old, and some are only few hundred years old, which makes them potentially vulnerable to liquefaction.



(a) Soil profiles in the north-west part of the CBD



(b) Soil profiles in the south-west part of the CBD

Figure 7. Characteristic soil profiles within the CBD

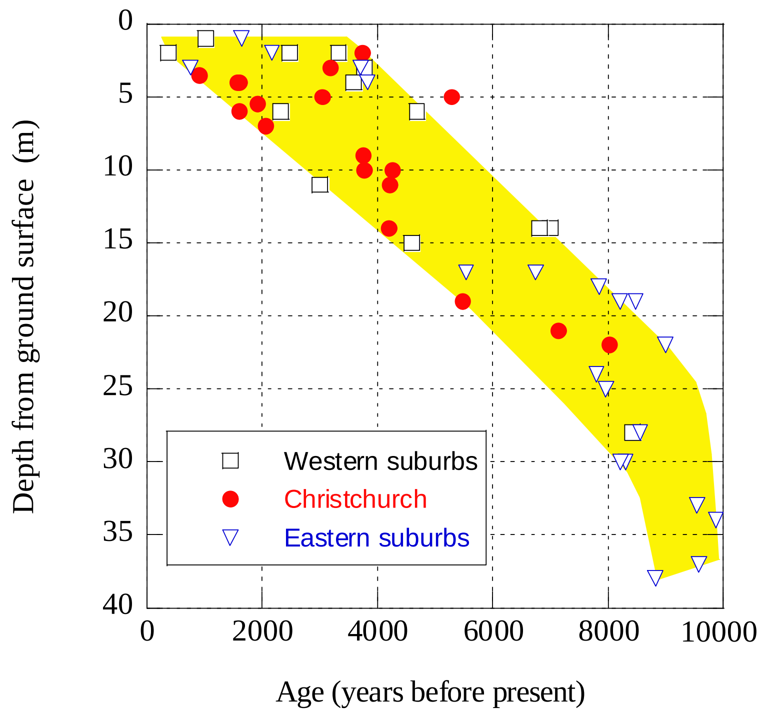


Figure 8. Age of soils overlying the Riccarton Gravel expressed as a function of depth (based on radiocarbon ages of selected soils samples from the Christchurch area reported by Brown and Weeber, 1992)

3.5 Summary of Key Features of CBD Soils

The key characteristics of the CBD soils can be summarized as follows:

- The top 20-25 m of the CBD soils are relatively recent alluvial soils overlying 300 m to 500 m thick gravelly deposits.
- The recent alluvial soils in the top 20 m of the deposits are the most important for foundations of multi-storey buildings and liquefaction evaluation. These soils comprise gravels, sands, silts, peat and their mixtures, and are highly variable both horizontally and vertically.
- The soils within the CBD are fully saturated below 1.0 m to 1.5 m depth
- Considering their composition (sandy soils and non-plastic silts), age (recent deposits, few hundreds to a few thousand years old) and depositional environment (river, swamp and marine sediments), these soils are generally considered susceptible to liquefaction, and in some cases (when deposited in a loose state) they have very low resistance to liquefaction.
- By and large, the foundation conditions within CBD are very complex and challenging for geotechnical engineers, particularly in regard to their performance during strong earthquakes.
- The presence of aquifers at depths of about 20 m to 25 m (and in some cases even at shallower depths) is a relatively unique feature that potentially may exacerbate the seismic response of the soils above the aquifers during strong earthquakes (by providing an additional mechanism for increase in the groundwater pressure through upward flow of water fed by the aquifers).

4 Observed Liquefaction and Response Spectra in CBD during the 2010 and 2011 Christchurch Earthquakes

4.1 Soil Liquefaction During the 22 February 2011 Earthquake

The series of earthquakes that hit Christchurch in the period between 4 September 2010 and 13 June 2011 caused repeated liquefaction through its suburbs and the CBD itself. The 22 February earthquake was the most damaging, inducing widespread liquefaction and lateral spreading in the eastern suburbs and within parts of the CBD. The liquefaction in the CBD adversely affected the performance of many buildings resulting in differential settlements, lateral movement of foundations, tilt of buildings, and some bearing failures.

Figure 9 shows the extent of liquefaction caused by the 22 February 2011 earthquake in wider Christchurch documented through a drive-through reconnaissance that was conducted in the period from 23 February to 1 March by the University of Canterbury (Cubrinovski and Taylor, 2011). The drive-through survey aimed at capturing surface evidence of liquefaction as quickly as possible and quantifying its severity in a consistent and systematic manner. Four areas of different liquefaction severity are indicated in the map: (a) moderate to severe liquefaction (red zone, with very large areas covered by sand ejecta, mud and water, large distortion (undulations) of ground and pavement surfaces, large cracks and fissures in the ground, and significant liquefaction-induced impacts on buildings), (b) low to moderate liquefaction (yellow zone, with generally similar features as for the severe liquefaction, but of lesser intensity and extent), (c) liquefaction predominantly on roads with some on properties (magenta zone, where heavy effects of liquefaction were seen predominantly on roads, with large sinkholes and 'vents' for pore pressure dissipation, and limited damage to properties/houses), and (d) traces of liquefaction (red circular symbols, with clear signs of liquefaction, but limited in extent and deemed not too damaging for structures). The suburbs to the east of CBD along Avon River (Avonside, Dallington, Avondale, Burwood and Bexley) were most severely affected by liquefaction, which coincides with the area where about 5000 residential properties will be abandoned (New Zealand Government, 2011).

Ten days after the earthquake, after the urban search and rescue efforts had largely finished, a comprehensive ground survey within the CBD was initiated to document liquefaction effects in this area. Figure 10 shows the resulting liquefaction documentation map for the CBD. The principal zone of liquefaction stretches west to east through the CBD, from Hagley Park to the west, along the Avon River to the northeast boundary of the CBD at the Fitzgerald Bridge. This zone is of particular interest because many high-rise buildings on shallow foundations and deep foundations were affected by the liquefaction in different ways. Note that this zone consists mostly of sandy soils and it largely coincides with the path of the Avon River and the network of old streams shown in the 1850s survey maps (Figure 4). Another zone of moderate to severe liquefaction was found in the south-east part of the CBD, though its effects were less significant in relative terms.

Even though the map shown in Figure 10 distinguishes the zone along Avon River as the most significantly affected by liquefaction, the severity of liquefaction within this zone was not uniform. In this zone, the manifestation of liquefaction was primarily of moderate intensity with relatively extensive areas and volumes of sand/silt ejecta. There were also areas of low manifestation or only traces of liquefaction, but also pockets of

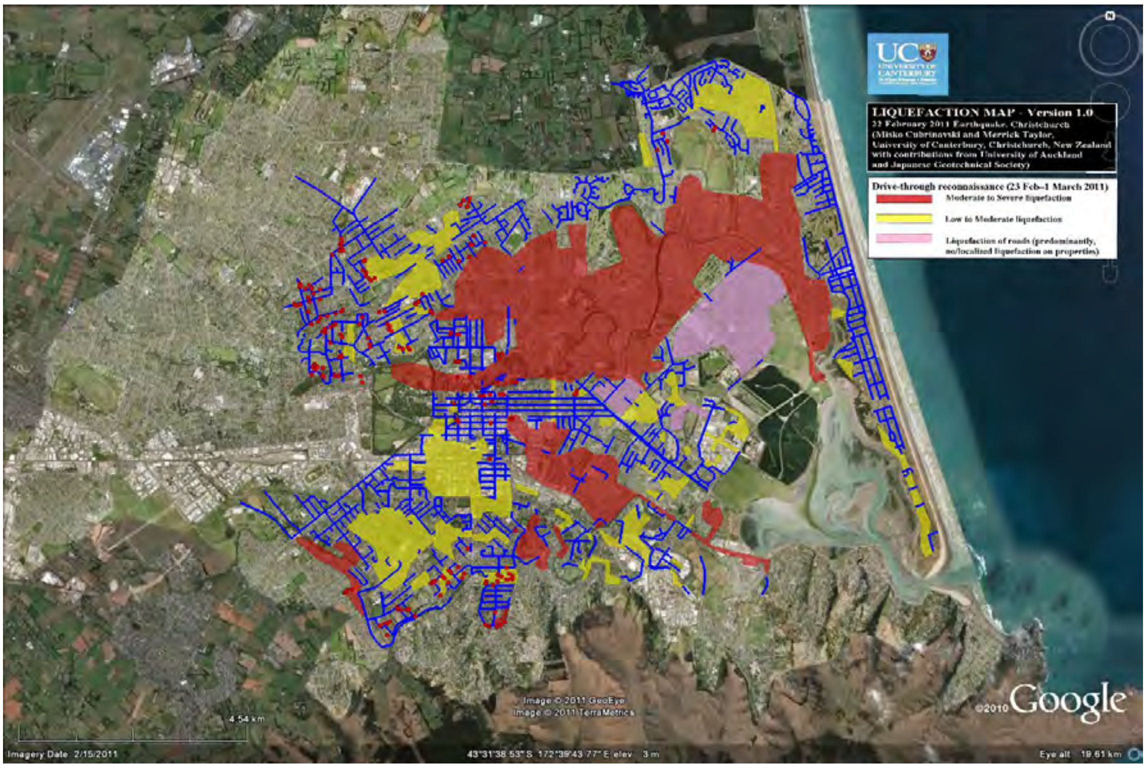


Figure 9. Preliminary liquefaction map of Christchurch from drive-through reconnaissance (Cubrinovski and Taylor, 2011); the map is not complete and shows only general overlay of areas (it cannot be used on property basis)

