Christchurch CBD: Lessons Learnt and Strategies for Foundation Remediation – 22 February 2011 Christchurch, New Zealand, Earthquake

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SUMMARY:
On 22 February 2011, a moment magnitude (M_W) 6.2 earthquake occurred, causing extensive damage to the Canterbury region of New Zealand. Buildings and infrastructure were affected as a consequence of land deformation, including ground oscillation, liquefaction induced settlement and lateral spreading. This earthquake event was characterised by intense shaking, and horizontal peak ground accelerations (PGAs) of up to 0.71g being recorded within the CBD.

The authors have been involved in the post-earthquake assessment of a number of multi-storey buildings within the Christchurch Central Business District (CBD).

This paper considers three aspects, firstly the damage observed to foundation elements; secondly, the reasons why the observed damage occurred; and thirdly, the foundation remediation options that were considered to return the subject buildings to as close to the pre-earthquake condition as practical or to meet the current Christchurch City Council regulations.

Keywords: Geotechnical earthquake engineering, foundation performance, foundation repair options

1. INTRODUCTION

Since September 2010, buildings in the Christchurch Central Business District (CBD) have been affected by a number of significant earthquake events. Significant foundation damage due to liquefaction and lateral spreading within the CBD occurred following the 22 February 2011 earthquake. The authors had been engaged to review the foundation performance of a number of multi-storey buildings within the CDB.

Evidence of liquefaction was identified in the vicinity of each of the buildings studied following the 22 February 2011 earthquake event. The serviceability of the buildings was compromised by the consequential differential settlement.

This paper summarises the observations made on four buildings in the CBD. It provides a comparison of the actual measured settlement of these buildings with the predicted liquefaction induced settlements. The foundation remediation options that were proposed to return these buildings to their condition prior to the earthquakes are also discussed.

2. CANTERBURY EARTHQUAKE SEQUENCE

At the time of writing, the Canterbury earthquake sequence included events occurring on 4 September 2010, 22 February 2011, 13 June 2011 and 23 December 2011.

On 4 September 2010, a moment magnitude M_W7.1 earthquake occurred near Darfield, approximately 40km west of the CBD. This earthquake caused damage in some areas of Christchurch; however little damage due to liquefaction is known to have occurred within the CBD.

A M_W6.2 earthquake occurred near Lyttelton, approximately 7km south east of the CBD on 22 February 2011. This earthquake caused widespread damage in central Christchurch, and damage due to liquefaction and lateral spreading occurred throughout much of the CBD.

Following this event, the authors and their colleagues undertook mapping of the extent and severity of land damage within the CBD. This mapping was based largely on observed surface manifestation of the liquefaction process and included lateral spreading, the presence of ejected material (groundwater, sand and silt), ground cracking and deformation of the ground surface. Further details are provided in Murahidy et al. (2012). A simplified plan indicating the extent and level of the observed land damage reported following the 22 February 2011 event is presented as Figure 2.1.

