Performance and Post Earthquake Assessment of CFA Pile Ground Improvement – 22 February 2011 Christchurch, New Zealand Earthquake

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SUMMARY:
The 'Press House' building in Christchurch, New Zealand is constructed on a foundation system comprising a composite piled raft foundation system incorporating a grid of Continuous Flight Auger (CFA) piles designed to mitigate liquefaction potential.

On 22 February 2011, a moment magnitude (M_W) 6.2 earthquake occurred, causing widespread damage and liquefaction throughout many areas of Christchurch. This event was characterised by intense shaking, with peak ground accelerations (PGAs) of up to 0.71g recorded nearby.

This paper summarises the original design, a post earthquake investigation of the condition of the CFA piles, and an assessment of the performance of the ground improvement.

Keywords: CFA Pile, ground improvement, liquefaction, earthquake.

1. INTRODUCTION

The 'Press House' building in Christchurch, New Zealand is constructed on a foundation system comprising a composite piled raft foundation system incorporating a grid of Continuous Flight Auger (CFA) piles designed to mitigate liquefaction potential.

A post earthquake assessment of the CFA piles was undertaken following the 22 February 2011 Christchurch earthquake, to assess the condition and likely future performance of the CFA piles.

This paper describes the original design philosophy, site observations following a number of severe seismic events, a summary of the post earthquake investigation of the condition of the CFA piles, and an assessment of the performance of the ground improvement/ foundation system undertaken by authors.

2. ORIGINAL DESIGN

2.1. Project Scope

In 2008, the authors were engaged by Ganellen (NZ) Ltd to undertake geotechnical investigations and provide recommendations for foundation design for the proposed redevelopment of three sites formerly occupied by 'The Press' newspaper ("The Press Precinct"). Stage 1 comprised the construction of a 7 storey office building, 'Press House'. Stages 2 and 3 comprised office buildings up to 12 stories in height that have yet to be designed and constructed.

2.2 Geological Conditions

Published geological information (Brown et al, 1992) describes the site 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 Riccarton Formation, a well graded gravel artesian aquifer, underlies the Springston Formation in the Christchurch area.

Geotechnical investigations undertaken at the site consisted of 2 machine drilled boreholes and 4 Cone Penetration tests. The general stratigraphy underlying the site comprises loose sand, silt and gravel deposits extending to approximately 6 m below natural ground level. This material is underlain by medium dense sand, grading to dense sand between 12 and 18m below natural ground level. Underlying this material, a layer of silt extends to a depth of approximately 24m, which is underlain by very dense sandy gravel of the Riccarton Formation. The generalised subsurface profile is shown in Figure 2.1.

The groundwater table generally fluctuates between 1.5 and 3.0m below ground level.