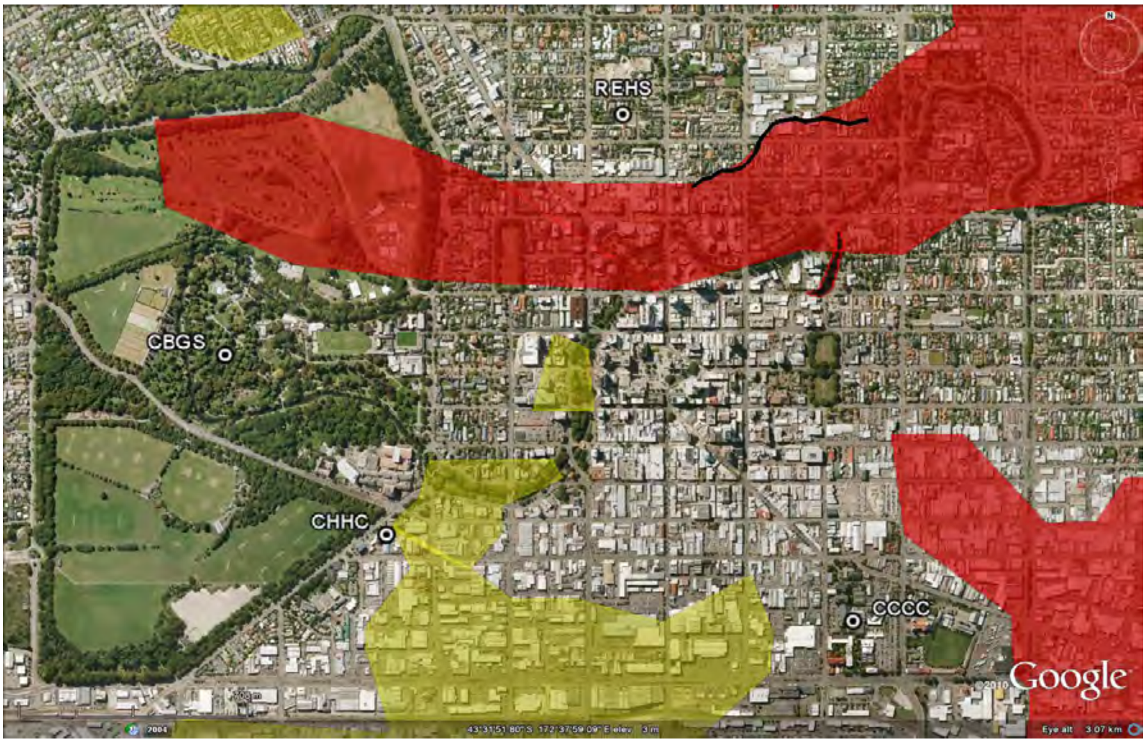


Figure 10. Preliminary liquefaction map indicating areas within the CBD affected by liquefaction in the 22 February earthquake

severe liquefaction with very pronounced ground distortion, fissures, large settlements and substantial lateral ground movements. The zones of more pronounced liquefaction do appear somewhat to "line up" with the old stream channels, which sheds some light on the reasons for variability in liquefaction manifestation. One should not expect though that all liquefaction features and zones of pronounced ground weakness could be explained with reference to the stream channels dating back to 1850s, because the earlier depositional history and re-working of surficial soils is also very relevant for their liquefaction susceptibility.

The north extent of the zone, which is shown by the thick black line in Figure 10, is a clearly defined geomorphic boundary (easily detectable change in the ground surface due to features of underlying soils) running east to west. This feature was marked by a slight change in elevation of about 1 m to 1.5 m over approximately 2 m to 10 m wide zone, and was characterized by ground fissures and distortion associated with gentle slumping of the ground surface and localized spreading towards the down-slope side. Ground cracks, fissures and a distorted pavement surface marked this feature, which runs continuously through properties and affected a number of buildings causing cracks in both the foundations and their structures. Liquefaction and associated ground deformation were pronounced and extensive on the down-slope side between the identified geomorphic feature and the Avon River, but noticeably absent on the slightly higher elevation to the north (upslope side away from the river). This feature is thought to delineate the extent of a geologically recent river meander loop characterized by deposition of loose sand deposits under low velocity conditions. A similar geomorphic feature was observed delineating the boundary between liquefaction damage and unaffected ground within a current meander loop of the river to the east of this area (Oxford Terrace between Barbados Street and Fitzgerald Avenue).

Liquefaction-induced lateral spreading occurred along the Avon River in the liquefied zone within the CBD, and the horizontal stretching of the ground adversely affected several buildings. Ground surveying measurements conducted at about ten transects on Avon River within the CBD after the 22 February earthquake indicated that at several locations, the banks of Avon River moved laterally about 50-70 cm towards the river, whereas at most of the other locations the spreading displacements were on the order of 10 cm to 20 cm. The zone affected by spreading was relatively narrow usually within 50 m from the Avon River, though at a few locations the spreading extended up to 100 m to 150 m from the banks. There were many smaller buildings suffering serious damage to the foundations due to spreading as well as clear signs of effects of spreading on some larger buildings both at the foundations and through the superstructure. Structures and foundations within the spreading zone are greatly impacted by the horizontal ground strains causing stretching of the ground, foundations and then the building itself.

4.2 Repeated Liquefaction within CBD during the 2010-2011 Earthquakes

Soil liquefaction repeatedly occurred at the same sites during the earthquakes producing strong ground shaking in Christchurch, and in particular during the 4 September 2010, 22 February 2011, and 13 June 2011 earthquakes. Figure 11 comparatively shows liquefied areas of Christchurch in these three events, as documented by field inspections. Note that only parts of Christchurch have been surveyed (coloured areas) and that the aim of the surveys was to capture general features and areas affected by liquefaction as observed from the roads, hence, the zoning is not applicable to specific properties.

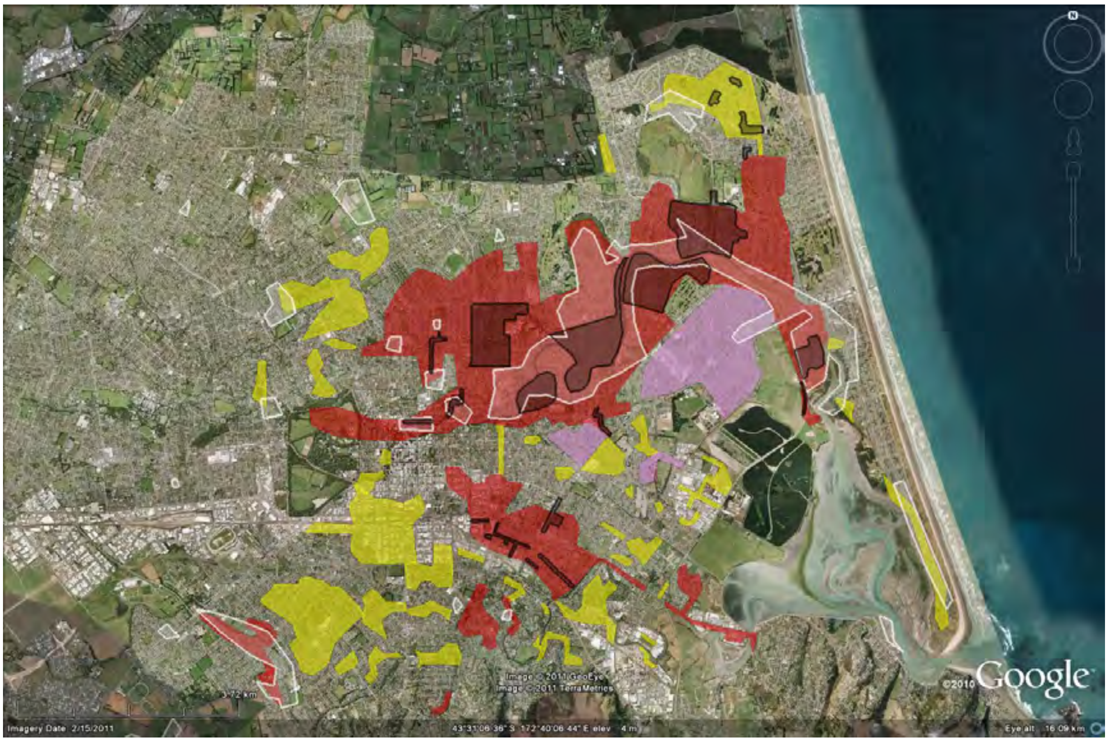


Figure 11. **Preliminary liquefaction maps documenting areas of observed liquefaction in the 4 September 2010 (white contours), 22 February 2011 (red, yellow, magenta areas), and 13 June 2011 (black contours) earthquakes; note that only parts of Christchurch were surveyed (coloured areas), and that the aim of the surveys was to capture general features and areas affected by liquefaction as observed from the roads, hence, the zoning is not applicable to specific properties**

The repeated occurrence of liquefaction at a given site during an earthquake is not surprising because liquefaction generally does not increase the liquefaction resistance nor prevents the occurrence of liquefaction of the site in subsequent earthquakes. The sequence of events in Christchurch has certainly proven this notion.

