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MINI-PILE CASE HISTORIES AND A TENSILE LOAD TEST ON AN INSTRUMENTED MINI-PILE

Schnabel
FOUNDATION COMPANY

Mini-Pile Case Histories and a Tensile Load Test on an Instrumented Mini-Pile

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Abstract

Three mini-pile case histories are presented. The first project used mini-piles to strengthen an existing building's foundation in order to add a new level. A small, short mast drill was used to install 159 mini-piles inside the occupied shopping mall. The piles were drilled through lagoonal silts and clays into a cemented coral formation. The second project utilized mini-piles and passive anchors to stabilize shallow fill slides along a narrow roadway. The piles acted as soldier beams supported by passive (untensioned) anchors. A powerful, compact, rotary drill was used to install the mini-piles without closing the roadway and with minimal disturbance to the surrounding landscaping. The third project used mini-piles to upgrade an existing building to meet current seismic standards. Seventy-eight piles were installed to support new shear walls inside the building. Most piles were post-grouted. Load test results are presented.

Results of a 1779 kN tension load test on a 15.2 m long, instrumented mini-pile are presented. Soils at the test pile location consisted of a medium stiff to stiff clay with occasional lenses of clayey sand and clayey gravel underlain by sandy gravel. The pile was installed in a 254 mm augered hole, and consisted of a 12.2 m long 168 mm casing and 15.2 m long rebar. Vibrating wire strain gages on the casing and the bar measured strain along the pile. Load transfer curves are presented.

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Shopping Center Expansion

A shopping center in Honolulu, Hawaii was constructed in 1956. A major renovation in 1998 added a level to the center and required the strengthening of the existing foundation for both vertical and earthquake forces. The shopping center was constructed over a reclaimed marsh and is underlain by lagoonal deposits consisting of coral sands and gravels in a matrix of very soft clays and silts. A very dense coralline reef formation underlies the lagoonal soils at a depth of 6.1 to 9.2 m below the ground surface. The coral is 3.0 to 4.6 m thick and grades from very dense to a highly cemented coral and limestone. Below the coral are weakly cemented coral sands and gravels with occasional lenses of very well-cemented coral. The original foundation for the shopping center consisted of single driven octagonal prestressed concrete piles bearing on the coral. The piles were assigned capacities of 667 and 1112 kN, which were insufficient to support the loads resulting from the new level.

One hundred and fifty-nine, 400 kN mini-piles were selected to support the new building loads within the existing shopping center. URS Corporation recommended using mini-piles since: (1) the shops had limited headroom, (2) compact, quiet, mini-pile drills would create the least amount of disturbance, (3) several piles could be completed during the evening shift when the shops were closed, (4) a 400 kN mini-pile could be designed to have similar deflection behavior to the existing concrete piles, and (5) the mini-piles would resist uplift forces.

Figure 1 shows a typical mini-pile used at the shopping center. The piles were installed inside the existing structure using short masted, electric, rotary drills. Two methods were used to install the mini-piles. Test piles and the initial piles on the project were installed by coring, with water flushing, a 194 mm diameter by 12.7 mm wall casing 0.6 m into the top of the cemented coral. A down-hole-hammer was used to clean the casing and to advance a 165 mm hole below the bottom of the casing. Then the hole was tremie grouted using a neat-cement grout, the #36 rebar (rebar sizes are metric, i.e. 36 mm) was inserted into the lower portion of the hole, and the #25 bent bar was set in the upper portion of the casing. Two conditions required the adoption of an alternate installation method. Caving ground below the casing was encountered in one area and air from the down-hole-hammer was observed flushing the grout out of a nearby drillhole. The alternate method installed the casing to the bottom of the pile by coring. A 203 mm hole was made by the coring bit on the casing. Then the casing was tremie filled with grout and the #36 rebar was set. Next, four lengths of casing were withdrawn and the #25 bent bar was set in the top of the mini-pile.

Two production piles were compression tested and two test piles were tension tested. Compression tests were performed on piles similar to the pile shown in Figure 1. Total movements for Mini-pile No. 20 and Mini-pile No. 63 under an 890 kN compressive load was approximately 5.1 mm. The tension tests were performed on piles consisting of an uncased #36 rebar. The two tensile pile tests had total movements of 23.5 and 24.4 mm, respectively, under a load 418 kN. Figure 2 shows the results of the tests. Differences in the total movements of the compression and tension piles reflect differences in the axial stiffnesses of the piles. Figure 2 compares the load-deformation behavior of the mini-piles with the load deformation behavior of two driven, precast concrete piles installed when the shopping center was originally built. The mini-piles had similar load-deformation behavior over the full range of test loads.

