**UPLIFT MICROPILE LOAD TRANSFER IN UNSATURATED MISSOULA FLOOD DEPOSITS**

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Despite the wide use of micropiles for rehabilitating and retrofitting existing structures, little is known regarding the load-transfer distribution of grouted micropiles in various soil deposits. The rehabilitation of a concrete-frame warehouse in Portland, Oregon, required provisions for the seismic restraint of a new shear wall; the installation of two micropiles was specified to meet project requirements. The micropiles were uncased, 19-m (63-ft) long, 200-mm (8-in) nominal diameter, and reinforced full-length with a 64-mm (No. 20) diameter hardened steel threadbar. Due to space limitations, a sacrificial, instrumented micropile was installed near the two production piles to verify design assumptions. This paper describes the geologic setting of the project within the unsaturated Missoula Flood deposits, instrumentation, load-displacement behavior, load transfer, and Beta-coefficients for the sacrificial micropile.

**INTRODUCTION**

Micropiles are increasingly used to support new and existing structures, and are particularly useful when seismically upgrading existing buildings. Cadden et al (2004) attribute the growing popularity of micropiles to advances in drilling equipment, techniques, and improvements in the fundamental understanding of micropile behavior. The proliferation of micropile use may also be ascribed to the ability to generate foundation support in low headroom, confined, and limited access areas. The role of micropile foundations installed for seismic retrofits is to improve the uplift and compressive resistance of existing foundations within tolerable displacements (Misra et al., 2004).

Due to contractor-specific equipment and drilling methods, verification of design assumptions is preferably performed using instrumented loading tests. For example, it is common practice to neglect the contribution of end-bearing resistance in some regions of the United States. Stuedlein et al. (2008) showed that micropiles loaded in compression can generate significant end bearing resistance, and that the load-displacement behavior of micropiles loaded in compression is significantly stiffer than those loaded in uplift. For those foundations that are required to support both uplift and compressive loading, it is usually more economical to verify capacity through an uplift loading test. It is commonly assumed that if the required uplift resistance is achieved within tolerable displacements, then the required compressive resistance will be achieved at the same or smaller displacements.

This paper summarizes the site conditions, field testing, analyses, and results of a load test conducted to support design of micropile foundations specified as part of a seismic retrofit. An instrumented uplift load test was performed on a 19-m (63-ft) long, 200-mm (8-in) diameter micropile. The sacrificial pile was installed and tested to confirm the design capacity of two production micropiles needed to support each end of a shear wall being added to a warehouse undergoing major renovations and seismic upgrades in Portland, Oregon. Strain gages were installed along the length of the test pile to evaluate the micropile load transfer. This paper provides the project background, geologic conditions, load test setup and instrumentation used to determine the behavior of the production micropiles. The load-displacement, load transfer, and estimated Beta-coefficients are provided. These results emphasize the importance and role of load tests in the effective
determination and understanding of micropile uplift resistance.

**PROJECT DESCRIPTION**

A 1920s era reinforced concrete warehouse in Portland, Oregon, required foundations to support a new shear wall as part of a seismic upgrade (Figure 1). The project structural engineer required that foundation elements at each end of the shear wall support 1,020 KN (230 kips) in compression and 670 KN (150 kips) in tension. The structural engineer also required that foundation movement be less than 20 mm (0.75 in).

![Figure 1. Boxlift Building.](image-url)

The magnitude of loads and property line restrictions prevented supporting the 10 m (33 ft) long shear wall on a shallow foundation. Micropiles were selected because of their ability to provide high compressive and tensile capacity and because they could be constructed inside the building with less than 3.5 m (11.5 ft) headroom and within 250 mm (10 in) of existing concrete columns.

Space restrictions inside the building prevented performing either proof or verification loading tests on the two production micropiles. As an alternative, the contractor proposed to construct a sacrificial pile outside the building and load test it to at least 200 percent of the maximum design load (2,040 KN [460 kip]). Strain gages were installed in the test pile as backup so that, if needed, rational modifications to the pile design could be made without needing to justify the changes with an expensive, second load test.