The repeated liquefaction was often quite severe and many residents reported that in some cases the severity increased in subsequent events. In addition to the inherent level of liquefaction resistance of soils (a specific strength property), whether liquefaction will occur or not, and what will be its severity, should it occur, depends on the severity of ground shaking caused by the earthquake. In this context, each of these earthquakes produced different ground shaking within the CBD. Table 1 summarises the peak ground accelerations (PGA) recorded at four strong motion stations within/close to the CBD (CBGS, CCCC, CHHC, REHS; locations listed in the footnote of Table 1) during five earthquakes producing damaging levels of ground shaking.

The simplified procedure for liquefaction evaluation enables us to combine two key features of ground shaking (i.e. its amplitude and duration) into a single parameter (CSR), and hence comparatively examine the severity of ground shaking or seismic demand on soils imposed by different earthquake events. More details around this procedure are given in Section 6, while here the results of the simplified analysis are briefly discussed. Table 1, in addition to the PGAs, also summarises the calculated geometric mean CSR values for the four strong-motion sites, for the five different earthquakes considered. These data suggest that the 22 February earthquake was by far the most severe with regard to triggering of soil liquefaction, with a severity of ground shaking nearly 1.5 to 2 times exceeding the second most severe event, the 4 September 2010 earthquake. Close third comes the M_w=6.0 earthquake of 2:20 pm, 13 June 2011. However, this earthquake was preceded by another earthquake (M_w=5.5) producing similarly strong shaking at 1:00 pm, 13 June 2011. Since these two earthquakes occurred within a short time interval of 80 minutes, the effects of liquefaction produced by the second shake were exacerbated because there were still elevated pore water pressures in the ground produced by the first earthquake when the second quake hit. By and large, the CSR values computed for the five events (in fact four, if we consider the cumulative effects of both 13 June earthquakes) listed in the table are consistent with the severity of liquefaction induced within CBD during each of these events (summarised in the column to the right).

Table 1. Peak Ground Accelerations (PGA) and adjusted Cyclic Stress Ratios to M_w=7.5 earthquake ($CSR_{7.5}$) recorded (computed) at four strong motion stations within/close to CBD, for five earthquakes in the period September 2010 – June 2011

Event	Geometric Mean PGA (g)				Geometric Mean Cyclic Stress Ratio, $CSR_{7.5}$ [a)]	Magnitude Scaling Factor MSF[b)]	General liquefaction manifestation within CBD
	CBGS	CCCC	CHHC	REHS			
4-SEP-10 M_w=7.1	0.16	0.22	0.17	0.25	0.11	1.150	Low to moderate
26-DEC-10 M_w=4.7	0.27	0.23	0.16	0.25	0.04	3.307	No liquefaction
22-FEB-11 M_w=6.3	**0.50**	**0.43**	**0.37**	**0.52**	**0.19**	1.562	Severe
13-JUN-11 M_w=5.3	0.18	-	0.20	0.19	0.05	2.431	Low to moderate
13-JUN-11 M_w=6.0	0.16	-	0.21	0.26	0.08	1.770	

a) $CSR_{7.5}$ = 0.65·(PGA/g)/MSF (at depth of water table)

b) MSF = $10^{2.24}/M_w^{2.56}$ (corresponding to the lower bound range recommended in Youd et al. (2001)

CBGS = Christchurch Botanic Gardens; CHHC = Christchurch Hospital; CCCC = Christchurch College; REHS = Resthaven;

4.3 Influence of CBD Soils on Response Spectrum

The deep alluvial soils of Christchurch influence the ground motions and their response spectra through amplification of some period components of the motion and de-amplification of others as the shear waves propagate from the base of volcanic rocks to the ground surface. The 300-500 m deep gravel formations amplify the motions in the range of their predominant periods between 1 and 3 seconds while they slightly de-amplify the high frequency components of the base rock motions. In the softer surface layers in the top 20-25 m of the deposits, large ground deformation is induced involving significant nonlinearity and liquefaction in some cases. These layers act as a filter that removes the high-frequency components and spikes while elongating the motion cycles and hence amplifying some of the long period components. This feature is illustrated in Figure 12 where response spectra of the ground motions recorded at LPCC and LPOC in the Lyttelton Port are shown. These stations are approximately 1 km apart, however, LPCC is located effectively on the volcanic rock, while LPOC is on top of approximately 30 m layer of silty and clayey soils. The ground motion at LPOC shows significant reduction of the low periods (high frequency) components, and conversely an amplification of the motion in the range of long periods, which are typical effects of soft deposits on the response spectrum.

The occurrence of liquefaction is also evident in the recorded acceleration time histories at many sites across Christchurch where, following the triggering of liquefaction in the first 5 to 10 seconds of shaking, elongated oscillation cycles are seen in the time histories (Figure 13). These are apparent through a spectral amplification at periods exceeding 2 seconds. Youd and Carter (2005) have reported similar observations in liquefaction-affected ground motions with bulges in the acceleration response at about 3 seconds. Considering the significant variation of response spectra, even within small distances, the records confirm that the surficial soil layers played an important role in defining the ground motion characteristics.

In addition to these two effects from the deep gravel formations and softer shallow deposits, the ground motions in the CBD are also influenced by basin effects due to the shallowing out of the surface deposits towards the base of the volcanic rock at the Port Hills and focussing of waves associated with the specific features of the fault rupture and its spatial and temporal propagation.

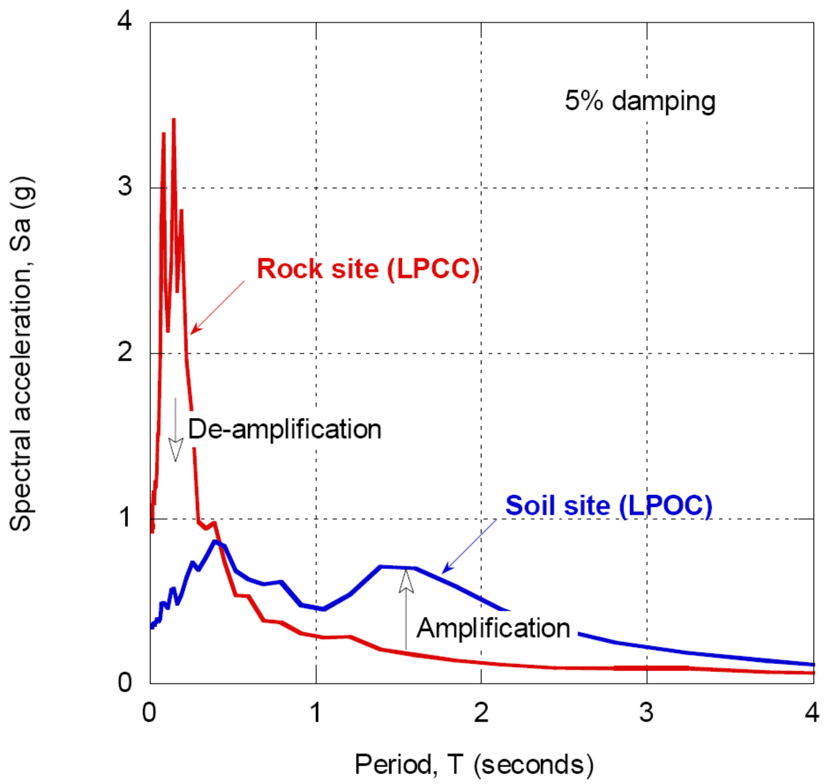


Figure 12. **Acceleration response spectra recorded on rock (LPCC) and soil (LPOC) in Lyttelton during the 22 February earthquake illustrating typical effects of alluvial soils on response spectra (5% damped, elastic spectra)**

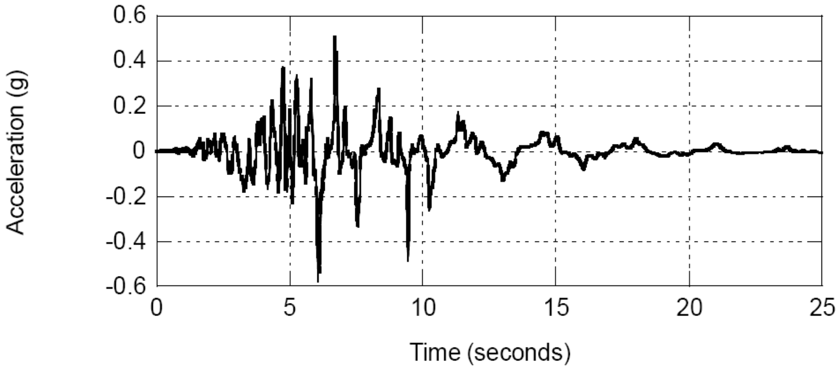


Figure 13. Acceleration time history recorded at the Botanic garden (CBGS) during the 22 February 2011 earthquake

Tasiopoulou et al. (2011) performed an advanced ground response analysis to simulate the recorded response spectra at CBD by using the recorded motion on rock at the Lyttelton Port (LPCC) as input motion. A generic soil model was adopted in their analysis with a surface layer of sandy soil up to 25 m depth (the top 17 m of which was modelled as liquefiable) underlain beneath by deep gravels up to 400 m depth. In general, good agreement was obtained between the recorded and the computed spectra in the CBD confirming that a realistic insight of the mechanisms of soil response during the Christchurch earthquakes have been gained from the analyses.

