GEOTECHNICAL SPECIAL PUBLICATION .NO. 124

GEOSUPPORT 2004

DRILLED SHAFTS, MICROPILING, DEEP MIXING, REMEDIAL METHODS, AND SPECIALTY FOUNDATION SYSTEMS

PROCEEDINGS OF SESSIONS OF THE GEOSUPPORT CONFERENCE:
INNOVATION AND COOPERATION IN THE GEO-INDUSTRY

January 29-31, 2004
Orlando, Florida

SPONSORED BY
International Association of Foundation Drilling (ADSC)
The Geo-Institute of the American Society of Civil Engineers

EDITED BY
John P. Turner
Paul W. Mayne

Published by the American Society of Civil Engineers

MODULUS LOAD TEST RESULTS FOR *RAMMED AGGREGATE PIERS*™ IN GRANULAR SOILS

Craig S. Shields[1], P.E. Member, ASCE, Brendan T. FitzPatrick[2], Associate Member, ASCE, Kord J. Wissmann[3], P.E. Member, ASCE

ABSTRACT: In the past five years, over 25 structures in the states of California, Oregon, and Washington, have been supported by *Rammed Aggregate Piers*™ constructed in granular soils. The piers are installed by drilling 60 to 90 cm diameter holes and ramming thin lifts of highway base course stone within the drilled cavities. The elements are used to support conventional shallow footings. The system is unique and innovative because it incorporates features associated with the design and construction of shallow and deep foundation systems. Accordingly, the design procedures include concepts derived from conventional shallow foundation design, historical stone column soil improvement system design, and cast-in-drilled-hole concrete shaft design. Unlike design values for drilled deep foundation systems, which are well-documented in the literature, parameter values for rammed aggregate piers are established from the results of modulus tests conducted at each project site.

This paper presents results of 19 rammed aggregate pier modulus tests performed at sites underlain by granular soils. Test results are correlated to matrix soil characteristics and length of the piers. This paper is of particular significance because it presents a database of in-situ modulus values used in the design of a cost-effective and increasingly popular drilled foundation system.

---

[1]Principal Engineer, Treadwell & Rollo, Inc., 510 14th Street, Third Floor, Oakland, CA 94612, csshields@treadwellrollo.com

[2]Associate Project Engineer, Geopier Foundation Company, Inc., 200 Country Club Drive, Suite D-1, Blacksburg, VA 24060, bfitzpatrick@geopiers.com

[3]President, Geopier Foundation Company, Inc., 200 Country Club Drive, Suite D-1, Blacksburg, VA 24060, kwissmann@geopiers.com

INTRODUCTION

The use of *Rammed Aggregate Piers™* as a cost-effective foundation support option has gained widespread acceptance over the past five years in the United States. The aggregate piers provide settlement control for the support of conventional shallow spread foundations. The design methodology for the rammed aggregate pier system combines design aspects used for shallow spread footings, historical stone columns, and cast-in-drilled-hole (CIDH) concrete shafts. Figure 1 shows a photograph of the Justice Center Parking Garage, Hillsboro, Oregon, which is supported on rammed aggregate piers.

FIG. 1: Structure supported on rammed aggregate piers in granular soils

The design methodology and project-specific performance of the rammed aggregate pier system are well documented in the literature (Lawton et al. 1994, Lawton and Fox 1994, Wissmann et al. 2000). However, the literature contains no detailed study of governing mechanisms or design parameter values to be used in granular soils. This paper presents a study of the rammed aggregate pier modulus test results for 19 piers installed in granular soils. The data is used to provide a database of pier performance categorized by matrix soil characteristics and pier length.

RAMMED AGGREGATE PIER CONSTRUCTION

The construction sequence of the proprietary *Geopier®* rammed aggregate pier system is shown in Figure 2. The aggregate piers are installed by drilling 60 cm (24 inch) to 90 cm (36 inch) diameter holes to depths ranging between 2 m and 8 m (7 feet and 26 feet) below working grade elevations, placing controlled lifts of well-graded highway base course aggregate within the cavities, and compacting the aggregate using a specially designed high-energy beveled impact tamper. The first lift consists of open-graded stone that is rammed into the soil to form a bottom bulb below the excavated shaft. The piers are completed by placing additional 30 cm (12 inch) thick lifts of aggregate over the bottom bulb and densifying the aggregate with the beveled tamper. In granular soils, casing is often required to maintain the stability of the sidewalls of the cavity during construction. A steel casing that is slightly larger than the drill tool is either vibrated or lowered into the cavity during drilling. The casing is withdrawn incrementally during aggregate tamping.

