

DESIGN AND CONSTRUCTION OF MULTI STOREY BUILDING FOUNDATION PILES IN LIQUEFIABLE GROUND IN CHRISTCHURCH, NEW ZEALAND

H Hendrickson¹

¹WSP Opus, 12 Moorhouse Avenue, Addington, Christchurch 8041. Email: helen.hendrickson@wsp-opus.co.nz

ABSTRACT

A five storey office building is being constructed at St Andrews Square in Christchurch. The building is founded on soft and liquefiable ground and is located 60 m from the Avon River. The ground conditions are variable both throughout the soil column and across the building. Due to the size of the building and the structural system, column loads are very high and vary considerably across short distances. Ground improvement, shallow foundations and deep foundations were considered. Due to the physical and environmental constraints of the site and construction advantages a bored pile foundation was chosen. The bored piles are designed to carry large vertical loads enabling wide column spacing which is favoured in office style buildings. The contractor also founded a tower crane on the piles to help with super structure construction.

The construction methodology involved open-hole technology which is relatively new in the NZ market. A partial length of temporary casing was used with the remainder of the pile open-hole augered using a combination of bentonite and synthetic polymer fluids for side wall support. It required international support on the fluid technology and pro-active risk management from the contractor and onsite staff. This type of pile is quick to construct which allowed part of the superstructure construction to begin before piling was completed. Design considerations including learnings from the Canterbury Earthquakes, pile capacity, negative skin friction and total and differential settlement are presented. The construction supervision required from the consultant in order to achieve Construction Review PS4 sign off is discussed.

1 INTRODUCTION

2.1 Background

A five storey office style building is being constructed in St Andrews Triangle in Christchurch (refer to Figure 1). The building superstructure is designed to importance level 3 and has a design life of 100 years in accordance with Structural Design Actions NZS 1170. The building incorporates supplemental viscous fluid dampers as part of the seismic resistance system. (Brown et al, 2016).

In order to support the structural system, a damage resistant and high capacity foundation is required. The performance of the foundation needs to be carefully considered to protect and ensure performance of the superstructure.

2.2 Ground Conditions

St Andrews triangle is in the Christchurch CBD. It is 60 m from the Avon River. The location of the site is shown in Figure 1.

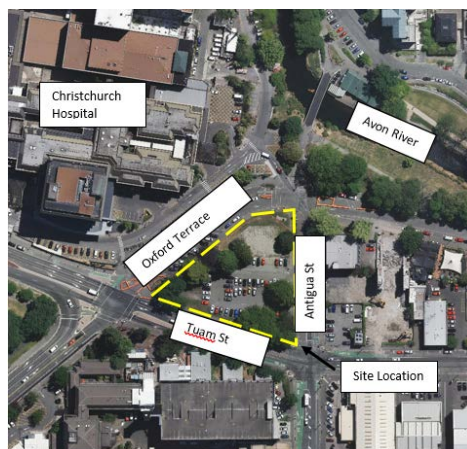


Figure 1: Site Location

Site specific investigations comprised a total of 6 boreholes, 2 cone penetrometer tests (CPTs) and Multi-Channel Analysis of Surface Waves (MASW) testing around the perimeter. Investigations were carried out in three rounds as the project progressed through the various stages. In addition to the site specific geotechnical investigations, there were three ECan well logs available on the site and numerous investigations in the form of CPT's, boreholes and MASW on neighbouring sites.

The site is underlain by several hundred metres of alluvial materials (Brown and Weeber, 1992). A geological cross section of the site is shown in Figure 2.