Figure 14 comparatively shows the horizontal response spectra for the four CBD stations and the Riccarton station (RHSC). The latter was included as a reference for ground motion recorded at a site that did not liquefy but which is located on the deep gravel formation. Two spectra are shown for each station, for the 4 September 2010 and 22 February 2011 earthquakes respectively. Superimposed in these plots are also the design spectra defined in NZS1170.5 for three soil sites: C (relatively stiff soils in the top 30 m), D (soft to very soft soils in the top 30 m) and E (extremely soft and liquefiable soils in the top 30m). Note that the shape of the design spectra for sites C, D and E, depicts the trend discussed previously in which soft and liquefiable soils amplify the ground motion at periods exceeding 2 seconds. For example, spectral values for Class E site at a period of 3 seconds are nearly three times higher than the corresponding values for Class C site.

The design spectra shown in these figures are for a 475-year return period earthquake which is also often referred to as the ultimate limit state (ULS) in design (see Section 5.2). By and large, during the 22 February 2011 earthquake the ground motions within CBD exceeded the ULS spectra for all site types. In some cases the recorded motions were two to three times above the ULS design level. The spectra essentially imply that structures within the CBD from 2 storey to 20 storey buildings experienced much higher loads than their design loads. The 4 September 2010 earthquake caused much smaller seismic loads within the CBD which were generally within the bounds of the code spectra with a few exceptions. In Riccarton, both quakes produced motions very similar to the design level ground motions. The performance of the CBD buildings and their foundations were largely consistent with the severity of ground motions produced by the two earthquakes and design objectives stipulated in current codes, except of course for the two fatal collapses of reinforced-concrete buildings and the collapses or partial collapses of a number of masonry structures. The performance of foundations of CBD buildings is addressed somewhat in detail in the following section.

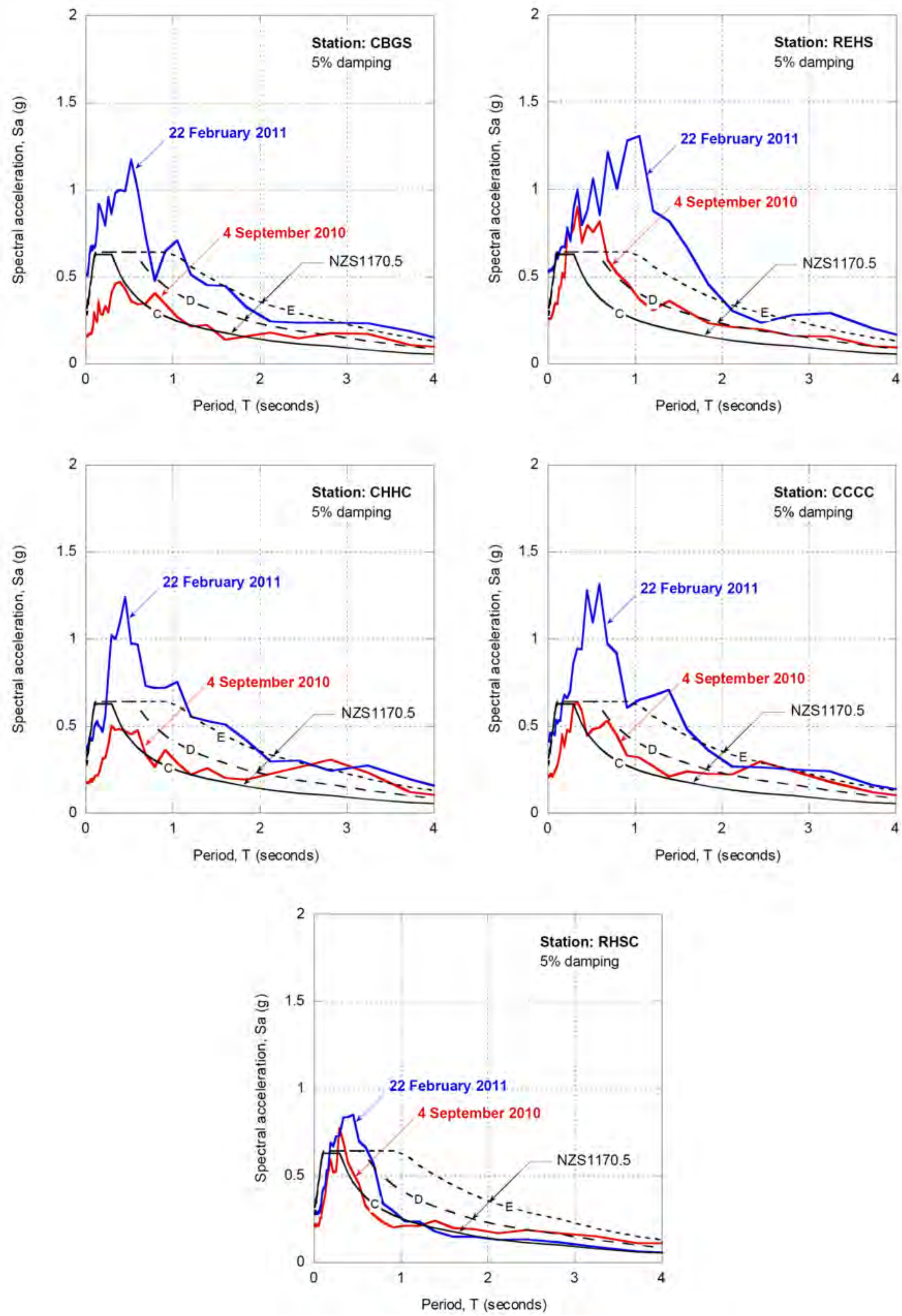


Figure 14. Comparison of acceleration response spectra (5% damped elastic spectra) recorded at five strong motion stations and design acceleration response spectra for a 475-year return period earthquake; red lines show recorded motions in the 4 September earthquake (geometric mean spectrum); blue lines show recorded motions in the 22 February earthquake (geometric mean spectrum); black lines show design spectra (NZS 1170.5) for Soil Class C (solid line), Soil Class D and Soil Class E (broken lines)

5 Typical Causes of Failure in the CBD

The non-seismic design of foundations is principally governed by gravity loads imposed by the weight of the building itself and the weight of the contents and occupants. Geotechnical engineers need to check and satisfy two principal criteria when designing foundations for normal conditions (under gravity loads). They have to ensure that the soil is strong enough so that it can support the building weight without catastrophically failing, and that the soil when loaded under the gravity and service loads will not deform more than is tolerable for the building and its normal use.

In cases when the soils close to the ground surface are strong and stiff enough, shallow foundations are built immediately under the walls and columns of the building at shallow depths. For taller and heavier buildings a raft (mat) foundation is often used since it provides a stiffer and stronger shallow foundation that spreads the building weight over a wide enough area of the underlying soil to keep settlements to an acceptable level. In addition to the control of global or total settlements, it is critically important to keep the differential settlements within acceptable levels since these settlements are very damaging to the building. In other cases when the top soils are too soft or weak, the building weight is transferred to sufficiently strong soils at greater depth (several to many metres below the ground surface), most often using piles.

An additional seismic assessment/design of the foundations is then conducted to ensure their satisfactory performance during earthquakes, as stipulated in the building design codes.

5.1 Typical Foundations of CBD Buildings

As mentioned previously and illustrated in Figures 5, 6 and 7, the CBD soils consist of different dominant soil layers in different areas of the CBD, with highly variable stratification and depth to competent foundation soils. This fact, together with the different requirements imposed on the foundation soils by buildings of different sizes/heights, where taller and heavier buildings require stronger and less deformable soils for their foundations, have led to a range of different types of foundations being used for the CBD buildings. The most commonly used foundation types are summarised in Table 2.

Table 2. Typical foundation types used within the CBD

Foundation type	*Building type*	*Foundation soils*
Shallow foundations (Isolated spread footings with tie beams)	• Multi-storey buildings • Low-rise apartment buildings	• Shallow alluvial gravel • Shallow sands, silty sands
Shallow foundations (Raft foundations)	• Multi-storey buildings • Low-rise apartment buildings with basement	• Shallow alluvial gravel • Shallow sands, silty sands
Deep foundations (shallow piles)	• Low-rise apartment buildings	• Medium dense sands (Soft silts and peat at shallow depths)
Deep foundations (deep piles)	• Multi-storey buildings	• Medium dense to dense sands (Areas of deep soft soils or liquefiable sands underlain by dense sands)
Hybrid foundations (combined shallow and deep foundations or combined shallow and deep piles)	• Multi-storey buildings	• Highly variable foundation soils including shallow gravels and deep silty or sandy soils beneath the footprint of the building

There is a wide range of foundation types used in Christchurch with shallow foundations ranging from small strip and pad footings to large very stiff rafts, and piles ranging in length from a few metres to about 25 m. With few exceptions, building foundations in the Christchurch CBD have performed satisfactorily under normal conditions confirming that the foundations have been designed adequately for static (non-seismic) gravity loads.

