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TENSION/COMPRESSION LOAD TESTING OF A MINIPILE

ESSAIS DE CHARGE A LA TRACTION/A LA COMPRESSION SUR UN MINI PIEU

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SYNOPSIS: Minipiles have been installed on many projects in many parts of the world to carry high working loads. However, information on the behaviour of minipiles when subjected to repeated tension and compression load testing and actual performance under seismic loading is limited. In one recent seismic retrofitting project in Vancouver, a combination of vertical and inclined minipiles were installed to increase the basal shearing resistance of shear walls. A testing program was designed and carried out to investigate the behaviour of a test minipile to 25 cycles of repeated tension and compression loading. Strict displacement tolerances were met in the course of repeated loading on the test minipile and subsequent proof-loading of 90 production minipiles.

INTRODUCTION

As part of the seismic vulnerability assessment for schools in Vancouver, British Columbia, an east Vancouver school was identified as needing priority attention. The school is an unreinforced masonry three storey structure built in 1913, with less seismically vulnerable structures added in the 1950s and 1960s. The design of seismic retrofitting for the school was carried out in 1991. The retrofit measures were to make the school safe for seismic ground motions with an annual probability of 0.0021 (i.e., a return period of 475 years), the same seismic design criteria for new structures in Canada. In Vancouver, the peak ground acceleration at this probability level is about 0.2 g with most contribution deriving from earthquakes with magnitudes between 6.3 and 7.2.

The structural retrofit measures included construction of stiff shear walls throughout the interior of the building to take up the seismic horizontal load. The static vertical loads of the building would not be transferred to shear walls as part of the retrofit design, with the result that shearing resistance developed at the base of the shear walls from self-weight was not sufficient to resist the seismic horizontal loads. It then became necessary to increase the resistance at the base of the shear walls by tying the wall footings to the foundation soils in some manner.

A number of alternative solutions were considered to increase the basal resistance of the shear wall foundations. These included concrete piers, soil anchors and minipiles. Initially vertical concrete piers were preferred because isolated piers could develop substantial horizontal resistance even at 5 m depth in the weak sandstone at the site. However, reduction in the lateral capacity due to the pile group effect and difficulties with installation made concrete piers less attractive than minipiles. Soil anchors were considered but were discounted early because loads could only be resisted in tension. The minipile emerged as the favoured solution because such piles can resist loads in compression and tension, can be installed relatively easily in confined areas, and can develop large resistance.

REVIEW OF MINIPILING PRACTICE

Minipiles are defined as drilled and grouted piles with diameters less than about 300 mm. They go by many names including pinpiles, paii radice, micropiles and root piling. Their fundamental characteristic is that they can be installed with equipment used for anchoring and grouting which allows them to be installed in confined spaces not accessible to conventional piling equipment. Minipiles have been used extensively in Europe but have seen little use in North America until the last decade. Bruce (1992, 1993) gives a useful summary of examples of minipile uses in the United States. Of note is the high capacity of minipiles which can carry working loads of up to 900 kN in soil and three times that in rock.

As minipiles have been used mainly to sustain static loads, there are very few case records documenting the behaviour of minipiles during repeated or even reversed (i.e., both tension and compression) loading. Dywidag (1992) reported on the performance of minipiles subjected to repeated loading from tests conducted in Germany in 1984-85. The test minipiles were 5 m long and 130 mm in diameter and were pressure-grouted (500 kPa) in moist, medium dense sand. The minipiles were subjected to about 24,000 cycles of alternating tension (120 kN) and compression (150 kN) before failure. Failure could have been a result of fatigue of the steel and grout and/or of degradation of the grout-soil contact.

Very little experience existed in Vancouver in 1991 with minipiles so a load test program was specified as part of the seismic retrofit contract and was conducted prior to the installation of the production piles. The objective of the load test program was to establish whether minipile deformations during 25 cycles of repeated compression and tension would be within the strict displacement tolerances imposed by the structural designer. Twenty-five cycles were selected because 25 is the approximate number of significant cycles expected for the design earthquake.