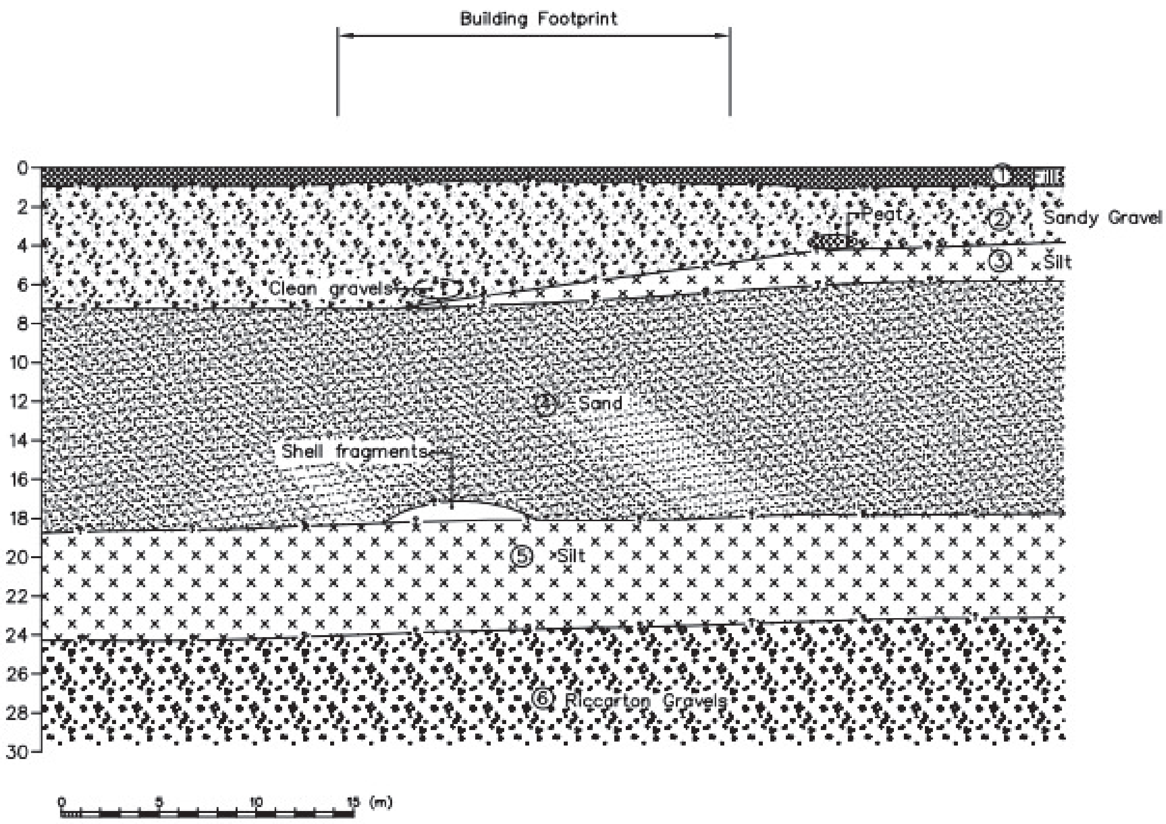

Figure 2.1. Generalised subsurface profile

2.3 Design Philosophy

2.3.1. Introduction

Geotechnical issues that were considered during the development of the concept design included seismic risk and associated ground shaking, liquefaction hazard, bearing capacity and settlement issues, and construction considerations.

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).

During concept design a number of foundation options were considered including shallow foundations, deep foundations and ground improvement.

2.3.2 Seismic Design Actions

The New Zealand Standard "AS/NZS1170 – Structural Design Actions" was used to derive the seismic design actions. The earthquake scenarios used in the foundation system design are presented in Table 1.1.

Table 1.1. Summary of the Earthquake Scenarios used in the Original Foundation System Design

	Serviceability Limit State (SLS)(1)	Ultimate Limit State (ULS)(1)
Return Period	25 years	500 years
Magnitude, M	7.5	7.5
Peak Ground Acceleration, PGA	0.06g	0.25g

(1) Importance level 2 structure with a 50 year design working life and Class D (deep or soft soils) soil assessment

2.3.3 Liquefaction assessment

A review of the underlying site geologic conditions and the potential seismic hazards identified that the effects of liquefaction should be considered for any future development of the site. The assessment of the liquefaction potential of the soils at the site was undertaken using both the CPT and SPT data collected at the site. Liquefaction analyses were undertaken using the method of Seed et al. (2003) to determine if the founding soils were likely to be susceptible to liquefaction.

The assessment indicated that liquefaction is unlikely to occur in the SLS earthquake scenario. In the ULS design case, the analysis indicated that various lenses between 2 – 8m depth were considered very likely to liquefy. Thin, isolated lenses between 8 – 11m depth are potentially marginally liquefiable, with a factor of safety of approximately 1.0 against liquefaction in the ULS earthquake scenario. This is shown in Figure 2.2.

For geotechnical analysis and design, it was assumed that in the ULS earthquake scenario a total cumulative thickness of 3 – 5m of soil liquefies between 2 – 9m depth, distributed as a series of interbedded lenses.

