Predicting axial performance of piles in pumiceous materials

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ABSTRACT

The design of piles is typically undertaken using empirical or semi-empirical methods developed for 'regular' silica sands or clays. The application of such methods to geological materials whose physical characteristics are unlike those on which the conventional design methods were derived can result in unconservative estimates of pile capacity. The pumiceous deposits that occur extensively throughout the central North Island of New Zealand are one group of materials where such standard methods of design must be used with caution. Pile tests undertaken as part of the Stella Passage Bridge and Tauranga Harbour Bridge projects have provided a rare opportunity to examine the performance of different pile types within pumiceous soils. Although the highly frictional nature of pumiceous soils can lead counter-intuitively to lower shaft capacities, the use of grouted or bored piles can give very good performance through the establishment of good soil-pile wall contact.

1 INTRODUCTION

Pile design is typically undertaken in accordance with procedures contained within codes, industry recommended practice such as the American Petroleum Institute RP2A or one of numerous textbooks on the subject. Being empirical or semi-empirical in nature, these standard methods can result in unconservative pile designs when applied to geological materials whose physical characteristics are unlike those on which the design methods have been derived.

One group of materials recognised as being problematic in this regard are the so called "crushable soils". This group of materials includes carbonate sands, some volcanic tephras, decomposed granites and mica-rich sands (Kuwajima et al, 2006). Within New Zealand, the most significant materials of this type are the pumiceous tephras and sediments that occur over extensive areas of the central North Island. The potential engineering problems associated with New Zealand's pumiceous deposits were bought into focus during the 1980's with the under-performance of the Stella Passage (Tauranga Harbour) Bridge piled foundations. The problems encountered in Tauranga were similar to those being experienced in Australia at that time, where offshore oil and gas platforms founded on steel tubular piles driven into calcareous marine sediments were often found to have axial capacities significantly lower than predicted by both laboratory testing and theory (Jewell, 1993).

Although research has provided some advancement in our understanding of the engineering characteristics of pumiceous materials (Wesley et al, 1998, Wesley et al, 1999, Pender et al, 2006), there is very limited guidance as to how the pile design process should be modified to account for these characteristics. Recent pile testing undertaken as part of the Tauranga Harbour Link (THL) Project has provided a rare opportunity to assess the relative performance of several pile types within pumiceous soils.

This paper uses the results of these tests, together with a review of the problems encountered during the piling of the Stella Passage Bridge, to identify the controls of pile performance in pumiceous soils as well as assessing the most appropriate construction methods.

2 GEOTECHNICAL CHARACTERISTICS OF PUMICEOUS DEPOSITS

Pumice exhibits physical and engineering characteristics significantly different to those of silica sand, on which most non-cohesive geotechnical design is based. Chief among these differences are lower particle strength but higher friction angles, void ratios and compressibility. Friction angles in excess of 40^o are common, with little difference in value being apparent between the loose or dense states (Wesley et al, 1998). Interlocking of the angular pumice particles also results in an apparent cohesion that can typically range from 5kPa to 20kPa. The shear strength and stiffness of pumiceous materials is also greatly influenced by the tendency of pumice particles to crush at even modest stress levels (Pender et al, 2006). This property has significant implications for both the axial and lateral capacity of piles in pumiceous materials.

3 FUNDAMENTAL CONTROLS ON PILE AXIAL CAPACITY

For uncemented granular materials, pile axial compressive capacity (Q_{ult}) is largely a function of the material's friction angle (ϕ'), the physical properties of the pile wall and the construction methodology. Given the high friction angles characteristic of pumiceous deposits, it would seem fair to assume that axial pile capacities could also be high. Experience from the construction of the Stella Passage Bridge clearly indicated however that the reverse was true (Murray-North, 1987).

4 OBSERVED PILE PERFORMANCE

The Tauranga Harbour Link Project, currently underway at Tauranga, involves the construction of a second harbour crossing between Tauranga and Mt Maunganui, as well as an extensive onshore viaduct system. The New Harbour Bridge is being constructed immediately adjacent to the existing Stella Passage Bridge, built between 1986 and 1988.