The mini-piles were completed in 1998 and the strengthened foundation is performing satisfactorily. Load testing demonstrated that the mini-piles could be designed and installed to match the deformation behavior of an existing foundation system. Mini-piles with a discontinuous rebar core performed satisfactorily.

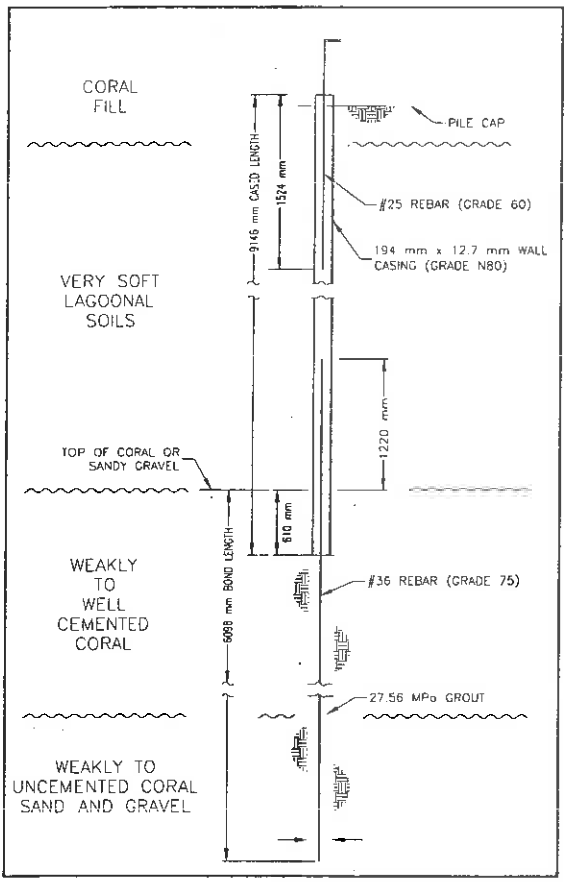

Figure 1. Shopping Center Mini-Pile

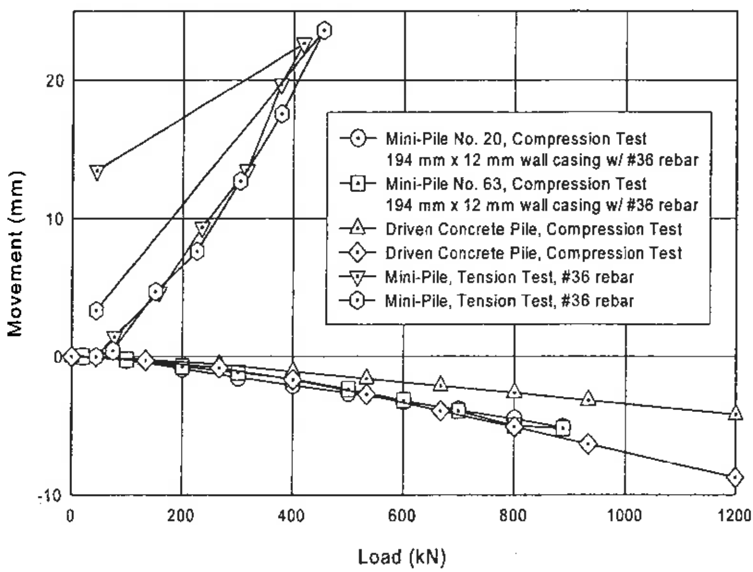

Figure 2. Shopping Center Load Test Results

Paumalu Road Repair

The Paumalu Access Road services a critical communication facility on the north shore of Oahu Island, Hawaii. Two fill sections along the roadway were considered susceptible to landslides. The owner of the facility wanted these areas treated to insure that a slide would not close the road. Schnabel Foundation Company proposed using a mini-pile and passive anchor system to stabilize a 43 m long and a 31 m long section of the roadway. Driven soldier beams or large diameter drilled shafts were not feasible due the presence of hard basalt boulders. Dames and Moore, the owner's geotechnical consultant, and the owner accepted the proposed method since the system could be installed from the existing roadway using compact drilling equipment while maintaining traffic and without cutting work benches in the slope in front of the wall.

Assuming the soil properties in Figure 3, limit equilibrium analyses indicated that the factor of safety (FS) for one potential slide area was 1.04 and the FS for the other area was 1.39. The owner required that the stabilization system raise the FS to 1.5. Additional design elements included: (1) neglecting the buttressing effect of the fill in front of the wall (cross-hatched area in Figure 3), (2) assuming that the critical failure surface is 4.1 m below the roadway at the wall, (3) applying a 14.4 kPa traffic surcharge to the roadway, (4) maintaining traffic during construction, (5) not disturbing the slope in front of the wall, (6) assuming the ground between the soldier beams is self-supporting since the fill in front of the wall may slide away in the future, and (7) using equipment capable of working on the sloping roadway.