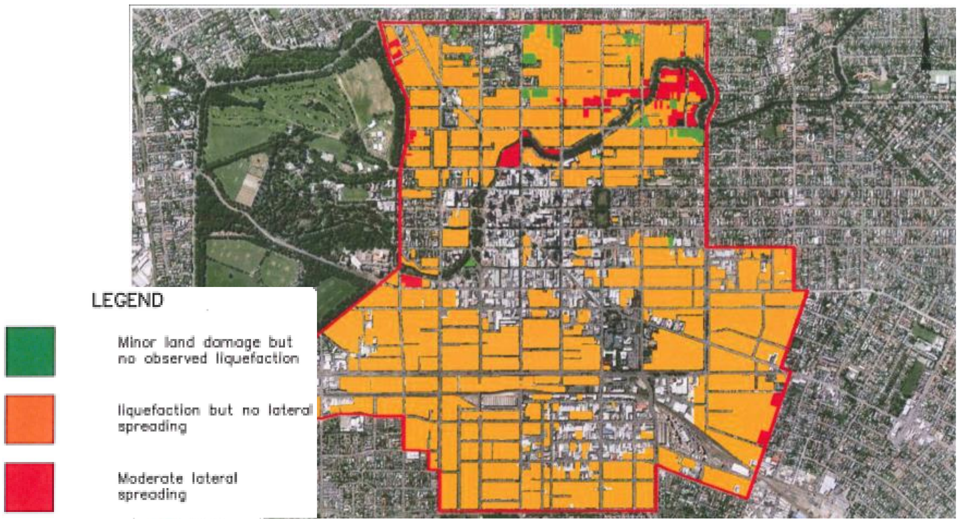

Figure 2.1. Simplified Plan indicating Observed Land Damage following the 22 February 2011 Event

It should be noted that large footprint buildings and thick pavements may have prevented significant formation of sand boils. Additionally, a relatively thick crust of non-liquefiable materials may have prevented surface expression of liquefaction in parts of the city.

An example of the damage caused by the liquefaction process during this event is evident in the photograph taken in the northern CBD (refer Figure 2.2).

Figure 2.2. Evidence of liquefaction in Christchurch, New Zealand

Two significant earthquakes, of $M_W5.6$ and $M_W6.0$ respectively, occurred in the Christchurch area on 13 June 2011. These earthquakes were centred near Sumner, approximately 10km south-east of the CBD. These earthquakes caused further damage in Christchurch and localised areas outside the city.

On 23 December 2011, a further two earthquake events occurred with magnitudes $M_W5.8$ and $M_W6.0$ respectively. These events also caused further damage in Christchurch.

However, neither the 13 June 2011 nor the 23 December 2011 earthquakes triggered widespread liquefaction in the CBD.

Figure 2.3 illustrates the distribution of earthquake events in relation to the Christchurch central business district that occurred between 04 September 2010 and 31 December 2011.