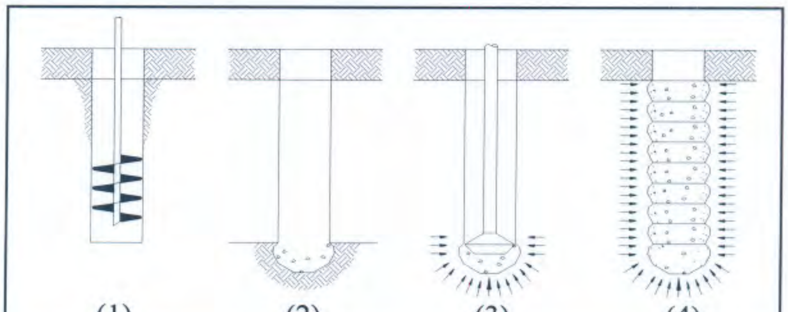

FIG. 2: Rammed Aggregate Pier Construction Process

During densification, the beveled shape of the tamper forces the stone laterally into the sidewall of the drilled cavity. This action increases the lateral stress in the matrix soil, thus providing additional stiffening and increased normal stress perpendicular to the perimeter shearing surface. The development of the increased horizontal pressure and the undulated shape resulting from construction provide an efficient mechanism for the development of shaft resistance along the perimeter of the pier. The installation of the very stiff aggregate piers, which exhibit high internal angles of friction (White et al. 2002, Fox and Cowell 1998), increases the composite shear resistance beneath the foundation, providing an increase in the allowable bearing pressure of the reinforced zone to values ranging from 240 kPa (5,000 psf) to 480 kPa (10,000 psf).

DESIGN METHODS

Rammed aggregate piers are installed in groups beneath conventional shallow foundations to increase the allowable bearing pressure for design and control foundation settlement. The design methodology uses a two-layer settlement approach that consists of evaluating the settlement of both the aggregate pier-reinforced soil (upper zone) and the unreinforced matrix soil (lower zone) below the aggregate pier-reinforced zone. The upper zone design methodology (see Eqs. 1 and 2) are described by Lawton and Fox (1994), Lawton et al. (1994), Lawton and Merry (2000), Wissmann et al. (2000), and Minks et al. (2001).

The settlement in the upper zone is computed using Eq. 1, where q_g is the stress applied to the aggregate piers and k_g is the stiffness modulus of the aggregate pier elements:

$$s_{uz} = \frac{q_g}{k_g} \qquad (1)$$

The stress on top of the aggregate piers (q_g) depends on the average bearing pressure of the rigid footing (q), the area coverage of the aggregate piers (R_a), and the stiffness ratio between the aggregate piers and the matrix soil (R_s):

$$q_g = q\left(\frac{R_s}{R_s R_a - R_a + 1}\right) \qquad (2)$$

The stiffness ratio, R_s, is the ratio of the stiffness modulus of the rammed aggregate piers (k_g) to the stiffness modulus of the matrix soil (k_m). The stiffness modulus of the aggregate piers is therefore an important parameter value because it plays a role in determining the top-of-pier stress (Eq. 2) and the upper zone settlement (Eq. 1). The stiffness modulus is typically established at each project site with a modulus test.

MODULUS TESTING

Test procedures

The modulus test set-up (Figure 3) is similar to a pile load test configuration and the test is performed in general accordance with ASTM D-1143. Rammed aggregate pier elements outfitted with uplift assemblages or steel anchors are installed to serve as reactions for the test beam. During the installation of the compression test pier, a steel telltale is positioned on top of the bottom bulb with sleeved telltale rods extending to the surface. This allows for deflection measurements to be made at both the top and the bottom of the pier. Following telltale installation, the test pier is constructed in the same manner as production piers. A concrete cap is constructed at the top of the completed pier to provide a platform for the hydraulic jack.

