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(2004) - Fifth International Conference on Case Histories in Geotechnical Engineering

Apr 13th - Apr 17th

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Cushing, Andrew G.; Stonecheck, Scott A.; Campbell, Bradley D.; and Dodds, Scott D., "An Evaluation of the Load-Displacement Behavior and Load Test Interpretation of Micropiles in Rock" (2004). *International Conference on Case Histories in Geotechnical Engineering*. 1.
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Proceedings: Fifth International Conference on Case Histories in Geotechnical Engineering
New York, NY, April 13-17, 2004

AN EVALUATION OF THE LOAD-DISPLACEMENT BEHAVIOR AND LOAD TEST INTERPRETATION OF MICROPILES IN ROCK

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ABSTRACT

This paper summarizes a series of never-before reported axial compression load tests conducted on single micropiles that are embedded in or constructed on rock. These data are augmented by load tests on similar micropiles that have been reported by others. The observed displacements at the maximum test load (Q_{MAX}) and reported unfactored design load (Q_{DL}) are summarized. In addition, the small-strain load-displacement behavior of these foundations is evaluated by comparing the initial tangent slope (IS) to the theoretical elastic slope (ES), which is calculated by modeling the micropile as a free-standing column exhibiting fully-composite behavior. The data demonstrate that the ES/IS ratio has a strong dependence on the slenderness ratio D[depth]/B[diameter]. The observed results for micropiles in rock are discussed in the context of the micropile load test acceptance criteria proposed by the Deep Foundations Institute (DFI, 2001). In addition, recommendations are proposed for the maximum acceptable vertical displacement under the unfactored design load for such micropiles.

INTRODUCTION

Micropile technology was first developed in Italy by Dr. Fernando Lizzi more than 50 years ago as a means of in-situ soil improvement, with particular application to structural restoration and slope stabilization. Since then, the use of this technology has flourished throughout the world, and is used on an ever-increasing basis here in the United States. The technology can be used in a wide variety of geotechnical conditions, ranging from soft clay to hard rock.

This paper presents a database of load tests in axial compression conducted on single micropiles embedded in or constructed on rock. The vertical displacements at the maximum test load (Q_{MAX}) and reported unfactored design load (Q_{DL}) are summarized. The observed results for micropiles in rock are discussed in the context of the micropile load test acceptance criteria proposed by the Deep Foundations Institute (DFI, 2001). In addition, recommendations are proposed for the maximum acceptable vertical displacement under the unfactored design load for such micropiles.

DFI MICROPILE ACCEPTANCE CRITERIA

The Deep Foundations Institute (DFI, 2001) proposed the following general acceptance criteria for load tests conducted on high-capacity drilled and grouted micropiles:

Vertical displacement at the unfactored design load (ρ_{DL})

The pile shall sustain the unfactored compressive or tensile design load (1.0 Q_{DL}) with no more than _____ inches *(to be determined by the engineer)* of total vertical displacement at the top of the pile as measured relative to the top of the pile prior to the start of testing.

Creep Behavior

Test piles shall have a creep rate at the end of the 1.30 Q_{DL} increment which is not greater than 0.040 inches/log cycle time from 1 to 10 minutes or 0.080 inches/log cycle time from 6 to 60 minutes and has a linear or decreasing creep rate. This creep criterion is identical to that applied to proof load tests on permanent soil and rock anchors (PTI, 1996).

Failure load interpretation and factor of safety at Q_{DL}

Failure shall not occur at the 2.0 Q_{DL} maximum compression and tension loads. Failure is defined as the load at which the slope of the load-displacement curve falls below 40 kips/inch.

This paper addresses the issues of tolerable vertical displacement at the unfactored design load (ρ_{DL}), in addition to the interpretation of the failure load (Q_f) to achieve a minimum factor of safety of 2.0 at Q_{DL}.