5.2 Seismic Design Philosophy

The general seismic design philosophy for building foundations focuses on two principal performance requirements technically termed the serviceability limit state (SLS) and the ultimate limit state (ULS).

For ordinary buildings, the SLS is associated with a frequent earthquake with return period of 25 years. There is a relatively high probability that such earthquake will occur during the lifetime of the building. Under the SLS earthquake (which produces a relatively small level of ground shaking) the building including its foundations should perform to a high standard and should remain in full service and occupancy. Hence, large ground deformation or soil liquefaction (with the exception of minor non-damaging liquefaction) should not occur under the SLS earthquake. For good foundation soils, this requirement is basically satisfied indirectly through the robust foundation design for gravity loads previously discussed. For soft or liquefiable foundation soils, additional considerations and measures are required to meet SLS performance requirements.

The ultimate limit state (ULS) is associated with a 475 year return period earthquake, which translates to a 10% probability for occurrence in 50 years. The key performance requirement under the ULS earthquake is to prevent loss of life, and hence the structure or parts of it should not collapse either inside or outside the structure. Thus, for a ULS earthquake, some damage and deformation of the foundations are acceptable and even expected, but not to a degree that may lead to a failure in the building which could endanger life. This is achieved through the seismic design provisions stipulated in the seismic codes for buildings applicable at the time of design/construction. While these codes have evolved over a relatively short period of time, over the past 20-30 years the codes have relied upon essentially the same design philosophy with respect to SLS and ULS performance requirements.

When examining the performance of the foundations or buildings themselves we have to place the 2010-2011 earthquakes in the context of this philosophy and ULS earthquake levels. The PGAs listed in Table 1 and spectral accelerations shown in Figure 14 show that the 4 September 2010 and several of the aftershocks produced ground shaking equivalent to the ULS earthquake for Christchurch, and that the 22 February 2011 earthquake substantially exceeded the ULS earthquake. Having this in mind, one may argue that the CBD buildings (with few notable exceptions) performed as designed and as expected to perform (in general terms, recognizing that some important anomalies have been observed). It is entirely another matter whether this philosophy is an appropriate one for the 21st Century New Zealand and beyond, which is an issue requiring a broad debate and involvement of the communities and different aspects of the society as a whole. One may argue that the current philosophy (which is shared internationally by the most advanced countries in earthquake engineering) does not address the issues of the overall impact of big earthquakes on a city or a country, and the need for a reasonably quick recovery from such events. In essence, it ignores some key aspects of resilience requirements.

5.3 Typical Performances and Ground 'Failures' Observed within CBD

As discussed in Section 4, two aspects of the deep alluvial soils under Christchurch have been demonstrated by the recent earthquakes. The first is the modification of the ground motion as illustrated by the soil effects on spectral accelerations (e.g. Figures 12 to 14). In places the shaking has been damped, or to some degree reduced/ cancelled out, while in other places it has been amplified to a marked degree. This in turn has affected the intensity of shaking of both the surface soils and the buildings. The second aspect is soil liquefaction which is a form of 'ground failure' because it produces large displacements and permanent ground distortion. Liquefaction affects the ground response as well as the performance of foundations and buildings through a complex process involving very large and rapid changes in loads and soil conditions over few tens of seconds. Soil liquefaction in a substantial part of the CBD adversely affected the performance of many multi-storey buildings leading to total and differential settlements, lateral movement of foundations, tilt of buildings, and bearing failures. Note that the term 'failure' does not imply collapse, but rather indicates excessive permanent displacements of ground or foundations that require either serious remediation measures/retrofitting or demolition/abandonment of the structure.

Differential Settlement, Tilt and Sliding

Several buildings on shallow foundations within the CBD are supported on loose to medium-dense sands and silty sands that liquefied during the 22 February earthquake. The liquefied foundation soils lost the capacity to support the buildings leading to non-uniform (differential) settlements of the foundations and tilt of the buildings. Uneven settlements across the footprint of the building inevitably induce structural deformations which are often damaging to the structure.

Figure 15 shows a three storey structure on shallow foundations illustrating this deformation mode. The building settled substantially at its front resulting in large differential settlements and tilt of the building of about 2 degrees. The building was also uniformly displaced laterally approximately 15 cm towards the area of significant liquefaction near the front of the building (i.e. to the right on the photo). This type of damage was commonly observed for buildings on shallow foundations in areas where the soils beneath the footprint of the building were not uniform, and only part of the foundation soils liquefied significantly.

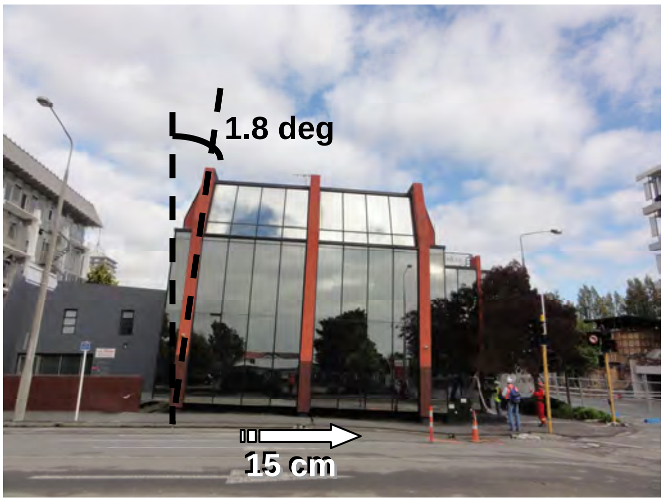


Figure 15. Liquefaction-induced differential settlement and sliding of a building

Figure 16, shows a six storey building at the same location (which faces the weak liquefied area to the left in the photo). This building is also on shallow foundations comprised of isolated footings with tie beams and perimeter grade beam. Differential settlements are indicated in the figure relative to the right-most column which is used as a reference. The differential settlement of the southeast corner (the left-most column in the figure) was approximately 26 cm. Again, effects of liquefaction were the most severe at the southeast corner of the building and gradually diminished throughout the footprint of the building towards north leading to substantial differential settlements and pronounced structural deformations. Both these buildings were considered uneconomic to repair and were (will be) demolished in the months following the 22 February 2011 earthquake. Other multi-storey buildings also suffered this type of damage, which in many cases was exacerbated by the 13 June 2011 earthquakes.

Figure 16. Liquefaction-induced differential settlement of a six storey building

Punching Settlement

Several buildings on shallow foundations located within the liquefied zone underwent punching settlements with some undergoing significant differential settlements and bearing capacity failures (sinking of the building in the soil). An example of such performance is shown in Figure 17 for a two storey industrial building. The building settled approximately 10-25 cm relative to the surrounding ground. There were clear marks of the mud-water ejecta on the walls of the building indicating about 25 cm thick layer of water and ejected soils due to the severe liquefaction. While the perimeter footings beneath the heavy walls were driven downwards causing the building to sink, the ground floors were subjected to uplifting forces by the groundwater pressures and the soil beneath the floor resulting in bulging and blistering of the ground floor. There are numerous instances of basements under low-rise buildings having moved upward because of the high water pressures below them exceeding the building weight.

Buildings on Shallow Gravels

Many of the high-rise CBD buildings are supported by shallow foundations sitting on shallow gravels. While gravels are relatively competent foundation soils, their thickness within the CBD is variable over short distances (often under the footprint of a single building) and so is the composition of the soils underlying the gravels. During

Figure 17. Punching settlement of a building in liquefied soils

earthquakes, these different soils will respond and settle differently due to various degrees of cyclic softening and compressibility. This large spatial and temporal variability in the response of the soils beneath the foundation will eventually result in differential settlements, tilt and permanent lateral displacements of buildings. These adverse effects are especially pronounced in transition zones where ground conditions change substantially over short distances. This complex foundation environment has sometimes led to the adoption of hybrid systems combining shallow and deep foundations, and piles of different lengths. The performance of such foundations under strong earthquakes is very difficult to predict unless a robust advanced seismic analysis is carried out.

Performance of Adjacent Structures

Two adjacent buildings shown in Figure 18 exhibited a number of features related to the above discussion. One of the buildings is on shallow foundations, while the other is on hybrid shallow and deep foundations with piles of different length. Both buildings suffered noticeable residual tilt. One would anticipate some degree of interaction between these two buildings during strong shaking and even substantial influence being exerted on the adjacent building through the foundation soils/system (i.e. structure-soil-structure interaction, e.g. Chen et al., 2010). In the interface zone, both buildings contribute to the stresses in the soil. Since the buildings have different foundations and oscillate differently from each other, they will impose different dynamic loads and stresses in the interface zone throughout the depth of the foundation soils. This in turn will change the deformations and pore water pressures in the interface soil zone and will influence the foundations and overall response of the adjacent structure.

Pile-Supported Structures

Several pile supported structures were identified in areas of severe liquefaction. Although significant ground failure occurred and the ground surrounding the structures settled, the buildings supported on piles typically suffered less damage. However, there are cases where pile-supported structures were damaged in areas that underwent lateral spreading near the Avon River.

