Behavior and mechanics of micropiles in rock

Comportement et mécanismes des micro-pieux dans la roche

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ABSTRACT

This paper presents advanced analyses of two instrumented axial compression load tests on micropiles founded in rock; one in Pre-Cambrian mica schist and one in Triassic mudstone. The design loads on these test micropiles were 2002 kN and 623 kN, respectively. Vibrating wire strain gauges were situated along the piles' lengths and monitored continuously during the load tests. Trends of measured strain versus pile top load indicated strain stiffening behavior in the cased portion of the pile and softening in the rock socket with increasing load. The load distribution in the rock sockets was found to be nonlinear and concentrated in the upper ¼ to ½ of the length even at the maximum test load. The mobilized bond shear stresses are similar to published values for similar geologic materials and environments.

RÉSUMÉ

Ce document présente le résultat des études menées sur deux micro-pieux instrumentés, soumis à des essais de compression axiale. Ces micro-pieux sont fondés dans un micaschiste précambrien et un schiste triassique. Les charges appliquées sur ces micro-pieux d'essai sont de 2002 kN et 623 kN. Des extensomètres sont installés le long des micro-pieux et les déformations sont suivies tout au long des essais de charge. Les mesures effectuées montrent, lorsque la charge augmente, un raidissement des contraintes dans la partie coffrée des micro-pieux et un ramollissement dans la partie ancrée dans la roche. La répartition des charges dans la portion ancrée dans la roche s'avère non linéaire et concentrée, même lors du chargement maximum, entre le quart supérieur et le mi lieu du pieu. Les efforts de cisaillement rencontrés lors de ces essais sont semblables aux valeurs publiées pour des études menées dans des environnements comparables.

Keywords : micropiles, rock sockets, bond shear stresses, load tests, strain gauges

1 INTRODUCTION

Detailed analysis and design of micropiles socketed into rock is frequently ignored and the mechanics under load are not well understood. However, these piles are more frequently designed to withstand very large structural loads, sometimes exceeding 2 MN for piles only 245 mm diameter. Proper understanding of the rock socket mechanics is critical for design and cannot be obtained unless strain gauge instrumented load tests are conducted. This paper presents the results of two instrumented compression load tests on rock-socketed micropiles. The first case is a 245 mm micropile socketed into Triassic-age mudstone in northern New Jersey, designed for a working axial capacity of 623 kN. The second load test is on a 2.0 MN working capacity, 245 mm micropile founded in the variable Pre-Cambrian age Wissahickon mica schist beneath the City of Philadelphia, Pennsylvania.

The results of these two heavily instrumented load tests demonstrate that the behavior of these composite micropiles is nonlinear with both structural response and geotechnical load transfer dependent on strain or load level. The cased portions of each pile exhibited stiffening behavior under increasing load with the secant modulus increasing. The rock sockets showed the expected softening behavior under increasing load with the secant modulus decreasing. Processing of the strain gauge data allowed the load distribution throughout the rock sockets to be examined, leading to the conclusion that the majority of the load was being resisted in the upper zone of the socket even at the maximum test load. Comparison of the mobilized bond shear stresses was made to published data for similar geologic materials and indicates that similar to higher values have been mobilized for these micropiles than was expected based on information available in the literature.

2 SUBSURFACE CONDITIONS AND TEST MICROPILE CONSTRUCTION

The first case history was located in the Meadowlands area of the northern New Jersey Piedmont, an area characterized by surficial swamp and varved silt and clay deposits over glacial till and mudstone-siltstone bedrock of the Passaic Formation. Figure 1 depicts the subsurface conditions at the micropile location. The micropile consisted of a 245 mm casing advanced just into the top of coreable rock and a 3.0 m long rock socket approximately 220 mm diameter. The rock socket was drilled with a hard formation tricone roller bit. The pile reinforcement consisted of a full length 43 mm Grade 520 threadbar. The grout compressive strength was 40 MPa at the time of the load test.

The micropile for the second case history was installed into medium grade metamorphic rock belonging to the Pre-Cambrian Wissahickon Formation in the City of Philadelphia. This unit consists of mica schist exhibiting a highly variable weathering pattern and thick bands of pure muscovite that control the in-situ properties. Figure 2 depicts the subsurface conditions around the test pile. The test pile was composed of 245 mm casing installed to a depth of 8.7 m, followed by drilling of a 6.1 m long rock socket using a down the hole hammer with a 200 mm percussion bit. The micropile was reinforced by a 57 mm Grade 520 threadbar in the cased zone and a 63 mm Grade 1035 threadbar in the rock socket. The neat cement grout had a compressive strength of 41 MPa at the time of the load test.