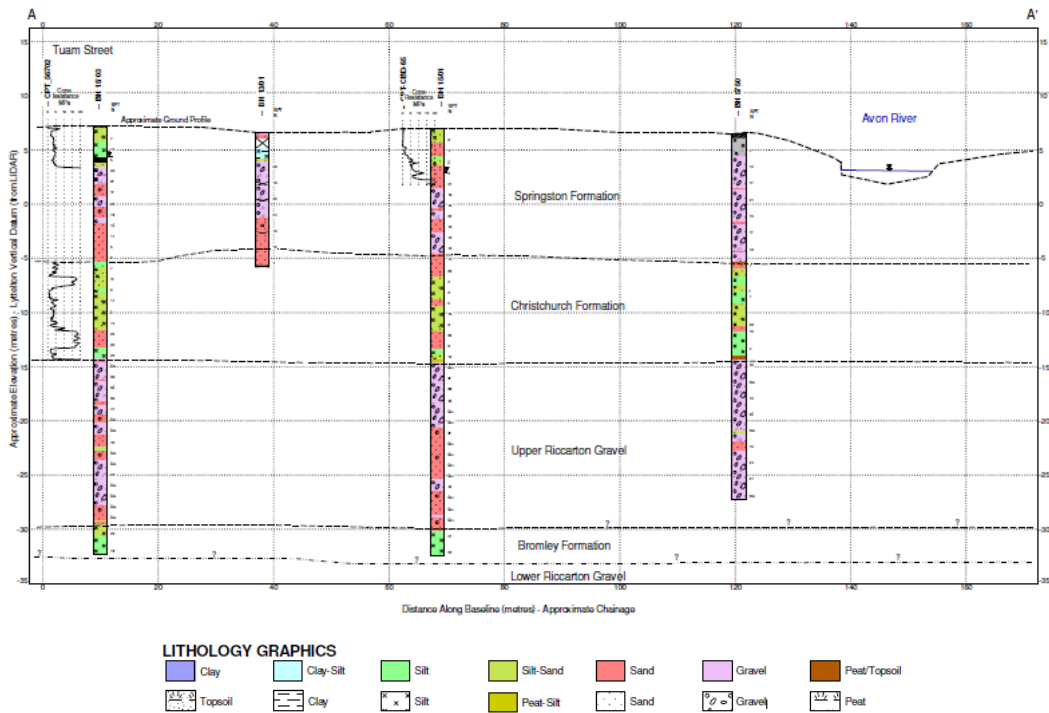


Figure 2: Geological Cross Section

The Springston, Christchurch and Bromley Formations were shown to consist of alluvial silts, sands and gravels and the Upper and Lower Riccarton Gravels were shown to comprise of sands and gravels of varying density.

Groundwater at the site is largely controlled by the nearby Avon River. The static ground water level was determined to range between 3.5 m and 4.03 m below ground level. Additionally, piezometers installed in three of the boreholes indicated artesian and sub artesian pressures of up to 1 m above ground level in the Riccarton Gravels confined by an overlying lower permeability layers containing clay and silt.

The liquefaction analysis undertaken indicated extensive earthquake induced free field subsidence between 250 and 300 mm following a ULS earthquake. (Mw 7.5 and peak ground acceleration (PGA) 0.44g). Liquefaction was predicted consistently across the site between the water table depth at around 2.5 m down to 20 m depth. Liquefaction potential and free field settlements were shown to vary across the site by approximately 50 % due to variations in thickness and density of the underlying soils.

The soil parameters interpreted from the SPT, CPT and MASW data and adopted for the pile design are summarised in Table 1.

Table 1: Soil Parameters Adopted in Design

Description	Depth (top of layer, m)	γ (kN/m ³)	S_u (kPa)	ϕ' (°)
SAND / SILT. Loose, soft, non plastic.	0	16 - 17	25 - 28	24 - 26
Sandy GRAVEL and Gravelly SAND (Medium Dense)	3.6	18 - 19	-	30 - 34
Sandy SILT (Firm) and Silty SAND (Medium Dense) (Christchurch Formation)	12.6	18	33	35 - 38
SILT, LP, organics. Soft to very stiff. (Christchurch Formation)	18.0	15 – 18	20 - 120	-
Sandy GRAVEL, sand and gravel. (Medium dense to very dense) (Riccarton Gravel)	21.4	19 – 21	0	36 - 40
Silt (Soft to Firm) peat, low plasticity (Bromley Formation)	35.0	17 - 18	104	0
Gravelly SAND (very dense) (Lower Riccarton Gravel)	38.6	18	-	36 - 38