Limit equilibrium methods determined the magnitude of the load required to support the fill with a FS of 1.5. Distributing this load to a rectangular apparent earth pressure diagram resulted in a pressure of 4.59H (kPa), where H is the height in meters. The mini-pile soldier beams and passive anchors were designed to resist this earth pressure. The piles were supported by the anchor at the top and the embedded portion of the pile below the failure surface. The required toe embedment was determined by summing moments around the anchor level. The resistance of the embedded portion of the piles was determined using relationships developed by Wang and Reese (1986). The FS against passive toe failure was greater than 3. The anchor force was determined by summing forces. A mini-pile spacing of 1.5 m center to center was selected. Resistance to soil flow between the piles was checked using a relationship presented by Wang and Reese (1986). A 168 mm diameter by 11 mm wall mini-pile casing in a 203 mm drillhole was selected to prevent soil flow between the piles and resist the bending moments resulting from the earth pressures. Mini-pile drillholes were 6.1 m deep, giving a minimum toe of 2.0 m. Passive anchors had a maximum design load of 150 kN, were spaced 3.0 m apart, and located at a 20 degree angle from horizontal. Pile toe embedment lengths were checked to make sure they could resist the vertical component of the anchor force. Any contribution of the grout to the flexural strength of the mini-pile was neglected.

Passive anchors were connected to the mini-pile soldier beams with a continuous, reinforced concrete cap beam. The cap beam was 610 mm high by 915 mm wide. The cap was designed as a horizontally loaded beam with a span equal to 3.0 m, the anchor spacing. Anchor heads were embedded in the beam and placed to limit torsional loading on the cap beam. Mini-piles were installed on alternate sides of the cap beam centerline. Staggering the piles, eliminated bending moments in the mini-piles as a result of the torsional load. Figure 3 shows a typical wall section.

Construction commenced in 1997 and was completed in 5 weeks. Stabilization work began with excavation for the cap beam along the edge of the roadway. Mini-piles were installed by drilling 203 mm holes using a track-mounted rotary drill equipped with a down-hole-hammer. Percussion drilling was necessary to penetrate the hard basalt boulders and rock. The steel mini-pile was set in the hole and the hole was tremie-grouted using a neat-cement grout.

After the mini-piles were installed, the passive anchors were installed. Creating a bench in front of the wall was not possible, so a drill guide was removed from a drill and hung from a crane out over the wall. This allowed the anchors to be drilled from the roadway surface. A 114 mm hole, 11 m long was drilled for each anchor. Then a neat-cement grout was tremied into the hole and a #32, Grade 60 rebar tendon was inserted into the grout filled hole. Four of the anchors were proof-tested to loads between 178 and 222 kN. The maximum movement during testing was 16 mm and the average maximum movement was 10 mm. The anchors were not tensioned. Passive anchors were used since the movements required to mobilize anchor load were small and casting the anchor head in the cap beam allowed the minimum size beam to be constructed.

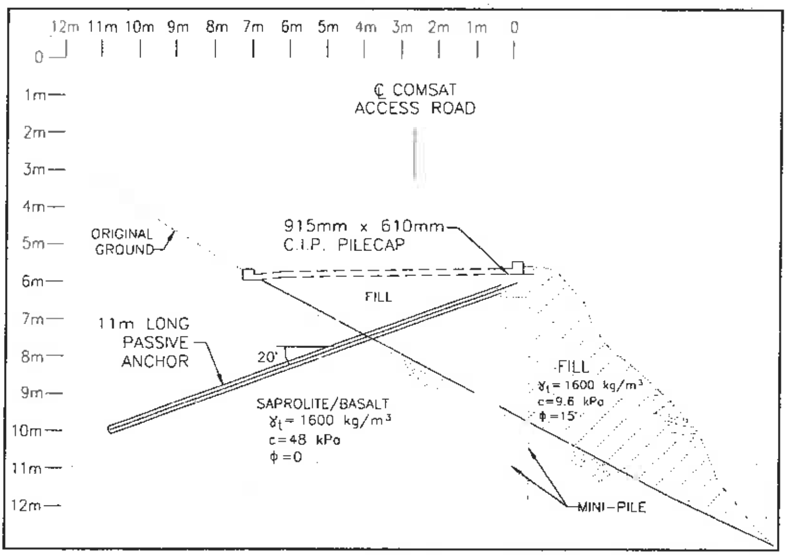

Figure 3. Paumalu Access Road Stabilization System

The stabilization system has performed satisfactorily for three years. This method of roadway stabilization is applicable where: (1) conventional soldier beams installation would be time-consuming and expensive because of difficult drilling conditions, (2) disturbance of the slope below the roadway is not desirable or allowed, and (3) the depth of the unstable soil is shallow, around 4.5 m.