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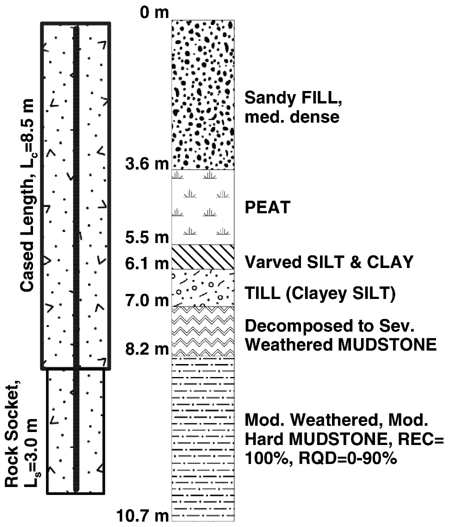

Figure 1 – Subsurface conditions and micropile schematic for case history no. 1 (Triassic mudstone)

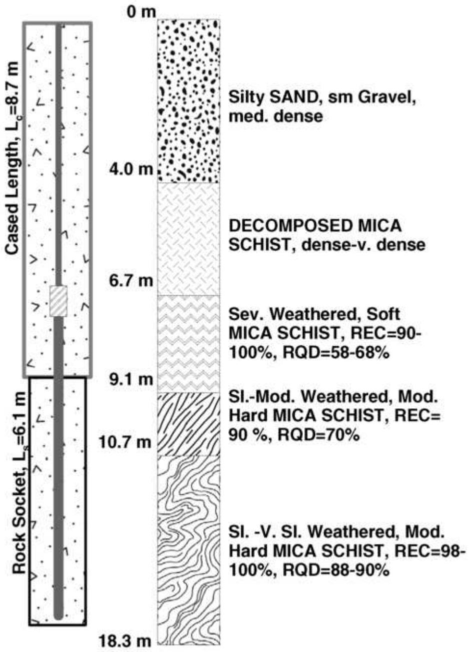

Figure 2 – Subsurface conditions and micropile schematic for case history no. 2 (Pre-Cambrian mica schist)

3 COMPRESSION LOAD TEST RESULTS

Both test piles were tested in axial compression to twice their design capacity in accordance with ASTM D1143. During both load tests, pile top settlements were measured with micrometer dial gauges with an accuracy of 0.025 mm and vibrating wire load cells were used. The micropile socketed into mudstone was loaded in increments of 25% of the design capacity to a maximum load of 1246 kN and held for 12 hours at that load. One load-unload cycle was performed during this process after reaching the 623 kN load. The maximum settlement of the pile was 6 mm and the permanent deflection was 1.5 mm after unloading. The pile in mica schist was tested in accordance with a quick-loading method with loads applied in 10% increments and held for 10 minutes each. The test load was 4.0 MN and was held for 60 minutes. The maximum and permanent settlements were 20 mm and 2 mm respectively. Load-settlement data from each test are presented in Figures 3a and 3b. From the shapes of the load-settlement and unloading data, it can be concluded that the micropile responses are largely elastic with nearly all deformation recovered upon unloading.

Each test pile was instrumented with vibrating wire strain gauges. The micropile in case no. 1 contained five pairs of embedment strain gauges mounted such that they were situated within the grout approximately 50 mm from the reinforcing bar. Two pairs of gauges were located in the cased length and three pairs in the bond zone. Eighteen strain gauges were installed in the case no. 2 micropile. Six spot-weldable gauges were welded directly to prepared locations on the centralized reinforcing bar and twelve embedment gauges were included. The cased and rock socket portions of the micropile contained six and twelve strain gauges, respectively. An automatic data acquisition unit was employed to continuously collect strain data during each load test.

4 MECHANICAL RESPONSE OF TEST MICROPILES

The strain response of each micropile indicates that the cased and rock-socketed portions have different mechanical responses to load. In Figure 4, two data sets of pile-top load versus measured strain are plotted for each case history. The strain gauge responses are plotted to resemble a conventional stress-strain relationship for illustrative purposes. Variations in pile composition, coupled with the differences in the surrounding geomaterials, would seem to dictate that the stress-strain response be different between the two pile cross sections. Assessment of these behaviors is important for understanding the relationship between pile secant modulus and strain.