4.1 Geology

The geology of the Tauranga Harbour and its immediate surrounds represents a complex interplay of marine and non-marine depositional environments. The stratigraphy consists of sands, silty sands, silts/clays and gravels of marine and fluvial origin as well as silt, sand and clay mixtures representing both primary and reworked volcanic tepha, as well as ignimbrites. An abundance of pumice, volcanic glass and quartz particles within these materials reflects an environment dominated by rhyolitic volcanic activity (Murray-North, 1987).

4.2 Construction of the Stella Passage Bridge

The existing 480m long Stella Passage Bridge is supported primarily by 1.5m and 1.8m diameter concrete piles formed within a permanent steel casing. The piles range in length from 24m to 34m (base of pile cap to pile toe). Each open-ended casing was fitted with an external cutting shoe and driven to a founding level determined by SPT N values (Murray-North, 1987). Once the internal soil plug had been removed by bailing and airlifting, a 1.5m long pre-cast concrete plug was driven approximately one pile diameter. The casing was then sealed with tremie-poured concrete and the water removed prior to the installation of a reinforcing cage and structural concrete (Murray-North, 1987).

Soon after construction of the piles commenced, it became clear that the casing was developing significantly less shaft capacity than had been calculated. Reported problems included the floating of the Pier 7 casing upon dewatering and excessive settlement of Pier 1 during launching of the first deck segment (Murray-North, 1988a,b). Load tests undertaken on seven of the production piles and a small diameter test pile confirmed the low shaft capacities developed (Murray-North, 1987). The results of these tests are summarised in Table 1.

Table 1: Estimated Shaft Capacity, Stella Passage Bridge Piles

Location	Shaft Capacity		
	Actual (kN)	Design (kN)	Actual/Design
Pier 1	500 to 1500	2000	25 to 75%
Pier 2	3000 to 3500	3500	86 to 100%
Pier 3	1200 to 1800	4000	30 to 45%
Pier 4	1200 to 1800	2600	46 to 69%
Pier 6	<400	4000	<10%
Pier 7	<200	4000	<5%
Pier 8	"low"	3800	?
Test Pile	850	1300	65%

Murray-North (1988a,b) subsequently concluded that:

- The low shaft capacity (or skin friction) was due primarily to the external cutting shoe forming an annulus between the pile and the sand.
- The highly frictional and interlocking nature of the pumiceous sand limited the extent to which sand displaced by the cutting shoe could collapse back against the pile and re-established skin friction.
- Pile performance was significantly improved by the driving of a base plug.
- Piles without driven or jacked plugs performed worse than what would have been predicted for a bored pile.

A programme of side and base grouting subsequently proved successful in rehabilitating the piles. The use of driven plugs clearly improved the overall performance of the piles by increasing the end bearing above that expected for a bored pile, however some caution would be required in estimating this bearing capacity during design as crushing of the pumice particles at the pile tip can affect pile load-displacement characteristics (Kuwajima et al, 2006).

4.3 Tauranga Harbour Link Pile Tests

The New Harbour Bridge and associated viaduct are being founded entirely on large diameter, base-grouted bored piles, although driven open-ended steel tubular (CHS) and H-piles are being used for the temporary works. The selection of a bored pile system stems partly from the poor performance of the driven piles used on the Stella Passage Bridge 20 years previously. As part of the ongoing design validation process, pile axial capacity tests have been undertaken on both driven and bored piles.

4.3.1 Temporary Works Piles

A staged programme of pile testing was undertaken on ten H-piles (310UC97) installed as part of the bridge launch plinths. All of the piles were installed by vibration to depths below ground level (bgl) ranging from 10m to 37m. PDA testing was undertaken on eight of the piles, whereas two piles were assessed by Hiley analysis. The two shallowest piles were PDA tested twice, first 2 days after installation and again 18 days later. The ultimate axial capacities of the tested piles are shown on Figure 1, together with the theoretical capacity vs depth curve. The following observations have been made:

- A significant "setup" is apparent within the two piles tested 18 days apart. The increase in ultimate axial capacity was approximately 16% and 45%.
- The two 10m long piles had ultimate capacities between 3 and 5 times greater than predicted. Despite being located only 3m apart, their individual capacities differed by 45%.