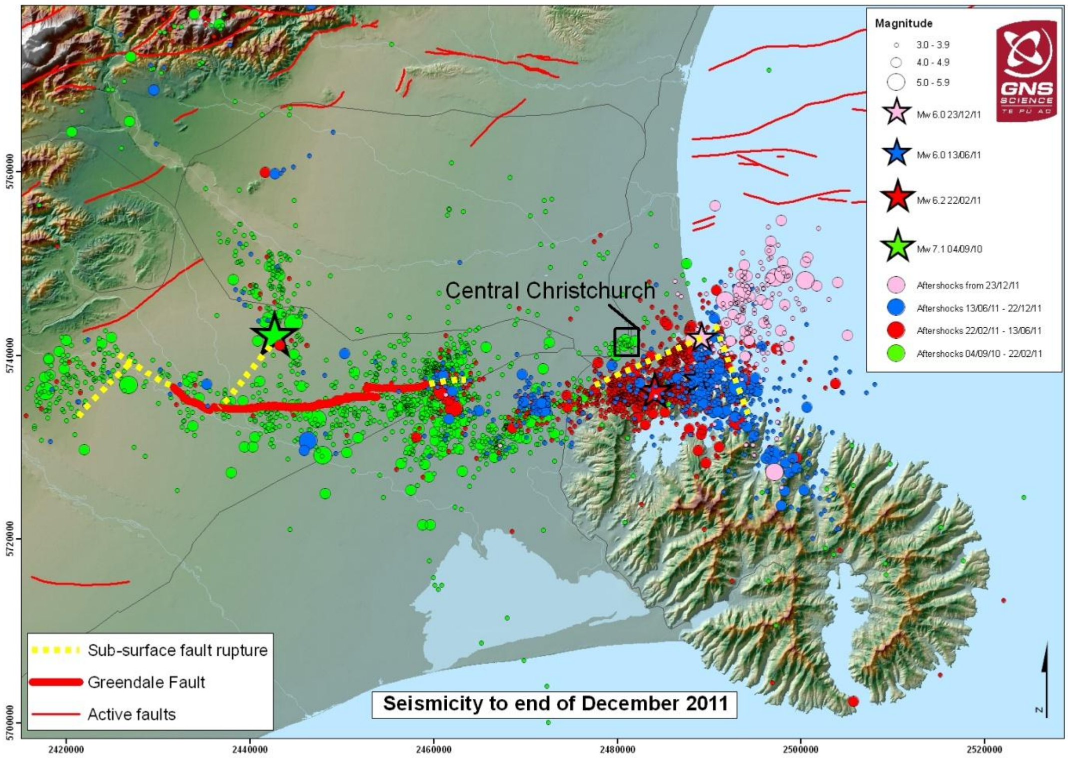

Figure 2.3. Regional seismicity map (from GNS)

3. BUILDINGS STUDIED

Four multi-storey buildings built on shallow foundations were selected as the basis for this study. The buildings were all located within the CBD. These buildings varied in height from six to over twenty storeys and were constructed between 1970 and 1990.

3.1. Building 1

The building was constructed in the mid 1970s and has a total of 13 above-ground stories with a podium extending over the Ground and First Floor Levels. The central tower portion of the building is supported on a prestressed concrete raft slab foundation. The building is located on a flat site bounded by streets to the north, south and west. The Avon River is situated approximately 50 metres away from the building.

3.2. Building 2

Building 2 comprises a 12 storey reinforced concrete building designed and constructed in the 1970s. Structural drawings indicate that the structure is founded on a 900 mm thick prestressed concrete raft foundation with a series of pad thickenings at the column locations. The Avon River is approximately 300 metres from the site.

3.3. Building 3

The building, constructed in the late 1980's and extends over twenty storeys with a single level basement. The building has a central tower surrounded by a two storey podium. The central tower is supported on columns on a grid pattern with a lift shaft in the plan centre of the tower. The building is supported on a heavily reinforced raft, which is thickened beneath the ring of columns supporting the exterior walls of the tower. The structure has been constructed on a river terrace adjacent to the Avon River.

3.4. Building 4

Building 4 comprises a 5 storey reinforced concrete frame structure with a ground floor carpark. The building was designed and constructed in the mid1970's. Structural drawings show that the building was predominantly supported by reinforced concrete pad footings and tie beams, with a concrete raft underneath the stairwell/lift shaft. The site is located approximately 100 metres from the Avon River.

4. GEOLOGICAL CONDITIONS

4.1 Published Geology

Published geological information (Brown et al, 1992) describes the CBD area as being underlain by Holocene age deposits known as the Springston Formation. The Springston Formation comprises units of river deposited alluvial gravel, sand and silt. The Christchurch Formation predominantly comprises marine sands, but locally includes significant gravel, finer material and shells. The Riccarton Formation, a well graded gravel artesian aquifer, underlies the Springston and Christchurch Formation in the Christchurch area.

4.2. Investigations Undertaken

Geotechnical investigations have been undertaken at three of the four above sites to confirm the published geology and to obtain a greater understanding of the liquefaction potential of the underlying silts and sand lenses. Investigations, generally comprising a combination of machine drilled boreholes, Cone Penetration tests and laboratory testing, were undertaken surrounding each building. At the fourth site, sufficient geotechnical investigation information was available from the design investigation to base a condition assessment on.

4.3. Subsurface Conditions

4.3.1. Buildings 1 and 2

At the location of Buildings 1 and 2, the generalised subsurface profile comprises interbedded gravel with sand or sandy gravel with sand lenses to a depth of approximately 6 to 8 metres. These materials are underlain by interbedded alluvial silts, sands and gravels (Yaldhurst Member of the Springston Formation) to a depth of approximately 16 metres. Marine silty sands and sands of the Christchurch Formation underlie these materials to a depth of between approximately 19 and 20 metres below existing ground level. The SPT blow counts from the investigations indicate this material is generally medium dense to very dense.

Underlying the Christchurch Formation sands is a layer of silt/sandy silt, followed by dense sandy gravels of the Riccarton Formation at a depth of approximately 22 to 24 metres below existing ground level. This generalised stratigraphy is shown in Figure 4.1.

