Micropiles for Seismic Retrofitting of Highway Interchange Foundation

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ABSTRACT

Micropiles are being installed for seismic retrofitting of the 580/980/24 freeway interchange in Oakland, California, U.S.A. This seismic retrofit project, costing approximately US $50 million and consisting of 156 bents, 226 columns and 56 hinges, will have almost 685 micropiles installed at completion. The retrofit strategy for this four level interchange provided for increasing the stiffness of the entire system, including the existing foundation. Specifically, the function of micropiles is to enhance the pull out capacity of the existing foundation system, and minimize the vertical deflection of the structures in a seismic event. For this project, micropiles were required to maintain the design loading at a maximum deflection of 0.5" (1.27 cm) deflection. Because the existing structures varied in height and loading, micropiles of different pull out capacities were designed. Pre-production micropiles were installed and tested in representative site conditions. Several different strategies were investigated at pre-production stage to achieve adequate micropile performance. These pre-production pile-load test results are used to study the relationship between micropile pull out capacity and micropile diameter, rebar size, casing diameter, and casing thickness.

1. INTRODUCTION

Micropiles are being increasingly utilized in foundation rehabilitation projects. These drilled, cast-in-place, small diameter grouted piles are especially suited for foundations with difficult access, restricted clearance, and poor ground conditions, where minimal disturbance to the existing structure is required. Consequently, micropiles were specified for seismic retrofitting of the 580/980/24 freeway interchange in Oakland, California, U.S.A. This U.S. $50 million seismic retrofit project of the interchange was started in 1997 to retrofit the existing substructure against the effects of a seismic event. Foundations were to be retrofitted with a combination of CIDH piles and micropiles. Micropiles were used due to space limitations and in an attempt to limit costs by reducing the size of the retrofitted footings. The retrofit strategy for this four level interchange provided for increasing the moment capacity of the foundation. Specifically, the micropiles are expected to enhance the pull out and compressive capacity of the existing foundation system, and minimize the vertical deflection of the structures in a seismic event.

As part of the design process, the contractor installed pre-production piles to confirm the design assumptions. Several different strategies were investigated at pre-production stage to achieve adequate micropile performance. In this paper we use the pre-production pile-load test results to study the relationship between micropile pull out capacity and micropile diameter, rebar size, casing diameter, and casing thickness. We also discuss some of the problems with the performance of the micropiles in the soft soils, and design changes implemented during construction.

2. PROJECT DESCRIPTION

The 580/980/24 interchange is a 4 level freeway interchange located in Oakland, CA, near the Hayward and San Andreas faults. Since the 1989 Loma Prieta earthquake which resulted in the collapse of

Cypress viaduct, the 580/980/24 freeway has become a significant travel route for commuters. In addition to the three freeways meeting at the interchange, a 4 track mainline for the Bay Area Rapid Transit (BART) system crosses the site. As seen from Figure 1, the interchange is located in a crowded urban area and consists of elevated ramps spanning over the freeways, BART, and several city streets. The BART rail system runs in the north-south direction parallel to route 24/980 and has a major stop at MacArthur station where all main lines meet just north of route 580. Further, the site is approximately 4 miles from the Hayward Fault, and 10 miles from the San Andreas Fault.

The interchange consists of 9 elevated connector ramps of various lengths and 156-bents, including T-bents, C-bents, multi-column bents, outriggers and double-deck bents. Construction of the original structures was completed in the late 1960s. Consequently, a number of structural components and connections are vulnerable to seismic events. Specifically, the foundations consist of under reinforced tapered piles. Given the nature of access to the existing foundation and space limitations, the needs to maintain traffic during construction and minimize effect on the BART tracks, micropiles were deemed to be suitable for foundation retrofit.

Accordingly, the project plan called for 685 micropiles to be installed in existing footings. Further, the project specifications called for the piles to have a maximum deflection of 0.5" at the design load. Design loads were variously specified at 300 kips, 350 kips and 400 kips. The project plans included Caltrans designed micropiles as well as provision for contractor proposed pre-approved alternate pile design. The project documents called for the piles to be drilled and then grouted before or after placement of steel casing. Pile lengths were to be determined by the contractor. Additionally, for the contractor proposed alternative micropiles the lengths, diameters and reinforcing sizes were to be determined by the contractor. Based on the micropile details contained in the contract documents and the subsurface investigation reports, the contractor was able to make assumptions as to the relative capacity of the soil. The contractor bid the project based on these details.