MICROPILE DATABASE

Table 1. summarizes a series of never-before reported axial compression load tests conducted on single drilled and grouted micropiles that are embedded in or constructed on rock. All of the piles consist of a grout-filled steel pipe casing. The following geometric and load-displacement data are provided:

- depth (D), measured from butt to tip
- diameter (B), defined as the outer diameter of the steel casing,
- pile slenderness (D/B),
- rock socket length (L_{socket}),
- theoretical elastic slope (ES),
- initial tangent slope (IS), defined as the first derivative of the load-displacement curve (evaluated at zero load),
- load and displacement at the maximum test load (Q_{MAX}, ρ_{MAX}), and
- the first derivative of the load-displacement curve ($dQ/d\rho$) at the maximum test load, Q_{MAX}.

All of the micropiles presented in Table 1 were installed to provide structural support either for bridge piers or bridge abutments, with the exception of piles SC-1 and SC-2, which form the foundation system for a building. To augment the database presented in Table 1, data reported by others for drilled and grouted micropiles installed in rock and loaded in axial compression are provided in Table 2. The load and displacement at the unfactored design load of the piles introduced in Tables 1 and 2 are provided in Table 3. Note that for piles FF-1 through FF-10, only Q_{DL} and ρ_{DL} were available.

TOLERABLE VERTICAL DISPLACEMENT AT THE UNFACTORED DESIGN LOAD

The general acceptance criteria for high-capacity drilled and grouted micropiles proposed by DFI (2001) consider the service limit state (SLS) by limiting the vertical displacement at Q_{DL} to an acceptable value (to be determined by the engineer), and that the pile pass a creep test. For piles installed in rock, creep is generally not a significant design concern and will not be discussed further in this paper. Therefore, the tolerable vertical compression displacement at the unfactored design load will be emphasized.

The unfactored design loads (Q_{DL}) of the micropiles that are presented in Table 3 are plotted against the corresponding vertical compression displacements (ρ_{DL}) in Fig. 1. The data demonstrate that there is a distinct linear relationship between Q_{DL} and ρ_{DL}, indicating that the vertical displacements at the unfactored design load are primarily the result of the elastic compression of the micropile. The mean relationship between Q_{DL} (kips) and ρ_{DL} (inches), as identified by the solid line in Fig. 1, can be expressed by Eqn. 1 below.

$$\rho_{DL}(\text{inches}) = \frac{Q_{DL}(\text{kips})}{817} \tag{1}$$

Equation 1 was developed using 54 data points, and can be used to determine the "most likely" (mean) value of vertical compression displacement of micropiles in rock at the unfactored design load. If Eqn. 1 is used in this fashion, the error in estimating ρ_{DL} can be expressed by an overall coefficient of variation (COV = Standard Deviation / Mean) of approximately 0.55, presuming that the applied compression load Q_{DL} is deterministic.

The observed upper bound of ρ_{DL} as a function of Q_{DL}, identified by the dashed line in Fig. 1., can be expressed by Eqn. 2 below.

$$\rho_{DL}(\text{inches}) = \frac{Q_{DL}(\text{kips})}{363} \tag{2}$$

Since Eqn. 2 represents the upper-bound to existing design practice at the SLS for acceptably-performing drilled and grouted micropiles in rock, the authors recommend that it serve as the maximum acceptable value of total vertical compression displacement at Q_{DL}. In the absence of additional test data at higher loads, Eqn. 2 should be limited to values of Q_{DL} less than or equal to 500 kips.

For comparison, a limited amount of load test data was collected on driven H-piles bearing on rock and loaded in axial compression (Table 4). These data are plotted on Fig. 1, and indicate that Eqn. 2 for drilled and grouted micropiles may also be applicable to these driven H-piles.

FAILURE LOAD INTERPRETATION

In general, the load-displacement curves obtained from axial load tests on deep foundations conform to one of the three curves shown in Fig. 2. Curve A of Fig. 2 reaches a well-defined peak load, normally interpreted as the failure load Q_f, after which the load decreases with additional foundation movement. For Curve B, the load reaches an asymptotic maximum value, also interpreted as Q_f.

