Tauranga Harbour Link foundation pile construction and testing

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ABSTRACT

The Tauranga Harbour Link project represents a major upgrade of the existing harbour crossing and approaches. It comprises a two lane incrementally launched bridge and major viaduct structure spanning road and rail links. This paper presents the details of the pile construction methodology using bentonite support fluid and presents a preliminary assessment of a load test on an instrumented preliminary pile.

1 BACKGROUND

The Tauranga Harbour Link project involves four laning of an existing causeway, the construction of a duplicate harbour bridge and a four lane flyover. The estimated cost of the project is about $255 million and is one of the largest transport projects constructed in New Zealand, and the largest in the Bay of Plenty.

The existing harbour bridge, constructed in the 1980's, was founded on 1.5m and 1.8m diameter piles. During construction of the original bridge problems were noted with the pile performances which indicated lower than expected end bearing and shaft friction capacities. Contemporary modifications to the design included lowering the pile toe levels and tube-a-manchette drilling and grouting of the pile shafts and bases. These modifications resulted in a successful conclusion to the original construction.

All the new bridges and flyover structures for the present project are to be founded on cast in-situ piles constructed under bentonite and with pressure grouted toes.

2 GEOLOGICAL SETTING

2.1 Investigation works

The development of the project has required several phases of investigations including boreholes, cone penetration tests, seismic cone soundings and in-situ shear vane profiling. Laboratory testing has included extensive classification, strength, compressibility testing and cyclic triaxial testing. The result of these various investigations is a relatively comprehensive set of data from which design parameters have been assessed and developed for the individual foundation piers.

2.2 Ground Conditions

Geologically the stratigraphy across the site is generally consistent in that it typically comprises a veneer of reclamation fill over Holocene-aged deposits which in turn overlie Pleistocene-aged and volcaniclastic derived sediments.

The ground investigation data identified that penetration resistances (SPT and CPT) generally increased with depth, but in the upper 10m to 20m relatively low resistances were recorded. The looser upper granular soils were identified as being susceptible to liquefaction based on SPT, CPT and shear wave velocity analyses. Airfall and estuarine deposits were identified from index test data to be generally 'safe' and non-liquefiable.