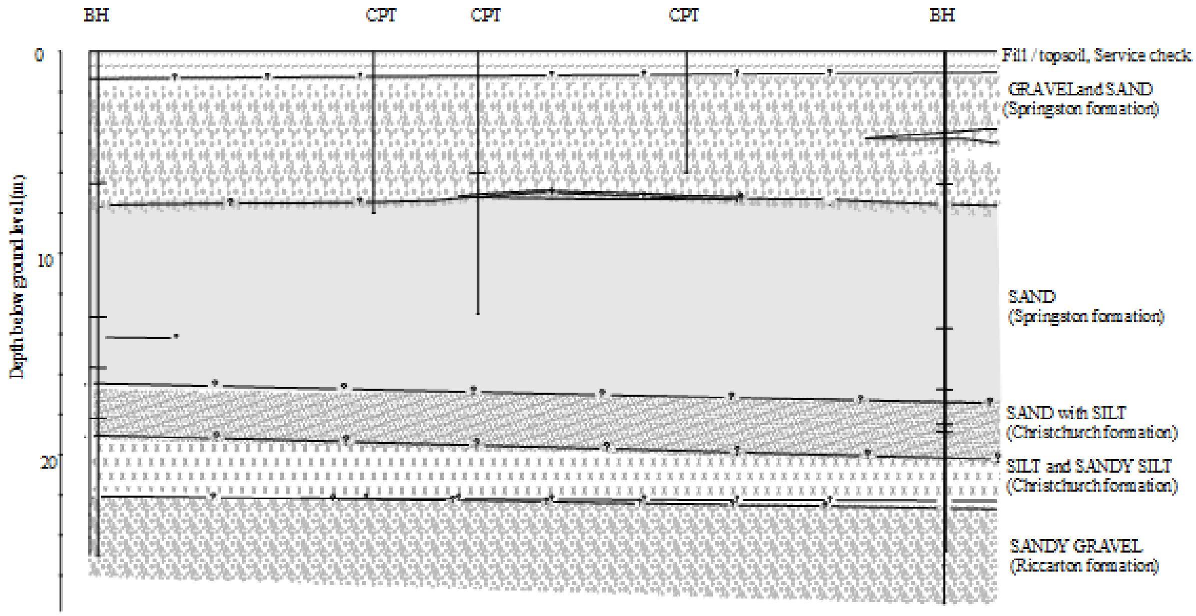

Figure 4.1 Generalised cross section

4.3.2. Building 3

The stratigraphy at the site comprises sandy gravel from approximately 2m below ground level that extends to between 5 and 12m below ground level and increases in thickness across the site from north to south. A sand layer underlies this to between 15.5 and 20 metres below ground level. A sandy silt layer overlies the gravel aquifer (Riccarton Gravels), which was intercepted at depths varying between 20.5 and 23.5m below ground level.

4.3.4. Building 4

Historical maps and published geological maps show that the site is within a previous channel of the Avon River. CPTs undertaken adjacent to the site indicate that the upper 3 to 5 metres depth consists predominantly of loose sands, silty sands and sandy silts. Below these sands, sandy gravels appear to be present to approximately 9 metres depth.

Below 9 metres dense sands are generally present. However layers of silty, possibly organic material are also apparent in a number of the CPT profiles, generally in the 9 to 13 metres depth range. Below 14 metres depth, the soil profiles revert to dense sands.

A silt or organic material was encountered overlying the gravel aquifer (Riccarton Gravels), which was intercepted at depths varying between 21 and 24.5m below ground level.

5. BUILDING DAMAGE: OBSERVED VS ESTIMATED SETTLEMENTS

Land and foundation damage assessments were completed using the following information sources:

- Site walkover inspections by Geotechnical Engineers to identify any evidence of liquefaction, lateral spreading and foundation damage;
- General observations from T&T staff immediately after the earthquake;
- Land damage identified and mapped using aerial photographs; and
- Survey data of vertical and horizontal offsets.