Table 1. Brayman-D'Appolonia Axial Compression Load Test Database of Micropiles in Rock

Pile ID	Location	D (ft.)	B (ft.)	D/B	L_{socket} (ft.)	ES (kips/ inch)	IS (kips/ inch)	ES/IS	Q_{MAX} (kips)	ρ_{MAX} (in.)	$dQ/d\rho$ (kips/in.) at Q_{MAX}
SC-1	State College, PA	87.0	0.583	149	10.0	355	489	0.73	350	1.08	167
SC-2	State College, PA	98.0	0.583	168	10.5	316	363	0.87	350	1.19	145
CC-1	Pittsburgh, PA	53.5	0.583	92	2.0	928	1267	0.73	300	0.60	396
CC-2	Pittsburgh, PA	51.0	0.583	87	2.0	974	704	1.38	300	0.70	268
CC-3	Pittsburgh, PA	51.0	0.583	87	2.0	974	927	1.05	300	0.41	679
219-1	DuBois, PA	48.0	0.583	82	0.1	694	495	1.40	372	0.90	561
219-2	DuBois, PA	48.0	0.583	82	0.1	694	1840	0.69	372	0.34	767
219-3	DuBois, PA	48.0	0.583	82	0.1	694	489	1.41	372	0.61	539
33-1a	Easton, PA	87.0	0.802	108	2.0	612	1220	0.50	400	0.49	476
33-1b	Easton, PA	87.0	0.802	108	2.0	612	1299	0.47	400	0.34	1111
33-2	Easton, PA	68.4	0.802	85	2.0	778	2500	0.31	400	0.26	2778
33-3	Easton, PA	72.3	0.802	90	2.0	736	4167	0.18	600	0.67	595
MFX-1	Washington Co., PA	38.4	0.802	48	19.0	1395	3472	0.40	1000	0.73	833
MFX-2	Washington Co., PA	66.0	0.802	82	10.0	806	2364	0.34	410	0.23	1625
MFX-3	Washington Co., PA	40.8	0.802	51	21.0	1303	2140	0.61	1092	0.52	2140
MFX-4	Washington Co., PA	92.9	0.802	116	18.2	573	1250	0.46	907	0.76	1250
FP-1a	Pittsburgh, PA	71.0	0.583	122	23.0	469	595	0.79	350	1.13	206
FP-1b	Pittsburgh, PA	71.0	0.583	122	23.0	469	489	0.96	350	0.72	341
FP-2	Pittsburgh, PA	71.0	0.583	122	23.0	469	1158	0.41	350	0.72	341
FP-3	Pittsburgh, PA	71.0	0.583	122	23.0	469	595	0.79	350	0.59	367
FP-4	Pittsburgh, PA	71.0	0.583	122	23.0	469	550	0.85	350	1.09	199
LR-1	Pittsburgh, PA	--	--	--	--	--	1357	--	300	0.27	950

Table 2. Axial Compression Load Test Database of Micropiles in Rock – Reported By Others

Pile ID	Location	D (ft.)	B (ft.)	D/B	L_{socket} (ft.)	ES (kips/ inch)	IS (kips/ inch)	ES/IS	Q_{MAX} (kips)	ρ_{MAX} (in.)	$dQ/d\rho$ (kips/in.) at Q_{MAX}
K-10-A(1)	Philadelphia, PA	61.0	0.583	105	14.0	810	1357	0.60	380	0.50	625
J.1-9-A(1)	Philadelphia, PA	61.0	0.583	105	14.0	810	1357	0.60	380	0.56	552
K-6-A(1)	Philadelphia, PA	118	0.583	202	14.0	358	950	0.38	380	1.04	297
L-9-A(1)	Philadelphia, PA	100	0.583	171	14.0	507	1188	0.43	380	0.88	347
I-1(2)	India	20.7	0.492	42	3.6	1297	4692	0.28	316	0.25	803
IL-1(3)	Chicago, IL	90.0	0.583	154	2.0	414	685	0.60	400	1.07	313
CH-1(4)	Chapel Hill, NC	46.8	0.635	74	12.0	--	1919	--	750	1.03	701
TP-B22(5)	Kuala Lumpur, Malaysia	180	0.820	220	52.5	395	2162	0.18	540	0.80	509
TP-C27(5)	Kuala Lumpur, Malaysia	136	0.820	166	26.2	634	2857	0.22	540	0.79	675
IL-2(6)	Chicago, IL	2.0	0.583	3.4	2.0	17614	9993	1.76	1000	0.71	700
IL-3(6)	Chicago, IL	4.0	0.583	6.9	4.0	8807	2551	3.45	800	1.38	317
IL-4(6)	Chicago, IL	6.0	0.583	10	6.0	5871	3166	1.85	1000	0.47	1070