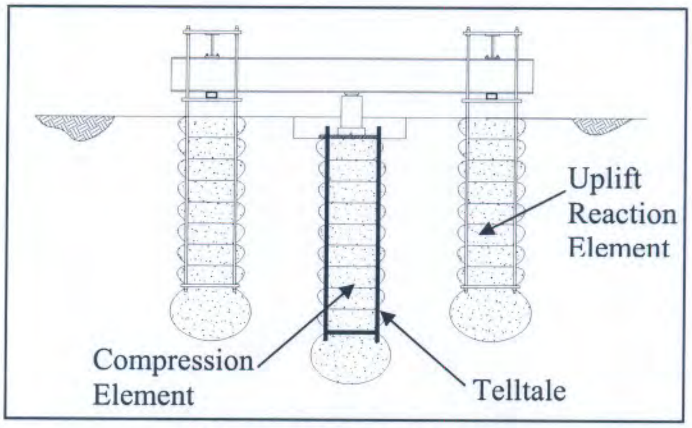

FIG. 3: Modulus Test Setup

The modulus test is performed by applying loads of up to 150 percent of the design stress to the top of the installed aggregate pier. During the application of loads, the deflections of the concrete cap at the top of the pier are measured. The deflections of the bottom of the pier are measured by monitoring the movement of the telltale rods at the surface. The rammed aggregate pier system is unique in the use of the telltale rods to measure deflections at depths within the piers. Plots of the stress versus deflection are constructed from the modulus test results to evaluate the stiffness of the modulus and deformation behavior of the aggregate pier.

Modulus test interpretation

The modulus test affords the opportunity to not only evaluate the stiffness modulus of the pier, but also to identify the governing behavior of the aggregate pier. The relationship between stress and deflection of the aggregate pier is typically characterized by a bi-linear response (Figures 4a and 4b). The stress level at the intersection of the two legs of the bi-linear stress-deflection curve is referred to as the inflection stress (σ_i). At stress levels less than the inflection stress the aggregate pier is characterized by elastic deformation. At stress levels greater than the inflection stress, the pier experiences non-recoverable plastic deformation. For foundation support, the top-of-pier stress for production elements is limited to values less than the inflection stress. Stiffness modulus (k_g) is defined as the ratio of the applied stress (σ) to deflection, and is expressed in units of F/L^3.

The respective movements of the top of the pier and the telltales at levels exceeding the inflection stress provide an indication of the governing deformation mechanism. The two types of modulus test responses are illustrated in the modulus test curves shown in Figures 4a and 4b. For an aggregate pier that undergoes plastic deformation with very little movement of the telltales, as shown in Figure 4a, the post-inflection stress deformation behavior results from radial bulging of the element into the matrix soil. The propensity for bulging is related to the in-situ matrix soil horizontal stress and the shear strength of the matrix soil. In some cases, bulging is attributed to the presence of an interbedded soft layer along the shaft. Bulging behavior is common in soft cohesive soils where resistance to bulging is low or in granular soils with very long shaft lengths where the shaft capacity exceeds the bulging resistance of the stiff or dense matrix soil. For an aggregate pier that undergoes plastic deformation with movement experienced by the telltale, as shown in Figure 4b, the post-inflection stress deformation behavior results from the development of tip stresses at the bottom of the pier. This type of behavior occurs when the applied load exceeds the frictional resistance along the perimeter of the shaft, which is common for short aggregate piers in granular soils where the bulging resistance exceeds the shaft capacity of the short pier.