5.1. Foundation Damage Assessment Results

5.1.1. Buildings 1 and 2

A verticality survey indicated that the tower portion of Building 1 was tilting towards the east. Measurements indicate that the western edge of the raft was approximately 30mm higher than the eastern edge, and the building leans towards the east by approximately 70mm at roof level. The raft foundation has settled, on average, by 50mm more than the foundations for the surrounding podium structure. No large scale bearing capacity failures were observed.

The survey results also indicate an approximate differential settlement of 60mm over a distance of 20m between the grade beam supporting the podium and the raft slab.

Verticality survey data for Building 2 indicates that the structure had tilted towards the south and west and has an overall tilt of approximately 50 to 70mm to the southwest over a distance of approximately 25m. The data also showed that in general the internal columns were at a lower relative level than the perimeter columns. However, the overall movement of the internal columns is in general agreement with the overall tilt of the building, though the individual columns have moved different amounts on different floors of the structure. This may be partially due to survey error.

5.1.2. Building 3

The verticality survey data for Building 3 indicates that the structure had tilted towards the south west. The greatest deviation over the 70m high building was to the north face, where it deviated 228mm from vertical. Maximum differential settlement measured at basement level (at the tops of the basement columns) was 93mm at the south west corner, relative to the north east corner. Damage to the foundations included a number of cracks through the raft slab. The cracking of the basement slab resulted in flooding of the basement.

5.1.3. Building 4

A brief survey of ground levels and beam soffit levels at 16 locations within the bottom floor carpark showed differential settlements of the beam soffits of over 300mm between the northwest and southeast of the building, and also considerable movements of the ground level.

This settlement of the ground beneath the building has caused differential settlements of the foundation elements. The photographs in Figure 5.1 illustrate the magnitude of differential settlement of the building.

Figure 5.1. Photographs showing corner of structure pre and post 22 February 2011 earthquake event (note height of metal grill above concrete wall, and corresponding damage to pavement)

The settlement has resulted in a loss of function of the structure (i.e. the building is no longer able to be used in the manner in which it was intended) and structural distress (i.e. beam and column cracking). The total and differential settlements due to seismic consolidation were safely tolerated by the structure without collapse.

5.2. Predicted Liquefaction Induced Settlement

5.2.1. Earthquake Scenarios

The three earthquake scenarios were used in the liquefaction analyses based on subsurface information for each of the subject sites. Two scenarios were derived from "NZS1170 – Structural Design Actions" assuming Importance Level 2 or 3 buildings with a 50 year design life. To meet the New Zealand Building Code requirements buildings are required to be designed to:

- Avoid collapse during a large earthquake (Ultimate Limit State, 500 year return period); and,
- Not suffer significant damage and retain amenity following a moderate earthquake (Serviceability Limit State, 25 year return period).

The NZS1170 scenarios were derived using Amendment 10 to the New Zealand Building Code, which includes an increase in the seismic hazard factor (Z) from Z = 0.22 to Z = 0.30, and increases the return period factor for the SLS event from Rs = 0.25 to Rs = 0.33 for the Canterbury region.

In terms of NZS1170, Class D subsoil conditions (deep or soft soils) were adopted due to the considerable depth to bedrock which is generally present in the Canterbury Region.

The third earthquake scenario that was analysed by the authors comprised the 22 February 2011 earthquake and the recorded ground motion from that event nearest the site. The earthquake scenarios are presented in Table 5.1.

Table 5.1. Summary of the Earthquake Scenarios used in the Liquefaction Assessment

	Serviceability Limit State (SLS)(1)	Ultimate Limit State (ULS)(1)	22 February 2011 Earthquake(2)
Return Period	25 years	500 – 1000 years	
Magnitude, M	7.5	7.5	6.2
Peak Ground Acceleration, PGA	0.11g	0.34 to 0.44g	0.36 to 0.6g

(1) Importance level 2 or 3 structures with a 50 year design working life, scenario developed using amended seismic hazard factor for Christchurch (in accordance with the changes to the Building Code that took effect on 19 May 2011).