(1)—Gallagher and Langan (2002); (2)—Davie and Senapathy (2002); (3)—Finno and Scherer (2000); (4)—Sanders, et al. (1999); (5)—Gue and Liew (1998); (6)—Finno, et al. (2002)

Table 2. (Continued)

Pile ID	Location	D (ft.)	B (ft.)	D/B	L_{socket} (ft.)	ES (kips/ inch)	IS (kips/ inch)	ES/IS	Q_{MAX} (kips)	ρ_{MAX} (in.)	dQ/dρ (kips/in.) at Q_{MAX}
LY-1(7)	Lynchburg, VA	--	0.500	--	--	--	--	--	300	0.13	--
WW-3(8)	West Whiteland, PA	34.8	0.583	59.7	9.8	958	1157	0.83	750	0.91	694
WW-4(8)	West Whiteland, PA	28.9	0.583	49.6	9.8	1154	1854	0.62	600	0.51	863
WW-5(8)	West Whiteland, PA	34.1	0.583	58.5	9.8	977	2911	0.34	600	0.51	1013
WW-6(8)	West Whiteland, PA	48.9	0.583	83.9	9.8	710	1416	0.50	870	1.00	578
WW-7(8)	West Whiteland, PA	110	0.583	189	9.8	309	624	0.50	685	1.30	552
WW-8(8)	West Whiteland, PA	80.4	0.583	138	9.8	425	393	1.08	685	1.89	344
WW-9(8)	West Whiteland, PA	44.9	0.583	77.0	9.8	775	1165	0.67	685	0.75	760
PR-1(9)	Providence, RI	65.0	0.500	130	8.0	--	--	--	220	0.70	--
TR-1(9)	Trafford, PA	36.0	0.417	86.4	0.1	--	--	--	40	0.06	--
TN-1(9)	Alcoa, TN	40.0	0.458	87.3	1.0	--	--	--	280	0.46	--
PA-1(9)	Warren Co., PA	44.0	0.708	62.1	15.0	--	--	--	448	0.40	--

(7)—www.technicalfoundations.com; (8)—Cadden, et al. (2001); (9)—Bruce (1989)

Table 3. Vertical Compression Displacements (ρ_{DL}) of Micropiles in Rock at the Reported Working (Design) Load (Q_{DL})

Pile ID	Q_{DL} (kips)	ρ_{DL} (in.)	Pile ID	Q_{DL} (kips)	ρ_{DL} (in.)
SC-1	175	0.375	IL-1	200	0.458
SC-2	175	0.439	CH-1	300	0.242
CC-1	150	0.259	TP-B22	270	0.264
CC-2	150	0.249	TP-C27	270	0.248
CC-3	150	0.185	IL-2	500	0.295
LR-1	150	0.123	IL-3	400	0.886
219-1	186	0.500	IL-4	500	0.226
219-2	186	0.139	WW-3	300	0.290
219-3	186	0.294	WW-4	300	0.170
33-1a	200	0.218	WW-5	300	0.170
33-1b	200	0.161	WW-6	300	0.250
33-2	200	0.113	WW-7	300	0.532
33-3	200	0.187	WW-8	300	0.827
MFX-1	400	0.230	WW-9	300	0.250
MFX-2	400	0.228	FF-1(1)	112	0.060
MFX-3	400	0.214	FF-2(1)	99	0.133
MFX-4	400	0.361	FF-3(1)	79	0.055
FP-1a	175	0.372	FF-4(1)	90	0.081
FP-1b	175	0.297	FF-5(1)	99	0.105
FP-2	175	0.263	FF-6(1)	99	0.220
FP-3	175	0.234	FF-7(1)	88	0.064
FP-4	175	0.402	FF-8(1)	135	0.040
K-10-A	190	0.240	FF-9(1)	79	0.096
J.1-9-A	190	0.240	FF-10(1)	135	0.037
K-6-A	190	0.420	FF-11(1)	90	0.098
L-9-A	190	0.350	FF-12(1)	108	0.107
I-1	121	0.033	FF-13(1)	108	0.131

