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Uplift load testing of grooved tension piles in Waitemata Group Rock for the Manukau Rail Link

D. M. Oosterbeek[1], BE(Civil), MIPENZ, CPEng

[1]Opus International Consultants Ltd, Auckland Office, P.O. Box 5848, Auckland, 1141; New Zealand, PH (+64-9) 355 9500; FAX (+64-9) 355 9585; EMAIL: darrell.oosterbeek@opus.co.nz

ABSTRACT

The results of static load testing in uplift of two preliminary test piles undertaken for the Manukau Rail Link project are presented. The test piles were 600mm in diameter with socket lengths of 3m within Waitemata Group Rock. Grooving of the rock socket was undertaken to provide a rough interface. Each test pile was loaded under maintained static conditions against a reaction frame supported by four screw piles to resist the test loads. Multiple load cycles were used within the predicted working stress range of the contract piles to test the performance of the piles under cyclic loading prior to imposing maintained static load increments to failure. Measurements of pile deformation were taken throughout the tests using strain gauges at the pile head. The results were analysed to determine ultimate shaft friction within the Waitemata Group Rock.

Keywords: bored piles, shaft friction, pile load testing.

1 INTRODUCTION

The Manukau Rail Link project involved the construction of a new rail trench to form an underground terminal station in Manukau City, Auckland. The new station is 7m underground and will be serviced by the first new rail route to be constructed in Auckland since 1930. The trench was formed top-down in Greenfield conditions using propped secant pile perimeter walls and a cast in situ reinforced concrete base slab. Bored and cast in place tension piles socketed into Waitemata Group Rock extend below the base slab to resist high groundwater uplift loads on the underside of the slab. Two maintained static load tests in uplift were carried out on pre-construction piles to verify the ultimate shaft friction between the Waitemata Group rock and bored piles.

The rail trench was constructed for Kiwirail by Leighton Contractors under a design and build contract with Opus as design consultant. Piling works were subcontracted to Brian Perry Civil and Piletech undertook the pile load tests.

2 GROUND CONDITIONS

2.1 Geological setting

The geology at the site comprises a veneer of basalt ash from the Auckland Volcanics overlying an approximately 12m thick sequence of Puketoka Alluvium. The alluvium typically consists of an upper stiff silty clay with loose liquefiable silty sand / sandy silt sequences below. Very weak Waitemata Group rock is encountered below the alluvium and is the founding material for the secant and tension piles.

2.2 Waitemata Group rock

The Waitemata Group rock at the site is a very thickly bedded sandstone-dominant rock with moderately thin mudstone interbeds. The average unconfined compressive strength of the rock as measured in the laboratory is about 2 MPa with a range from 1 MPa to 4 MPa. The sandstone ranges in grain size from fine through medium to a pebbly course-grained rock. Fractures observed were widely spaced, infrequent and generally dipping from sub-horizontal to 30 degrees.

2.3 Groundwater conditions

Groundwater measurements were taken from piezometers installed in investigative boreholes at the site. Piezometers were screened in the upper soils and in the Waitemata Group rock, and measurements of groundwater levels were slightly lower in the Waitemata Group rock, indicating separate aquifers. Significant annual fluctuations in groundwater levels were observed within the upper soils with measurements indicating that winter high water levels rose to 1m below original ground level and fluctuations of 3m to 4m below this level were observed in summer periods.

3 PILE DESIGN

3.1 Philosophy

The rail trench is situated at a previously Greenfield site, and is up to 7m deep at the terminus. The trench was constructed by installing hard/soft bored secant piles around the perimeter from original ground level, casting reinforced concrete props at grade and subsequently excavating to the underside of the base slab. The perimeter walls and cast-in-place base slab provide a watertight trench and consequently significant uplift forces are considered to act on the underside of the base slab due to the groundwater levels at the site.