Seismic Upgrade of an Existing Building

An existing industrial building in Emeryville, California needed new foundations to support shear walls required for upgrading the building to meet seismic requirements for multi-unit, residential buildings. Subsurface Consultants Inc., project geotechnical engineer, and Steven Tipping and Associates, project structural engineer, designed mini-pile supported pile caps for the shear walls. Mini-piles were selected since the work had to be completed inside the existing building with restricted overhead and lateral clearances.

The warehouse is located near the former shoreline of San Francisco Bay. A simplified soil profile and a schematic drawing of a typical mini-pile installed for the project are shown in Figure 4. Soils below the plastic silty clay are alluvial fans brought down from the Berkeley Hills. These deposits are comprise of interfingered lenses of silty and sandy clays, clayey sands, clayey gravels, silty sands and sandy gravels. Occasional lenses of soft clayey silt, sand and peat are present down to a depth of 5 m. The groundwater level varied from 2.4 to 6.7 m below the ground surface.

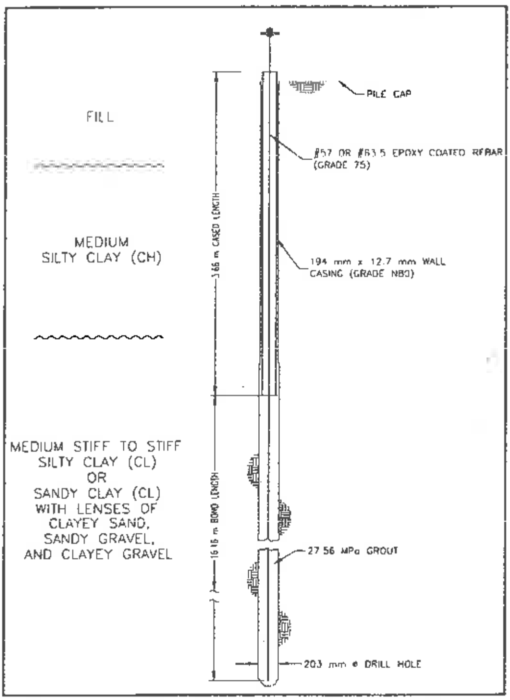

Figure 4. Soil Profile and Mini-Pile Details for an Industrial Building in Emeryville, California

New pile caps within the building were designed to resist maximum seismic forces of 5515 kN downward and 890 kN upward. Individual mini-piles, with allowable compression capacities between 712 and 890 kN and allowable tensile capacities between 534 and 667 kN were grouped as necessary to develop the design loads in the pile caps.

The cased portion of the pile was fabricated using two 1.83 m lengths which were threaded together during installation. The #57 or #63.5 epoxy-coated rebars were assembled from either 2.13 m or 3.05 m sections depending on overhead clearance. Rebar sections were connected using un-coated steel couplers. Couplers were protected from corrosion with heat-shrinkable sleeves.

Test holes installed adjacent to the building demonstrated that uncased drilling techniques could be used at the site. Production holes were drilled using continuous flight augers rotated by a compact, high torque rotary drill. The drill had an adjustable mast with a minimum drill height of 2.15 m.

After drilling the holes and removing the augers, the holes were tremie grouting using a neat-cement grout. Grouting continued until clean grout was observed flowing from the top of the hole. The casing was placed in the grouted hole and held at the proper elevation. Epoxy coated bar sections were fitted with centralizers and a regrout tube and lowered into the grout fill hole through the casing. Rebar sections were coupled together and a heat-shrinkable sleeve was installed around each coupler to protect them from corrosion. Completed mini-piles were 203 mm in diameter and 19.8 m long. Each mini-pile was regrouted approximately 24 hours after the initial grouting. Regrout pressures varied between 26 and 65 kPa with an average pressure of 43.5 kPa. Attempts were made to regrout 5 piles more than once. Two piles refused to take any grout, and three piles were regrouted a second time. One of the piles was regrouted a third time.

Tests piles were installed prior to working inside the building. Seventy-eight production mini-piles were installed and 20 production piles were tensioned tested. Figure 5 shows the results of a compression test on Test Pile 1, a tension test on Test Pile 2, and two tension tests performed on the same production pile, Mini-Pile 82. The initial test on Mini-Pile 82 was performed after one regrout and the second test was performed after three regrouts.