(1)—www.fondedile-foundations.ltd.uk/prodpalidata.html

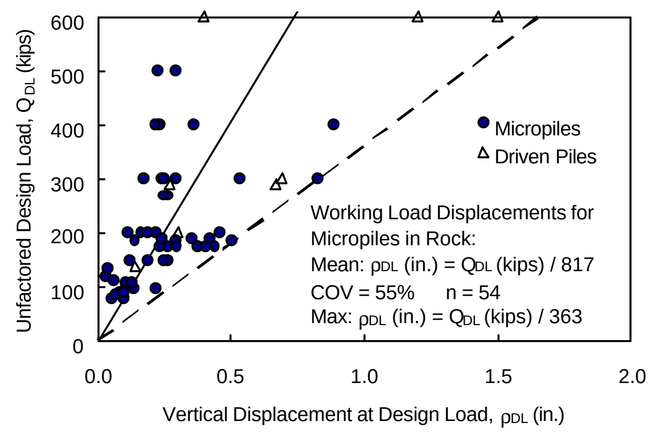

Fig. 1. Vertical Compression Displacements (r_{DL}) at the Unfactored Design (Working) Load (Q_{DL}) for Micropiles in Rock

Table 4. Vertical Compression Displacements (ρ_{DL}) of Driven Piles Bearing on Rock at the Reported Working Load Q_{DL}

Pile ID	Location	Q_{DL} (kips)	ρ_{DL} (in.)
11(1)	Chicago, IL	600	0.40
15(1)	Dearborn, MI	300	0.69
16(1)	Dearborn, MI	290	0.67
17(1)	Milwaukee, WI	600	1.20
18(1)	Milwaukee, WI	600	1.50
3-1(2)	Lackawanna, NY	140	0.14
3-2(2)	Lackawanna, NY	290	0.27
3-3(2)	East Chicago, IN	200	0.30

(1)—AISI (1975)
(2)—Bethlehem Steel (1979)

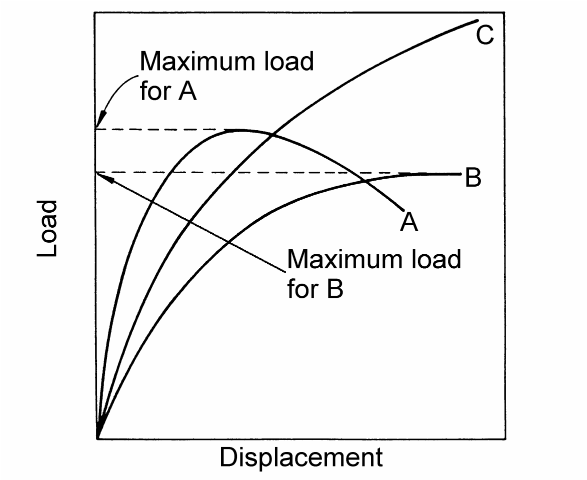

Fig. 2. Load-Displacement Curves for Deep Foundations

However, most load-displacement curves take the form of Curve C, in which no clearly defined peak load is achieved. For such cases, the failure load is difficult to assess. All of the load-displacement curves for the load tests presented in this paper are of type C.

DFI Failure Load Criterion

As discussed previously, the DFI (2001) micropile acceptance criteria defines the failure load Q_f as the point on the load-displacement curve at which the slope of the curve ($dQ/d\rho$) begins to fall below 40 kips/inch. The axial compression load test data presented in Tables 1 and 2 indicate that, at the maximum test load Q_{MAX}, the smallest value of $dQ/d\rho$ encountered was 145 kips/inch (Pile SC-2), with a corresponding ρ_{MAX} of 1.09 inches. Therefore, none of the micropiles presented in this paper reached the failure load Q_f, as defined by DFI (2001).

From the data presented in Tables 1 and 2, values of $dQ/d\rho$ at the maximum test load Q_{MAX} are plotted against the corresponding values of ρ_{MAX} in Fig. 3. These data indicate that a relationship exists between these two parameters for micropiles in rock under axial compression. The mean trendline shown in Fig. 3 indicates that a value of $dQ/d\rho = 40$ kips/inch corresponds to a value of $\rho = 1.83$ inches. However, the micropile with the largest value of ρ_{MAX} presented in this paper (1.89 inches for micropile WW-8) has $dQ/d\rho = 344 > 40$ kips/inch. Considering these facts and the data presented in Fig. 3, it is reasonable to assume that the DFI (2001) failure criterion of $dQ/d\rho < 40$ kips/inch corresponds to an absolute vertical compression displacement of approximately 2.0 inches for micropiles in rock.

