Integrating Foundation Design for a Low-Damage Superstructure

N. Barounis, A. Brown & G. Saul

Opus International Consultants, Christchurch

2015 NZSEE Conference

ABSTRACT: A five storey office building that incorporates supplemental fluid viscous damping as part of the seismic system is currently under construction at 12 Moorhouse Ave, Christchurch. The building has been conceived utilising a low-damage design philosophy, which required a foundation system that utilised damage resistant principles. The site presents difficult ground conditions, with weak upper soils, extremely variable intermediate gravel layers, and liquefiable lenses throughout the soil column.

This paper presents and discusses the factors considered for pile design, the financial benefits of PDA testing as well as the strategy developed for quality assurance.

The issues encountered during construction will be presented, including dealing with artesian groundwater pressures developed within the Riccarton Gravels at founding level. The methodology and results of PDA testing are explained (hammer mass, drop height, testing sequence) along with the CAPWAP analysis of the bearing capacity of the Riccarton Gravels for this type of pile.

1 INTRODUCTION

1.1 Background

A five storey office building is currently under construction at 12 Moorhouse Avenue in Christchurch that incorporates supplemental fluid viscous damping as part of the seismic resisting system. The design has been based on performance based design principles and utilises a low-damage design philosophy throughout to achieve the following seismic performance criteria. Damage to non-structural elements is limited for the Serviceability Limit State. Damage to non-structural elements is acceptable for the Ultimate Limit State. Substantial damage to non-structural elements is expected in the Maximum Credible Earthquake with repairable damage to structural elements and collapse prevention ensured.

To meet the seismic performance criteria above, the structure required a foundation system that met the same damage resistant principles as the superstructure, although the site presented difficult ground conditions. This site has weak upper soils, extremely variable intermediate gravel layers, liquefiable deposits extending to 22m depth, average annual groundwater level at 1.2m below ground level and artesian conditions in the underlying Riccarton Gravels.

Deep piles were proposed for the foundation system with the Riccarton Gravel formation being the target for end bearing support.

These site ground conditions created three significant challenges on this project:

- Cost implications due to the target depth for the piles being at least 25 metres;
- Construction difficulties for the first 22m of the pile excavation due to the collapsible nature of the formations present; and,
- Adequate performance of the constructed piles in terms of bearing capacity and serviceability of approximately 5000kN and 500kN ultimate compression and tension capacity respectively.

The solution was the use of bored piles down to the Riccarton Gravel formation.

1.2 Ground Conditions

The 12 Moorhouse avenue site is located approximately 2km south west of the centre of Christchurch and occupies an area of approximately 4,500 square metres (see Figure 1).

Figure 1. Location of the site.

The initial site investigations comprised five CPTUs extended to refusal at 13m depth and two boreholes to 27m and 31m depth respectively. During the initial site investigation stage, a building that was to be demolished covered the majority of the footprint area of the proposed building, consequently, the investigations were located around the perimeter of the proposed building.

The ground conditions encountered at the site and the adopted geotechnical parameters for pile design are summarised in Table 1. The liquefaction analysis undertaken indicated extensive earthquake induced subsidence between 130 and 200mm following a ULS earthquake with M7.5 and PGA 0.46g. The liquefaction is mainly situated between 14m and 22m depth.

Table 1. Average ground conditions at the site and pile design parameters

Soil description	Top of formation (m)	Bottom of formation (m)	γ (kN/m^3)	C´ (Kpa)	Φ´
Silty Sand, loose, interbedded with soft Silty Clays	0.00	11.5-13.5	18	0	26°
Gravel, medium dense, fine to coarse	11.5-13.5	15.0-17.0	18	0	32°
Clayey Silt, very soft, with sand lens up to 2m thick	15.0-17.0	20.0	17.5	0	22°
Sand, medium dense, fine	20.0	21.5	18.5	0	29°
Riccarton Gravel	21.5	-	19.0	0	38°
Groundwater depth	1.0m bgl				

1.3 **Load demand and pile capacity at the design stage**

The new IL2 building is to be five storey viscous damped steel moment resisting frame structure on a network of reinforced concrete ground beams on piles. The load demand on the piles requires both tension and compression capacity from the pile. The structural analysis undertaken indicated an ultimate demand of 2,500kN in compression and 300kN in tension.

The foundation options considered for support of the building included ground improvement of the upper liquefiable layers and piling to depth. Ground improvement was dismissed as a feasible option at an early stage due to cost. The piling options assessed at the design stage included bored piles, screw piles and driven piles. The 900 mm diameter bored pile option was adopted for the following reasons:

- It is considered the most robust solution that provided sufficient bearing capacity and serviceability performance with the piles founded in the Riccarton Gravel.
- The ground borne vibrations to the adjacent building at 20 Moorhouse Avenue were minimised compared to driven pile or screw pile options.
- It provides an acceptable cost solution, even though the ground conditions are unfavourable in terms of constructability.

