

Adapting a Driven Pile Design to meet the Challenges encountered during Construction

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ABSTRACT

A recent multi-storey commercial development undertaken in Christchurch, New Zealand following the 2010/11 Canterbury Earthquake Sequence presented a range of geotechnical foundation constraints. These included loose, liquefiable silts and sands layered with dense, non-liquefiable gravels; predicted large post-seismic settlements; and significant demand loads from the multi-storey superstructure; all of which had to be addressed via an economic capital cost design solution. A relatively “straightforward” driven concrete pile foundation design was designed for the development, which required adaption during construction due to latent conditions and difficulties encountered. When construction commenced, several piles refused above their target depth. Where this was due to factors such as localised variations in the soil profile, adverse pile performance during driving, or the striking of buried obstructions, automatic acceptance of these piles could not be guaranteed. Careful investigation as to the cause of these issues, and reanalysis of the piled foundation design was necessary to resolve the problems, meet the design requirements and account for the constraints of construction on the site. Various tools were used throughout this process, including high-strain dynamic PDA testing, CPT investigations and engineering judgement. The solutions ranged from acceptance of the refused piles ‘as-is’, to predrilling through surficial fill for the installation of remedial piles. Ongoing critique and assessment of the PDA results, including variation of the testing schedule as construction progressed resulted in value engineering opportunities, including the reduction of the overall number of piles compared to the original design. This ultimately offset the pile remediation costs such that both a successfully constructed foundation and a net cost saving on the original consented design were achieved.

Keywords: Driven concrete piles, PDA testing, construction challenges

1 INTRODUCTION

The 2010/11 Canterbury Earthquake Sequence saw extensive damage to many structures in the Christchurch Central Business District (CBD). This prompted their subsequent demolition and clearance, opening the way for new developments, albeit with a much greater awareness of the performance of buildings in seismic events. This project was a product of that redevelopment opportunity and involved the construction of a retail precinct comprising a new eight storey car park building (Building A), the extension of an existing two to five storey retail and commercial building (Building B) and two new four and three storey retail and commercial structures (Buildings C and D respectively). Due to the size of the project, the client had a strong focus on economic efficiency, in addition to meeting a rigid timeframe for the completion of the works due to their contractual obligations to the site’s future tenants.

1.1 Geological Environment

Geotechnical investigations at the site comprised six boreholes to 25 metres below ground level (mbgl) depth or greater. These identified a fill layer at the surface arising from demolition of the previous structures that occupied the site, overlying layers of silt, sand and gravel of the Springston formation. The Riccarton gravels that persist across the city beneath the Springston formation were identified at a uniform depth of 23 mbgl. This soil profile is summarised in Table 1.

Table 1. Soil profile beneath the site determined from machine boreholes

Layer	Top of Layer (mbgl)	Thickness of Layer (m)	Description	Formation
1	0.0	2.2 – 3.2	Medium dense to very dense Sandy GRAVEL	Fill
2	2.8	0.0 – 1.1	Soft to firm Clayey SILT	Springston
3	3.3	1.0 – 1.3	Loose to medium dense Silty SAND	
4	4.4	3.3 – 3.7	Medium dense to very dense Sandy GRAVEL	
5	7.9	5.6 – 6.4	Loose to medium dense Silty SAND	
6	13.8	4.0 – 4.4	Medium dense to dense Silty SAND	
7	18.0	1.0	Soft SILT	
8	19.0	2.0 – 2.8	Loose to dense Silty SAND	
9	21.1	2.0	Soft to firm SILT	
10	23.1	–	Medium dense to very dense Sandy GRAVEL	Riccarton

Piezometers installed in several of the boreholes identified the groundwater table depth to vary from 3.3 to 5.0 mbgl.

Cone Penetration Tests (CPTs) were also completed across the site. These were used to investigate the liquefaction potential of the soil profile and the results were correlated against the soil layers described in Table 1. Below the non-liquefiable crust, layers 2, 3, and 5 were identified as being susceptible to liquefaction.

1.2 Foundation Design

The foundation design selected for the site consisted of 300 mm square driven precast concrete piles. The piles were arranged in square groups ranging from single piles to 16 piles, in a 4 x 4 arrangement. The piles resisted the demand loads of the superstructure and were connected on top by a concrete pile cap, with each cap linked by concrete foundation beams. The ground floor slab was structurally disconnected from the rest of the superstructure and founded on a reinforced gravel raft installed beneath it, laterally confined by the foundation beams.