The design specifications were to be evaluated through a rigorous testing plan that included the installation of 9 pre-production piles, and the testing of at least 2 production piles per footing. Prior to the start of contract work, the contractor installed and tested 3 micropiles without post grouting. The test results for these showed that the soil strength was actually less than implied by the contract documents. These piles were then post-grouted with 24 sacks of cement grout, which lead to improved performance. These modified piles reached capacity, but did not meet the deflection criteria. The contractor then proceeded to install the pre-production micropiles required per the contract. In these pre-production piles, larger reinforcing bars, casing thickness and diameters were used. These piles were also post-grouted with 24 sacks of cement grout. With the post-grouting, the micropiles exhibited improved performance and were able to maintain the design loads with maximum allowable deflection in most cases.

3. SUBSURFACE CHARACTERISTICS

The project area is located near the east shore of San Francisco Bay, California, USA, in the Coast Ranges geomorphic province. This province is characterized by northwest-southwest trending mountains and valleys, and is dominated by similarly trending faults and other structures. San Francisco Bay and Santa Clara Valley to the southeast occupy a structural depression bounded by the Santa Cruz Mountains on the southwest and the Diablo Range and Berkeley Hills on the northeast.

The project site is on the western margin of an alluvial plain sloping down westerly from Berkeley Hills. The plain comprises of alluvial fan deposits from the hills consisting of coarse-grained sandy deposits, fine-grained silt and clay deposits, and shallow water deposits. According to Radbruch (1957) Bedrock is identified as Franciscan complex and is estimated to be at depth in excess of 400 feet (120 m).

Project area is located within a well-known seismically active region. Several large, historically active faults are capable of causing severe shaking at the project site. The Hayward fault, located along the southwestern foot of the Berkeley Hills, is the nearest major active fault, and is therefore considered to be the controlling fault. The Hayward fault is an active branch of the San Andreas fault system and extends northward about 62 miles along the western base of the East Bay foothills to San Pablo Bay, and passes about 4 miles east of the project area. The Hayward fault has generated two historic earthquakes of Richter magnitude ~7.0, in 1836 and 1868, respectively. The estimated slip rate of Hayward fault is 9mm/year and it has the potential of generating up to 7.4 magnitude earthquake (Slemmons, 1982, USGS, 1990).