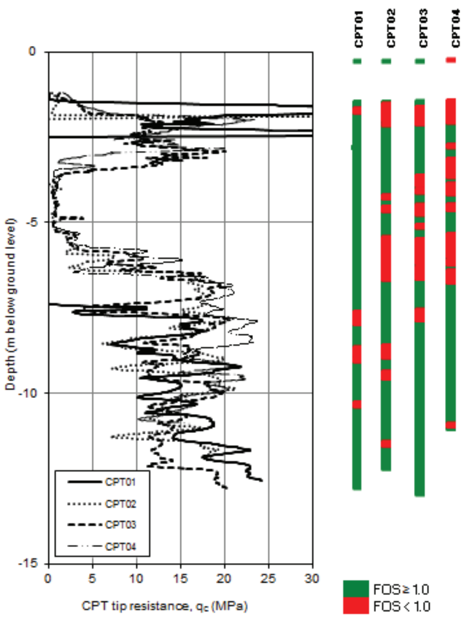

Figure 2.2. Summary of CPT-based liquefaction analysis for ULS earthquake (1/500yr)

2.3.4 CFA Ground improvement design

A CFA pile ground improvement/ shallow foundation system was selected to serve the following key functions:

- Reduce the potential for liquefaction to develop in loose granular soils during seismic loading; and,
- Improve the bearing capacity and reduce settlement for shallow foundations, for both static and seismic design load cases.

The design philosophy is summarised in the following sections:

<u>Design for liquefaction mitigation</u>

The following four steps were undertaken during the design of the CFA ground improvement for liquefaction mitigation:

1. Estimation of the cyclic stress ratio (CSR) at which the soil between piles would liquefy based on the CPT data;
2. Assessment of the proportion of the imposed cyclic stress which must be concentrated in the piles to limit the CSR in the soil between the columns to below the triggering value for the ULS design earthquake. In the analysis the authors aimed to provide a factor of safety against liquefaction of greater than 1.1 for at least 95% of the soil volume. This required that at least 75% of the cyclic shear stress is concentrated in the CFA piles;
3. Determination of the CFA pile spacing required to carry the proportion of imposed cyclic stress calculated in Step 2 was based on the relative shear stiffness of the piles and ground. The shear stiffness of the concrete piles times their cross sectional area had to be at least 3 times that of the soil stiffness times the area. The analyses indicated that for 600mm diameter CFA piles, a spacing of 2.8 – 2.9m (on a triangular grid) was sufficient.
4. Determination of the design action envelopes for pile structural design. This allowed the structural designer to ensure that the pile had sufficient shear strength to carry the concentrated cyclic stresses, and sufficient bending strength to withstand the imposed cyclic ground displacements.

Information presented in Baez and Martin (1993) was applied in developing the design.

<u>Design for bearing capacity and settlement</u>

The foundation system comprised the CFA pile ground improvement, in combination with a series of wide & stiff foundation ground beams. Geotechnical finite-difference software FLAC was used to analyse the performance of the system and examine various aspects of the foundation response.

The first analysis modelled the ground beneath the building as a series of unit-cells, consisting of a single pile, the surrounding tributary soil and foundation above. Due to the symmetry of the unit-cell an axisymmetric analysis was possible, to allow the 3-dimensional interaction of the pile and soil to be assessed. This analysis provided an initial assessment of the settlement vs. load response of the foundation, confirming the proportion of vertical-load sharing between the pile and surrounding ground.

The second analysis modelled a 2-dimensional cross section along the building gridline where the most severe foundation loads are applied. This enabled the distribution of bearing pressures beneath the foundation beam to be assessed, to confirm the bearing capacity and settlement response of the foundation for the imposed loadings. Two key loadcases were considered: service loading (deadload + live load, G+Q), and seismic over-strength loading on the foundations for the ULS design earthquake scenario in the east and west directions (G+Q+Eu).

In addition to these finite-difference analyses, simplified hand-calculation checks were also performed to confirm the predicted magnitude of settlements and load sharing between the piles and surrounding ground.