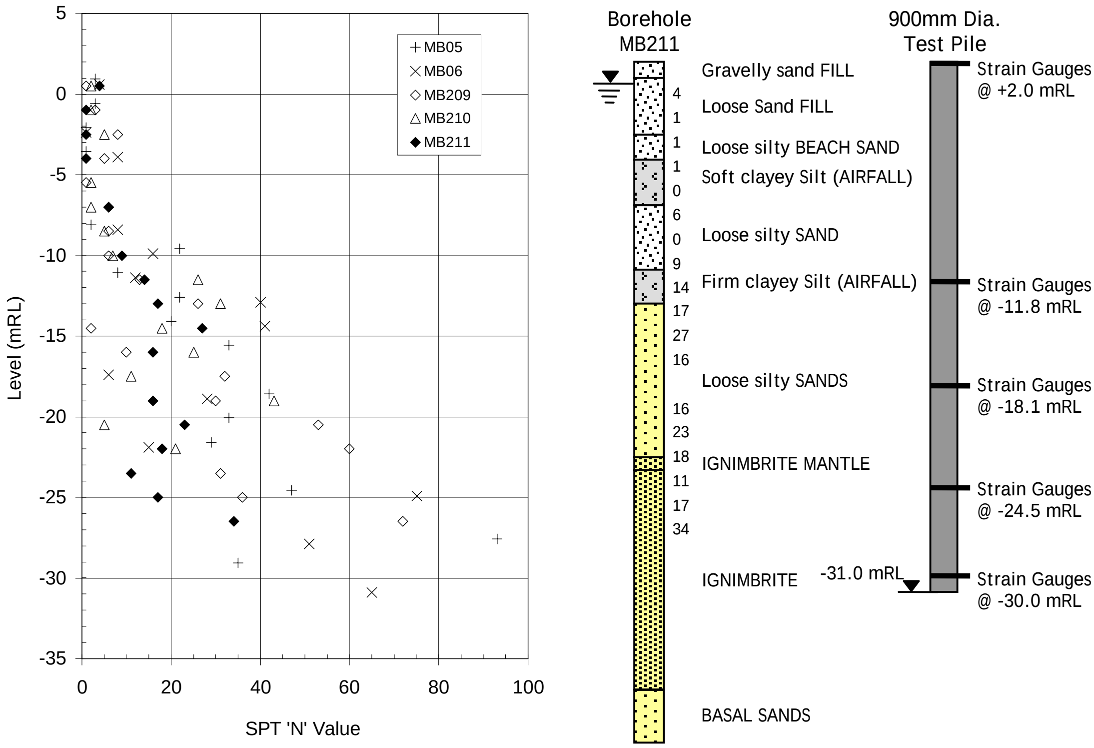

Figure 1: Geological profile near the test pile

Suitable horizons for pile founding levels could be identified in Ignimbrite and dense sand layers, typically encountered below 40m depth, where penetration refusal (SPT 'N' >50) was consistently observed. A summary profile of SPT 'N' values in the vicinity of the test pile is presented on Figure 1.

The SPT and CPT resistances recorded in the airfall soils were identified as being lower than otherwise anticipated for these soils based on conventional correlations with laboratory strength tests. These observations are consistent with research findings that relatively high friction strengths (and apparent cohesion values) can be attained for pumiceous soils despite their apparent low relative densities (Hind 2008). The apparent mismatch between frictional strength and penetration resistance is considered to be due to the crushable nature of these soils. These conditions were also found to be critical in the performance and remedial works designs for the original Harbour Bridge construction, as detailed in the following section.

3 FOUNDATION CONSTRAINTS

The poor quality of the upper soils, and the potential for widespread liquefaction and lateral spread towards the coastal margins ruled out consideration of shallow foundation options for the present project.

The piles for the existing Harbour Bridge were initially constructed using driven steel casings with oversize cutting shoes with a driven pre-cast plug to enhance the end bearing. Contemporary investigations into the poor performances of the original pile designs identified that the annulus left around the original oversize casing shoe did not close after driving, despite the 'sandy' nature of the soils resulting in poor shaft friction capacities. Testing of the material indicated relatively high soil strengths despite their low bulk density and relatively high void ratios. The high strength but crushable nature of the soils was attributed to the pumiceous nature of the volcaniclastic derived sediments. Disturbance to the fabric of these soils during pile formation was also a factor in the low pile capacity observed.

The design of the present piles has therefore been developed recognising the above constraints and construction issues. A minimum pile toe founding level was stipulated by the Principal and this was generally consistent with a formation level within very dense (SPT N>50) Ignimbrite or Sands. In addition to minimum founding depths, minimum design depths for liquefaction were also a given design condition. Pile load considerations therefore included liquefaction to depths to up to approximately 22m, resulting in large negative skin friction and lateral load/displacement considerations. Large diameter bore piled foundations with pressure grouted toes were therefore considered appropriate for the new bridges and flyovers for the present project.

3.1 Pile Design

One of the principal considerations for the design of the foundations to the New Harbour Bridge was adoption of a single large bored pile foundation at each pier. This design provided for significant benefits for construction including omission of extensive temporary works associated with construction of pile caps for the eleven piers within the estuary. The remaining structures also adopted a minimum number of larger diameter piles. The resulting unfactored pile design loads ranged from 4MN to 18MN for the abutments and piers. Design negative skin friction loads on the 1.5m to 2.3m diameter piles typically ranged from 0.5MN to 2MN.