Figure 18. Residual tilts of adjacent buildings

In other cases, such as the building shown in Figure 19, the ground floor garage pavement was heavily damaged in combination with surrounding ground deformation and disruption of buried utilities. The structural frame of the building supported by the pile foundation with strong tie-beams apparently suffered serious damage, though this damage cannot be attributed to a poor performance of the foundation. The settlement of the surrounding soils was substantial, about 30 cm on the north side and up to 17 cm on the south side of the building, after the 22 February earthquake. Across this building to the north, is a 7 storey reinforced concrete building on shallow foundations that suffered damage to the columns at the ground level. This building tilted towards south-east as a result of approximately 10 cm differential settlement caused by the more severe and extensive liquefaction at the south, south-east part of the site.

It is interesting to note that in the vicinity of these two buildings, the site liquefied during the 4 September 2010. Following the extensive liquefaction in the 22 February 2011 event, there was extensive liquefaction again during the 13 June 2011 earthquakes. Deformations are cumulative with every liquefaction event, and the ground around the building in Figure 19 settled an additional 20 cm in the June earthquake. In the worst spot, the cumulative settlement of the ground exceeded 50 cm.

Effects of Pronounced Ground Weakness

At several locations within the CBD, well-defined zones of ground weakness were localized over a relatively small area (part of a block), but sometimes continuous features run over several blocks adversely affecting a number of buildings and foundations. Within these weak zones, surface cracks, fissures, and depression of the ground surface, as well as substantial volumes of water and sand ejecta were evident. There was a marked difference in the performance between buildings of similar types and construction detail that were literally 20-30 metres apart, one sitting on the bad stretch of the heavily liquefied soil and the other on a slightly higher level with no signs of liquefaction or ground distress. The buildings sitting on the higher ground showed no evidence of damage, whereas uneven settlements and tilts were commonly observed for buildings on shallow foundations sitting in the liquefied zone.

Lateral Spreading

In areas affected by lateral spreading, the horizontal stretching of the ground adversely affected several buildings causing damage to the foundations and superstructure, lateral movements and tilt of buildings. The effects of spreading within the CBD were localized, but quite damaging to buildings and services within the affected zone. Typical stretching of the foundations resulting in damage of the structure (opening of expansion joints) is shown in Figure 20.

Figure 19. Substantial settlement of surface soils due to liquefaction; deep pile foundations prevented significant settlement of the building

Figure 20. Stretching of foundations due to lateral spreading resulting in opening of the expansion joints

6 Comparison of Extent of Liquefaction between 2010-2011 Earthquakes and a M_w=8.0 Alpine Fault Event

In soils susceptible to liquefaction, the strong ground shaking produced by earthquakes causes rapid build-up of excess pore water pressures (increase in the groundwater pressure) and eventual soil liquefaction through a complex dynamic process. While the ground shaking affects the development of liquefaction in a number of ways, there are two key parameters of the ground shaking that practically define whether liquefaction will occur or not at a given site. These are the amplitude of ground shaking (i.e. the size of the ground oscillation/movement) and the duration of shaking (or the number of significant cycles of shaking). In the simplified procedure for liquefaction evaluation (Seed and Idriss, 1982; Youd et al., 2001), the peak ground acceleration (PGA) is used as a measure for the amplitude of ground shaking while the earthquake (moment) magnitude (M_w) is the proxy for the duration of shaking (i.e. significant number of stress cycles).

Figure 21 depicts such a relationship between the earthquake magnitude (M_w) and the number of significant cycles of shaking (N_C). It suggests for example that, on average, a magnitude M_w=7.5 earthquake has 15 significant cycles. A large magnitude M_w=8.0 earthquake has 22 cycles while a magnitude M_w=6.0 has only 5 significant cycles. It simply reflects the fact that the size of the earthquake magnitude is related to the size of the fault rupture and hence the duration of shaking. Using this simple concept, we can examine the potential impact on Christchurch, and the CBD in particular, of a magnitude M_w=8.0 earthquake generated by the Alpine Fault, and compare it to the liquefaction induced in the 22 February 2011 earthquake. The calculations summarised below are preliminary and specific to triggering of liquefaction and should be restricted to such use only.

Using this method, we can calculate the required peak ground acceleration (PGA) that a magnitude M_w=8.0 Alpine Fault earthquake will have to produce in the CBD in order to induce liquefaction within the CBD similar to that observed in the 22 February earthquake. In the calculations, the magnitude scaling factor (*MSF*) accounts for the

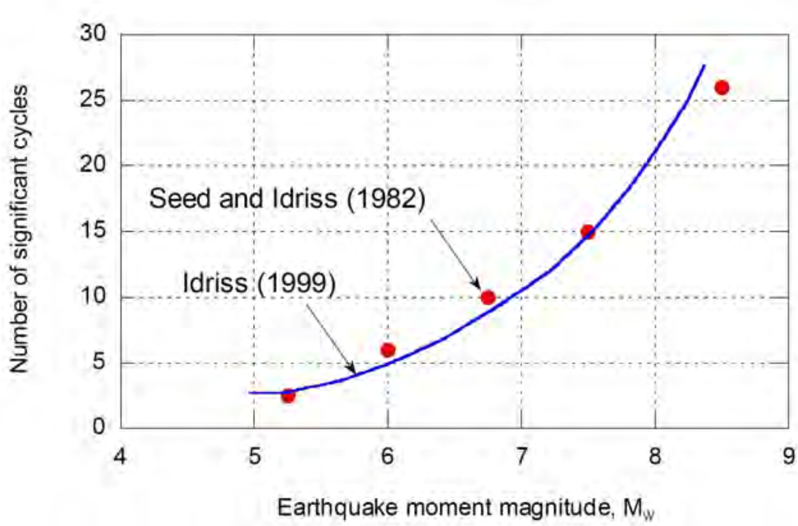


Figure 21. Relationship between the number of significant cycles and earthquake magnitude (M_w) (reproduced from Idriss and Boulanger, 2008)

different number of significant cycles associated with different earthquake magnitudes and also for the seismic ground response specific to liquefaction evaluation. The calculation and interpretation are a bit more demanding and for details the reader is referred to Youd et al. (2001), and Idriss and Boulanger (2008). Three different expressions for calculating *MSF* were employed to allow for uncertainties and differences in interpretation, as summarised in Table 3. The results of the calculations are summarized in Table 3 for the records obtained at the four strong motion stations within or in the vicinity of the CBD (CBGS, CHHC, CCCC and REHS).

The calculation implies that a magnitude M_w=8 Alpine Fault earthquake would produce similar liquefaction effects to those observed during the 22 February 2011 earthquake if it produces PGAs within the CBD half the size of those recorded in the February earthquake. In other words, the Alpine Fault event will have to produce PGAs within the CBD in the range between 0.165 to 0.25g in order to induce liquefaction effects similar to those observed in the 22 February 2011 earthquake. The specific PGAs required to be produced by the hypothetical M_w=8.0 Alpine Event within the CBD are listed in Table 3 (back-calculated from the recorded CBD stations), and are illustrated in Figure 22 (with the yellow band).

Results from probabilistic seismic hazard analysis suggest that a magnitude 8.0 Alpine Fault earthquake will produce PGAs in the range between 0.06g and 0.17g (shown by the blue band in Figure 22). These estimates are based on median PGAs from seismic hazard analysis for a site Class C (PGA ≈ 0.05g to 0.06g.) (GNS Reference), and allow for amplification of ground motion (almost by a factor of two) due to local site and basin effects (resulting in a PGA ≈ 0.1g) and uncertainties (± one standard deviation, or multiplication factors of 0.6 and 1.7 respectively).

As shown in Figure 22, the simplified method suggests that a M_w = 8.0 Alpine Fault event will induce less liquefaction than the 22 February 2011 earthquake. While this outcome appears reasonable in average terms, one has to acknowledge that there might be cases in which worse effects and poor performance will result from the much prolonged duration of shaking caused by the Alpine Fault event.

A similar comparison presented in Figure 23 shows that a M_w = 8.0 Alpine Fault event could induce similar level of liquefaction to that caused by the 4 September 2010 earthquake.