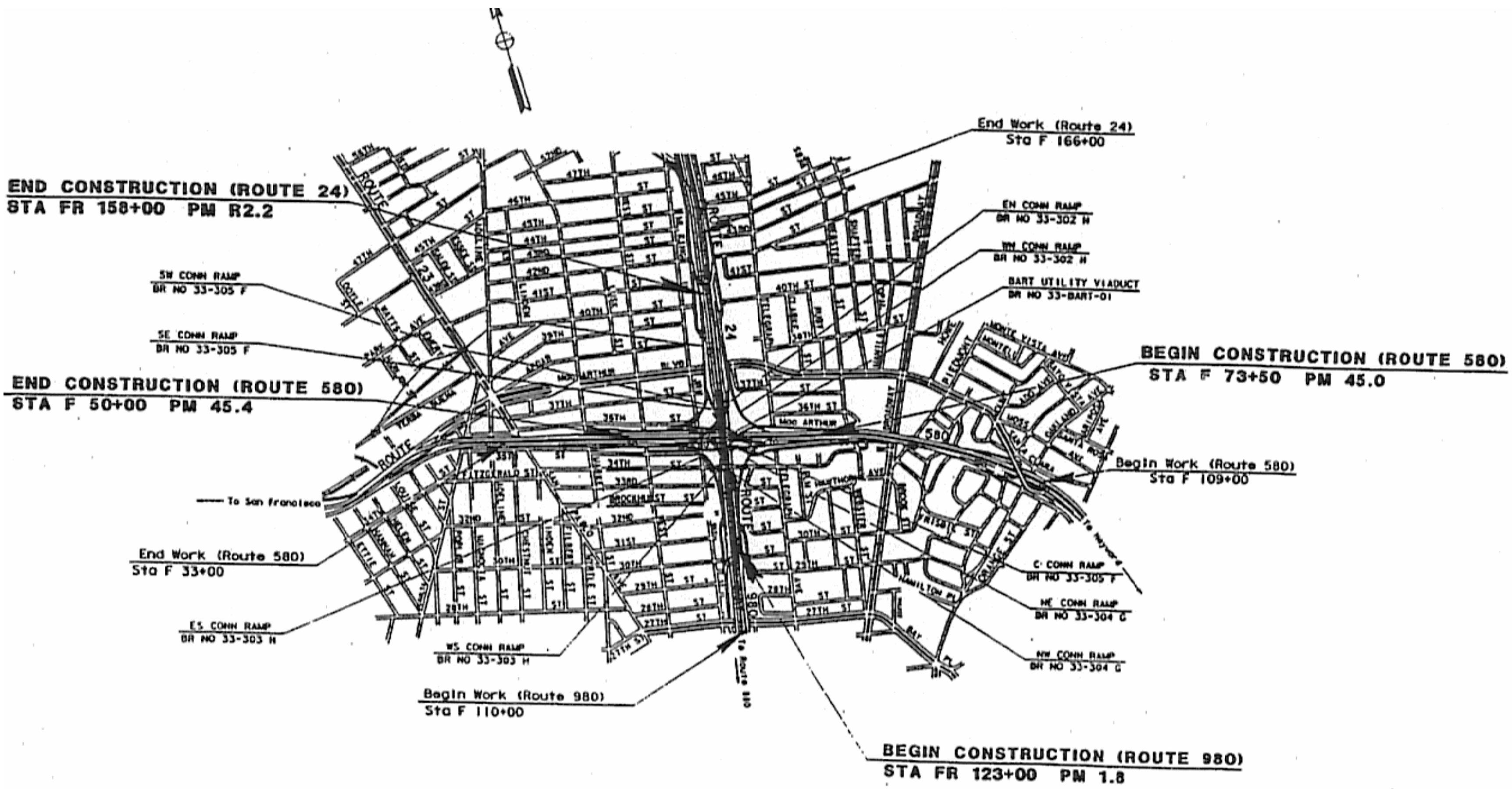

Figure 1. Project Loaction

For seismic ground response analysis, the computer program SHAKE was used to simulate wave propagation from rock motion to the ground surface. A maximum surface spectral acceleration of 1.2g was used for the project design based upon a rock outcrop spectrum for a moment magnitude 7.25 Hayward fault earthquake. The performance criteria for the seismic retrofit was set at non-collapse, limited service, and significant damage.

Subsurface conditions at the site consist of layers of medium to dense silty clay of medium to high plasticity, interspersed with layers of fine to coarse medium dense to dense silty sand and gravel, extending to a depth of approximately 400 feet (120 m). Groundwater surface at this site varies from elevation 20 to 60 feet (6 m to 18 m). Typical subsurface profiles along the highways 580/980/24 separation are presented in Figures 2 and 3.

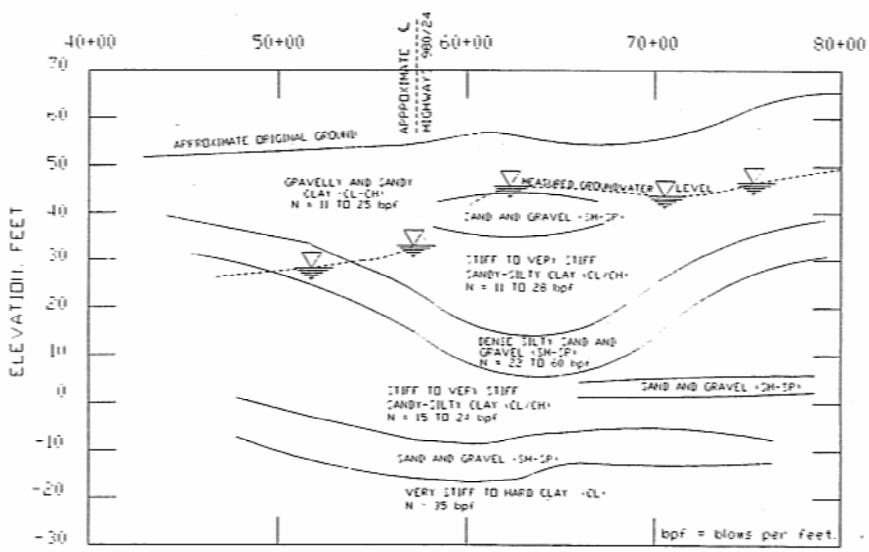

Figure 2. Typical soil profile at the site along highway 580.