The foundation designs provide for bored-cast in-situ piles constructed through a temporary casing, albeit for the over water piles the casing is permanent, using bentonite support for drilling below the casings. The construction of the piles under bentonite should result in minimal disturbance to the otherwise very dense Ignimbrite and Sands at and below the pile toe formation levels. Likewise, for design purposes no reduction of skin friction was provided for within the design, albeit for the deeper deposits the effective skin friction was limited to 100kPa. Allowing for possible unconsolidated sediments accumulating within the toe of pile from the bentonite suspended soils the base grouting was designed to ensure that mobilisation of the pile base capacities was not compromised. A design ultimate end bearing capacity of 12MPa was therefore adopted.

4 PILE CONSTRUCTION

The construction of such large diameter piles to depth of up to 65m in unstable soils below the ground water level is only possible with the use of a support fluid to maintain the stability of the hole. Potential fluids included polymers and naturally occurring sodium bentonite; the later was selected based upon the prevailing ground conditions and project size.

The basic construction methodology comprises:

a) Installation of short temporary casing
b) Excavation of pile with bore filled with bentonite drilling fluid
c) Cleaning of bentonite to remove sands
d) Installation of reinforcement cage
e) Placing concrete via a tremie pipe
f) Removal of temporary casing

All the piles are installed with grouting tube

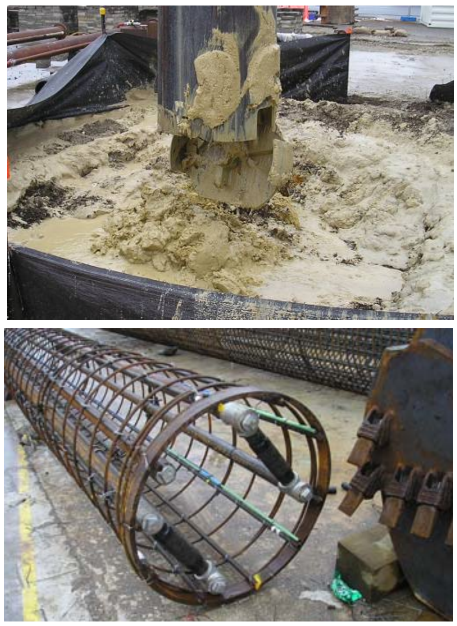

Figure 2b: Pile spoil grouting pipe work

circuits (Figure 2) to enable pressure grouting of the pile toes. The grout circuits are 'cracked' within 24 hours of the concrete pour and then grouted in multiple stages to limiting grout volume, pressure and pile uplift constraints. The grout tubes are also used for full length cross-hole sonic integrity testing.

4.1 Stability assessment

The static stability of a pile bore filled with bentonite requires the evaluation of 3D effects within the soils; there are a number of empirical limit state methods to assess the "active" soil pressure at the interface with the bentonite. The Huder (1972) design approach has been adopted as illustrated in Figure 3. Whilst reasonable assessment of the short-term soil parameters is important, the critical parameter is the difference in level between the bentonite and groundwater level. This is recognised in the ICE Specification for Piling and Embedded Retaining Walls (2007) which recommends the bentonite should be 1.5m above the groundwater level. Allowance needs to be made for the drop in bentonite level as the digging tool is removed. Furthermore, during excavation there are dynamic effects as the digging tool moves up and down the pile bore that locally reduce the stabilising bentonite pressure.