Settlement

Figure 2.3 shows the predicted accumulation of settlement with depth beneath the west and east edges of the building at the selected gridline. These plots show that little settlement accumulates over the upper 9m of the soil profile, as the majority of the vertical loading is carried by the CFA piles. For the various cases analysed, approximately 15-20mm settlement is predicted to occur in the medium-dense to dense sands which extend for approximately 9m beneath the toe of the piles. A further 15-20mm settlement is predicted to occur in the deep silt layer which is present at approximately 18-24m depth. Our model assumes that the gravelly soils inferred to exist below 24m depth are relatively incompressible. As the majority of this settlement occurs at significant depth, the resulting differential settlements at the ground surface are expected to be minor.

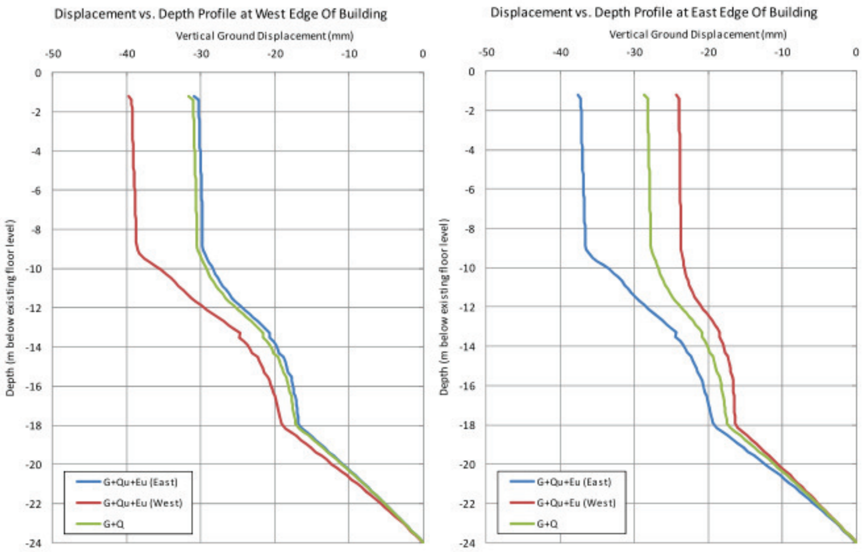

Figure 2.3. Predicted displacement versus depth profiles along selected gridline

A study was also undertaken to check the effect of potential localised areas of softer ground at individual pile locations. Three sensitivity cases were analysed, considering soft ground beneath a line of columns, a line including the core walls, and a line including the side walls.

These analyses show a slight increase in the predicted peak ground beam moments if there is soft ground beneath a heavily loaded column/wall, and a moderate increase in the building-edge settlement if there is soft ground under the side walls. This is shown in Figure 2.4.

Due to the good control of differential settlement offered by the rafting action of the stiff and wide foundation beams in combination with the ground improvement, the structural designer was able to detail the structure to accommodate the estimated differential and total settlements.

Bearing capacity

The bearing capacity of the foundation system was assessed by generating load vs. displacement curves for the various loadcases being considered.

For the static case under service loads (G+Q), a design factor of safety of at least 3.0 was targeted against bearing capacity failure. At three times the service load the graphs indicated that there would be a slight softening of the load vs. displacement response (due to piles being forced to carry more of their load in end-bearing rather than shaft friction, and sharing more load with the surrounding soil beneath the ground beam), but that the foundation would still perform well.

Similar results were achieved for the seismic cases (G+Q+Eu), where a factor of safety of at least 1.2 was required to correspond to the capacity reduction factor of phi Φ=0.85 (generally adopted for resistance of foundation loads from seismic over-strength actions).