GROUND CONDITIONS

Ground investigation and subsequent excavations revealed that the uppermost soils at the school were loose to medium dense sand and gravel fill generally less than 1 m thick. The fill was underlain by uniform fine to medium grained sand to 2 m on which the spread footings of the school building were founded. Weathered sandstone was encountered beneath the shallow soils at varying depth, with occasional large boulders near the contact. The sandstone is a weak Tertiary rock that occurs beneath much of Vancouver. The depth to "sound" rock (moderately decomposed or better) at the school was variable, due to erratic weathering and/or the presence of wide sub-vertical weathered joints. In general, sound rock was present at about 4 m to 6 m depth. The length of minipiles was specified to be 13 m, which was well into the sound but weak sandstone. Groundwater was generally absent in the soils and in the upper few metres of the completely decomposed sandstone. Figure 1 shows the log of a test hole which was drilled and cored within 10 m of the minipile load test setup.

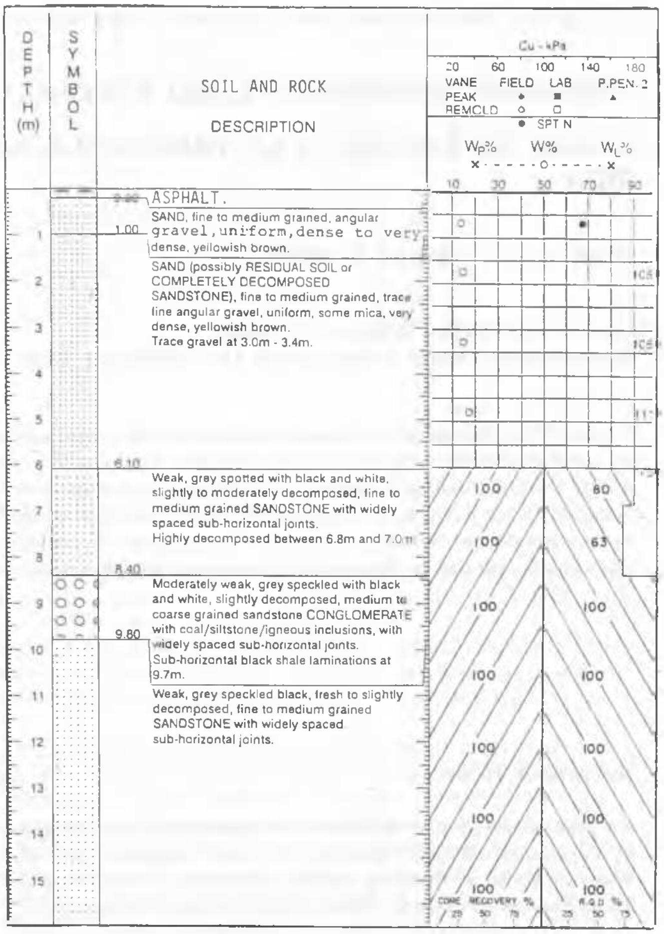

Figure 1 Test Hole Log (10 m from Test Minipile)

MINIPILE DESIGN CONSIDERATIONS

The minipiles have to resist overturning moments and lateral loads of the shear wall footing during seismic loading. Vertical minipiles are included beneath the shear wall footings to resist the seismic overturning moments and inclined minipiles will resist the seismic lateral loads. Initially, the use of vertical minipiles with steel casing in the upper 5 m to resist the lateral loads were considered; however, even with the steel casing providing additional moment resistance, the lateral load capacity of vertical minipiles was considerably smaller than inclined piles and not enough vertical piles could be installed to take up the lateral loads. Inclined piles have performed poorly in past earthquakes and have generally failed at the pile/pile cap connection. In a group of inclined and vertical piles, the inclined piles take up most of the horizontal seismic loading because horizontal loading is resisted in the relatively stiff axial direction. A brief review of past failures of inclined piles indicated that, in most cases, the piles penetrated soft soils and/or had a free-standing length. It was judged that relatively large cyclic deformations occurred under these conditions imposing large stresses on the pile/pile cap connection. In this project, the piles were installed in dense ground and would likely experience much less seismic deformations than those in the past failure cases. Low seismic displacements combined with a strengthening of the connection were used to justify the use of inclined piles to resist seismic loads on this project.