The construction difficulties for bored piles generally increase as the excavation depth increases, especially when they are bored through liquefiable soils. For tackling these difficulties a steel partial casing and open hole excavation with innovative polymer support fluids were proposed to be used for the pile construction. The construction sequence is described in Section 2.

The static capacity for the 900 mm diameter and 25m long bored pile was analysed with Allpile software, using the procedures described in the Foundations and Earth Structures Design Manual 7.02 (NAVFAC). The following were also considered:

- A reduced angle of friction of 38° was adopted for the Riccarton Gravel after considering the combined effects of excavation disturbance and accompanied relaxation and artesian water pressures. The φ´ values can go up to 45° for gravel, however Poulos and Davis (1980), recommend the φ´ value after pile installation to be reduced by up to 3°. Due to the artesian pressure, an upper value of 41° is recommended which is then reduced to 38° as per Poulos and Davis recommendations.
  - A critical depth equal to twenty diameters was adopted with the effective stress contribution limited to 18m.
  - Groundwater level taken at ground level (slightly conservative).

The results from the Allpile analysis are presented in Table 2 for the 900mm pile diameter and the constructed 1.03m diameter of the pile.

Table 2. Pile capacity results from Allpile

Pile diameter	Ultimate side friction	Pile settlement at ultimate side friction	Ultimate end bearing (kN)	Pile settlement at ultimate end bearing	Ultimate bearing capacity (kN)
900mm	1763kN	15mm	4221	107mm	5662 at 107mm
1030mm	1788kN	16mm	4410	119mm	5894 at 119mm

The load-settlement graph obtained from Allpile is shown in Figure 2. This result is considered a lower bound estimation for the pile capacity. A 900mm diameter pile requires sufficient PDA testing to enable a strength reduction factor Φr of 0.55 to be adopted as per AS 2159.

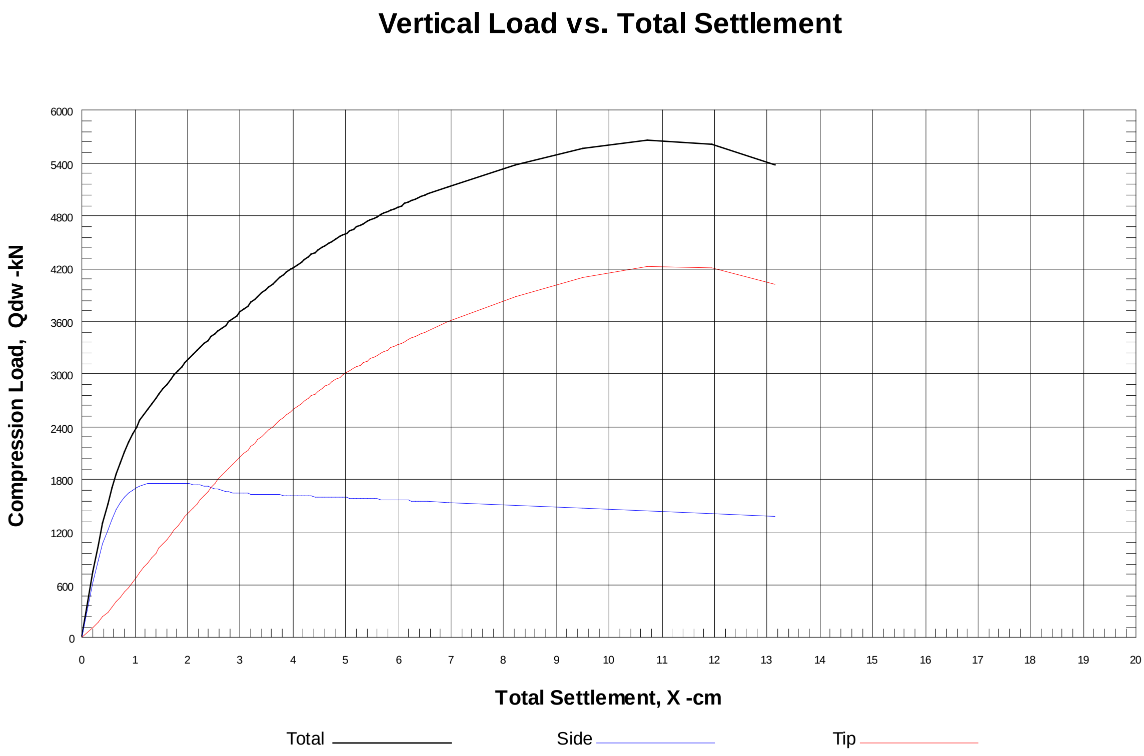

Figure 2. Load-settlement graph from Allpile for D=900mm and L=25m.