Seismically induced lateral spreading is not expected to occur at the site based on observations in the Canterbury Earthquake Sequence and the absence of surface manifestation of liquefaction of the soils on the riverside. This is supported by geotechnical information from neighbouring sites and the CBD wide shear wave velocity profile showing shear wave velocities in the order of 220 to 320 m/s in the upper 10 m along Oxford Terrace. In order to recognise the potential for lateral ground movements such as cyclic ground motions or kinematic displacements regardless of the absence of lateral spread, a fully ductile pile design was adopted and pile capacity checked for a permanent lateral displacement of 200 mm in any direction applied to the top of the pile.

2 SELECTION OF FOUNDATION TYPE

A foundation optioneering process was undertaken to select the most appropriate foundation type for the building.

A range of conceptual foundation types were developed for consideration including a basement, shallow foundations with or without stone column or controlled modulus column ground improvements and piled foundations.

Following preliminary analysis, it became evident that due to the risk of liquefaction of the underlying soils and the resulting risk of loss of bearing capacity, shallow foundations without ground improvement or a basement are unlikely to support the large loads from the 5 storey building, particularly in seismic and post seismic load cases. This would likely lead to intolerable differential settlement and possible tilting of the building. As the building walls were positioned against the boundary of the site, it was also unlikely that there would be sufficient confinement available for stone column ground improvement to be effective. Confinement from additional measures such as perimeter rows of CFA piles or controlled modulus columns would be required.

The remaining options were considered and developed in more detail. Piled foundations or ground improvements were considered with sub-options developed for each. The options considered are summarised as

- Option 1a – Shallow Piles
- Option 1b – Deep Piles
- Option 2a – Stone Column ground improvements with CFA Piles along Antigua Street
- Options 2b – Stone Column ground improvements with CFA Piles
- Option 2c – Ground Improvement in the form of Controlled Modulus Columns

Considerations such as likely static and seismic performance, space requirements, constructability, environmental constraints, design and construction programme and cost were assessed for each potential option. Due to the proximity of the site to the existing hospital, driven piles were deemed inappropriate. Shallow bored piles (Option 1a) into an intermediate gravel layer were not recommended due to liquefaction below this layer resulting in a risk of punching and low pile tip capacity which could result in excessive settlement.

Options 2a to 2c were not also not recommended due to settlement risks from liquefaction of the underlying layers. These options would also require additional CPT testing with predrilling, laboratory testing and shear wave MASW testing in the middle of the site. In addition to this, a thick foundation raft would be required with thickening around the edges. Uplift during seismic loading would need to be resisted by tension piles around the external foundation which would further complicate the design and construction.

Bored piles into the Riccarton Gravels were selected as they presented confidence in the ability to resist uplift and compression loads both in static cases and during seismic loading in the future. The concept and design for the piles was

able to be developed quickly following site investigations. Construction time for piled foundations was reasonable and more certain than for ground improvement.

A configuration of 43 piles founded in the Riccarton Gravels was selected and 1.2 m and 0.9 m diameter piles were used. The piles were spaced on a 9 m grid layout and range in length between 25 m and 26.5 m below ground level. Piles were founded 3 to 4.5 pile diameters into the Riccarton Gravels.

3 PILE DESIGN AND LOADING

The pile loads provided by the structural engineers were maximum load envelopes covering a range of loading cases. The loads are summarised in Table 2.