- One of the 25m long piles had an ultimate capacity only 50% of the design value and its neighbouring pile, despite being installed only 15m apart.
- The 37m long piles had the highest ultimate capacities. Despite being located on a 15m grid, their capacities also varied significantly.
- There is no clear relationship between pile length and ultimate capacity in the upper 25m.

The relatively poor performance of the 13m to 25m long piles was thought to be due to external splicing plates forming an annulus along part of the pile. Flexing of the piles during installation is likely to have also contributed to this effect. Extending these piles to a total length of 37m by the use of flat butt welds resulted in the required capacities being achieved, although only 2 of the 4 piles tested exceeded the theoretical capacity for that depth.

PDA testing of a 45m long, 0.6m diameter driven CHS pile gave an estimated ultimate axial capacity of 2,200kN. This was only 60% of the theoretical unplugged capacity of the pile. The installation and driving of an internal plug increased the estimated ultimate capacity to more than 13,000kN, well in excess of the theoretical value. The results of this test have clear parallels with the observations made at the Stella Passage Bridge some 20 years ago.

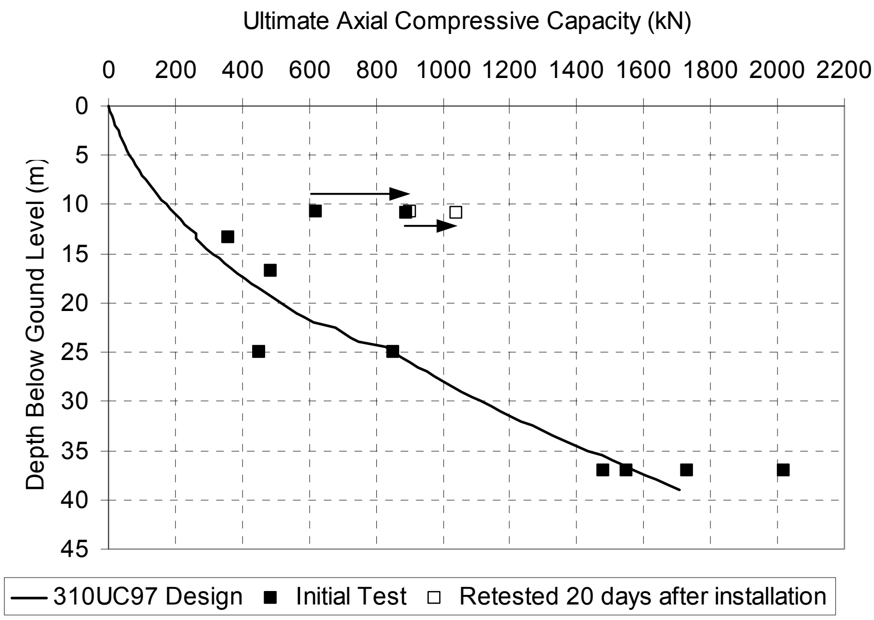

Figure 1: H-Pile Test Results

4.3.2 Bored Pile

A pile load test was undertaken on a 0.9m diameter bored and base grouted pile using two adjacent 1.5m piles as reactions. A summary pile head load-displacement curve is shown in Figure 2. The test pile was loaded in cycles of 1.0, 2.0 and 2.5 times working load. At peak applied load, the pile head had displaced approximately 10mm. Measurements from the instrumented pile indicate that the majority of the observed movement can be attributable to shaft resistance being taken up primarily by the elastic compression of the pile shaft (Wharmby et al, 2008).