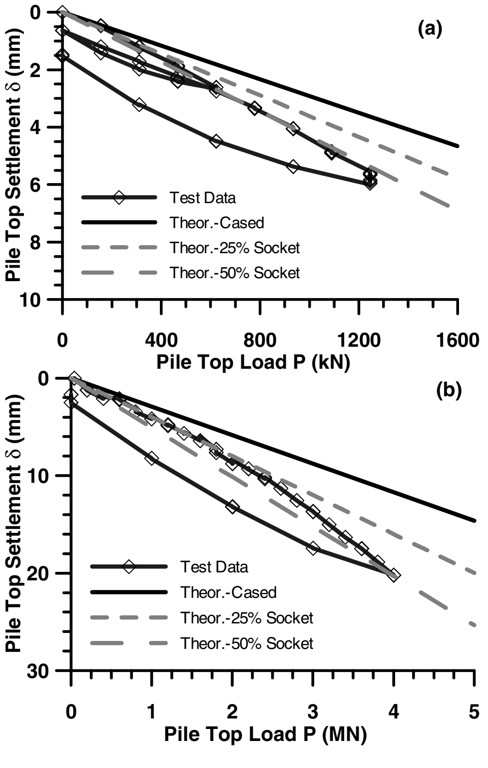

Figure 3 – Load-settlement plots for axial compression load tests (a) case no. 1, and (b) case no. 2

For both test piles, the average strain at each gauge level within the cased portion of the piles decreases with increasing load. Strain "hardening" is the apparent mode of behavior. The concave-up shape of the two data sets indicate that as the applied load is increasing the incremental strains are decreasing, leading to an increased stiffness or modulus. Based on the work of Fellenius (1989) and the author's experience with instrumented micropiles, this behavior is unexpected. Strain "softening" was the expected response for even the cased portion of this micropile because of the known decrease in grout stiffness with increasing strain. The observed hardening behavior is thought to be a result of the extremely high degree of circumferential confinement offered by the thick high strength pile casing. As the load is transferred to the central reinforcing bar and grout within the casing, radial strain of the grout results in circumferential stresses being generated within the casing wall, mobilizing additional confinement and stiffening this portion of the pile. The relative fixity of the casing tip at the top of sound rock only adds to this behavior.

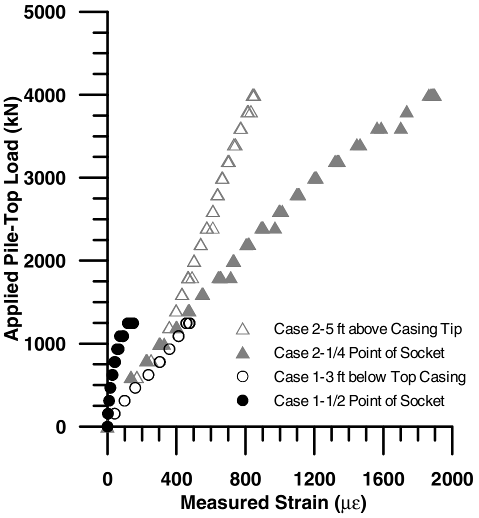

Figure 4 – Typical strain responses for cased length and rock socket of each test micropile

The plotted strain gauge data within the rock sockets indicate that the predominant pile response is that of a strain "softening" composite. The concave-down shape of the load-strain curves in Figure 4 denotes that the incremental strains are increasing with each load increment, leading to a decreasing socket modulus. This was the expected load-strain response for the rock socket and consistent with behavior observed by the author in other micropiles, particularly those in soils. The reason for the strain softening behavior is that the pile is assumed to be significantly stiffer than the surrounding rock mass. In this instance the pile-rock mass system permits some degree of radial strain and the pile compresses axially. Accumulating volumetric and shear strains in the rock socket, accompanied by microscale cracking of the grout, cause the degradation of the pile modulus with the increasing strain levels and consequently, a strain softening pile response.

5 LOAD TRANSFER RESPONSE IN ROCK SOCKETS

Load transfer from the micropile to the rock mass is accomplished primarily through bond shear at the rock to grout interface. The strains that develop and are measured within the pile are used to calculate the load at each gauge level using

$$P=\varepsilon \cdot A_p \cdot E_p \quad (1)$$

where A_p is the cross sectional area at the gauge level and E_p is the composite secant modulus of the pile. The secant modulus is in itself a complex nonlinear physical property of the composite micropile whose estimation is related to the data presented in Figure 4. The tangent modulus method of Fellenius (1989) was used to determine the relationship between secant modulus and strain within the pile. The mobilized bond shear stress τ can then be calculated for the zones between strain gauge using

$$\tau_{mob}=\Delta P/(\pi \cdot d \cdot \Delta L) \quad (2)$$

where ΔP is the interpreted load difference between two adjacent gauge levels, d is the diameter of the micropile in that zone, and ΔL is the length. Figures 5 and 6 present the load distribution for each test micropile along with the mobilized bond shear stresses for multiple load increments of each compression load test. The known loads at the top of each pile are determined from the vibrating wire load cells used during the tests.