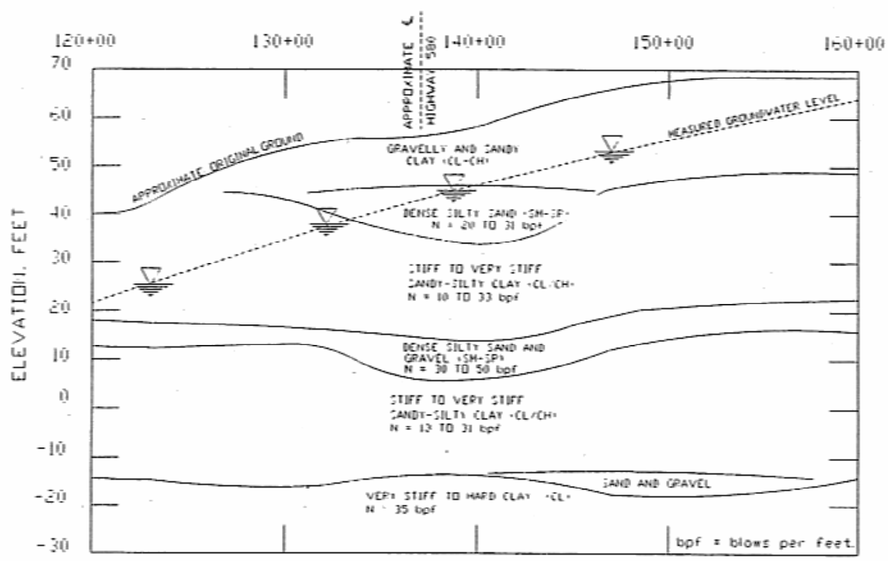

Figure 3. Typical soil profile at the site along highway 980/24.

Further, Figure 4 shows a selection of the penetration values obtained from the Standard Penetration test (SPT). The SPT values typically range between 10 and 20 which may be described as stiff clays and silts with an unconfined compressive strength ranging from 15 to 30 psi (100 to 200 kN/m^2).

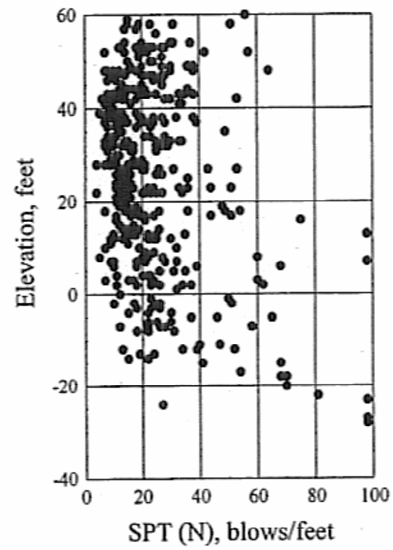

Figure 4. Typical standard penetration test blow counts (N) at the site.

4. MICROPILE INSTALLATION METHOD

Micropiles used in this project may be classified as Type D based upon the grouting method used for installation (see Schaefer et. al. 1997). A schematic of a typical installed micropile is shown in Figure 5.

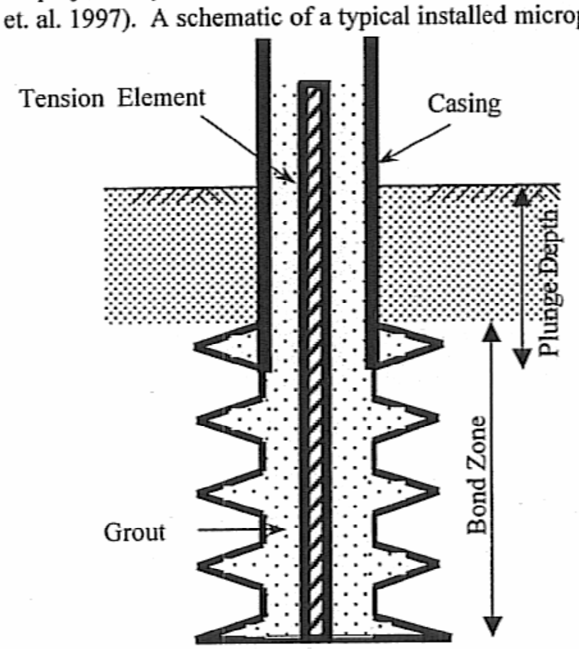

Figure 5. Schematic diagram of installed micorpile.