Table 3. Peak ground accelerations of the 22 February 2011 (M_w=6.3) earthquake converted to equivalent PGAs for M_w=8.0 event

MSF expression used	Geometric Mean PGA (g)				Multiplication factor used for PGA, $MSF_{8.0}/MSF_{6.3}$
	CBGS	CCCC	CHHC	REHS	
Expression 1[a)]	0.321	0.275	0.234	0.334	0.64
Expression 2[b)]	0.270	0.232	0.198	0.282	0.54
Expression 3[c)]	0.225	0.193	0.165	0.235	0.45

a) $MSF = 6.9\exp\left(\frac{-M}{4}\right) - 0.058$ (Idriss and Boulanger, 2008)

b) $MSF = \frac{10^{2.24}}{M_W^{2.56}}$ (Lower bound MSF recommended in Youd et al., 2001)

c) Upper bound MSF recommended in Youd et al., 2001

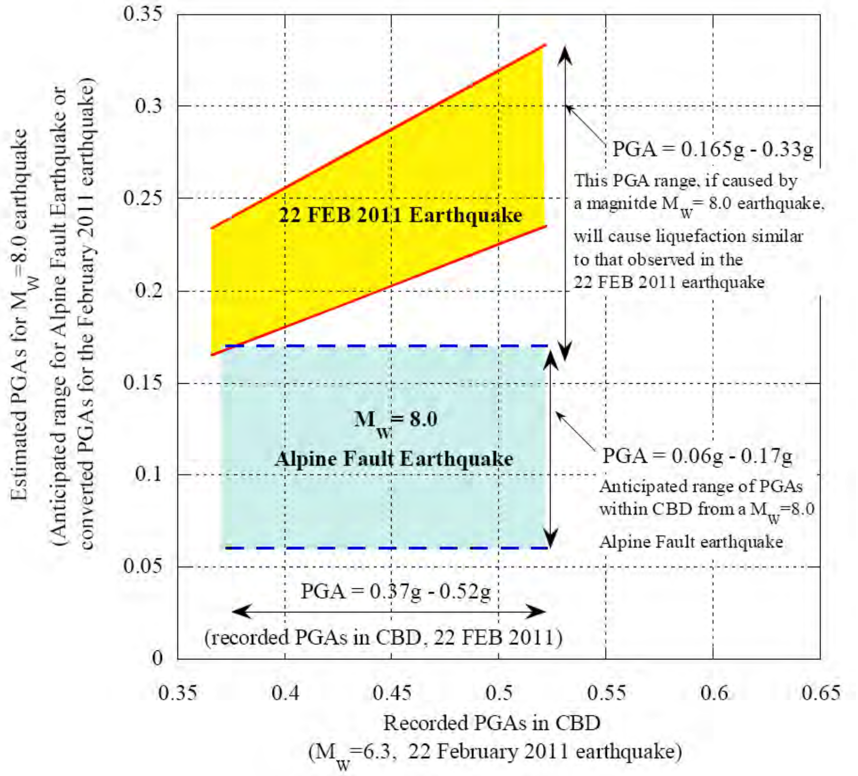


Figure 22. **Comparison of anticipated range of PGAs within CBD from a M_w=8 Alpine Fault event (blue zone) and range of PGAs from an M_w=8 event causing liquefaction similar to the 22 February 2011 earthquake (yellow zone)**

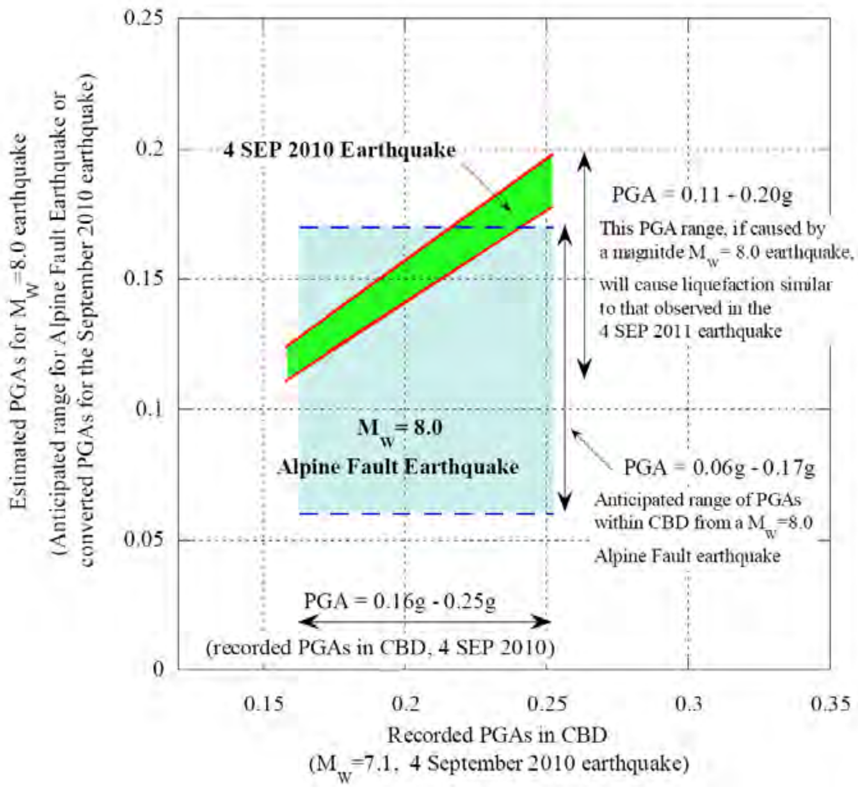


Figure 23. **Comparison of anticipated range of PGAs within CBD from a M_w=8 Alpine Fault event (blue zone) and range of PGAs from an M_w=8 event causing liquefaction similar to the 4 September 2010 earthquake (green zone)**

7 Typical Methods of Founding Buildings which Would Avoid such Failures

7.1 General considerations

Traditionally, the design of foundations for multi-storey and high-rise buildings has been governed by several factors balancing the uncertainties in predicting soil behaviour with the performance objectives and required cost for achieving those objectives. For gravity loads this can be achieved by conventional means (design practice) as proven by the CBD building foundations which performed satisfactorily under normal conditions (in the absence of strong earthquakes), with few exceptions. In other words, the actual settlements of the buildings under gravity loads were in the range of the predicted values, and the serviceability of the buildings was as designed and expected.

For seismic design, the uncertainties both in loads (ground motion characteristics) and soil behaviour (how the soil is going to deform and modify the ground motion) are significant. Importantly, one needs to consider the building (superstructure), its foundations and the underlying/supporting soils as one system, and understand how these individual but critical components will interact. One should understand the behaviour of the system during strong ground shaking and what will be the contribution of the foundation soils and the foundation itself to this behaviour. In case of good ground conditions (strong and stiff soils, or rock), the conventional methods focussing on the performance of the superstructure alone are appropriate since the effects of the soils and foundations on the building response will be relatively small. In case of deep alluvial soils, however, the effects of the foundation environment could be significant and potentially detrimental to the response of the building including relatively large and unacceptable residual deformations (settlement, tilt, and lateral displacements). For important structures, this calls for comprehensive geotechnical investigations of the site and robust design methodology considering the soil-foundation-superstructure system including use of in-depth analysis to scrutinize the performance of the system.

Considering the best practices internationally, the issues of foundations on deep alluvial soils have been addressed in two ways, either by following the above methodology and employing site-specific investigations and design, or by avoiding locations with difficult soil conditions. The former has been adopted in areas where complex soil conditions are prevalent and representative for that environment, whereas the latter has been followed in areas where alternative and better ground conditions are readily available.

7.2 Required geotechnical investigations

Because of the variable nature of the alluvial soils, it is essential to identify what soils are present and what is their spatial distribution under the site of interest so that an appropriate foundation can be designed. For important structures, this typically involves both field and laboratory testing of soils.

Field testing is required to (a) identify the different types of soils and their stratification under a site, (b) evaluate in some fashion the strength and compressibility characteristics of each layer, and (c) assess the behaviour of the soils and the site as a whole during strong earthquake excitations. The specific types of tests will depend on the soil types. For example, liquefaction will be an issue to address for sandy soils susceptible to liquefaction, whereas cyclic softening will be of principal concern for clayey and peaty soils. The number of required tests and their distribution at the site will be also highly variable. Sites with relatively uniform soil profiles across the site and areas where geotechnical engineers have good understanding of soils would require fewer tests to quantify the soil properties at the site in question, and confirm the appropriateness of use of other data from the area/neighbourhood. In case of highly variable soil conditions where the soil profile changes substantially over short distances, a larger number of tests would be required to develop good understanding of the soils and identify the most competent/appropriate layer for the foundations.

For the assessment of seismic behaviour, it is important to conduct appropriate field investigations to evaluate the in-situ state of the soils and provide a nearly continuous log of the soils at the site. This should then be verified and enhanced by selected soil sampling and testing to characterise the key soils, evaluate principal parameters of soil behaviour, and identify key issues of concern. For Christchurch CBD soils, for example, the questions that come to mind are: how large will be the settlements of the alluvial gravel layer, how high will be the excess pore water pressure within this layer, what will be the shear strains (lateral displacements) in the top 10-20 m of the soil deposit, what will be the relative contribution of deeper sandy soils underlying the gravel layer, are those deep layers going to liquefy or not, and if yes, what will be the consequence of liquefaction of these soils in terms of transient (changing in time) and permanent (residual) deformations. A number of similar questions can be asked for the peat layers near the ground surface, but focussing towards the cyclic softening of these soils imposed by the ground shaking and its impact on their deformability. Since, the alluvial soils change rapidly both horizontally and vertically, some considerations also have to be given to the interaction between different soil layers during earthquake shaking. All of this calls for the use of more comprehensive approaches in the testing programme, interpretation and analysis. It is a site-specific exercise that requires a comprehensive effort, covering soil behaviour somewhat in detail as well as a holistic approach in the evaluation of the building performance by considering the response of the soil-foundation-building system through detailed review and analysis.

Such an approach will significantly reduce uncertainties in relation to soil composition and expected dynamic behaviour. The process will inform the designer and will provide critical feedback on the anticipated behaviour of the system and all of its critical components. One may argue that added cost in investigations and analyses always leads to greater insights and it may save money (or provide the evidence to compel one to invest more money for a better performance).

The field investigations will have to be spaced appropriately and dense enough to pick up any potential weak ground zone with pronounced poor performance and expected large damage levels in strong earthquakes. The land damage evidence within CBD compiled from the 2010-2011 earthquakes provides good guidance on the location of such zones.