According to the data, the majority of the load in each micropile is transferred to the rock socket, where it is rapidly distributed to the rock mass via the grout to rock interface. The amount of load transferred to the overburden soils and weak rock is about 12% for case no. 1 and 22% for case no. 2. The average bond stresses mobilized at maximum test load were 35 to 50 kPa for the weak soils above the micropile in mudstone and 170 kPa for the more competent soils and decomposed rock above the mica schist of case no. 2.

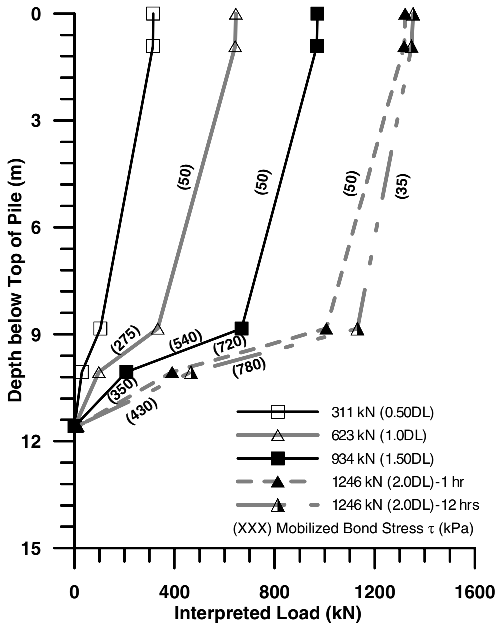

Figure 5 – Load distribution and bond shear stresses for case no. 1, micropile in mudstone

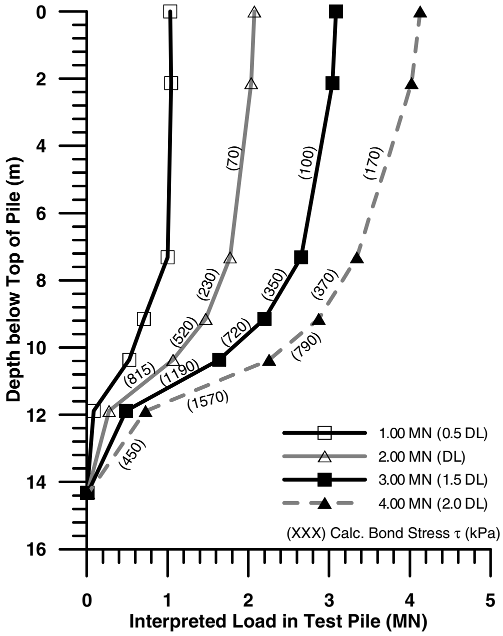

Figure 6 – Load distribution and bond shear stresses for case no. 2, micropile in mica schist

Of the load that reached the top of the rock socket for each pile, the majority of the load is dissipated through nonlinear shear stress transfer in the upper ¼ to ½ of the socket length. As the pile-top load increases, the distribution tends to shift down and to the right of Figures 5 and 6, indicating that the loads are being distributed deeper into the rock socket. The slope of the load distribution segments also tends to decrease as the mobilized bond shear stresses increase, but that the slopes never become constant or increase, indicating that no slippage is occurring in that zone. No load is seen to generate at the lowest strain gauge levels just above the rock socket bottoms.

Considering that the loading and unloading responses of each micropile, as depicted in Figure 3, are largely elastic and recoverable, the mobilized bond stresses in the rock sockets are still within serviceable limits and do not represent ultimate or failure conditions. The bond shear stresses that generated for each case are influenced by the rock quality and consistency as well as the inferred roughness of the rock socket due to installation methods. For case no. 1, the micropile in mudstone, the largest τ mobilized at design load, 623 kN, was 275 kPa, and this increased to 780 kPa at test load, 1246 kN. It is also worth noting that for the rock socket in mudstone, the quality of the rock was found to be relatively consistent during drilling. For case no. 2, the rock quality increased significantly down the length of the rock socket, leading to the distribution shape seen in Figure 6. The largest mobilized τ occurred from a depth of 10.4 to 12 m, where the rock mass strength was increasing rapidly with depth. In this zone, t_{mob} increased from 815 to 1570 kPa between the design load of 2 MN and test load of 4 MN. The rate of increase between load increments was also consistent, indicating that the grout to rock mass interface was still behaving linearly in this zone. In contrast, the zone immediately above this for case no. 2, from 9.2 to 10.4 m showed a decreasing rate of bond stress mobilization from between pile-top loads of 2 MN, 3 MN, and the test load of 4 MN. The rate of increase was 200 kPa between 2 and 3 MN and only 70 kPa between 3 and 4 MN, indicating that the top of the rock socket may have been approaching an ultimate condition resulting in progressive bond shear stress failure.