Test Piles 1 and 2 were 18.3 m long. They were similar to the pile shown in Figure 4 and they were fabricated using a #63.5 rebar. Both piles were regrouted once. Under a maximum compressive load of 1601 kN, Test Pile 1 moved a total of 10.6 mm and crept at a rate of 0.5 mm/decade (0.5 mm/log cycle of time) during a 10 minute load hold. Test Pile 2, the tension pile, moved a total of 21 mm under a 1334 kN load and had a creep rate of 0.6 mm/decade of time. Total movements of Test Pile 1 (compression test) are approximately half of the total movements of Test Pile 2 (tension pile) at the same load. This reflects the differences in the axial stiffnesses of compression and tension piles. When a compressive load is applied to a mini-pile the load is carried by the steel and the grout. Test Pile 1 had a transformed grout and steel area of 6452 mm^2. When the tensile load was applied to Test Pile 2, the load was only carried by the steel bar since the grout cracks. The steel area of Test Pile 2 was 3167 mm^2. Since the bar area was about half the composite area, the tensile movements should be about twice the compressive movements.

Figure 5 also shows the results of the tension tests performed on Mini-Pile 82. The initial test was performed after one regrout and the final test was performed after three regrouts. Test results on Mini-Pile 82 show that regrouting increased the capacity of the pile and may have increased its axial stiffness.

Nineteen other production piles, one pile per cap, were tension tested on this project. Tension tests are relatively economical to run and the pre-production tests demonstrated that similar piles would fail under a tension load before they would failure under a compressive load.

Several observations were made at the conclusion of the project: (1) there was no clear trend or relationship between the volume of regrout or regrout pressure and capacity, (2) load-carrying capacity increased as the number of phases of regrouting increased, and (3) mini-pile grout and rebar behave as a composite section under compression loading.

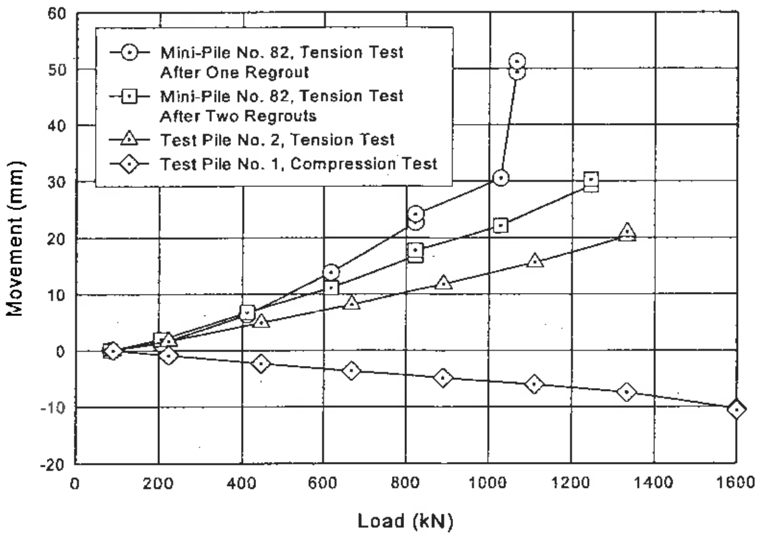

Figure 5. Mini-Pile Test Results at Industrial Building, Emeryville, California

Tension Test on an Instrumented Mini-Pile

An instrumented mini-pile was installed and tested at the project described in the proceeding section of this paper. At the test pile location, the existing building was constructed on 1 m of fill. The fill was underlain by a 1.7 m thick strata of medium silty clay (CH). Below the plastic clay, was an 8.2 m thick deposit of stiff silty clay (CL). The stiff silty clay had occasional lenses of clayey sand, clayey gravel and silty sand. Underlying the stiff clay was a dense sandy gravel (GM). Groundwater was located 6.1 m below the ground surface.

The test pile was 16.1 m long with 15.2 m below the ground surface and 0.9 m extending above the ground. Because many seismic retrofit projects limit allowable movements, the test pile was designed with a larger steel area than that required for load-carrying capacity. For example, the California Department of Transportation frequently limits the maximum total displacement during a load test to 12.7 mm. The test pile had a #57, Grade 75 rebar over its full length. A 168 mm diameter by 22 mm wall casing was installed over the upper 13.1 m of the pile. The upper portion of the pile had a steel area of 12,647 mm^2, about 5 times the area of the #57 bar. Geokon Model VK-4100 weldable, vibrating wire strain gages were applied to the casing and the #57 rebar. The gages were located at 1.52 or 1.83 m intervals.