2 PILE CONSTRUCTION

2.1 Further site investigations program

Opus geotechnical engineers were involved at an early stage for this project. The multiple role of the geotechnical engineer included the following tasks:

- Evaluate the proposed piling methodology
- Identify and compare the excavated material with the material identified at the initial site investigation stage and offer advice if substantial differences were observed
- Monitor the pile construction and advise the final depth of the piles
- Liaise with the contractor and detect any construction issues that may occur from unforeseen ground conditions or any other issue
- Coordinate quality assurance program for the piles including integrity and PDA testing

A secondary site investigation was conducted prior the commencement of pile construction. This provided confirmation of the typical pile founding directly based on the building footprint. The site investigation comprised of three extra boreholes undertaken by the contractor to 32m depth including SPT testing between 18m and 32m depths.

The interpretation of the SPT tests indicated that the appropriate founding depth for the piles was 27m below ground level. Thus the initial pile depth of 25m was further increased by 2 metres. The Riccarton Gravel proved to be consistent in terms of relative density for a depth of at least five pile diameters below this depth.

2.2 **Excavation and hole support**

Construction involved installing 39, 900mm in diameter and 27m long piles. The upper 2m were likely to encounter poorly graded gravels and sands that would present stability issues for an open hole. Additionally the length between ground surface to 14m depth was expected to have a lateral variation between pile locations that could possibly present stability issues for an open hole. For these reasons, the temporary steel casing was used for the upper sections of the pile. Between 6.5m and 15m depth the gravely soils transition into plastic soils and this was considered the best point to end the casing. The length of 12m provided an adequate solution at an acceptable cost.

Figure 3. Excavation of typical pile.

The piles were constructed by first installing the 12m long open hole casing by means of vibration applied at the top of the casing. Excavation commenced from within the casing installed to its full depth with an open hole and then with the support of polymers. The excavation continued for the length between 12m and the final pile depth being uncased but under the support of the piling fluid. Excavation was by rotary bucket excavator with a self-closing gate (refer Fig. 3).

A solids free piling fluid was recommended by the contractor to promote settling of the cuttings in surface tanks without large volumes required for extended retention times, as the site footprint was restrictive. The fluid consisted of chemicals that provided viscosity and control of any formation clays, filtration control and viscosity for the lower sand sections, pH adjustment, and hardness removal in the event of lost circulation.

At the depth of between 15m and 17m, the excavation rate was slowed as the clay layer at that depth acts as an impermeable barrier/cap to the top of the Riccarton Gravel. It is usually this layer, which if excavated or drilled at a high rate, that may cause the artesian groundwater to abruptly flow upwards resulting in hole instability and cavings, making concreting difficult or even impossible. Sufficient time was allowed for the soil to stabilise and mix with the piling fluid which was enhanced with bentonite added directly to the polymer at the base of the hole. A steady rate of excavation was achieved until the top of the Riccarton Gravels were reached. The excavation for each pile continued successfully to the depth of 27m with an average of three piles constructed every week.

This is a relatively new technique in Christchurch and this was one of the first sites it has been used by the contractor.

2.3 **Integrity testing**

The need for integrity testing of the piles was identified during the design stage. The Australian Standard (AS) 2159 was implemented and both the pulse echo method and sonic logging method were adopted. The Individual Risk Rating (IRR) as per AS 2159 was assessed and the Average Risk Rating (ARR) was calculated as 2.45, classifying the overall risk as low. The pile testing requirements for serviceability a minimum of 1% of piles to be tested, equivalent to 1 pile. However, a greater level of Quality Assurance (QA) was deemed as necessary due to the innovative excavation method. Therefore, instead of one pile, seven piles were tested with pulse echo method and one pile with cross-hole sonic logging (CSL) method.

The echo pulse method or low strain impact integrity method is the most widely used non-destructive test described in the American Society for Testing and Materials (ASTM) standard test method D5882-07 (2013). It involves the measurement and analysis of the velocity response of a pile induced by a hand held hammer applied on the pile head surface. An accelerometer is attached to the shaft. The echo pulse test results did not identify any deficiencies.