If the failure load is defined at a vertical compression displacement equal to 2.0 inches, then the corresponding ρ_{DL} will be less than or equal to 1.0 inch (for $FS = Q_f/Q_{DL} \geq 2.0$). The allowable compression displacement at Q_{DL}, expressed by Eqn. 2, must also be checked. In addition, the unfactored

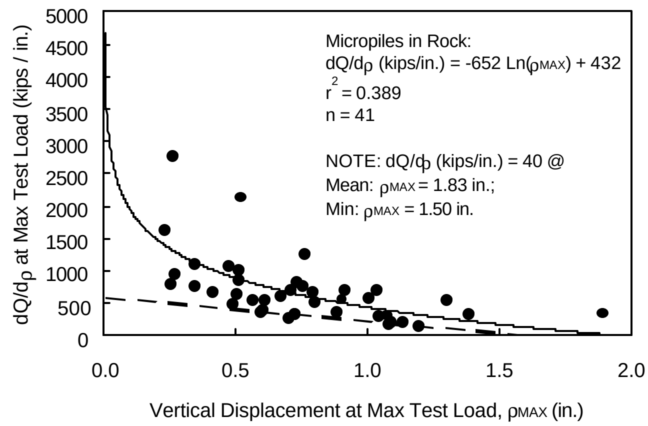

Fig. 3. $dQ/d\boldsymbol{r}$ at Q_{MAX} Versus the Vertical Compression Displacement at Q_{MAX} ($\boldsymbol{r}_{MAX}$) for Micropiles in Rock

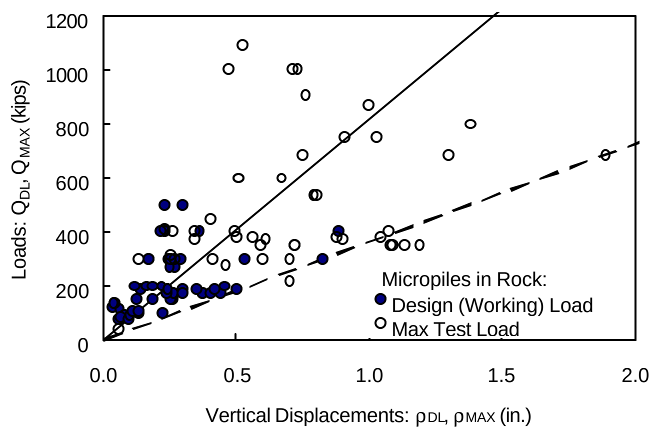

Fig. 4. Comparison of Vertical Compression Displacements at Q_{DL} and Q_{MAX} for Micropiles in Rock

design load Q_{DL} must not exceed the allowable structural capacity of the pile.

If the load test is terminated prior to the vertical compression displacement reaching 2.0 inches, then Q_f should be taken as Q_{MAX}.

An evaluation of the data presented in Tables 1 and 2 indicate that, from a geotechnical perspective, the maximum design load for micropiles in rock is governed primarily by the allowable vertical compression displacement at Q_{DL} rather than the application of $FS \geq 2.0$ on Q_f. This concept is exhibited graphically in Fig. 4, where the values of ρ_{MAX}, for the most part, fall within the displacement limit expressed by Eqn. 2.

Discussion of Other Interpretation Methods

The authors recognize that a number of other load test interpretation procedures exist to evaluate Q_f for deep foundations. The Davisson (1972) procedure, which was originally developed for driven H-piles, identifies Q_f as the load corresponding to the intersection of the load-displacement curve with the elastic line, which is constructed at a specified displacement offset from the origin at an inclination equal to the elastic slope (ES).

