SOME OBSERVATIONS ON AXIAL COMPRESSION CAPACITY OF MICROPILES

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ABSTRACT. This paper examines the results of eighteen full-scale field tests on micropiles and integrates the results into an approximate analysis procedure to estimate micropile axial compression capacity. Both cohesive and cohesionless soils are included in the evaluation. The results show significant improvement in capacity with micropile grouting, but additional load tests data will be needed to refine this analysis procedure.

INTRODUCTION

Micropiles have a high ratio of pile circumference to cross-section area, and therefore they rely essentially on side resistance for load transfer. Tip resistance is negligible in most cases. Micropiles have been used widely for foundations, underpinning, slide stabilization, etc. Their development generally is attributed to pioneering work in Italy during the 1950s (Lizzi, 1982).

Specific analysis and design methodologies for micropiles basically are not available. Instead, empirical rules and procedures developed for larger-diameter drilled shafts generally have been used. However, load testing of micropiles has demonstrated that this approach usually results in conservative designs. Economies normally are realized only after site-specific loading tests.

In this paper, the behavior of micropiles is examined in cohesive and cohesionless soils under axial compression loading. Case histories were examined, and an evaluation was made of the test results, focusing primarily on the prediction of side resistance using conventional geotechnical parameters and analyses. A more detailed evaluation will be presented at the Transportation Research Board meeting in January 2000.

CASE HISTORIES AND EVALUATION OF GEOTECHNICAL ANALYSIS PARAMETERS

The database for this study included eight load tests from three sites in cohesive soils and ten load tests from five sites in cohesionless soils. The micropiles were grouted, typically type B and probably type D. For evaluation, the required geotechnical parameters are the undrained shear strength (s_u) for cohesive soils and the effective stress friction angle ($\bar{\phi}$), overconsolidation ratio (OCR), and in-situ coefficient of horizontal soil stress (K_o) for cohesionless soils. Some of the case histories had the necessary parameters (typically cohesive soils), but many lacked these geotechnical parameters (typically cohesionless soils). Therefore, it was necessary to use theoretical and empirical correlations (Kulhawy and Mayne, 1990) between the necessary parameters and those that were available, as described below.

EVALUATION OF SIDE RESISTANCE BY ALPHA METHOD FOR COHESIVE SOILS

The undrained side resistance for cohesive soils commonly is evaluated by the α method, which is given as (e.g., Kulhawy, 1990):

$$Q_S = \pi B \alpha \int_0^D s_u(z)\, dz \tag{1}$$

in which α = empirical correlation factor, B = shaft diameter, D = shaft depth, s_u = undrained shear strength, and z = depth. The axial compression capacity is deduced from the L_2 method (Hirany and Kulhawy, 1989). The value of α then can be back-calculated from the field load test results as follows:

$$\alpha = \frac{Q_s(L_2)}{\pi B D s_u} \tag{2}$$

in which $Q_s(L_2)$ = interpreted side resistance using the L_2 method, and s_u = mean undrained shear strength over depth D.

The side resistance Q_s also can be predicted from Equation 2 using available α correlations. For example, Chen and Kulhawy (1994) developed an α_{CIUC} for drilled shafts that links specifically to the soil undrained shear strength determined by CIUC triaxial tests. However, it must be remembered that the α factor is a lumped constant of proportionality that relies on many factors, including construction techniques, drilling disturbance on the soil, roughness of the interface between concrete and the soil, pore water pressure changes that occur during loading, geotechnical soil properties, and the method used to assess s_u. Figure 1 shows the normalized unit side resistance (from the micropile load test results / computed capacity for drilled shafts by α method) versus depth ratio. This figure shows that, for D/B > 100, micropiles and drilled shafts are essentially the same. However, at shallower depths, there is an apparent increase for micropiles over larger-diameter drilled shafts, by a typical factor on the order of 1.5 with values as high as 2.5.

EVALUATION OF SIDE RESISTANCE BY BETA METHOD FOR COHESIONLESS SOILS

The side resistance evaluated by the beta method is provided by the interface friction of the soil-pile interface over the shaft depth. To account for the variation of the soil properties with depth, the soil profile was divided into different soil layers. The average geotechnical parameters were estimated at the mid-depth of each layer, assuming the effective stress friction angle and overconsolidation ratio (OCR) based on the general description of soil consistency.