Table 2: Pile Loads

Load Case	Pile Axial Load (kN)	
	900 mm dia piles	1200 mm dia piles
SLS Static Case (G + Q)	237 – 2296	237 – 3599
ULS Static Case (1.2G + 1.5Q)	300 – 2906	300 – 4556
ULS Seismic Compression (G + EQ)*	120 – 3214 (118 – 570)	1513 – 4611 (282 – 728)
Overstrength*	982 – 3807 (762)	2222 – 6041 (987 – 2019)

*Uplift loads in brackets

Due to the wide column spacings, pile loads are relatively large. In static cases, the highest loads were underneath the core of the building. In seismic cases, the inertia loads were carried mainly by the outer piles. Loads are particularly large underneath columns with viscous dampers attached meaning considerable differences in loads between neighbouring piles are possible. Pile performance in terms of total and differential settlement between piles as well as the ability to resist uplift loads are important factors for structural performance. A typical sketch of the structural system is shown in Figure 3.

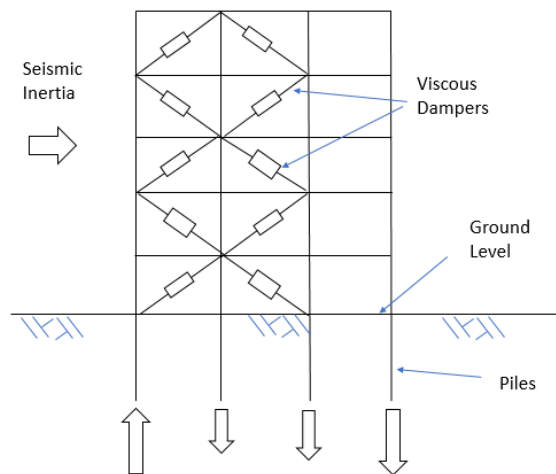


Figure 3: Typical Sketch of Structural System

4 PILE CAPACITY AND SETTLEMENT CALCULATIONS

Pile vertical capacity was calculated and checked using the method presented in NAVFAC (1986) and implemented using the software AllPile, the methodology presented in FHWA (2010) and Salgado (2008). Calculated shaft friction capacities varied somewhat across the 3 deep investigation borehole locations at the site. A maximum ultimate end bearing of 5 MPa was adopted for the Riccarton Gravel formation to account for artesian water pressures, soil relaxation and soil disturbance during pile excavation. This is approximately equivalent to a friction angle of 38° when using traditional bearing capacity equations. A strength reduction factor (ϕ_r) of 0.56 was calculated using the methodology outlined in AS 2159.

Negative skin friction was predicted to occur due to densification of liquefied soils resulting in free field subsidence of the upper layers. Negative skin friction load was calculated assuming cases of 0 kPa and 10 kPa through the liquefied soil layers and full skin friction through non-liquefied layers. Additional settlement due to potential down drag loads was also estimated. Liquefaction triggering analysis showed that the most critical negative skin friction case occurred between the

SLS and ULS shaking levels where liquefaction is not fully developed and deeper layers are able to consolidate without the overlying layers experiencing a build-up in pore pressure and reduction in skin friction.

Three methods were considered for pile capacity analysis. NAVFAC (1968) and Salgado (2008) produced similar values of pile ultimate capacity as they rely on variations of the traditional bearing capacity equations. FHWA (2010) relies on a simplified formula of $0.6 \cdot N_{60}$ (tsf) for the ultimate end bearing capacity and a much smaller value for end bearing and a higher pile settlement. However, the skin friction was higher than the other two methods.

The end bearing values calculated using the three methods suggest the NAVFAC calculations are reasonable but the predicted settlements using this method were smaller than those predicted using the other two methods. Therefore, pile settlements were evaluated using the method outlined in FHWA (2010) with an end bearing limit of 2.88 MPa at a settlement of 5% of the pile diameter. FHWA (2010) was also used to predict the sensitivity of the piles to differential settlement particularly with negative skin friction.

A comparison of the three methods is shown in Table 3 for a 1.2 m diameter pile embedded a minimum of three diameters into the Riccarton Gravels.