Analysis of stain gauge data suggests that mobilised skin friction values along the pile shaft ranged from 40kPa up to 150kPa, with an overall average of 90kPa (Wharmby et al, 2008). This is significantly higher than the range of 2kPa to 67kPa estimated using the effective stress method. Whilst some of this difference can be attributed to the adoption of conservative

ϕ values in design, most appears to be due to the sands developing significantly higher skin frictions than would be expected for uncemented sands. An unknown proportion of this strength may be related to particle locking under higher pre-consolidation pressures. The relatively high strength of the sands is supported by the fact that samples from shallow depths are able to be cored and hand trimmed for triaxial testing.

Pile load transfer (t-z) back analysis results using the skin friction values estimated from the pile tests closely matched the observed pile head load-displacement response (Figure 2). The back analysis also indicates the presence of a softer stress-strain response i.e. the pumiceous sands have a significantly greater displacement to peak skin friction than is typical for silica sands.

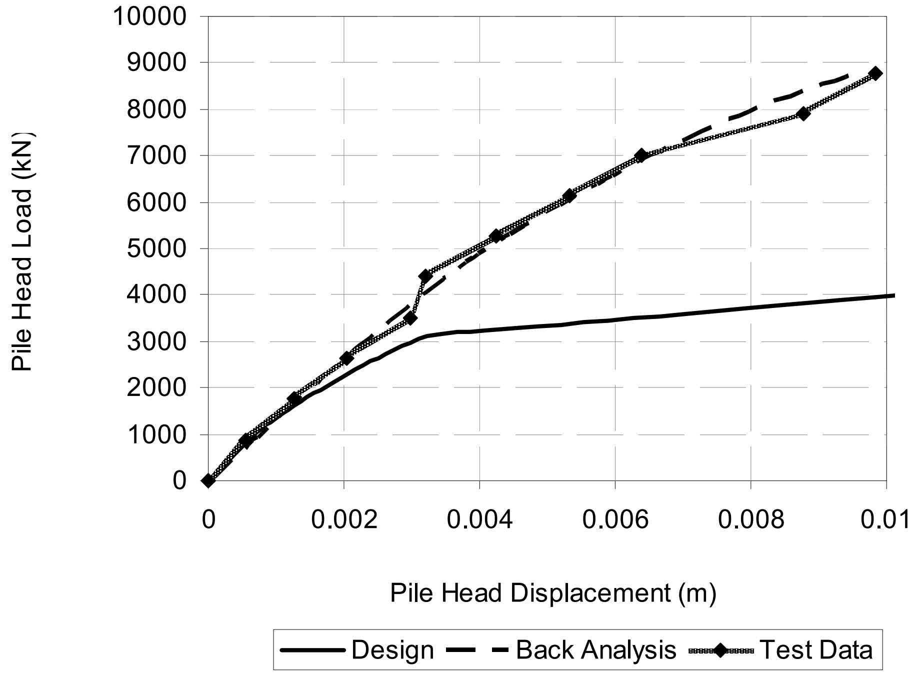

Figure 2: Bored Pile Test Data and t-z Analysis Results

5 CONCLUSIONS

The low compressive strength and interlocking nature of pumice gives it engineering characteristics significantly different to those of silica sands. Extensive pile testing undertaken for the Stella Passage Bridge and Tauranga Harbour Link Projects have highlighted the tendency for the development of shaft capacity within driven piles to be somewhat erratic and highly dependant upon installation method. This does not preclude the use of driven steel piles within pumiceous soils, although the actual capacities achieved may vary significantly between adjacent piles. The loss of shaft friction is thought to be primarily related to the formation of an annulus between sections of the pile and the soil as a result of either external protrusions or pile flexing during installation. The physical properties of the pumiceous sands, like those of calcareous sediments, limits the extent that the sand can fall back against the side of the pile to re-establish skin friction. These problems appear to be overcome if the driven piles are post-grouted or substituted with bored and base grouted piles. The use of driven base plugs appears to significantly improve overall performance, although grain crushing is expected to result in a softer than standard load-displacement response.

These findings are very similar to those obtained from a number of investigations into the substandard performance of piles installed in calcareous sediment (Jewell, 1993). The extensive body of research and development into calcareous pile design could serve as a guide to the performance of piles in pumice.

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