**GEOLOGY AND SUBSURFACE CONDITIONS**

The project site is located in the northern Willamette Valley, within the Portland Basin. Catastrophic flood deposits resulting from late Pleistocene-aged Missoula Floods described by Bretz et al. (1956) cover much of the Portland Basin, including the project site. The floods occurred after repeated failure of ice dams located on the Clark Fork River in northwestern Montana. The floods released approximately 2,100 km$^3$ (500 mi$^3$) of water during each event, flooding portions of eastern Washington, the Columbia Gorge and the Willamette Valley (Bretz et al. 1956; Allen et al. 1986). The flooding occurred at least 40 times during the Pleistocene (16,000 to 12,000 years ago), depositing boulders, cobbles, gravels, sands and silts.

The results of a piezocone penetration test (CPTu) performed near the test pile are presented in Figure 2. The soil profile consists of about 20 m (65 ft) of fine-grained Missoula flood deposits underlain by much older and much stronger gravel deposits.

In the upper 11 m (36 ft) of the site, the fine-grained flood deposits consist primarily of loose to medium dense, unsaturated, interbedded silts and sandy silts of low to moderate plasticity (Figure 3). From 11 m (36 ft) to 20 m (65 ft), the flood deposits are mostly medium dense to dense, unsaturated, fine- to medium-grained silty sands and sandy silts. Groundwater is at least 30 m (100 ft) below the ground surface.

**PILE INSTALLATION**

The test pile was drilled open-hole without casing using a Beretta T46 drilling rig equipped with a 200-mm (8-in) diameter, three-wing drag bit. Air flushing was used to remove soil cuttings.

The pile was reinforced with a 64-mm (No. 20) diameter threaded bar with 1,040 MPa (150 ksi) yield strength. The thread bars were cut in 7.5 m (25 ft) segments and coupled during installation. A 12-mm (0.5-in) diameter tremie tube was attached full length to the bar. Centralizers were spaced at 3 m (10 ft) intervals. Grouting was carried out using Type I/II Portland cement with no additives. Project specifications called for a
loading test setup, methods, and instrumentation

The micropile load test setup is shown in Figure 4. Five vibrating wire sister bar strain gauges from RST Instruments Ltd. were used to observe the load transfer in the test micropile. The published accuracy and resolution of the gauges were 6 με and 0.4 με, respectively. The measurable strain for the gages ranged from 1000 to 4000 με. Each gage was pre-stressed at the factory at about 2500 με. The strain gauges were attached to the central thread bar as it was lowered into the drill hole, and were installed at depths of 18.5 m (61 ft), 15.2 m (50 ft), and 11.3 m (37 ft) below the ground surface. In addition, two gages were installed 180 degrees opposite each other at 0.9 m (3 ft) below ground surface. This near-surface gage pair was installed as a check on the applied load as well as to determine the secant modulus of the constructed pile. It is assumed that the presence of the 12.5 mm (No. 4) sister bar did not interfere with the load in the adjacent central bar and the analysis of imposed strains. Data

water-cement ratio of 0.45. Pumping pressures began at 140 KPa (20 psi) and increased to about 660 KPa (95 psi) as the height of grout increased in the drill hole.
from the five gages were acquired using a Model DT2055 RST Instruments Ltd. data logger and a laptop computer.

Figure 4. Photo of load test setup. Building undergoing renovation is on right.

Axial load applied to the pile was measured using a calibrated 2700-KN (600-kip) load cell. Calibrated jack pressures were also monitored from the 2700-KN (600-kip) hydraulic jack. Pile head movement was measured using two dial gauges and a backup wire and mirror system.