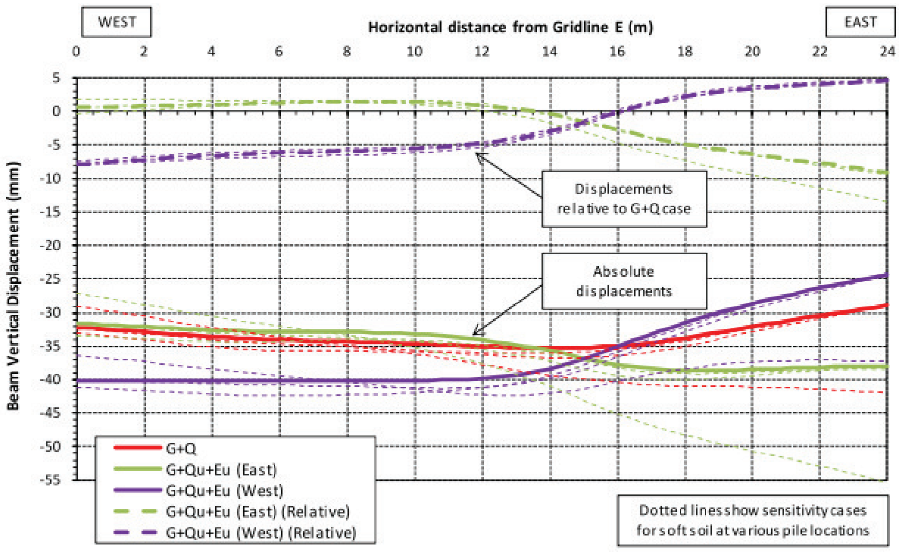

Figure 2.4. Predicted displacement along selected gridline

2.3.5 Design summary

The final design comprised 210 No. 600mm diameter CFA piles as shown in Figure 2.5. This layout includes a zone of closely-spaced piles around the perimeter of the building footprint. This zone is provided to improve the stiffness of the foundation at the edges (important for limiting seismic tilting), and to minimise the negative effects of liquefied soil outside the building footprint.

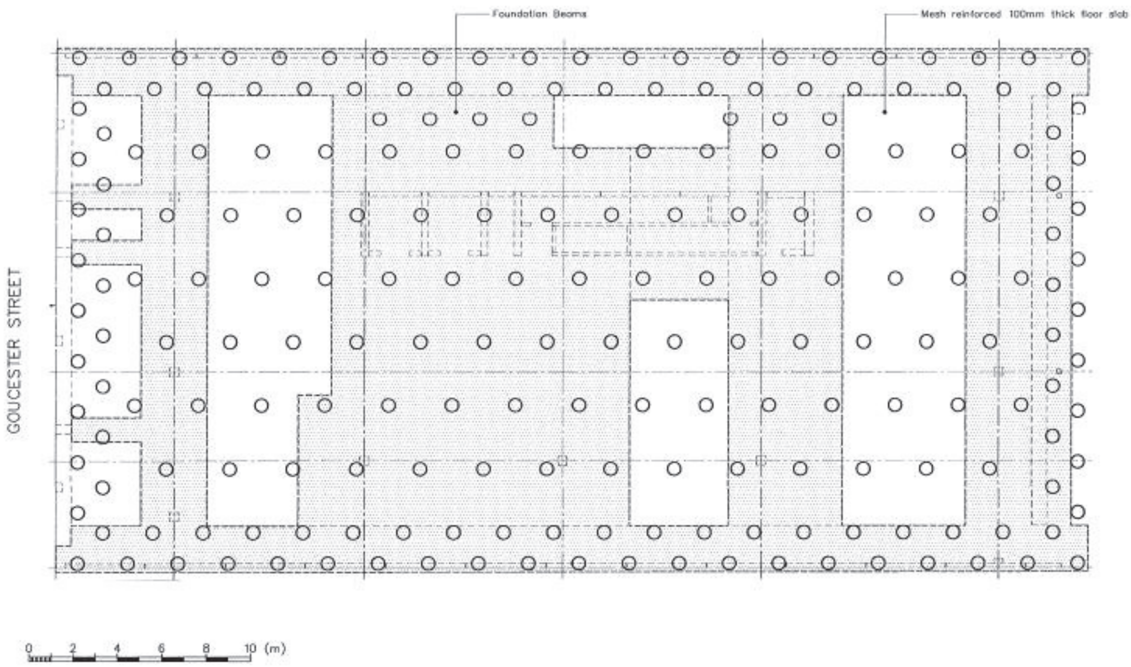

Figure 2.5. Final CFA Pile Design Layout

3. SYSTEM PERFORMANCE DURING 22 FEBRUARY 2011 EARTHQUAKE

3.1 Introduction

At the time of the 22 February 2011 earthquake, the construction of the structure was nearing completion, with internal finishing being undertaken.