The mechanics of load transfer and mobilization of bond stress in the rock sockets are strongly influenced by the method of socket construction, overall socket roughness, and a dilatant response at the interface between grout and rock. For case no. 1, a hard formation tricone roller bit was used to drill out the socket in mudstone, resulting in a very rough interface. For case no. 2, a down the hole hammer with a carbide percussion bit was used to create the socket in variable quality mica schist, resulting in an interface that may have ranged in roughness. Socket roughness has been examined in the past by notable researchers in drilled shaft design and construction, most notably Seidel and Haberfield (1994). In assessing the socket performance, it is assumed that the rock socket is significantly stiffer than the surrounding rock mass. For case no. 2, nearby rock pressuremeter data indicated that the ratio of pile to rock mass stiffness, E_p/E_{rm}, was approaching 50. In these instances, the combination of socket roughness combined with limited radial expansion of the grout under confined axial loading leads to a dilatant response at the interface. It is postulated that as the rock socket is compressing under load it expands radially and increases the normal stresses at the interface. This mechanical response, when coupled with the presence of asperities caused by the drilling method and rock quality, creates a strong dilatant response at the interface, increasing the apparent bond stress until either the asperities begin to fail or grout microcracking begins to allow release of the normal stresses. A reduced rate of bond stress mobilization or slippage would then initiate, allowing progressive bond failure.

6 COMPARISON OF MOBILIZED BOND SHEAR STRESSES WITH PUBLISHED VALUES

The bond stresses mobilized at design and maximum test load for the two case histories discussed are similar to or greater than values found in the literature. Table 1 below summarizes some of the published data for rock types similar to that examined in case histories nos. 1 and 2. It is interesting to note that very few values of ultimate bond stress have been measured in the field and of those measured, most were for tension anchors. For the mudstone data, many design values were similar to that used for case no. 1. The mobilized bond stresses are similar for this rock type. For the mica schist, the maximum mobilized value for case no. 2 is higher than any of the three sets of published data for the Philadelphia area (Partos et al (1989), Koutsoftas (1981), and Yang et al (2004). For cases nos. 1 and 2, it is believed that the use of working bond stress upwards of 600 and 1000 kPa, respectively, is safe for design and should result in predominately elastic rock socket performance provided the rock quality is similar to that observed for these projects.

Table 1 – Summary of published values of mobilized, design, and ultimate bond stresses in rock similar to those discussed in this paper

Mudstone/Shale	τ_{mob} (kPa)	τ_{des} (kPa)	τ_{ult} (kPa)
Case No. 1	780 (630)$^{+}$	280	-
Hanna (1982)	-	120	370
Hanna (1982)	-	260	520
Hanna (1982)	880	630	-
PTI (2004)	-	-	500-1000*
Mica Schist	τ_{mob} (kPa)	τ_{des} (kPa)	τ_{ult} (kPa)
Case No. 2	1570 (940)$^{+}$	550	-
Partos et al (1989)	550	-	-
Koutsoftas (1981)	1360	480	-
Yang et al (2004)	450	220	-
Hanna (1982)	2160	1740	-

$^{+}$ values in ( ) are average mobilized values for rock socket

* recommended value to assume for design

7 CONCLUSIONS

The instrumented micropile load tests discussed in this paper have allowed for significant insight into the behavior and mechanics of rock sockets in Triassic mudstone and Pre-Cambrian mica schist bedrock situated in the northeastern United States. Confinement due to heavy-walled pile casing resulted in stiffening behavior under load for both test piles, while the expected softening behavior due to the neat cement grout was observed for the rock sockets. Consideration of the load-settlement data and the calculated load distributions indicated that the rock socketed micropiles performed elastically and did not suffer any significant debonding or socket slippage even at test loads up to 4 MN. Nearly all load was transferred to the upper ¼ to ½ of the rock sockets and no load observed at the tip. The average mobilized bond stresses for each of the rock sockets examined can be safely used for working design given the elastic, recoverable nature of the instrumented test micropiles. These mobilized bond stress values from cases nos. 1 and 2 are higher than those commonly seen in regional practice.

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