Table 3: Calculation Method Comparison

Method	Ultimate End Bearing Capacity (kN)	Comments
FHWA (2010)	3,257	Side resistance calculated by Beta Equations for individual soil layers. End bearing capacity calculated by traditional bearing capacity factors with modification factors by Kulhawy. Found to be higher than the other methods. Nominal unit base resistance = $0.6 \cdot N_{60}$ (tsf) to account for soil stress release during drilling and cleanout. Recognises bearing capacity theory is less reliable in drilled shafts. Considers averaged soil properties for two diameters below the pile tip Simple formula for prediction of settlement $0.6 \cdot N_{60}$ at 5 % of pile diameter Comprehensive and prescriptive load settlement relationship specified in Appendix C. Settlement is reliant of the percentage of the load capacity which is mobilised.
Salgado (2008)	9,200	Effective stress based calculation of side resistance with a modified K factor. Pile displacement and end bearing capacity related to relative density of soils.
NAVFAC (1968)	9,900	Effective stress based calculation of side resistance with a modified K factor. Relies on traditional bearing capacity equations for tip resistance. Considers compression of pile ($s = QL/EA$) in settlement calculation. Settlement calculated considering load transmitted to side and to base separately then adds all three settlements. (In accordance with Vesic (1977)). Method included in AllPile software.

Average Pile Capacities for the 25 m – 26.5 m long piles are reported in Table 4.

Table 4: Calculation Method Comparison

Pile Capacity	Ultimate Geotechnical Pile Capacity (kN)	
	900 mm dia piles	1200 mm dia piles
Ultimate Side Friction	4010	5340
Ultimate End Bearing	1830	3260
Ultimate Bearing Capacity	5840	8600
Pile Design Capacity (k) ($\phi_r = 0.56$)	3270	4820

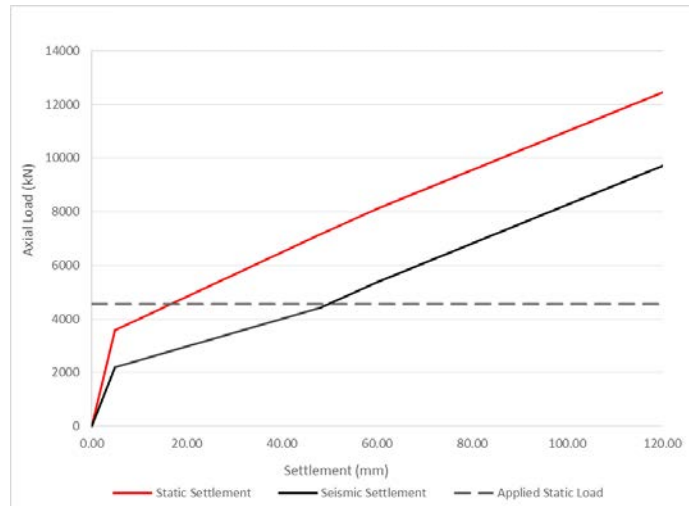


Figure 5: Typical Load Settlement Relationship Developed Using FHWA (2010)

Load settlement relationships were used to determine the expected total settlements for individual piles and differential settlements between neighbouring piles. The analysis carried out shows that for static load cases, the pile shaft friction carries the applied load and settlements are tolerable (< 20 mm). For seismic cases, liquefaction was assessed to occur and the load is transferred to the pile tip and pile settlements are much larger. Where predicted settlements were more than the structural tolerances or likely to increase the ductility demand of the ground beams, pile depths were increased and an updated load settlement relationship considered in the structural design. This was an iterative process and required close co-ordination between the geotechnical and structural engineers.

The final pile depths ranged between 25 m and 26.5 m across the site.

5 PILE CONSTRUCTION

5.1 Methodology

Piles were constructed using an innovative open hole fluid supported technology. From previous experience in similar ground conditions, 12 m long partial temporary casings were adopted to support the highly variable upper layers. The casing was installed using a vibration technique to 10.5 metres depth leaving an upstand of 1.5 m. Excavation was carried out using a rotary bucket. First within the casing, then below the casing with side wall support achieved by using synthetic polymer drilling fluid. The cage was then lowered into the polymer supported hole and concrete pumped using a tremie pipe. Following concreting, the temporary casing was withdrawn and re-driven at the location of the next pile. This methodology was proven to be effective in similar ground conditions for a similar building project in Christchurch.