Following our discussions with the project team a series of intrusive investigations were undertaken to assess the condition of the CFA piles following the earthquake events.

3.2 Canterbury Earthquake Sequence

At the time of writing, the Canterbury earthquake sequence included significant events occurring on 04 September 2010, 22 February 2011, 13 June 2011 and 23 December 2011.

On 04 September 2010, a M_W7.1 earthquake occurred near Darfield, approximately 40 km west of the site. This earthquake triggered liquefaction in some areas of Christchurch; however no liquefaction is known to have occurred at the site or surrounding neighbourhood.

A M_W6.2 earthquake occurred near Lyttelton, approximately 7 km south east of the site on 22 February 2011. This earthquake caused widespread damage in central Christchurch, and liquefaction occurred throughout much of the central business district. The nearest surface evidence of liquefaction was observed approximately 150m east of the site. The soil profile at this location was similar to that of the subject site.

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 10 km south-east of the site. These earthquakes caused further damage in Christchurch and localised areas outside the city. No liquefaction was noted in the general vicinity of the site.

On 23 December 2011, a further two significant earthquake events occurred, of M_W5.8 and M_W6.0 respectively. These events also caused further damage in Christchurch; however, no liquefaction was noted in the general vicinity of the site.

3.3 Visual Assessment Following Seismic Events

There was no surface evidence of liquefaction as a consequence of the Canterbury earthquake sequence in the direct vicinity of the site. Sand boils appeared approximately 150m east of the site with a similar soil profile.

Survey measurements indicated that the south side of the building settled by up to 30mm relative to the remainder of the building. This differential settlement did not result in visible cracking of the building. This settlement could be as a result of liquefaction beneath the CFA pile ground improvement. The 30mm measured differential settlement will include any which occurred during the building construction.

Foundation damage due to settlement occurred at an adjacent theatre complex; however other two storey buildings in the area did not appear to have foundation damage. A six storey building founded on shallow pad footings, located approximately 200m east of the site, suffered significant foundation damage due to liquefaction occurring in the loose sands underlying the ground surface. The soil profile at this location was similar to that of the subject site.

The subject 7 storey building's foundations performed well. A 6 storey building on shallow foundations on a similar soil profile performed poorly.

3.4 Intrusive Investigations

The intrusive investigations included the following:

- Coring of four CFA piles and inspection of the recovered drill core;
- Rising head tests to infer the degree of cracking sustained by the pile by quantitatively measuring the intrusion of groundwater; and,
- Down hole video camera inspection to investigate the source of groundwater intrusion and to identify any obvious cracking.

Further details regarding these investigations are presented in Sutherland and Murahidy (2012).

3.5 Assessment of CFA Pile Performance During Canterbury Earthquakes

The CFA pile ground improvement appears to have successfully mitigated the liquefaction risk at this site in the 22 February 2011 earthquake. This assessment is based on:

1. The minimal settlement indicated by the post earthquake level survey. The measured differential settlement of 30mm could be attributed to liquefaction below the depth of the ground improvement, plus possibly some during construction settlement;
2. The lack of observed damage to the structure; and,
3. The lack of observed evidence of liquefaction present on the site following the earthquake.

4. ASSESSMENT OF FUTURE PERFORMANCE

The design of the ground improvement has been revisited to assess the expected future performance of the foundations system. This included allowance for increased earthquake shaking hazard as a consequence of aftershocks associated with the Canterbury Earthquake Sequence.