A 254 mm hole was augered for the test pile. Groundwater encountered during augering mixed with native soils to create a soil slurry during the drilling of the lower portion of the hole. The drillhole did not collapse during augering. After the augers were removed from the hole, a 19 mm tremie grout tube was inserted to the bottom of the hole. The hole was tremie grouted using a neat-cement grout. A substantial amount of grout was pumped to remove soil spoils and slurry from the hole. Grouting continued until clean grout was observed flowing from the top of the drillhole. The steel casing, with threaded joints designed to develop high tensile strength, was inserted into the drillhole in 3.05 m sections. After setting the casing, the #57 bar and a 19 mm regrout tube were installed. The regrout tube allowed the lower 3.0 m of the mini-pile to be regrouted.

The mini-pile was regrouted, 12 and 18 hours after the #57 bar was set. The first regrout took 0.55 m^3 of grout at a grout pressure of 2.76 MPa. No grout was injected when the second regrout was attempted. The mini-pile was tension tested 6 days after regrouting.

Figures 6 and 7 show the load-deformation results of the test. Figure 6 shows the total movement at the maximum test load, 1779 kN, was 21 mm and residual movement was 10.7 mm. Movements at constant load represent the creep movements of the pile during load holds. Figure 7 shows that the creep rate exceeded 2 mm per decade, the common creep failure rate, at the 1779 kN load. The plot of creep rate verses load indicates that the creep rate was increasing approximately linearly up to a load of 1560 kN.

Figure 8 shows the measured strains in the bar and the casing. At the same location, the strains in the casing were slightly larger than the strains in the bar. There is no clear explanation for the differences in strain readings. These differences made determination of load from strain readings along the composite pile unreliable. However, applied load at the top of the pile was known. Applied load was measured using a calibrated jack and a calibrated load cell. Load measurements with the jack and with the load cell were within 2 % of each other at upper load increments. Load over the lower portion of the pile could be calculated from the measured strains in the bar below the bottom of the casing.

Figure 9 shows the load distribution computed assuming the applied load at the top of the pile and the load computed from the strains in the bar below the bottom of the casing. Figure 9 shows that the pile transferred load along its complete length at low loads. Load is transferred to the bottom of the pile since it is axially very stiff relative to the ground. Table 1 gives the load transfer rates computed for the cased portion and the bar portion of the pile. These rates were determined from the slopes of the load distribution curves in Figure 9. Table 1 and Figure 9 show the load-carrying capacity along the cased portion of the pile was fully mobilized at a test load of about 1000 kN. At loads above 1000 kN, additional load-carrying capacity was developed along the lower portion of the pile. At the maximum test load, 1779 kN, the load transfer rate for the cased section of the pile was 63 kN/m (bond stress 79 kPa) and 313 kN/m for the bar section of the pile. The cased portion of the pile was in a stiff silty clay with occasional lenses of clayey sand, clayey gravel and silty sand, and the bar section of the pile was in a dense sandy gravel. These load transfer rates and bond stress are considered reasonable for these soils.