(2) Peak ground acceleration interpolated from available accelerometer data recorded at Christchurch Hospital and Christchurch Botanic Gardens.

5.2.2. Liquefaction Analysis

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

Analyses were performed to evaluate the liquefaction potential of the loose to medium dense sands and non-plastic/low plasticity silts found in our boreholes and CPT soundings utilising the methods recommended by Cetin et al. (2004) and Moss, et al. (2006). The three earthquake scenarios described above, and ground water levels of between 1.0 and 1.6 m below ground level were assumed in our analyses.

The seismic settlement of the liquefiable layers identified was computed using the methodology presented by Ishihara and Yoshimine (1992).

5.2.3. Estimated Liquefaction Induced Settlements

The results of the analyses undertaken for Building 1 and 2 indicate the following:

- Most of the medium dense sand layer, located at a depth of between 8 and 16m below the existing ground surface is likely to liquefy under ULS earthquake loading; and,
- A significant portion of this layer is likely to liquefy under SLS earthquake loading.

The computed magnitudes of settlement that may occur at the ground surface at the locations of Buildings 1 and 2 are summarised in Table 5.2.

Table 5.2. Summary of liquefaction induced free field settlement estimates (Buildings 1 and 2)

	Free field settlement estimates		
	Serviceability Limit State (SLS) M= 7.5, PGA = 0.11g	Ultimate Limit State (ULS)) M = 7.5, PGA = 0.34g	22 February 2011 Earthquake M = 6.2, PGA = 0.46g
Building 1	0 – 40mm	25 – 75mm	25 – 75mm
Building 2	80 – 120mm	250 – 300mm	200 – 220mm

The assessment undertaken for Building 3 predicted that the gravelly sands, and silty sands in the layers below the ground water table and above the Riccarton Gravels, where the SPT value was less than N=22, are likely to have liquefied during the 22 February 2011 earthquake. There is a high risk of these layers liquefying in a future ULS earthquake, and a low risk of liquefaction in these layers in a future SLS earthquake. The free field settlement estimates are presented in Table 5.3.

Table 5.3. Summary of liquefaction induced free field settlement estimates (Building 3)

Free field settlement estimates		
Serviceability Limit State (SLS) M= 7.5, PGA = 0.11g	Ultimate Limit State (ULS)) M = 7.5, PGA = 0.44g	22 February 2011 Earthquake M = 6.2, PGA = 0.6g
0 – 20mm	130 – 420mm	50 - 250mm

CPTs undertaken adjacent to the site of Building 4 indicate that the upper 3 to 5 m depth consists predominantly of loose sands, silty sands and sandy silts. There is a high risk that these materials will liquefy in a future ULS earthquake, but a low to moderate risk of liquefaction in these layers in a future SLS earthquake. The liquefaction induced total free field settlements estimated are presented in Table 5.4.

Table 5.4. Summary of liquefaction induced free field settlement estimates (Building 4)

Free field settlement estimates		
Serviceability Limit State (SLS) M= 7.5, PGA = 0.11g	Ultimate Limit State (ULS) M = 7.5, PGA = 0.34g	22 February 2011 Earthquake M = 6.2, PGA = 0.50g
0 – 150mm	75 - 350mm	50 – 325mm

Greater settlements were predicted at the southeast of the site, due to a higher thickness of liquefiable loose sands/silts in the upper layers. Settlements to the southeast of the building have caused building distress and loss of serviceability.

5.3. Comparison Between Estimated Liquefaction Induced Total Settlement and Measured Site Settlements

A comparison between the estimated total settlements, estimated differential settlements and the measured differential settlements is given in Table 5.5. Total settlements (relative to pre earthquake levels) were not measured at the time of the investigations as the survey datums had not been reestablished following the 22 February 2011 earthquake event.