Bored 600mm diameter reinforced concrete piles were adopted relying on friction at the pile/rock interface to resist the uplift forces on the base slab. The piles are spaced in a 4.5m grid which effectively reduces the span of the base slab significantly. This minimises the shear forces and bending moments in the base slab and consequently the thickness of the slab. Due to annual fluctuations in groundwater levels at the site and intermittent train loadings on the base slab, the piles were designed to mitigate possible degradation of shaft friction due to cyclic loading between tension and compression about a mean tensile force.

Based on the findings of M.F.Randolph (1988) and H.G.Poulos (1988b) from testing of grouted pile performance, the socket lengths of the piles within the Waitemata Group rock were designed to limit the serviceability limit state shear stresses at the pile/rock interface to not more than 25% of the unit ultimate shaft resistance in order to avoid degradation from cyclic loading.

3.2 Assessment of shaft friction

The unit ultimate shaft friction available at the pile/rock interface was assessed using the relationship proposed by A.F.Williams et al (1980) and shown in Figure 1. This relationship is based on testing to failure of test piles with a range of diameters within weak sedimentary rock and relates the unit ultimate shaft resistance to unconfined compressive strength. This was considered an appropriate relationship for this project based on similarity of rock type and strength. From this relationship a unit ultimate shaft friction of 430 kPa was assessed based on a lower bound unconfined compressive strength of 1 MPa. For the assessed Ultimate Limit State (ULS) tensile load of 2,100 kN and Serviceability Limit State (SLS) tensile load of 1,580 kN, a socket length of 9.2m was adopted for the contract piles (giving an average working shear stress of about 90 kPa at the pile/rock interface).

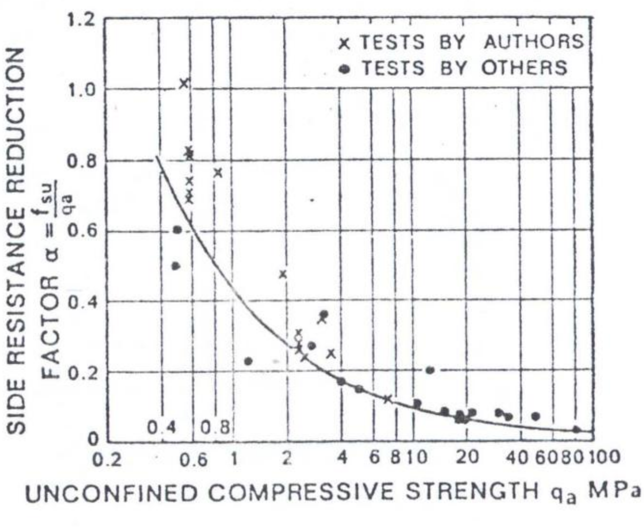

Figure 1: Side resistance reduction factor reflecting variation of rock strength (after Williams et al, 1980)

4 PILE CONSTRUCTION

Two test piles were constructed from original ground level prior to contract piling works commencing, primarily to verify the shaft resistance available within the Waitemata Group rock. The piles were constructed with 3m long 600mm diameter sockets within competent rock and 750mm diameter cased auger holes from ground level to the top of the rock sockets. Groundwater flow into the pile holes was not significant and the pile holes were able to be pumped dry for inspection. The piles were isolated from the surrounding soil above top of rock level using 600mm diameter sleeves. A reduced pile socket length in comparison to the contract piles was adopted to ensure the test equipment was capable of applying the anticipated failure loads to the piles and that the reaction piles were not overloaded. In order to ensure a roughened pile shaft, a drilling bucket with a grooving tool was utilised after the required rock socket was achieved. Total lengths of the test piles were approximately 15m.

5 PILE LOAD TESTS

5.1 Test setup and procedure

The setup for each of the pile tests was designed and assembled by Piletech, and involved the use of steel I-beams to form a reaction frame and four steel screw piles supporting the frame at each end of the beams-on-ground. The screw piles functioned as reaction piles to resist the compressive load applied to the reaction frame by a hydraulic jack applying tension (uplift) to the pile. The setup is shown in Figures 2 and 3.