MICROPILE LOADING TEST AND LOAD TRANSFER

Loading Test Procedure

The loading test, performed in accordance with ASTM D 1143, was conducted using 20 equal increments of approximately 107 kN (24 kips) to a maximum load of 2120 kN (476 kips). Load increments were generally held for 10 minutes, with a 30 minute hold at the 150 percent of the target compressive design load (1020 KN, 230 kips). The load-displacement curve, corrected for elastic elongation of the portion of the bar above the ground surface, is shown in Figure 5.

Figure 5. Load-displacement curve for uplift micropile loading test.

Development of Strain within Micropile

The development of strain within the micropile as a function of applied pile head load at the instrumented depths is presented in Figure 6. Note, Figure 6 does not present the strain history of the gage located at 18.5 m depth, as it showed very little variation in strain throughout loading. The strain history indicates a smooth, monotonic increase in strain with applied load up to approximately 1350 kN (300 kip) and 1700 kN (380 kip) for the 0.9 m (3 ft) and 11.3 m (37 ft) gage locations, respectively, after which the strain response indicates behavior inconsistent with the observed load-displacement response.

It is suspected that the strain gages along the embedded sister bars debonded or otherwise malfunctioned at the loads noted. In order to assess the load transfer of the micropile, the expected strain history was estimated for higher loads based on experience with strain histories observed in augered cast-in-place pile loading tests. The estimated strain history for higher loads is shown in Figure 6.

Development of Load Transfer Distributions

Since the materials comprising micropiles consist of both grout and steel, the composite modulus for any given micropile must be carefully determined. The modulus of concrete decreases with increases in strain; therefore, no uniform value of modulus may be used for the entire micropile length or over the range of applied loads. In order to determine the observed load-transfer distribution, the axial is estimated by fitting a line between the ratio of...
stress-strain relationship was determined using the procedure specified by Fellenius (1989, 2009). In this method, the tangent modulus, $E_t$, the change in axial stress to the change in axial strain, $d\sigma/d\varepsilon$, and axial strain, $\varepsilon$:

$$E_t = \frac{d\sigma}{d\varepsilon} = m \cdot \varepsilon + c$$  \hspace{1cm} (1)

where $m$ and $c$ comprise the fitted slope coefficient and intercept, respectively. Integrating Eq. (1) yields the appropriate expression for the axial stress at the observed axial strain, provided the strain gage pair is relatively unaffected by shaft resistance (Fellenius, 2009). The gage pair placed 0.9 m (3 ft) from the ground surface was used to determine the appropriate composite modulus, as the shaft resistance at this location was negligible. The fitted slope coefficient $m$ and intercept $c$ were found equal to -0.00462 GPa/$\varepsilon$ (-0.670 ksi/$\varepsilon$) and 36 GPa (5220 ksi), respectively. Note, the shallow slope coefficient is representative of the high area of steel present in the composite micropile section ($A_s = 9.8$ percent of nominal pile cross section area), and is much greater, for example, than an augered cast-in-place pile.

**Observed and Estimated Load Transfer**

The load transfer along the micropile shaft during the uplift loading test, shown in Figure 7a, was computed from the observed and estimated strain response and used the fitted slope coefficient and intercept, described above. The load transfer observed during the loading test indicated that the shear resistance in the loose to medium dense silts and sandy silts in the upper 11 m (36 ft) reached a maximum value at an applied micropile head load of approximately 1170 KN (263 kip). The fact that a limiting value of shear resistance was obtained is shown by the parallel lines in adjacent load transfer curves for this soil unit. The load transfer observed in the medium dense to dense, silty gravel, silty sands, and sandy silts located below 11.3 m (37 ft), however, indicated significantly greater shearing resistance, and did not reach a limiting resistance during the loading test.
The presence of residual loads in drilled foundations, such as drilled shafts, micropiles, and augered cast-in-place piles has been increasingly recognized. The break in the load transfer curve between the 11.3 m (37 ft) and 15.2 m (50 ft), and 15.2 m (50 ft) and 18.5 m (61 ft) gage locations indicated either that a significant change in soil strength existed within the soil profile (as expected from the CPTu profile) or that a residual load existed within the micropile. The unit shaft resistances presented reflect those observed during testing, and do not consider the effect of a possible residual load. Instead, the change in load transfer is attributed to a change in soil stiffness as indicated by \( q_t \) values in the CPT log (Figure 2).