To improve the constructability of the foundations, a consistent design was adopted across the site, with only the lesser demand loads on the piles of Building B enabling a shorter pile length. 15.0 – 15.5 mbgl was selected as the target depth range for Buildings A, C and D, resulting in pile cast lengths of up to 17 m, with the piles bedding within layer 6. Building B utilised a target depth range of 5.8 to 6.3 mbgl, with the piles bedding within layer 4. These target depth ranges were selected to achieve a minimum penetration of the pile within the founding layer of at least three pile diameters whilst also maximising the extent of stress bulb development within the same competent bedding material. Hence, there was minimal opportunity to reduce the pile penetration depth without promoting the risk of the piles bearing on liquefiable soils.

Each of the piles was designed to behave as an individual pile, rather than in a block mechanism, when subjected to each of the ULS loading conditions considered. These conditions included static loading in compression in both a pre and post-liquefaction environment; and seismic loading in compression and tension. The pile capacity was assumed to result from combinations of toe and shaft resistance in each case. Lateral design of the piles was completed by the Structural Engineer and so will not be commented on further.

2 PROBLEMS ENCOUNTERED DURING CONSTRUCTION

The multi-layered, liquefiable nature of the soil profile posed problems during construction of the piled foundations. The increase in density of the soil profile, both in the upper fill layer (layer 1) and the upper gravel layer (layer 4), when compared to the layers around them, meant that the piles had the potential to refuse prior to reaching their target depth range.

Pile refusal was the result of attrition on the pile during the driving process. Every time the pile head is struck, energy is transferred through the pile in the form of mechanical waves. The driving process fails the soil beneath the pile toe and along its shaft, and the pile displaces itself downwards. In instances where the strength of the soil exceeds that of the pile itself, the energy is absorbed by the pile instead, incrementally damaging it. Over the driving history of the pile, where the latter scenario

was prevalent, the pile damage accumulated to the level that the pile failed structurally, resulting in refusal of the pile to continue to drive when subjected to further hammer blows. Examples of the damage observed in the piles are included in Figure 1.



Figure 1. Pile damage to a pile that was (a) driven to the target depth range; and (b) refused above the target depth range

Whilst driving a pile to refusal would normally imply that the pile has sufficient capacity, this capacity could only be relied upon under pre-liquefaction static conditions when the pile is subjected to compressive loading. The performance of the pile under the other loading cases could not be guaranteed, particularly when considering the effects of liquefaction, both along the installed length of the pile and of the soil immediately beneath the refused pile's toe. Liquefaction presents the risk of both nullifying the shaft capacity of layers of soil along the pile's length and of a punching failure mechanism beneath the toe of the pile. Likewise, acceptance that the pile's integrity had not been compromised during its refusal was also required.

The majority of piles on the site successfully reached the target depth; refusal was noted for only 51 of the 746 piles (6.8%) driven on the site. Those encountered in Building A are shown in Figure 2. Of those piles which refused prematurely, there did not appear to be a trend with respect to depth or location. There were instances of pile failure early in the driving process, in and immediately below the fill layer (layer 1), and at depth (in layer 4, 5 and just above the target depth range in layer 6). Sometimes it was the later installed piles in a given group that refused, which was likely due to densification of the surrounding soil by the installation of nearby piles, however, at other times, there was refusal of the first pile installed, with subsequent piles in the group driving without incident. There were instances of two piles diagonally opposite from each other refusing, while the adjacent piles reached the target depth range. Likewise, there were also instances of the majority of piles in whole groups adjacent to each other all refusing at similar depths.

3 SOLUTION METHODOLOGY

3.1 General Process

When assessing each pile which refused prematurely, it was necessary to understand the causes of pile refusal and its potential impact to the structure, through a process of data collection, review and consideration of alternative remediation options.

Central to every discussion was the significance of each refused pile. In each case it was necessary to consider:

- Did sufficient structural redundancy exist to compensate for a pile which may be under capacity?
- If not, what load cases were affected, and what was the magnitude of the potential deficiency in capacity?