Pile displacement in uplift was measured at the pile head using strain gauges fixed to an independent frame. The load was applied to the steel reinforcement bars of the test pile using a hydraulic jack and was applied incrementally up to a test working load of 450 kN that would apply approximately the same average working stress to the rock socket as the contract piles. This load was held for 15 minutes to measure creep movement before incrementally decreasing back to a nominal load. This procedure was cycled ten times to investigate the effect of cyclic loading within the anticipated working stress of the pile, and after holding the working load on the tenth cycle for 500 minutes, the load was increased incrementally to either the yield strength of the reinforcement bars or to geotechnical 'failure' (where attempts to further increase the load are unsuccessful and excessive uplift displacement occurs).

Figure 2: Pile test setup

Figure 3: Connection detail

5.2 Test results

The load-displacement curve for Test Pile 1 indicates that the rock socket reached 'geotechnical' ultimate failure at approximately 2500 kN. For a 3m rock socket this equates to a unit ultimate shaft friction of approximately 440 kPa. Test Pile 2 reached a maximum test load of 2900 kN with significantly less displacement, and no indication of failure is observed in the load-displacement curve. The results are presented in Figure 4.

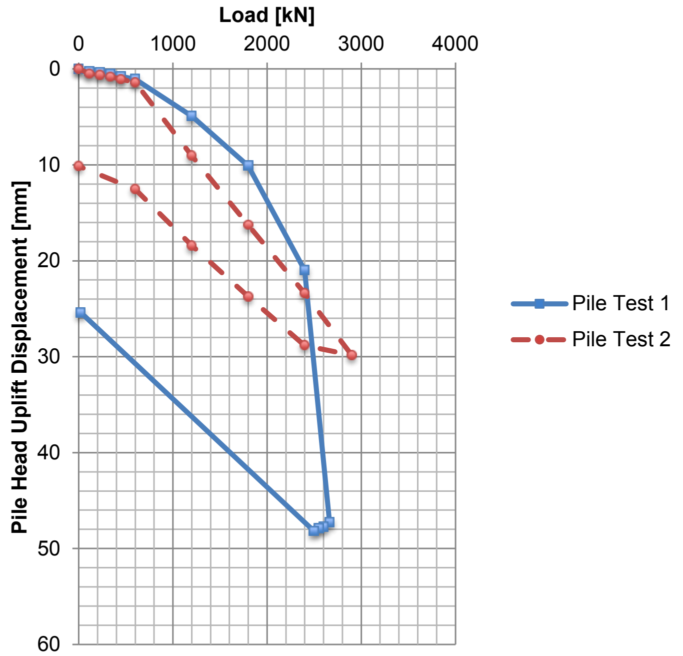

Figure 4: Pile test load-displacement curves

Figure 5 presents the load and displacement curves plotted against time for Test Pile 1. It can be observed that creep was negligible during load holding periods within the working load cycles, the degree of displacement was essentially identical for each load cycle to the working load, and displacement effectively returned to zero after each load cycle.

Also of note is that as the test load was increased beyond the working load of the pile, the degree of creep for each load holding period increased until the pile was unable to resist further tensile loads (endeavours to increase the test load resulted in large displacements).