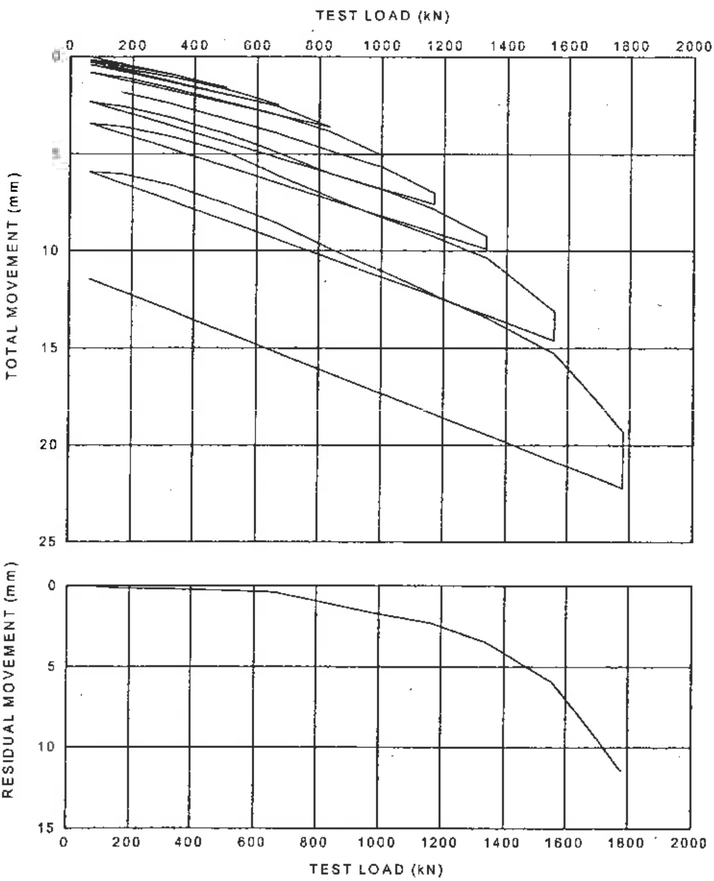

Figure 6. Total and Residual Movement Curves for High Capacity Test Pile

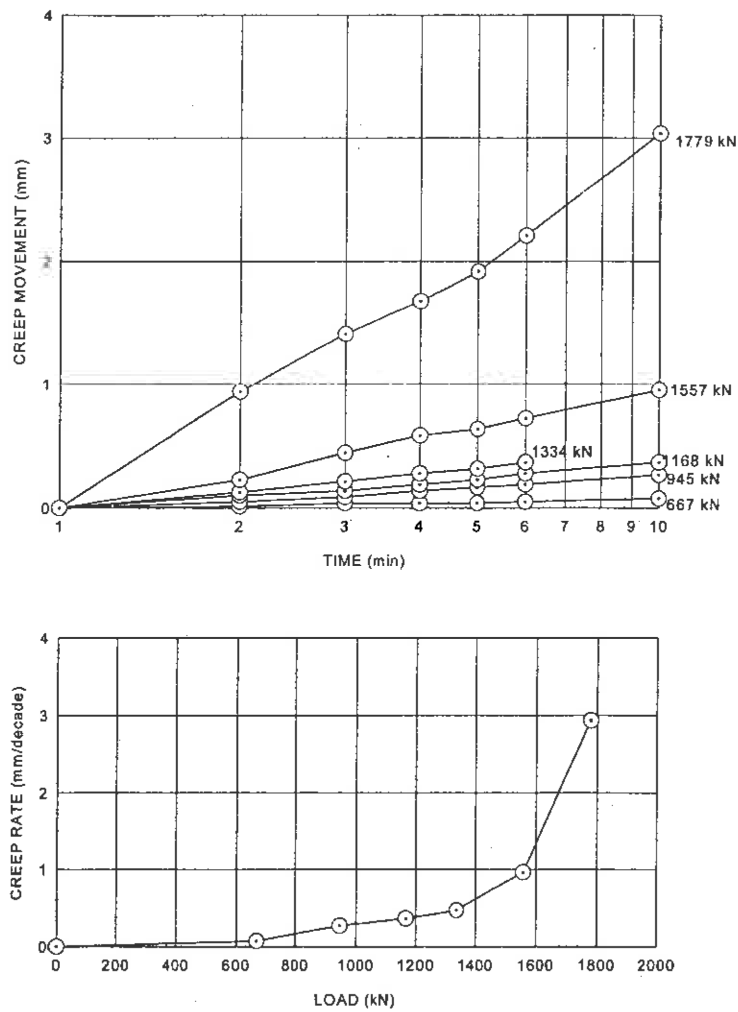

Figure 7. Creep Movement and Creep Rate Curves for High Capacity Pile

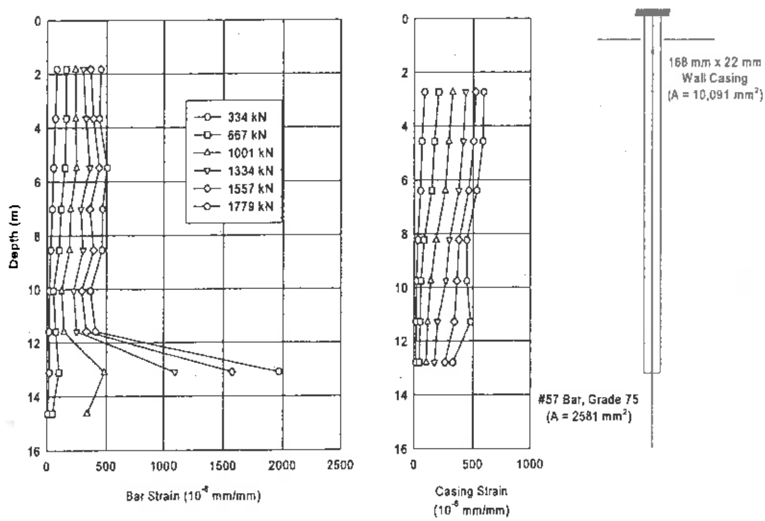

Figure 8. Bar and Casing Strains for High Capacity Pile

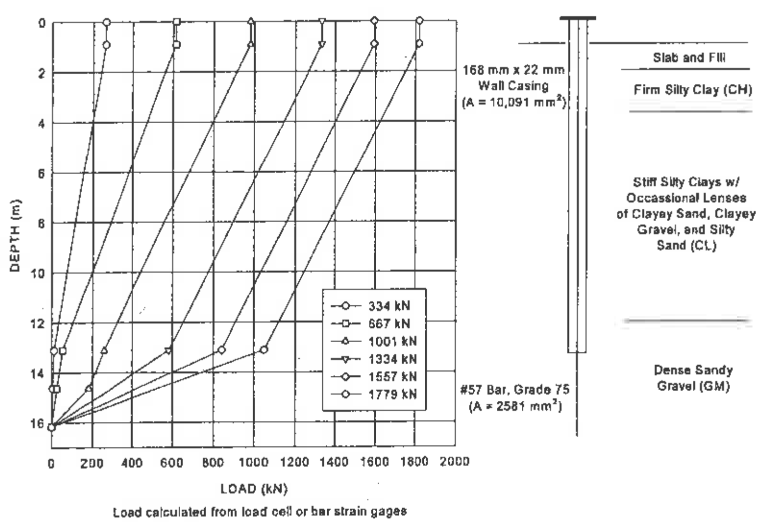

Figure 9. Load Distribution Curves for High Capacity Pile

Table 1. Computer Load Transfer Rates for Cased and Bar Section of the Mini-Pile

Desired Test Load (kN)	Measured Test Load (kN)	Computer Load Transfer Rates (kN/m)	
		Cased Section	Bar Section
167	100	7.88	1.17
334	262	20.57	3.50
500	424	32.83	7.15
667	614	45.81	16.49
834	800	55.00	38.66
1001	981	59.38	76.60
1168	1167	61.57	123.72
1334	1334	61.57	173.91
1557	1593	61.57	251.10
1779	1819	63.03	312.96

Figure 9 assumes that the load distribution curves for the cased section decrease linearly between the top of the pile and the bottom of the casing. The reasonableness of the load distribution curves in Figure 9 was checked by comparing the elastic movements of the mini-pile computed from the load distribution curves shown in Figure 9 with the measured elastic movements during the tests. Measured elastic movements are determined from Figure 6. They are the total movements minus the residual movements. Figure 10 shows that the elastic movement curves are similar. Therefore, the load distribution curves in Figure 9 are judged to be reasonable.

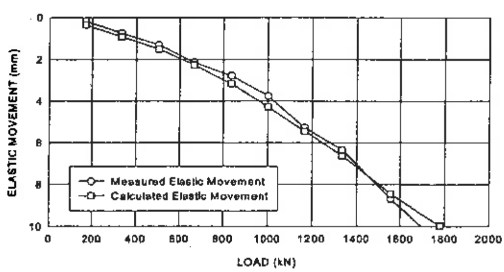

Figure 10. Measured and Computed Elastic Movements for High Capacity Pile

Observations

1. Mini-piles were successfully used to strengthen an existing foundation, as part of a slope stabilization system, and in a seismic upgrading of abuilding.
2. Mini-piles can be installed using specialized equipment within existing structures.
3. Mini-piles can be installed in occupied space.
4. Mini-piles can be installed along existing roadways while maintaining traffic.