The production minipiles have a diameter of about 130 mm, a length of 13 m and central threaded steel bar with a nominal diameter of 57 mm. The steel yield stress was specified to be less than 415 MPa, to avoid the brittle nature of high-yield bars. The specifications called for the drill holes to be pressure grouted with non-shrink grout with a water-cement ratio between 0.38 and 0.5 and to have a minimum 7-day compressive strength of 35 MPa. The axial design load for the piles was specified to be at least 535 kN with less than 6 mm of deformation in tension and 3 mm in compression. The contract called for each of 94 production minipiles to be proof-loaded to the design load in tension with deformations less than the specified value.

The load test program called for loading the minipile in alternate tension and compression for 25 cycles at a number of load levels. The selected load amplitudes were 535 kN (the design load), 625 kN, 710 kN and 960 kN. At the 26th cycle of each load increment, the loads were maintained for 30 minutes, or until creep deformations ceased. The test pile was loaded in tension to 93% of the yield strength of the threadbar, i.e., 1,220 kN.

TEST MINIPILE INSTALLATION

A 130 mm-diameter test minipile was installed to 13 m depth outside the school building with a rotary percussion drill rig. Since the groundwater table was at the soil/sound rock interface or lower, the drill hole did not need to be cased. A threadbar of 57 mm diameter, with a yield load of 1,334 kN, was placed in the hole to 13 m depth. A threadbar with a yield strength higher than that specified for the production piles was used to induce failure at the grout-soil interface as much as possible. The threadbar was spliced from three sections, using couplers capable of transmitting both tension and compression. Grout was tremied under gravity into the bottom of each hole via a grout line attached to the threadbar. The contractor demonstrated that the test minipile could develop design loads without pressure grouting and tremie placement of grout under gravity was subsequently adopted for the production minipiles also. Four reaction minipiles were installed in a square pattern around the test pile. The reaction piles had a diameter of 45 mm, central threadbars with yield loads of 600 kN and installation depths of 9.5 m.

TEST SET-UP

The load test set-up devised by the contractor was simple but effective (Figure 2). Loads were applied to the test pile by three donut-shaped hydraulic jacks through a cross-beam. The two smaller jacks at either side of the cross-beam were used to apply tension to the test minipile. They were operated using a common pump via a manifold, to minimize differential movement between them. For tension loads, the central locknut through the test pile threadbar was tightened and the two side locknuts was left loose. The smaller jacks reacted against steel beams which transmitted forces to the ground.

Figure 2 Tension/Compression Load Test Set-Up

The middle jack was used to apply compression loads to the test pile. The test pile threadbar fit through the central hole of the jack. The test pile was put into compression by operating this hydraulic jack, with the centre locknut left loose and the side locknuts tightened. Lifting of the cross-beam was prevented by the four reaction piles, which were connected to the cross-beam via ground beams and steel threadbars. Alternate tension and compression was applied by operating the side and middle jacks and loosening/tightening the side and middle locknuts.

Two dial gauges were used to record axial displacement of the central threadbar, one on either side and close to the ground surface. The dial gauges were seated on a remotely supported wooden reference beam, the movement of which was continuously monitored.

TEST PROGRAM RESULTS

The axial displacements measured at the pile top during compression and tension loading are shown in Figure 3. Figure 3 shows the displacements at maximum tension (quarter cycle), at zero load just after tension (i.e., half-cycle), at maximum compression (three-quarter cycle) and just after compression (full cycle).

The cyclic displacement was greater in tension than in compression. It can be seen from Figure 3 that the pile top displacement generally increased as the number of cycles increased as reflected by the flatter slopes of the load-displacement plots at higher loads as shown in Figure 3. However, the cyclic compression displacement under a given load stayed practically constant up to a load level of 630 kN, and increased as the load amplitude went over 710 kN.