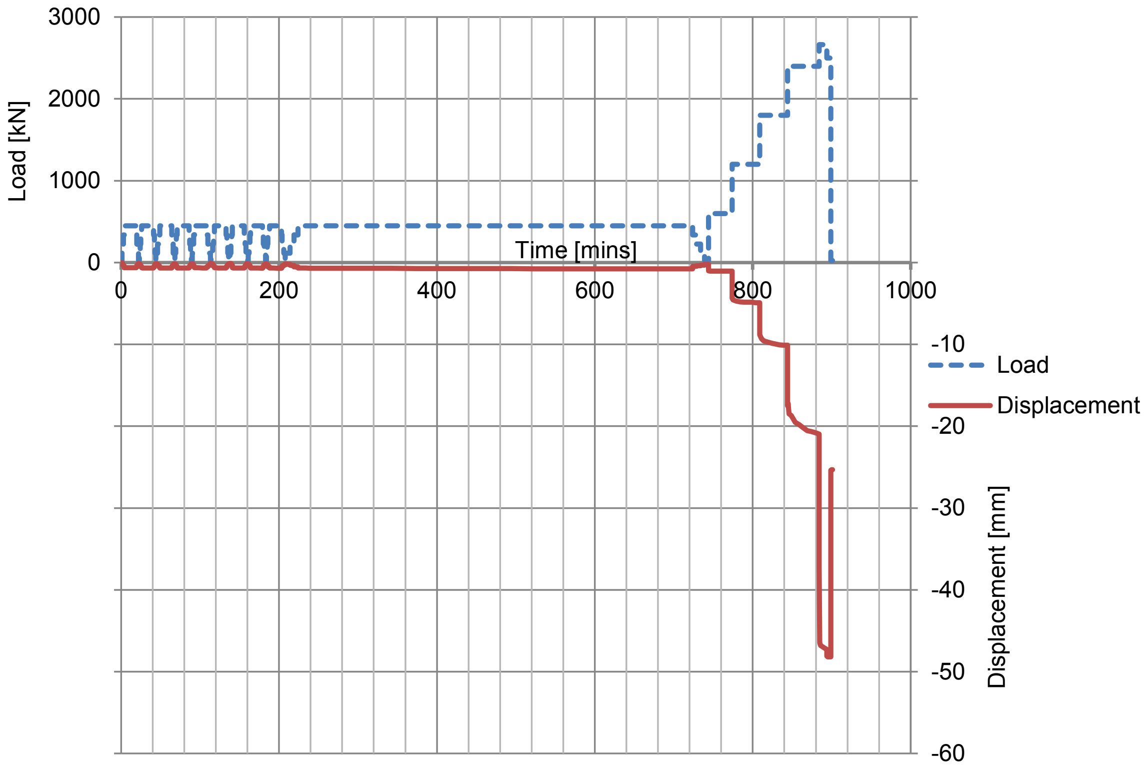

Figure 5: Load and displacement versus time for Test Pile 1

6 CONCLUSION

The purposes of the pile load tests were to verify the ultimate shaft resistance of bored piles in the Waitemata Group rock and investigate the effect of cyclic loading within the anticipated working stress range for the rock. Test Pile 1 was successful in mobilising the unit ultimate shaft friction at the pile/rock interface, a value of 440 kPa. Test Pile 2 was not successful in mobilising the unit ultimate shaft friction, and the results indicate a higher shaft friction capacity was achieved for Test Pile 2.

The design philosophy relied on an established relationship between unconfined compressive strength of the rock and unit ultimate shaft friction that was derived from historical test data. Site specific data was considered important in order to verify that these relationships were applicable to Waitemata Group rock. The ultimate capacity proven by Test Pile 1 compared extremely well with the predicted capacity based on a lower bound UCS strength (440 kPa measured versus 430 kPa predicted) whilst Test Pile 2 was tested to a greater load and did not reach ultimate capacity.

These two piles were essentially identical in dimensions and construction methodology and the variation in results could be attributed to the variability in rock strength (a range of 1 to 4 MPa) that was identified by laboratory UCS testing.

Testing of cyclic loading, albeit limited to 10 cycles per test within the working stress range of the piles, did not result in degradation of the rock/pile bond. Degradation of shaft resistance as a result of cyclic loading is related to the magnitude of the loads applied in comparison to the ultimate shaft resistance. For the Manukau Rail Link project, working stresses on the piles were limited to not more than 25% of the assessed unit ultimate shaft resistance. Based on the pile load test results it is considered that the working stresses were limited sufficiently to mitigate the effects of cyclic loading on the piles.

7 ACKNOWLEDGEMENTS

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