5. Mini-piles can be designed to match the load-deflection behavior of existing foundation elements.
6. Mini-piles can be installed in difficult drilling conditions where conventional pile installation methods may be prohibitively expensive.
7. Cased mini-piles with a discontinuous bar core successfully carried compression test loads of 890 kN.
8. Compressive load in a mini-pile is carried by the grout, steel core and casing.
9. Tensile load in a mini-pile is carried by the steel core and the casing.
10. Load-carrying capacity can be evaluated using tensile tests. Mini-piles are less axially stiff in tension than in compression. They will move more under tensile load than under a compressive load.
11. Regrouting can be used to increase the load-carrying capacity of a mini-pile, and regrouting may be used after the pile has been tested.
12. Mini-piles with high load-carrying capacities are feasible. An instrumented pile with an ultimate tensile capacity of 1779 kN was installed and tested. The pile moved 21 mm under the maximum test load. At a load of 1557 kN, the creep rate was less than 2 mm/decade and the total movement was approximately 12.5 mm.

Acknowledgments

Inspectors from Dames and Moore and Subsurface Consultants, Inc., and field crews from Schnabel Foundation Company were responsible for constructing these installations in accordance with the design requirements and in a workmanlike manner. Support for the instrumented test pile was provided by Schnabel Foundation Company. A special thank you to Ted Geilen of Schnabel Foundation Company for making the high-capacity mini-pile test a success, and we want to acknowledge Ron Reindl of Subsurface Consultants, Inc. for his assistance with the test data for the production piles installed on the seismic retrofit project. Mary Bradshaw and Matt Niermann's preparation of the figures in acknowledged.

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