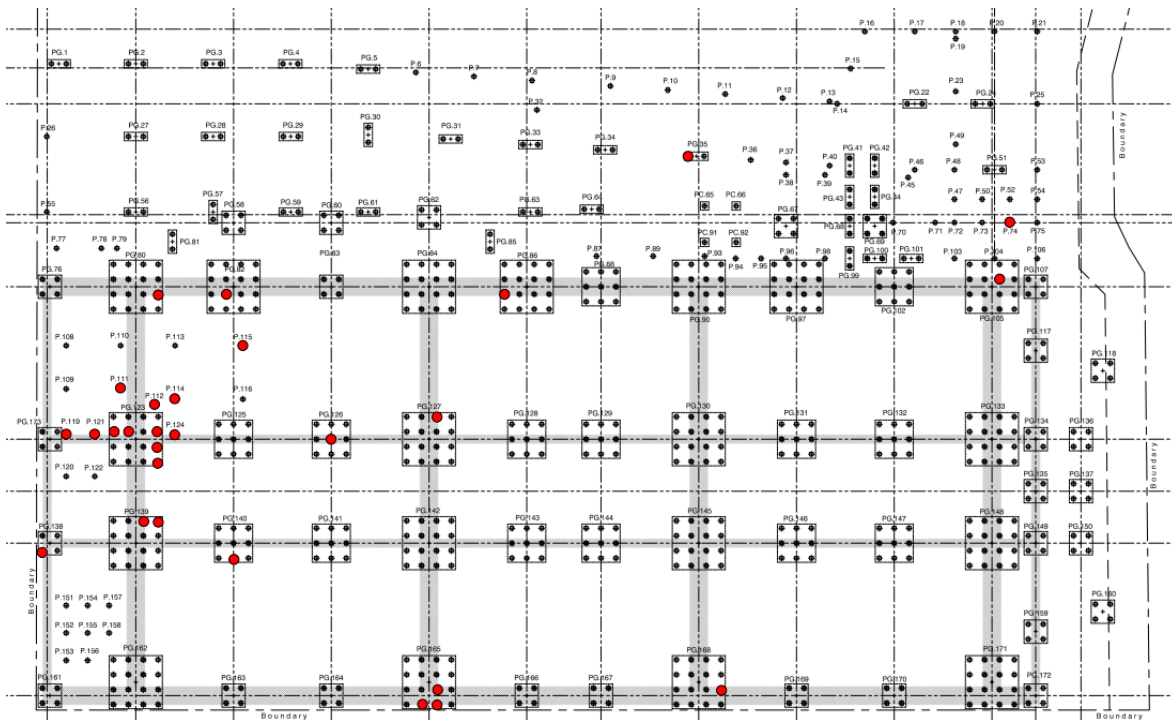


Figure 2. Locations of refused piles on Building A, shown in red; pile plan by Wilson and Hill Architects Ltd. (2015)

The answers to these questions informed the process thereafter. In some instances, the pile could be accepted 'as-is', on the basis that there was sufficient redundancy amongst nearby piles to compensate for the lower capacity of an isolated pile. Alternatively, where the refused pile was designed to carry large loads with minimal redundancy provided by surrounding piles, an evaluation of its expected performance became critical.

Communication between the client, the contractor, the geotechnical engineer and the structural engineer was also of high importance throughout the process.

3.2 Pile Driving Analyser (PDA) Testing

The main method for analysis of the expected performance of piles on the site was high-dynamic PDA testing. This was completed on 5.9 % of the number of piles in each building's foundation, with 55 tests completed in total on 44 different piles. The testing involved the attachment of two accelerometers and two strain-gauges to the top of the pile during driving. The records collected by these instruments were correlated to measurements of the pile set (depth of penetration of the pile per hammer blow); rebound (temporary compression of the pile when struck); and the driving hammer characteristics (type, weight and drop height) through a Case Pile Wave Analysis Program (CAPWAP) analysis. The results of this analysis predict the ULS capacity of the pile, the distribution of this capacity between toe resistance and shaft resistance, and the integrity of the installed pile.

The initial purpose of the PDA testing was to validate the theoretical pile capacity determined during the design phase. This enabled acceptance of the piles that successfully reached the target depth range, but didn't allow for decisions regarding remediation of the refused piles to be made. In order to address these, an explanation for the refusal of each pile had to be found, by way of examining local variability in ground conditions.

A review of PDA results during construction identified a general trend of higher pile capacities in the areas where pile refusals were encountered. Based on the assumption that two PDA tests of comparable piles in comparable soil profiles should produce comparable results, localised variations in the soil profile were inferred to be the main reason for variations in the measured PDA capacities. As these variations were also a likely cause for the early refusal of piles, it was justifiable on both a geotechnical and economic basis to consider individual PDA results to be indicative of the soil profile and the capacity of nearby piles. Thus, adjacent PDA tests were used to inform the pile remediation

process of refused piles, providing estimates of the expected capacities of the other piles in the group and therefore establishing whether there was sufficient additional capacity in additional piles to compensate for the pile which had refused prematurely.

A secondary benefit of this methodology was found through the extrapolation of it to all pile groups on the site. In many instances it was found that the pile capacities measured using PDA testing were sufficiently in excess of the demand loads on the pile group to allow for downsizing of the original group. An overall saving of 137 piles from the originally consented design was thereby achieved. This provided cost savings and reduced the foundation construction time, savings that offset the cost of the data collection and remediation of the piles for which it was still required, and the construction delays resulting from that remediation respectively.