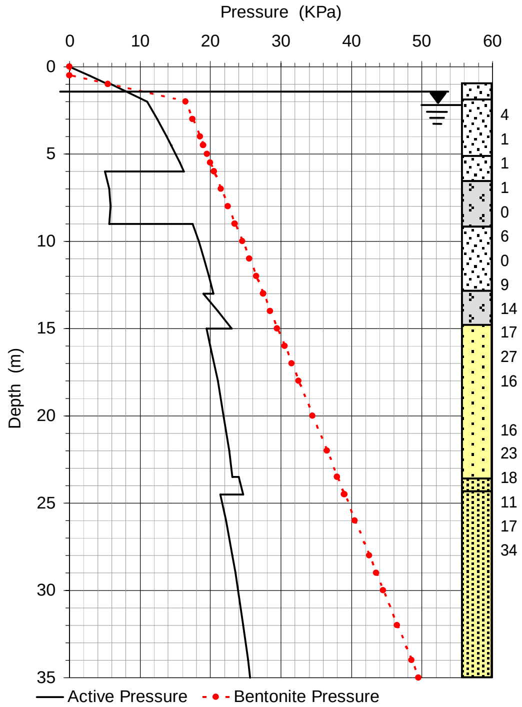

Figure 3: Bore stability assessment

4.2 Use of bentonite

To use bentonite as a support fluid requires a significant site set up and distribution network; the primary items include high shear mixers, hydration & storage tanks, desanding / mud conditioning plants and pumps. With only 3% to 5% solids, chemical testing of the water with pre-treatment if necessary, high shear mixing and hydration are required to ensure the bentonite fluid performs effectively. Guidance on appropriate fluid properties of the bentonite support fluid is given in the ICE Specification (2007) which was developed collaboratively by Consultants and Practitioners to meet foundation engineering requirements.

The fluid is susceptible to contamination as a result of mixing with ground water or spoil as excavation proceeds. For the bentonite to remain wholly effective it is important to routinely monitor the fluid properties and chemical state. For example, chlorides can cause the clay particles to flocculate resulting in the loss of water into the ground and the build up of a filter cake on the side on the pile bore.

Maintaining a suitable head of bentonite above the ground water level is critical with regard to the stability of the pile bore. The use of temporary casing provides the opportunity to both stabilise the upper deposits and safely allow the bentonite level to fluctuate above ground level as digging tools are inserted and removed. The digging tools are specifically designed for use in bentonite to minimise turbulence based upon the lift speed of the digging tools and the fluid properties of the material.

During the placement of the pile concrete, using a tremie, the bentonite is displaced and a degree of scour takes place as the concrete flows upwards. The sand content of the bentonite needs to be measured to ensure minimal sand settles out and the effectiveness of the tremie concreting process is maintained.

5 PILE LOAD TESTING AND RESULTS

5.1 Test and anchor pile construction

A 900mm diameter x 33m long sacrificial test pile was selected for load testing using two 1500mm x 36m long diameter permanent works piles for the load reaction. Whilst the diameter of the test pile was smaller than any of the works piles, the arrangement allowed testing of the pile to a significant and practically achievable load.

Figure 4: Test pile reaction frame

The ultimate capacity of the test pile was estimated as 10,500kN, of which approximately 70% was assumed to be generated in end bearing (≤12MPa) and 30% in skin friction (≤100kPa). The Principal had specified a global factor of safety of 3 for the pile design, which for the test pile equated to a working load of 3500kN.

The 10,500kN test frame (figure 4) and jacking system relied upon the tension capacity of permanent works piles was used to provide the reaction. Whilst these had a combined capacity of 14,200kN, these were monitored to ensure the performance under permanent load was not compromised; the initial maximum test load of 7000kN was increased to 8775kN (2.5 x the working load).

The test pile was constructed over an extended time period of 5 days which provides a good reference for the performance of the deep 2300mm diameter piles which take longer to construct than a short 900mm diameter pile and so the potential time effects are replicated. Base grouting of the test pile was carried out through the two circuits (ref. Figure 2) with 38 litres of grout (equivalent to 60 litres per m^2 of the pile toe area) placed at pressures of up to 35 bar.

5.2 Instrumentation

The test pile was fully instrumented, with provision for redundancy to safeguard against equipment failure. The instrumentation comprised 3No. extensometers and 12No. strain gauges located over the depth of the pile (ref. Figure 1) Furthermore, 4No. dial gauges at the pile head measured the pile head movement.