Tension produced more residual displacement than compression. Figure 4 shows that residual displacement of 0.5 mm to 5 mm existed after the tension half-cycle. Interestingly, this residual displacement was "cancelled" to a large extent when undergoing the following half-cycle of compression, for load amplitudes at 710 kN or less. In other words, whereas repeated tension loading tends to induce cumulative permanent displacements, repeated tension and compression loading at low to moderate load levels resulted in very small net permanent displacements unless the load amplitude is very high.

The amount of creep was of the order of 0.1 mm after 30 minutes even at a high load level of 960 kN, and was considered negligible.

PRODUCTION PILES

Ninety-four production minipiles with lengths of 13 m to 17 m were installed to anchor and support the interior shear walls. The work was carried out round the clock to meet the tight construction schedule. The inclination ranged from vertical to 40° to the vertical. The piles were installed with a rotary percussion drill rig equipped with a down-the-hole hammer system. Pile installation was performed in confined areas with headroom less than 3 m and in some cases the piles had to be installed through existing footings or boulders. About 95 percent of the production piles were

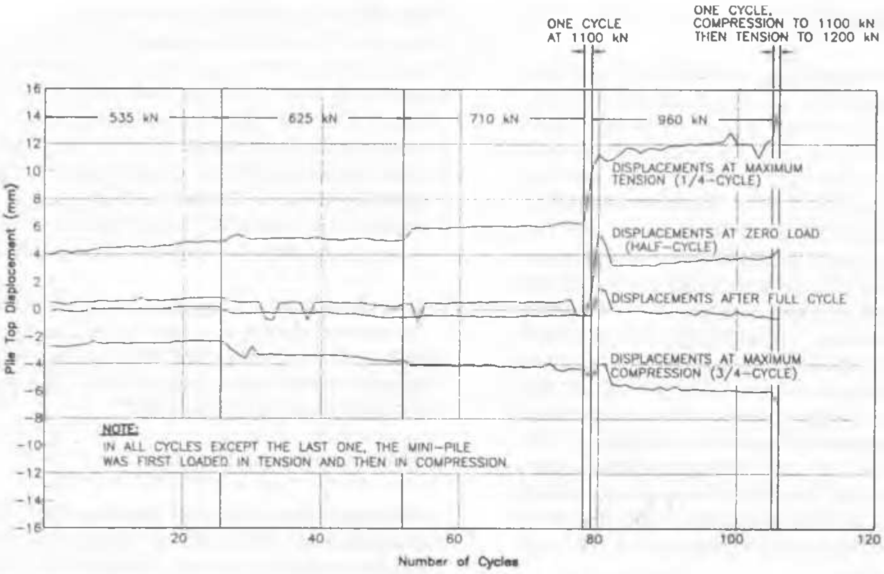

Figure 3 Cyclic Displacement at Pile Top

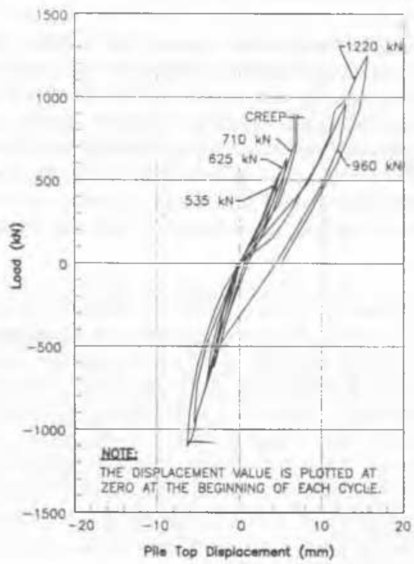

Figure 4 Displacement Loci of Last Cycle at Each Load Level

proof-loaded to 100 percent of the design load (i.e., 535 kN) in compression and gave deformations between 3 mm to 6 mm in general.

CONCLUSIONS

The test pile program confirmed that a design load of 535 kN for the production piles could be applied with an elastic deformation less than 6 mm. The minipiles developed these high capacities without pressure grouting. Pressure grouting in larger diameter piles would result in higher capacities. However, the ability of the pile/pile cap connection to transfer the load to the minipiles without failure would have to be confirmed by a test program or earthquake performance data.

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