3.3 Cone Penetrometer Test (CPT) Investigations

The secondary method for analysis of the expected performance of piles on the site was CPT investigations. The liquefaction susceptibility of the site was the primary driver for selection of the target depth range for the piles. This susceptibility was based on an average profile for the site, but as with the pile capacities, localised variations to this were possible. Furthermore, densification of the soil profile through the driving process was expected, but the benefits of this could not be quantified without supplementary CPT testing after the pile driving was completed.

CPT testing in the vicinity of the refused piles was used as a means of justifying ‘exceptions to the rule’ provided by the soil profile determined during the geotechnical investigations stage of the project. CPT testing was limited to areas where it was necessary to prove that ground conditions were more favourable than the site-wide ground model. Furthermore, due to their relative cost, they were used sparingly and only when other considerations, such as thicker than average gravel layers observed in the closest machine borehole, indicated a high likelihood of the method being successful.

3.4 Engineering Judgement

Applying engineering judgement and previous experience in similar conditions was a crucial element to the remediation process of every pile refusal. There were instances where it aided the decisions made using the PDA results, or the application of CPT investigations. In the former it was used to optimise the locations for the testing, to accept the test results and to determine the calculation methodology by which the raw results were transformed into estimated capacities for each of the load cases considered. In the latter it was used to interpret the results and to set limits on the zone of piles to which the results should be extrapolated to. However, there were also instances where it alone provided a means to justify acceptance of a refused pile or a recommendation for a remediation solution.

In the instance of a cluster of refused piles in Building A (seen on the left side of Figure 2), pile refusal had occurred for all piles at very similar depths and all in close proximity of the target depth range. A localised reduction of the minimum target depth by 150 mm was justified on the basis of this clustering, the nearby PDA test results and the expected soil densification in this area from the driving of many piles in close proximity to each other. This reduction enabled acceptance of 11 of the 12 refused piles in that area, avoiding the pile remediation costs and time delays that would otherwise have been required.

Likewise in Building C there were several instances of piles refusing within the demolition fill of layer 1. These piles exhibited very extreme degrees of damage, including snapping of the pile at its middle in one case and a 700 mm shortening of the pile from crushing in another. Judgement in these cases suggested that buried obstructions were the most reasonable explanation for the refusal, rather than the accumulation of variability in the soil profile. As such, excavations through the fill layer in these locations revealed a buried foundation beam and a buried concrete over-pour from construction of the previous building on the site, and a denser than expected layer of fill that a 30T excavator bucket couldn't penetrate.

3.5 Remediation Solutions

For 42 of the 51 piles which refused prematurely, close scrutiny of PDA testing and engineering judgement enabled acceptance of the refused pile 'as-is'. A further three piles refused at a shallow enough depth, including the two in Building C that struck buried obstructions, to enable them to be extracted, with subsequent driving of a replacement pile being completed without incident.

For the remaining piles, a decision was made regarding whether to undertake further CPT testing to gather additional information, or to proceed directly to the installation of remedial piles. This decision was a question of economics, scheduling and risk; weighing the potential cost-benefit of gathering further information to inform a remedial decision against the risk of the additional testing not returning favourable data. The risk of further refusal of remedial piles also had to be taken into account.

Different stages of the project called for different risk appetites. Early on in the project, the potential for decisions to have a large impact on cost or program was higher, and therefore the risk tolerance was also higher, and installing remedial piles was viewed more favourably for the construction program savings it allowed. Piles intended for use elsewhere were able to be re-purposed for installation as remedial piles without the need to delay the project, on the expectation of future pile savings from ongoing PDA testing in other areas. However, later in the project these options were not available.

In one case, predrilling was performed to aid the installation of a remedial pile and several other piles included in the original design to ensure they reached the target depth range. This could only be undertaken where there was sufficient additional capacity in the ground to compensate for the loss of shaft friction arising from the predrilling.

An additional five piles were required to provide remediation for the groups containing the six remaining pile refusals. These were installed around the group at locations chosen in consultation with the structural engineer to ensure a balanced load distribution for the group.

4 CONCLUSIONS

A redevelopment in the Christchurch CBD employed the use of a driven pile foundation solution for the structures on the site. However, pile refusals above the target depth range required adaptation of the originally consented design during construction. These refusals were managed through a process of data collection, review and alternative remediation option consideration for each pile.

Different methods were applied to justify the acceptance or rejection of each refused pile. These methods included PDA testing, CPT investigations and engineering judgement. Each method was extended beyond its usual application to understand the every refused pile and the likelihood of cumulative variations in the soil profile ("exceptions to the rule" provided by the ground model) being responsible for their refusal.

Application of these tools in this manner was proven to enable manipulation of a piled foundation design to meet the demands of construction and overcome the challenges encountered. Furthermore, in addition to minimising the extent of pile remediation, the PDA test analysis was also extended to justify downsizing of many of the original pile group designs, achieving a net cost saving for the client on the original consented design.

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