All instruments performed very well throughout the test procedure. Optical surveys were also undertaken to ascertain movement of the reaction piles and reference beam.

Figure 5: Instrumentation

5.3 Results

The overall load-settlement response for the test pile (ref. Figure 6) shows no indication that failure load is being approached. With less than 10mm settlement at 2½ times the design working load the performance of the test pile has exceeded design expectations.

The majority of the pile head movement can be attributed to mobilisation of shaft friction resistance (Figure 7) and elastic compression of the pile shaft; the ultimate shaft capacity was nearly fully mobilised. The strain gauge data suggests that mobilised skin friction along the pile shaft ranged from 40kPa up to 150kPa; the peak value is 50% higher than the limiting design value adopted. Furthermore, the overall average of 90kPa is 3 times higher than assumed for design purposes based on conventional effective stress shaft friction relationships.

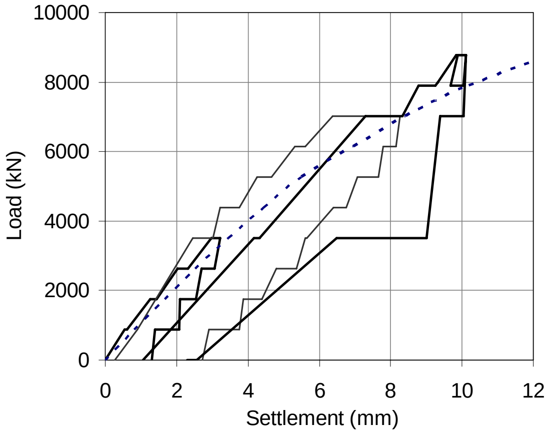

Figure 6: Pile load-settlement curve

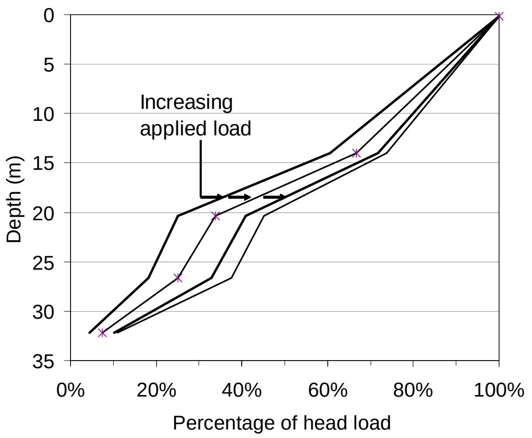

Figure 7: Load distribution along shaft

Analysis of the load-settlement curves to predict ultimate pile capacities based on hyperbolic–load-settlement characteristics (Fleming 1992; Chin 1978) yielded ultimate capacities of between 15,000kN to 17,000kN. The broken line in figure 6 represents the correlation after Fleming. These ultimate loads equate to an average shaft friction of approximately 90kPa and a base resistance of 12MPa. The design base resistance of the piles has therefore been confirmed but the skin friction values back analysed are higher than anticipated. The higher skin friction values achieved may have resulted from a combination of an underestimate of the shear strengths of the pumiceous soils, based on penetration test data, and the load reversal characteristic for base grouted piles (Fleming, 1993).However, peak values of 150kPa appear exceptionally high and therefore it may be concluded that potential grout leakage beyond the pile toe may have also effectively and locally enhanced the effective pile diameter.

6 CONCLUSIONS

The construction of the test pile and reaction piles has been achieved successfully using bentonite to maintain the stability of the uncased pile bore in the prevailing Tauranga soils with a high ground water level. A maximum test load of 8775KN was applied to the 900mm diameter pile with observed deflections of 10mm at the pile head and 1.5mm at the pile toe prior to termination of the test due to movement of the of the reaction piles. Whilst testing to a higher proof load would have been preferable, analysis of the data indicates that the pile has an ultimate capacity of 15MN to 17MN which corresponds to an ultimate base capacity of 12MPa and 90KPa average shaft friction.

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