As the first stage of micropile installation, the steel pile casing is drilled to the full pile depth. Water is pumped through the open casing tip and returns around the casing perimeter (rotary external flush drilling) as the casing is spun into the ground. Thus the drill cuttings are returned to the ground surface by annulus flush as the drilling proceeds. To assist casing penetration into the soil, the pile casing is tipped with cutting teeth. The drilled shaft diameter may be enlarged either by utilizing a larger casing, or by increasing the amount of time the water is allowed to flush the cutting from length of the pile shaft. In order to verify the shaft-hole is clear of cuttings, pile depth is measured while placing the grout tremie tube and the reinforcing bar. Any drill cuttings that have settled at the shaft-hole bottom are removed by re-flushing the shaft-hole.

Once the shaft-hole has been advanced and cleaned to full-depth, the tension element (reinforcing bar) with attached post grout tubes is placed along with centralizers. The casing is then tremie filled with neat cement grout from the tip to the top of the pile. Subsequently, the casing is removed in sections from the tip of the pile to top of the bond zone. During casing removal, the grout is pressurized to about 70-100 psi, filling the micropile bond zone with pressurized grout, in order to enhance pile bonding with the adjacent soil. The casing is then plunged back into the pressure grouted bond zone to a predetermined elevation. The installed pile is typically post-grouted after 24 hrs by pumping neat cement grout through the pre-installed post grout tubes. A post grout pressure of 600-800 psi is first used to fracture the initial grout. Subsequently, a post grout pressure of ~200 psi is used to inject the grout into the surrounding soil. The grout used in the micropiles is expected to have a 7-day strength of 500 psi. The grout specific gravity is specified at 1.89±0.02, which is obtained by mixing one 94 pound bag of cement with 5 gallons of water. Thus the grout water-cement ratio works out to be 0.44 by weight.

5. MICROPILE PULLOUT TEST RESULTS

To confirm and refine the design assumptions 20 pullout tests were conducted on pre-production piles. During the pullout test the complete load-displacement are recorded as the pile is loaded to failure. A typical load-displacement curve is shown in Figure 6. The following parameters were varied during the pre-production pile tests: pile length, casing size, tension element size, and degree of post-grouting.

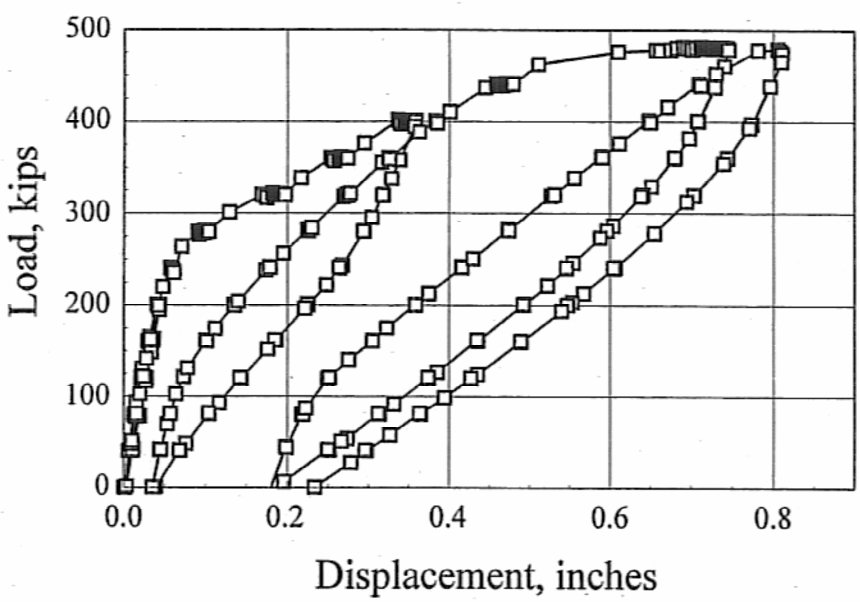

Figure 6. Sample of load-displacement test result for pre-production micropiles.