Table 1 summarizes the observed unit shaft resistance values, \( r_s \), for the instrumented micropile at various head displacements. Consistent with the observed load transfer, little additional shearing resistance was mobilized between a head displacement of 10 to 25 mm (0.4 to 1 in) within the upper loose to medium dense silts and sandy silts, whereas significant resistance is mobilized over the same head displacement range in the stronger medium dense to dense silty gravels, silty sands, and sandy silt layers below 11 m (36 ft).

Table 1 also provides the range in ultimate bond strength values suggested by the FHWA (2005) for a Type A micropile (i.e., gravity grouted). Although the test micropile was grouted under pressure, the pressure was small in comparison to typical Type B grouting pressures (e.g., 0.5 to 1.0 MPa). The observed unit shaft resistances appear to fall along the upper bound of values suggested by FHWA (2005). Additionally, the observed bond strength from 11 to 15 m depth do not represent ultimate conditions; the ultimate bond strength over this depth range would likely
be significantly higher had the micropile been loaded to geotechnical failure.

**Calculated $\beta$–Coefficients**

It is common to compare unit shaft resistances for deep foundation elements on a normalized effective stress basis. For example, Jeon and Kulhawy (2001) present $\beta$-coefficients for a variety of micropiles. The vertical effective stresses, $\sigma_{vo}'$, were estimated for the present load test case at the depths corresponding to the midpoints between the upper three gage locations to determine the average Beta-coefficients over the instrumented lengths, given by:

$$\beta = \frac{r_s}{\sigma_{vo}'}$$

Figure 8 presents the Beta-coefficients derived from the observed unit shaft resistances over the range in observed micropile head displacements. As shown in Figure 8, the peak unit shaft resistance was reached within 20 mm (0.8 in) head displacement in the upper loose to medium dense soils, and indicated slight softening thereafter. The back-calculated Beta-coefficients for depth interval of 11.3 to 15.2 m (37 to 50 ft) show that initially, the resistance along the micropile was not mobilized. Once the applied load mobilized the micropile along this length, the shearing resistance increased gradually. As shown in Figure 8, the loading test did not mobilize peak shearing resistance in the pile below 11.3 m (37 ft). Note that the measured Beta-coefficients, greater than 1.0 for higher micropile head displacements, are common for drilled and grouted piles in unsaturated sandy silts, silty gravels, and silty sands.

Thus, the relatively soft, yet constant pile stiffness determined from the load-displacement curve (Figure 5) was the result of complex load transfer along the length of the micropile, in which portions of the resistance indicated a slight softening, and other portions had not yet mobilized peak resistance.

Analysis of the load test results indicated that the proposed micropile design met the project structural engineer’s load support requirements. No modifications to the proposed production piles were required. After approval of the load test results by city building officials, the contractor successfully installed the two production piles without performing further loading tests.

**CONCLUSIONS**

This paper presents the results an uplift loading test on a 19-m (63-ft) long, 200-mm (8-in) diameter, instrumented micropile installed within unsaturated silts and sands. Load-displacement response was measured at the pile head. Strain measurements were made at depths of 18.5 m (61 ft), 15.2 m (50 ft), 11.3 m (37 ft), and 0.9 m (3 ft) below ground surface. The strain measurements allowed estimation of the elastic modulus of the micropile and load transfer along the pile length. The observed unit shaft resistance was tabulated and Beta coefficients estimated.

Sister bar strain gage instrumentation was easily installed. Valuable insight into micropile behavior was obtained at modest cost. The detailed information collected from the instrumented loading test on the sacrificial pile allowed the piling contractor to construct two production piles inside the confined building space without needing to perform further proof or verification testing.
REFERENCES