Photographs from Left – Pile Rotary Auger Bucket, Pile Temporary Casing showing drilling mud with Hold down and hold up bars for casing and tremie pipe about to be lowered into hole, Casing Withdrawal, Completed piles with site concrete for ground beam pour.

Piles were constructed on a two-day cycle. On day 1, the casing was installed and excavated to a few metres above the founding depth. On day 2, the excavation was continued to founding depth, the cage installed and concrete poured. This methodology had several benefits. It allowed the suspended soils to settle out of the drilling fluid overnight and then able

to be cleaned out the next morning. It also allowed for excavation to be undertaken slowly in the Riccarton Gravels where artesian water pressures would destabilise the hole if water was able to flow upwards abruptly.

As the method is relatively new in the New Zealand market, supporting international experience and advice on drilling fluid design and adapting the fluid physical and chemical characteristics to the site conditions as construction progressed was required. Specialist fluid mixing, pumping and storage equipment was also required onsite.

The drilling fluid worked to stabilise the hole by providing a hydraulic head greater than that of groundwater. This head applies a lateral force to the side walls and improved stability. Due to the open graded nature of the gravels, the fluid flowed radially out of the hole meaning new fluid had to be pumped constantly into the hole to maintain the hydraulic head. The fluid was also viscous and had a specific gravity of around 1.1 - 1.2 which further improved the effectiveness.

When necessary, bentonite was added to the synthetic polymer fluid to further improve the stabilising effects. As bentonite works by forming a cake around the sides of the hole when it is placed in large concentrations, the bentonite concentration had to be kept below 4% to prevent a reduction of pile side friction. Bentonite has also been used successfully on other open hole construction projects.

The construction methodology allowed piles to be constructed at a rate of 3 piles per week. This allowed some of the other foundation works such as ground beam construction to begin part way through the piling operation. The contractor was also able to found a tower crane on the early piles which further improved construction efficiency.

5.2 Additional Investigations During Construction

During the design, it was identified that ground conditions at and below the pile tips varied across the site meaning there was a risk of reduced end bearing capacity and increased settlement in some piles. Performance observations of piled foundations in the Canterbury Earthquakes showed that piled foundations with weak or liquefiable layers below the pile tip suffered differential settlement, this was exacerbated when skin friction was lost in overlying liquefiable soils. (Canterbury Earthquake Royal Commission, 2012) and (MBIE, 2018). As there were no deep boreholes in the middle of the site, the soil conditions for a minimum of 10 diameters below the pile tip for piles in the building core needed to be confirmed and an additional three proof holes to 40 m target depth were carried out.

A zone of weaker silty material was encountered below the building core in the proof holes. The load settlement analysis indicated total and differential settlements at this location were unacceptable, particularly as the piles in the core were the most heavily loaded. Various options were considered including installation of additional piles, stiffening of ground beams and altering the structural system of the building to redistribute the loads from the core of the building. It was decided that lengthening of the piles was the most cost-effective solution. Pile lengths were increased until the predicted differential settlement was acceptable for the performance of the structure. It was decided that four of the piles underneath the core of the building would be extended by 3 m to 29 m below ground level to address this.

5.3 Quality Assurance and Construction Observations

To achieve adequate confidence of construction quality, reasonably extensive on-site quality assurance and pro-active risk management was required. Pile side wall instability was a major risk to pile construction. A side wall collapse at any time can result in a build-up of loose materials on the base of the pile hole, in addition to this, sands and silts suspended in the drilling mud may settle out and accumulate on the bottom of the hole. It was determined by basic volumetric calculations that the sand content of the drilling fluid must not exceed 5% at any time. Fluid samples were collected from every pile prior to pouring of the concrete to assess the sand content at three depths using a one-way ball valve. Where the sand content exceeded 5%, fresh fluid was added to the hole to encourage the sand to settle out and the base of the hole was cleaned.