The CSL method was applied on one pile which was also pile driving analyser (PDA) tested. The test was conducted according ASTM standard test method D6760-08. Four 50mm diameter steel access tubes were installed along the length of the reinforcement cage, forming a square. The CSL test involves passing an ultrasonic pulse through the concrete. Source and receiver probes in water filled access tubes emit and receive the signals as the probe cables are pulled upwards towards the surface over a depth measurement wheel. The evaluation of the integrity of the concrete between and probes involves measurement of wave travel time between source and receiver along six different travel paths. Longer travel times can be associated with irregularities in the concrete mass, while a complete loss of signal is an indication of a significant defect such as a void in the concrete mass.

For checking the integrity post PDA testing, the CSL test was also repeated a few days after the initial PDA test. This was deemed as necessary in order to assess if any cracks formed in the pile from using the drop hammer during the PDA test. Both pre and post PDA tests did not identify any evidence of changes in the concrete quality or defects along the pile shaft.

The integrity tests also did not identify any evidence of deficiencies in the seven piles tested.

2.4 **PDA testing**

Dynamic pile analysis testing (PDA) was performed in accordance with high strain dynamic pile testing method ASTM D4925-12 and Appendix B, AS 2159. All testing equipment has been calibrated by Pile Dynamics USA.

Pile top force and velocity were measured with four strain transducers attached 1.53m below the pile top, which were connected to the pile driver analyser model PAX 8. Pile properties such as combined section modulus, pile length, length below sensors and pile embedment were entered into the PDA unit. The piling drop hammer was then positioned on top of the pile and provided 12 hammer blows with a drop height varying between 0.5m and 1.5m. The mass of the hammer used was 4,800kg, approximately 1% of the target ultimate capacity of 5,600kN.

In order to mobilise the full capacity of the pile, a permanent set of 3mm was required. The observed set ranged between 0.1mm and 3mm from blow to blow. The analyser measured pile top velocities, displacement and forces and computed various parameters such as stresses within the pile max compression stress (CSX), max tension stress (TSX), energy transfer ratio (ETR) and maximum energy applied on the pile (EMX).

The best blow in terms of data quality was analysed with the Case Pile Wave Analysis Program (CAPWAP). The pile analysed was 26.8 metres long. The pile had a steel casing of 15.4mm thickness and 12 metres long and it was heavily reinforced with 32mm reinforcement and 50MPa quality concrete.

The results from PDA are summarised in Table 3.

Table 3. Key parameters from PDA test (L=26.8m)

Length below gages	25.3m	Peak average stress at pile top CSX	7.9MPa
Maximum force FMX	5037kN	Max local stress at pile top CSI	10.2MPa
Maximum displacement DMX	3mm	Degree of bending	1.29
Maximum energy transfer EMX	11.2kN m	Max case tension stress TSX	2.9MPa
Hammer efficiency ETR	15.9%	Compression stress at the bottom of the pile	5.9 MPa
Max pile top velocity VMX	1.04m/s	Observed set and final displacement DFN	0.1 to 3mm

3 CAPWAP ANALYSIS

Use of the PDA for capacity evaluation requires the captured data to be analysed with Case Pile Wave Analysis Program (CAPWAP) software. The CAPWAP analysis included allowance for a 0.1m increase in diameter between 11.4m and 26.8m depth indicated from concrete volumes. Thus, the constructed pile had a 30% increased area.

The PDA test was conducted on an actual working pile of the building, thus concerns about the integrity of the pile were raised. The pile was heavily reinforced on the top and was permanently steel cased to 12m for protecting the pile from cracking by using the drop hammer. The number of blows was limited to avoid overstressing the pile either in tension or compression. Tensile stresses can crack a reinforced concrete pile. A 25mm thick plywood sheet was placed on the top of the pile to reduce the contact stresses induced by the hammer.

The analysed blow had an observed set of 0.1mm, which means that the pile was not fully mobilised by the hammer. This is consistent with the measured low level of hammer efficiency of 15.9% and the CAPWAP result of distribution between shaft and end bearing.

The CAPWAP results are shown in Figure 3 and Table 4. The CAPWAP load distribution analysis indicates that the pile shaft resistance contributed 4594kN (76%) to total activated pile resistance, and the toe resistance contributed 1415kN. The result from the PDA is considered as a lower bound estimate and it indicates only the resistance activated. It is noted that the dynamic testing estimates the pile capacity at the time of testing.

As the pile was an actual working pile there was reluctance to use a larger pile hammer to fully mobilise the pile due to the risk of cracking.