For the purposes of analyzing the pullout test results we consider two-design criteria, one based on ultimate pile capacity and the second based upon pile displacement. The ultimate pile capacity P_u may be obtained as follows

$$P_u = \pi D L C_u \tag{1}$$

where C_u is the ultimate pile-soil interface shear strength, L is the pile bond length and D is the pile diameter. The pile bond length is taken to be the drill depth less some free length over which pile-soil interaction is assumed to be zero. The free length may vary from 5 to 15 feet, however, in general it is not easily specified. In addition, the pile diameter is considered to be 3 inch in excess of casing diameter considering that the pressurized grout seeps into the adjacent soil. Based upon pile displacement criterion, the pile capacity is taken to be the pullout force required for achieving 0.5-inch pile deflection.

Effect of pile length: For the purposes of studying the effect of pile length on observed pile behavior, we consider piles is three groupings of drill depths 53-54 feet, 58-59 feet and 64 feet. Table 1 gives the ultimate capacity and the pile capacity at 0.5 inch pile displacement.

Table1. Effect of pile length on micropile capacity.

Pile Length Feet	Casing Depth Feet	Ultimate Pile Capacity Kips	Pile Capacity @ 0.5 inch displacement Kips	Casing thickness inch	Reinforcing bar diameter inch
53-54	24	348	311	0.5	2.25
58-59	26	370	335	0.5	2.25
64	29	477	404	0.75	2.50

The pile capacities shown in Table 1 are averages of three pile tests each; all of which have been post-grouted using 24 bags of cement. Further the casing sizes and tension elements of the piles in 53-54 feet and 58-59 feet groupings are identical at 7-inch diameter, 0.5-inch wall thickness, and #18 bar size (2.25 inch diameter). The casing size and tension elements of the piles in 64 feet groupings are 7-inch diameter, 0.75-inch wall thickness, and #20 (2.5 inch diameter) bar size. The casing depths for the three groupings are, however, different. The casing depth is expected to have a greater influence on the pile capacity at 0.5-inch displacement than the ultimate pile capacity. Interestingly the pile capacities of 53-54 feet piles and 58-59 feet piles do not differ substantially, while that of 64 feet piles are substantially higher. It is believed that the increase in pile capacity is primarily from larger casing and reinforcement bar. This conclusion is substantiated by the nearly identical pile capacities obtained for two piles with identical casing of 9-inch diameter and 0.5-inch wall thickness, identical post-grouting of 24 bags but with drill and casing depths of 69 and 29 feet, and 74 and 34 feet, respectively. Apparently, the pile length has little effect when other parameters are equal.

Effect of casing diameter: The effect of casing diameter may be evaluated by considering the pile-soil interface shear strength C_u developed during pullout test. Using Eq. 1 and the measured ultimate pile capacities P_u, the pile-soil interface shear strength C_u may be estimated. Analysis of the measured pullout capacities is shown in Table 2. In this analysis, a free length of 10 feet is assumed. The average values of C_u are based on the number of tests shown in the table.

Table 2. Effect of casing diameter on ultimate pile-soil interface shear strength.

Casing Diameter inch	Number of Tests	Post-grout Bags of cement	Average C_u psi
7	3	0	16.4
7	11	24	21.9
9.5	2	24	17.4

As expected, the post-grouting increases the pile-soil interface shear strength. However, the increase is substantially less for 9.5-inch casing than the 7-inch casing even though same amount of grout is used. Also, the average C_u of post-grouted 9.5-inch pile is considerably less than the identically post-grouted 7-inch pile. It is believed that the flow of grout out of a 9.5-inch diameter pile would be different than the 7-inch pile, consequently the grouted pile-soil interface properties of the two pile types would be different.

Effect of post-grouting: Although the mechanism by which the post-grout improves micropile capacity is not well understood. Nevertheless, the post-grout is expected to improve the shear strength of the pile-soil interface. The effect of post-grout upon pile-soil interface strength may be evaluated by comparing C_u calculated from measured P_u using Eq. 1. Analysis of measured data yields an average C_u=16.4 psi without post-grout and C_u=20.6 psi with post-grout using 24 bags of cement. Thus with this degree of post-grouting a 25% improvement in pile-soil interface property is obtained.

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