4.1 Seismic Hazard Revision

Three earthquake scenarios have been analysed by the authors during the foundation performance assessment (refer to Table 4.1). These analyses allowed for the recent changes to the Department of Building and Housing Compliance Document Clause B1/VM1 that were published on 19 May 2011 for Canterbury, in which the seismic hazard factor for Christchurch was increased. In addition, an analysis was undertaken for the 22 February 2011 earthquake (based on ground motions recorded near the site).

Table 4.1. Summary of the Earthquake Scenarios used in the Liquefaction Assessment

	Serviceability Limit State (SLS)(1)	Ultimate Limit State (ULS)(1)	22 February 2011 Earthquake
Return Period	25 years	500 years	
Magnitude, M	7.5	7.5	6.2
Peak Ground Acceleration, PGA	0.11g(2)	0.34g(2)	0.48(3)

(1) Importance Level 2 structure 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) Refer Table 1.1. This compares with 0.06g and 0.25g which formed the basis of the original design.
(3) Peak ground acceleration recorded nearby.

4.2 Liquefaction Assessment

Liquefaction analyses have been carried out based on the CPT results (from CPTs carried out as part of the original geotechnical investigation) using the method of Seed et al. (2003). Settlement estimations have been undertaken using the method of Ishihara and Yoshimine (1992).

The analysis indicates negligible liquefaction induced settlement within the improved ground block is likely to occur; however we cannot discount the possibility of some cyclic densification settlement. This settlement is expected to be within the structural tolerance.

Table 4.2. Summary of Liquefaction Assessment

		SLS (M=7.5, PGA=0.11g)	ULS (M=7.5, PGA = 0.34g)	22 February 2011 (M=6.2, PGA = 0.48g)
Materials within ground improvement block	Cumulative thickness of liquefied layers	Negligible	Negligible	Negligible
	Estimated liquefaction induced total settlement	Negligible	Negligible	Negligible
Materials below ground improvement	Cumulative thickness of liquefied layers	Negligible	Up to 0.4m thick	Up to 0.2m thick
	Estimated liquefaction induced total settlement	Negligible	Less than 50mm	Less than 25mm (1)

(1) Post earthquake level survey indicated 30mm differential settlement. Estimated and measured settlements are the same order of magnitude, supporting the settlement analysis undertaken.

4.3 Overall Future Performance

In order to assess the ground improvement performance in the 22 February 2011 earthquake, and likely performance in a future earthquake, we have reviewed the ground improvement design with respect to the earthquake scenarios presented in Table 4.1.

In general, the capacity of the foundation system is consistent with design specifications. The expected performance in a future seismic event is summarised in the following points:

1. Under an SLS design earthquake scenario, liquefaction is not expected to occur within or beneath the ground improvement block. It is possible that cyclic densification of material beneath the ground improvement block may lead to minor settlement. However this is within serviceability limits.
2. Under a ULS design earthquake scenario, some minor liquefaction of material beneath the ground improvement block may occur. However, this is not expected to result in settlement which would cause global collapse.

4.4 Shallow Foundation Performance

The performance of the shallow foundations is highly dependent on the performance of the ground improvement to provide suitable bearing. Based on our observations and analyses of the ground improvement, the shallow foundation capacity can be considered to be as designed.

5. CONCLUSIONS

The composite piled raft system performed well through the Canterbury earthquake sequence. We are aware of similar grids of piles being applied to stiffen ground with the objective of mitigating liquefaction. But we are not aware of other examples which have been tested by severe earthquake shaking.

There was no evidence of widespread liquefaction beyond the improved ground and thus we cannot say that the grid of piles stopped liquefaction, but the system did provide reliable support to the 7 storey building. This was likely to be as consequence of a combination of:

- The stiff piles limiting the development of liquefaction;
- The stiff ground beams distributing loads;
- The piles transferring load away from any local weak ground near the surface; and
- A design with inherent redundancy.

Similar sized buildings in the vicinity on shallow foundations performed poorly.

ACKNOWLEDGEMENT

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