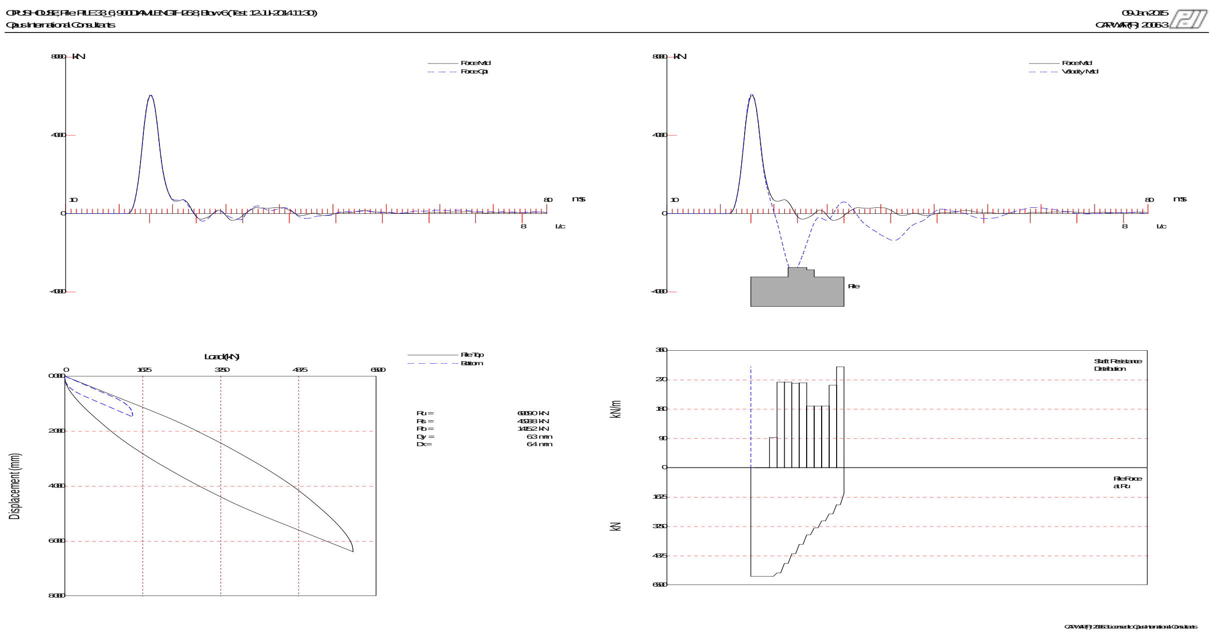

Figure 3. CAPWAP results for the 26.8m pile.

Table 4. CAPWAP analysis results

Blow number	6
Compression shaft resistance R_s	4594kN
Toe resistance R_b	1415kN
Compression capacity	6009kN
Deflection at working load of 2,500kN	1.8mm

The PDA also monitored the applied stresses and provided additional assurance that they were within acceptable limits. This was consistent with the CSL testing subsequently undertaken.

The pile integrity was also assessed by the PDA by means of the integrity parameter Beta, which also indicated no damage along the pile shaft.

The PDA test undertaken is equivalent to an end of drive situation as there was no restrike test undertaken. The reason for not undertaking a restrike test was the probability of induced cracking from the hammer blows. The time between concreting the pile and the PDA test was fourteen days and is considered sufficient to allow pore pressures to dissipate, pile set up to occur and concrete to sufficiently cure.

4 CONCLUSIONS

The capacity and integrity testing for the construction of bored piles is a process that should be always adopted and followed when liquefiable ground conditions prevail at a site. The option of such testing should be always assessed irrespective of the scale of the project as certain cost benefits may arise from the use of AS 2159. An assessment of IRR and ARR and use of PDA with integrity testing as per AS 2159 may prove to save substantial costs from the increase in the strength reduction factor Φg.

The results from such tests can help both structural and geotechnical engineers to verify initial pile design and check if the basic principles behind the pile design are met.

The dynamic testing of foundations by the PDA method is a great tool for assessing the pile capacity. Low strain and CSL methods are integrity methods that give a reliable answer at an acceptable cost.

The quality assurance program for pile integrity and capacity verification links together the geotechnical engineer/designer with the structural engineer/designer and the pile contractor in a constructive way.

Such quality assurance programs help to establish good communication and cooperation between geotechnical and structural engineers for the benefit of the project in terms of delivery time, unforeseen costs and contractor efficiency. It also helps the structural engineer to understand the importance of geotechnical input into the pile design and the geotechnical engineer to design the pile tailored to the needs of the structure. The result of this cooperation for the given project was the successful delivery of good quality piles on time and within the specified budget.

5 REFERENCES

Opus International Consultants, 2013. *12 Moorhouse Avenue Geotechnical Assessment Report.*

Poulos, H.G. & Davis, E.H. 1980. *Pile foundation analysis and design*. John Wiley and Sons, New York.