The drained side resistance is given by the beta method as (e.g., Kulhawy, 1991):

$$Q_S(\beta) = \pi B\,(K/K_0)\int_0^D \bar{\sigma}_V(z) K_0(z)\tan[\bar{\phi}\cdot\delta/\bar{\phi}]\,dz \qquad (3)$$

$$= \pi B\int_0^D \beta\,\bar{\sigma}_V(z)\,dz \qquad = \pi B\int_0^D f(z)\,dz$$

in which K/K_0 = factor that represents the change in the in-situ stress by construction method, $\delta/\bar{\phi}$ = interface roughness factor, $\bar{\sigma}_v$ = vertical effective stress, $\beta = K \tan\delta$, and $f(z)$ = unit side resistance. To calculate the side resistance, $\delta/\bar{\phi} = 1.0$ was assumed to model a rough soil-concrete interface. Since the case histories did not report the effective stress friction angle and OCR, the typical values in Table 1 for effective stress friction angle and OCR as a function of soil consistency were used to calculate the coefficient of horizontal soil stress, K_o, as given below (Kulhawy and Mayne, 1990):

$$K_0 = (1-\sin\bar{\phi}_{tc})\,OCR^{\sin\bar{\phi}_{tc}} \qquad (4)$$

The K/K_0 value has not been calibrated for micropiles, so a value of 1.0 that is based on good quality construction for drilled shafts was used for a first-order estimation. The field average beta (β_m) can be computed from the following:

$$\beta_m = \frac{Q_S(L_2)}{\pi B D\bar{\sigma}_{vm}} \qquad (5)$$

in which $Q_s(L_2)$ = interpreted side resistance from the L_2 method and $\bar{\sigma}_{vm}$ = mean vertical effective stress. As the depth ratio increases, β decreases and approachs the normally consolidated (NC) range, given by:

$$\beta_{NC} = K_0\tan\delta \approx (1-\sin\bar{\phi})\tan\bar{\phi} \qquad (6)$$

assuming $K/K_o = 1$ and $\delta/\bar{\phi} = 1$.

Table 1. Typical Values of Effective Stress Friction Angle ($\bar{\phi}$) and Overconsolidation Ratio

Soil Type	$\bar{\phi}$ (deg)	Overconsolidation Ratio
Loose Sand	28 – 32	1 – 3
Medium Dense Sand	32 – 38	3 – 10
Dense Sand	38 – 45	10 – 20

The predicted beta (β_p) can be computed from the following:

$$\beta_p = K_o(K/K_o)\tan[\bar{\phi} \cdot \delta/\bar{\phi}] \tag{7}$$

To evaluate β_p, the soil profile along the shaft depth was divided into different soil layers, and the average K_o and ϕ values were taken at mid-depth of each layer to calculate the predicted β for that layer. The value of δ/ϕ for cast-in-place shafts was taken as 1.0. Since the value of K/K_o depends on the construction method, local variations in K_o, cementation, and soil characteristics, it is difficult to define a unique relationship for β versus depth ratio. Figure 2 shows the normalized unit side resistance (from the micropile load test results / computed capacity for drilled shafts by β method) versus depth ratio. This figure shows that, for D/B > 100 or so, micropiles and drilled shafts are approximately the same. However, at shallower depths, there is an apparent increase for micropiles over larger-diameter drilled shafts, by typical factors in the range of 1.5 to 2.5 with values as high as 6. No differentiation by micropile type is evident.

SUMMARY

A database was developed of case histories of micropiles from eight sites with eighteen axial compression load test results. It was used to examine the micropile axial compression capacity.

It appears that there is a significant difference in load-carrying capacity between micropiles and drilled shafts that results primarily from the micropile pressure-grouting installation effects on the state of stress in the ground. An increase in "effective diameter" also is likely for micropiles, but this effect could not be evaluated from the data. The results show that micropiles can have a significant increase of capacity over larger-diameter drilled shafts at shallower depths with D/B < 100 or so. In cohesive soils, the typical increase is on the order of 1.5 with values as high as 2.5. For cohesionless soils, the typical increases are in the range of 1.5 to 2.5 with values as high as 6. No micropile type difference is evident. At greater depths with D/B > 100 or so, micropiles and drilled shafts appear to have similar capacity.