The hardness of the pile hole base was verified by sounding the hole using a weighted tape. Where the response from the pile toe was not satisfactory, pile clean out was required. Pile arisings from the last bucket were also checked to confirm the founding materials are as expected.

5.4 Pile Non Destructive Integrity testing

The pile testing schedule was developed in accordance with the risk based procedure set out in AS 2159. The individual risk rating for the site was calculated to be 2.48 indicating an overall low risk level and required a minimum of 5% to 15% of the piles to be tested.

Due to the open hole construction method, a higher level of quality assurance was adopted and pile integrity testing in the form of 3 cross hole sonic logs and 7 pulse echo tests were undertaken. Cross hole sonic logging verified the strength of the concrete and confirmed there were no inclusions while pulse echo testing was used to verify pile length and pile toe responses. The interpretation of the data collected during the testing is dependent on the soil stratigraphy and construction methodology. Test results were verified using construction data such as onsite measurements, pile excavation logs and

concrete placement records. Where test results did not match the construction records, discussions were held with the contractor and the geophysical testing consultant to resolve the issues and confirm the piles had been constructed as per the specification.

6 CONCLUSIONS

An investigation and options assessment led to the selection of piled foundations over ground improvement for preliminary design of a five storey office building in Christchurch. Bored 900 mm and 1200 mm piles founded at 25 m to 29 m deep have been designed and constructed to support the building.

The building is founded on challenging ground conditions including shallow ground water, liquefiable ground, variable ground and artesian water pressures. The piles were designed with learnings from the Canterbury Earthquakes, pile capacity, negative skin friction and total and differential settlement.

The construction methodology used involved the use of temporary casing to partial depth, and open hole pile fluid technology to support the hole below the casing. This required careful construction to ensure pile side wall collapse did not occur, the base of the pile was clean and hard and built as per the specification.

A higher level of pile integrity testing than specified in AS 2159 was also required due to the construction methodology. Cross hole sonic logging and pulse echo tests were used to verify pile integrity. Specific quality assurance checks required to achieve these criteria and a Construction Review PS4 sign off include the checks above as well as monitoring of the sand content of the drilling fluid and monitoring the bentonite content of the fluid.

Integrated structural and geotechnical design by a single consultant team allowed the design to be robust and enabled any design and construction changes to be readily accommodated, and was a critical part of the success of the project.

7 REFERENCES

- Brown, A. G, Uno, M., Thompson, J.J and Stratford, J. W (2016). Seismic and Financial Performance of Fluid Viscous Dampers Relative to BRBs: A Case Study. Proceedings of the 2016 New Zealand Society for Earthquake Engineering Conference. Christchurch, New Zealand.
- Brown and Weeber (1992). Geology of the Christchurch Urban Area. Scale 1:250,000. Institute of Geological and Nuclear Sciences. Geological Map 1. 1 sheet + 104 p. Institute of Geological Sciences Limited, Lower Hutt, New Zealand.
- Canterbury Earthquake Royal Commission (2012). Volume 1: Summary and Recommendations in Volumes 1-3, Seismicity, Soils and the Seismic Design of Buildings
- Federal Highway Administration (2010). Drilled Shafts: Construction Procedures and LRFD Design Methods NHI Course no. 132014 Geotechnical Engineering Circular No. 10
- Ministry of Business and Employment (MBIE) (2018). Part C: Assessing, repairing and rebuilding foundations in TC3
- Salgado (2008) Analysis of Single Piles: Challenges and Solutions. Proceedings of the 12th International Conference of International Association for Computer Methods and Advances in Geomechanics
- Standards New Zealand (2004). NZS 1170.5:2004. Structural Design Actions Part 5, Earthquake Actions. Standards New Zealand.
- Standards Australia (2009). AS2159 Piling – Design and Installation.