The offset proposed by Davisson (1972) is equal to 0.15 inch + [B (inches) / 120], where B = pile diameter. This offset is based on the presumption that the ultimate tip resistance of a 12 inch deep H-pile is mobilized at a tip displacement equal to 0.25 inch. Davisson (1972) conceded the possibility that the tip displacement required to mobilize the ultimate tip resistance for such a pile might exceed 0.25 inch. In reality, the displacement required to mobilize ultimate tip resistance depends on the nature of the pile cross-section. For deep foundations with solid, circular cross sections, tip displacements of 0.05B to 0.1B typically are required to mobilize the ultimate tip resistance. For the micropiles considered in this study, B ranges from approximately 6 to 10 inches. Therefore, the tip displacements required to mobilize ultimate end bearing can range from 0.3 to 1.0 inch. If the elastic compression of such piles is considered, the total vertical displacement of the pile butt can equal or exceed 2.0 inches without experiencing full geotechnical failure. Therefore, the use of the Davisson (1972) procedure can result in evaluations of Q_f that are overly conservative.

In the slope-tangent method, the initial tangent slope (IS) of the load-displacement curve is used rather than the elastic slope (ES). However, ES is not necessarily equal to IS. Values of ES/IS from the micropiles presented in Tables 1 and 2 are plotted against the slenderness ratio (D/B) in Fig. 5. As D/B increases, ES/IS tends to decrease. Clearly, variation in ES/IS can have a significant influence on the interpreted failure load Q_f.

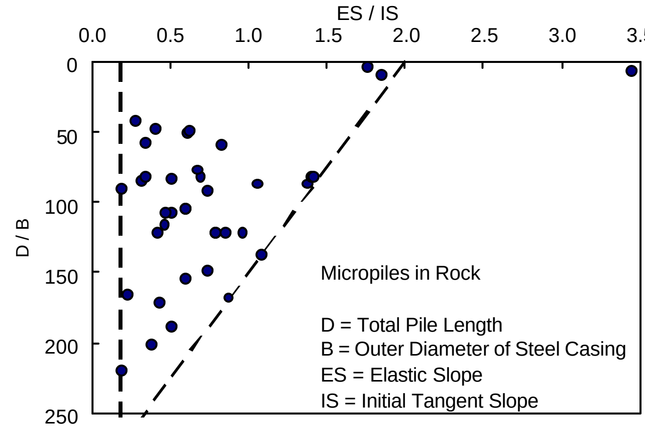

Fig. 5. ES/IS versus D/B for Micropiles in Rock

SUMMARY AND RECOMMENDATIONS

This paper has reported the results of a study of the load-displacement behavior of drilled and grouted micropiles embedded in or constructed on rock. A series of never-before reported axial compression load tests conducted on single micropiles is presented and subsequently augmented by load tests reported by others.

An evaluation of the unfactored design load (Q_{DL}) and corresponding vertical displacement (ρ_{DL}) for the micropiles discussed herein has resulted in the recommendation that the allowable vertical compression displacement be limited to Q_{DL}(kips)/363 for micropiles in rock.

An evaluation of the failure load interpretation method set forth by the Deep Foundations Institute (DFI, 2001) was also conducted. The data presented in this paper suggests that the DFI (2001) failure criterion of $dQ/d\rho < 40$ kips/inch can reasonably be associated with a total vertical compression displacement of approximately 2.0 inches. The application of a factor of safety FS = 2.0 to the value of Q_f will typically result in $\rho_{DL} < 1.0$ inch. However, the recommended limitation on ρ_{DL} described above must be checked independently. In addition, the design load Q_{DL} must not exceed the allowable structural capacity of the pile.

If the load test is terminated prior to the displacement reaching the DFI (2001) failure load criterion, then Q_f should be taken as Q_{MAX}.

From a geotechnical perspective, the maximum design load for micropiles in rock is governed primarily by the allowable displacement at Q_{DL} rather than the application of FS ≥ 2.0 on Q_f. This concept is best exhibited by the data presented in Fig. 4.

The findings set forth in this paper apply only to drilled and grouted micropiles in rock under axial compression, and are solely those of the individual authors.

ACKNOWLEDGMENTS

The authors would like to recognize Joseph W. Premozic and Christopher J. Lewis of D'Appolonia and Daniel D. Uranowski of the Brayman Construction Corporation for their assistance in the preparation and review of this manuscript.

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