The factors noted above should be investigated carefully when extrapolating design values from larger-diameter drilled shafts into micropile design practice. These values will need to be re-evaluated and refined as more and better quality data become available.

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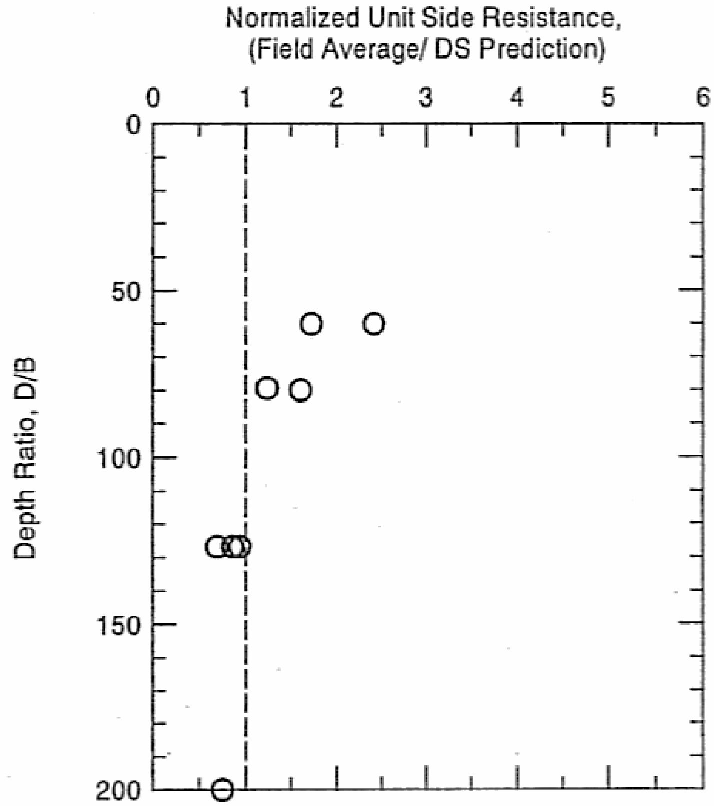

Figure 1. Normalized Unit Side Resistance vs. Depth Ratio in Cohesive Soils

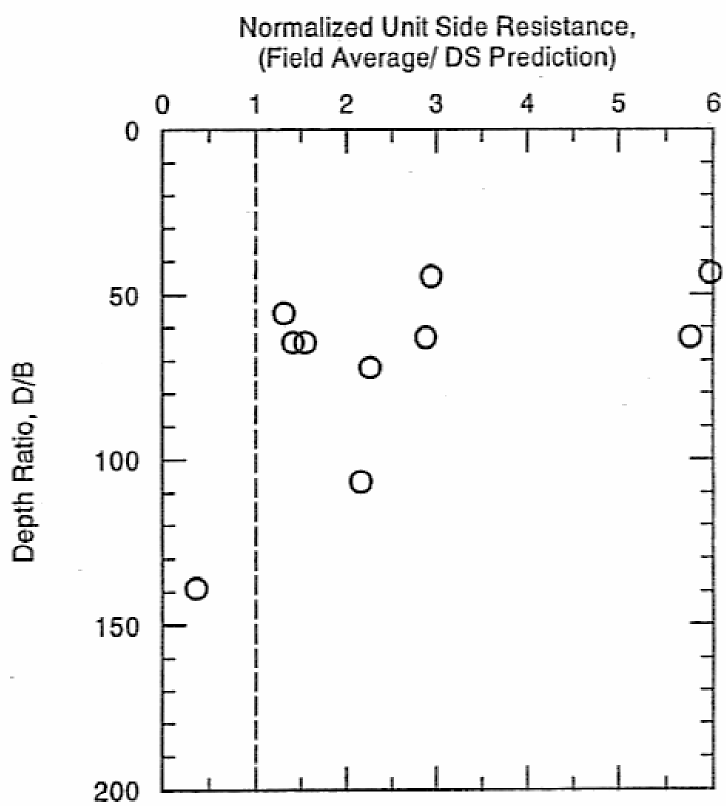

Figure 2. Normalized Unit Side Resistance vs. Depth Ratio in Cohesionless Soils