7.3 Types of foundations required

Robust shallow foundations often accompanied by ground improvement and deep pile foundations reaching competent foundations layers at large depths are appropriate for founding buildings on deep alluvial soils. These types of foundations have shown to provide an improved and satisfactory performance during strong ground shaking caused by earthquakes. Attention to details and selection of appropriate ground improvement methods and pile types are important for achieving these higher performance objectives.

A wide range of ground improvement methods are available using soil densification, solidification, drainage or underground walls to reduce the deformability of shallow soft soils and to reduce the potential for liquefaction and its effects on buildings (NZGS, 2010; Kramer, 1996). Such ground improvement measures will provide stiff and strong soils to support shallow foundations and will ensure much smaller and acceptable level of displacements (e.g. settlements, differential settlements, tilt and lateral displacement) and deformations both during the shaking and post event (residual ones). Again, site-specific and structure-specific considerations are needed to meet the requirements particular to the structure and address the key issues specific to the site in question. This ground improvement, when combined with robust foundations which are tied-together and work as a strong and stiff unit will improve the performance and minimise the adverse effects of differential movements of the foundations on the superstructure. The stiffening of the foundation soils may in some cases allow more seismic energy to enter into the superstructure and hence these foundations will have to be accompanied by a more robust design of the superstructure where energy dissipation mechanisms, damage-control devices or other structural solutions will ensure adequate performance of the building itself, and the system as a whole.

For deep foundations, it is critical to ensure that the piles reach sufficient depth and transfer loads to competent bearing stratum. Ductile piles that have significant lateral capacity and ensuring sufficient residual capacity to carry vertical loads post event, even if damaged during the earthquake, are essential. Robust pile caps and tie-beams rigidly connecting the pile tops will ensure that the foundation works as a single unit and is spreading the loads more uniformly through the foundation members and foundation soils thus ensuring smaller deformation and impacts of the foundation subsystem on the superstructure. The pile construction method also should be included as part of the design as different pile types have different deformation characteristics and interaction with the soils. Again, such foundation design has to be accompanied by adequate structural design of the superstructure itself to ensure good performance of the building and the system as a whole.

7.4 Analysis and verification of seismic performance for important structures

In the above process, it is critically important to ensure good communication between the geotechnical engineer and structural designer to better define the system and understand the role of each component in the seismic performance/behaviour. To achieve this goal, considerations of the dynamic response of the integrated soil-foundation-structure system should be given which will provide feedback to the designer on the effects of complex phenomena and interactions that are otherwise difficult to anticipate and quantify.

8 Conclusions

Alluvial Soils

The Canterbury Plains are built of complex inter-layered soil formations deposited by eastward-flowing rivers from the Southern Alps towards the Pacific coast. Relatively recent but numerous episodes of flooding by the Waimakariri River, and reworking of soils by the Avon River and Heathcote River have influenced the present day surficial soils. In the top 20 m to 25 m, the CBD soils consist of recent alluvial soils including gravels, sands, silts, peat and their mixtures. The soils are highly variable within relatively short distances, both horizontally and vertically. The following characteristics are of particular importance with respect to their liquefaction resistance:

- Considering their composition (sandy soils and non-plastic silts), age (recent deposits, few hundreds to a few thousand years old) and depositional environment (river, swamp and marine sediments), these soils are generally considered susceptible to liquefaction, and in some cases (when deposited in a loose state) they exhibit very low resistance to liquefaction. The high water table within the CBD and to the east of it makes the development of liquefaction and its consequences (liquefaction-induced damage) more likely and more severe.
- In general, the foundation conditions within CBD are complex and challenging for geotechnical engineers, particularly in regard to their performance during strong earthquakes. The presence of aquifers at depths is a relatively unique feature that potentially may exacerbate the seismic response of the soils above the aquifers during strong earthquakes.

CBD Building Foundations

The strong ground shaking triggered by the series of earthquakes in the period 4 September 2010 and 13 June 2011 caused widespread liquefaction throughout the suburbs of Christchurch and within the CBD. The 22 February 2011 earthquake was particularly damaging for the CBD buildings and their foundations. The principal zone of liquefaction stretching west-east along Avon River affected several high-rise buildings on shallow foundations and deep foundations in different ways.

- Buildings on shallow foundations, supported on loose to medium-dense sands and silty sands that liquefied, suffered differential settlements and residual tilts. The uneven settlements were often damaging to the structure. Several buildings underwent punching settlements and bearing capacity failures (sinking of the building in the soil).
- Pile supported structures in areas of severe liquefaction, particularly when the piles reached competent soils at depth, generally showed less differential and residual movements.
- Multi-storey and high-rise buildings supported on shallow foundations sitting on shallow gravels showed mixed performance. The variable thickness of the gravel layer and underlying soil layers resulted in some differential settlements, tilt and permanent lateral displacements. These adverse effects were especially pronounced in transition zones where ground conditions change substantially over short distances.
- There is evidence that hybrid building foundations (consisting of shallow and deep foundations or piles of different lengths) performed relatively poorly during the earthquakes. Structure-soil-structure interaction of adjacent (multi-storey) buildings was another response feature that somewhat influenced the performance of the foundations.

- Within the CBD, zones of ground weakness (either localized over a relatively small area or sometimes continuous over several blocks) manifested pronounced ground distortion and liquefaction that adversely affected a number of buildings and their foundations. There was a marked difference in the performance of buildings only 20-30 metres apart, one that sat on the bad stretch of the liquefied soil and the other on a ground showing no signs of liquefaction or ground distress.
- The effects of lateral spreading within the CBD were localized but quite damaging to buildings causing sliding and stretching of the foundations and the structure.

Types of Investigations and Foundations Required

Since the alluvial soils change rapidly over short distances, it is important to conduct appropriate field investigations to evaluate the in-situ state of the soils and provide nearly continuous log of the soils at the site. This data should then be enhanced by selected soil sampling and testing to characterise the key soils and soil behaviour. There is a need for well thought and executed investigations, interpretation and analysis. The field investigations will have to be spaced appropriately to pick up any potential weak ground zone or change in soil characteristics. The above is usually a site-specific exercise that requires a development of good understanding of site conditions and soil characteristics, as well as due consideration of the response of the integrated soil-foundation-building system. In this process, it is critically important to address the uncertainties associated with soils, earthquake loads and the adopted analysis approach. One may argue that added cost in geotechnical investigations and analyses always leads to greater insights and added value in the performance.

Robust shallow foundations often accompanied by ground improvement and deep pile foundations reaching competent foundations layers at large depths are appropriate for founding buildings on deep alluvial soils. These types of foundations have shown to provide an improved and satisfactory performance during strong earthquakes. Attention to details and selection of appropriate ground improvement methods and pile types are important for achieving the performance objectives. The foundations should be stiff and strong enough to ensure settlements and damage within acceptable levels, and provide sufficient residual capacity to carry vertical loads post earthquake.

The stiffening of the foundation soils and the foundation itself may in some cases allow more seismic energy to enter into the superstructure and hence these foundations will have to be accompanied by a more robust design of the superstructure where energy dissipation mechanisms, damage-control devices or other structural solutions will ensure adequate performance of the building itself, and the system as a whole.

One should recognize that deep alluvial soils, including potentially liquefiable soils, are not unique to Christchurch, but are a relatively common feature of many other cities and towns throughout New Zealand (and internationally). Similarly, the current design philosophy for buildings is shared both nationally and internationally, and it is therefore prudent to consider the lessons learned from the recent Christchurch earthquakes within this wider context, and to use them as a key advantage in achieving better performance in future earthquakes and a more resilient New Zealand.

Acknowledgements

The authors would like to acknowledge the contribution of Dr. Brendon Bradley (University of Canterbury) and Graeme McVerry (GNS Science) in defining the characteristics of the ground motion for the Alpine Fault event. Very helpful reviews and comments have been provided by Prof. Jonathan Bray (University of California, Berkeley) and Dr Elisabeth Bowman (University of Canterbury). Dr Matthew Hughes helped with the preparation and polishing of some of the figures. The University of Canterbury PhD students Merrick Taylor and Simona Giorgini provided overall assistance in the damage inspections of the CBD building foundations and sites.

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Geotechnical Considerations – Foundations on Deep Alluvial Soils

The Canterbury Earthquakes Royal Commission (the Commission) sought advice from Associate Professor Misko Cubrinovski of the University of Canterbury and Ian McCahon, Principal, Geotech Consulting Ltd about ground conditions in the Christchurch central business district (CBD), including:

- a general review of the alluvial soils found in the CBD and their performance and effects on building foundations in the recent Canterbury earthquake sequence;
- liquefaction and lateral spreading;
- general concepts that should be followed in the design of foundations for buildings on these soils.

The report entitled “Foundations on Deep Alluvial Soils” dated August 2011 is published on the Commission’s website: www.canterbury.royalcommission.govt.nz. It contains an Executive Summary which gives an overview of the content.

The authors note that the report, while containing technical information, describes the geotechnical phenomena, and their effects in non-technical language for a general audience.

The report will be peer reviewed by Professor Jonathan Bray of the University of California at Berkeley. The Commission will publish the peer review on its website when it is available.