Table 5.5. Comparison of estimated liquefaction induced settlements to measured settlements

	22 February 2011 Earthquake event (M=6.2, PGA = 0.36 to 0.6g)		
	Estimated total settlement	Estimated differential settlement	Measured differential settlements
Building 1	25 – 75 mm	Up to 50 mm	Up to 50 mm
Building 2	200 – 220 mm	Up to 25 mm	Up to 100 mm
Building 3	50 – 250 mm	Up to 200 mm	Up to 100 mm
Building 4	50 – 325 mm	Up to 275 mm	Up to 300 mm

As shown in Table 5.5, the measured differential settlements were less than 100mm for Buildings 1 to 3. This illustrates the effectiveness of the gravel layer to provide rafting effects. This layer however was not sufficient to prevent differential settlements of the foundations, which have resulted in structural distress to the buildings.

6. REPAIR STRATEGIES CONSIDERED

A number of repair strategies were considered to return the subject buildings to as close to the pre-earthquake condition as practical or to meet the current Christchurch City Council regulations. The options considered are presented in Table 6.1. However, due to the presence of adjacent buildings at each site, the only practical options comprised compaction grouting, and jet grouting, or retrospective piling combined with hydraulic lifting.

Table 6.1. Repair strategies considered for various buildings

Method		**Risks and issues**	**Benefits and opportunities**
Jet grouting	Grout is jetted out into the ground from a drilled hole. A grouted soil column is produced and repeated to produce a secant pile wall. Walls constructed in a cellular pattern across the site.	Possibly more expensive than compaction grouting. Jet grouting has not been used in Christchurch soils.	Can be undertaken in confined spaces or within buildings. Create an improved zone of material which would be less likely to liquefy in a design ULS earthquake event. More reliable in silty materials than compaction grouting.
Compaction grouting	Injection of low slump grout under pressure forming spheres of grout and applying lateral pressure to the adjoining ground to induce compaction.	Solution requires a trial to confirm effectiveness in site materials. Effectiveness reduced by presence of silt.	Can be undertaken in confined spaces or within buildings. Create a zone of soil with modified properties that is less likely to liquefy in a design ULS earthquake event. Can be used as a re-levelling technique.
Chemical grouting	Involves injecting liquids under pressure, into the pores and fissures of the ground. The liquid consists of a mixture of mortar and chemical products such as polyurethane, acrylate or epoxy.	Unlikely to be effective in areas where there is a high silt content Control of the chemical grout would also be required to restrict the lateral movement of the grout towards the waterways.	Can be undertaken in confined spaces or within buildings. Create a zone of soil with modified properties that is less likely to liquefy in a design ULS earthquake event.
Retrospective piling	Installation of piles at the location of the existing footings.	Expensive	Building support extended below soils with a potential for liquefaction and settlement.

7. CONCLUSIONS

Many buildings constructed within the Christchurch CBD were founded on shallow foundations, often sitting on a gravel or sandy gravel layer present within a few metres of the ground surface. The depth, strength characteristics and continuity of this gravel layer varies throughout the CBD. Liquefiable sand and silty sands were often present beneath the gravel layer, but the effects of this material on foundation performance was often not taken into account (due to the age of the buildings, the phenomenon of liquefaction had not been identified, or not well understood).

The examples of post-earthquake foundation assessments presented in this paper used visual reconnaissance and new investigation data to assess the assessment of the potential for liquefaction and likely associated free field ground settlement. These assessments have led to the conclusion that for many buildings, shallow foundations are not appropriate to provide the level of building performance expected by many building owners.

It should be noted however, that all of the foundations examined for this paper met the requirements of the New Zealand Building Code, in that the foundations did not fail, causing collapse of the structure. While the foundations performed adequately, such that they did not result in building collapse, the buildings suffered sufficient deformation such that their amenity function has been impaired. This has resulted in either a decision to demolish the building and rebuild, or to repair and relevel the building. Table 6.1 lists repair strategies considered for the buildings.

Where a new building is now required, shallow foundations are generally not considered to be an acceptable option. Options being put forward are either ground improvement of the potentially liquefiable soils or deep piles.

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