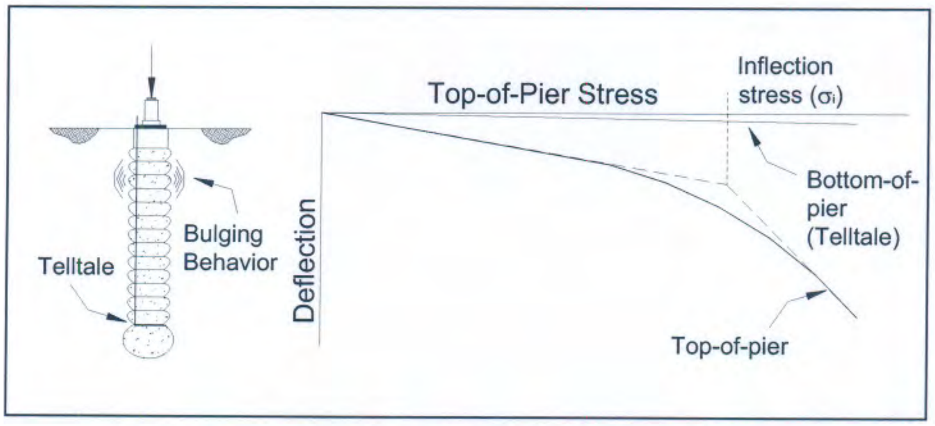

FIG. 4a: Modulus Test Results for Bulging Behavior

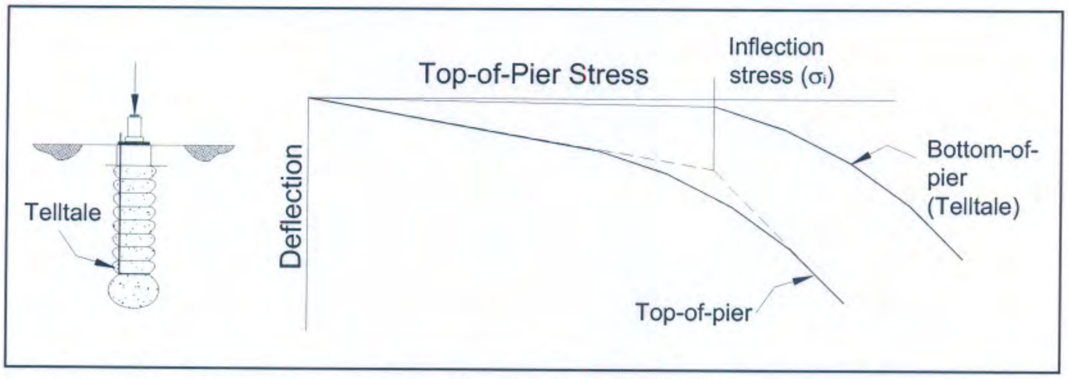

FIG. 4b: Modulus Test Results for Tip-Stress Behavior

Inflection Stress for Piers Characterized by Tip Stresses at High Stress

The estimated inflection stress for piers characterized by the development of tip stresses may be calculated by evaluating the ratio of the total shaft resistance along the perimeter of the pier (Q_s) to the cross-sectional area of the pier (A_g):

$$\sigma_i = \frac{Q_s}{A_g} \qquad . \tag{3}$$

The shaft capacity of the pier is calculated as the product of the unit shaft friction (f_s) and the area of the assumed cylindrical shearing surface along the sides of the pier:

$$Q_s = f_s d\pi H_s \qquad , \tag{4}$$

where d is the pier diameter and H_s is the shaft length. The unit frictional resistance (f_s) is calculated as the product of the average effective horizontal stress (σ'_h) and the tangent of the matrix soil friction angle (tan ϕ'_m). Past research has shown that the post-construction lateral pressure is increased to a value approximately equal to the Rankine passive earth pressure until a maximum lateral pressure of 120 kPa (2,500 psf) is reached (Handy 2001). As a result of the high lateral stress, significantly greater frictional resistance is developed along the aggregate shaft compared to other ground improvement systems.

CASE HISTORIES

A total of 19 modulus test results were obtained from West Coast project sites in granular soils (Table 1). As shown in Table 1, the majority of the soils are classified as naturally occurring silty sand (SM) or silty sand fill. Some of the sites are characterized by silty sand / sandy silt (SM/ML) soils. Five sites are characterized by poorly graded sand with silt (SP-SM) soils. Average SPT $(N_1)_{60}$-values recorded prior to pier installation within the upper zone of soil requiring aggregate pier reinforcement ranged from 4 blows per foot to greater than 40 blows per foot. The angles of internal friction of the matrix soils (Column 5 of Table 1) for each site are based on correlations with SPT $(N)_{60}$-values (Terzaghi et al. 1996) or project-specific laboratory shear tests when available.

TEST RESULTS AND INTERPRETATION

Inflection Stress for Different Deformation Mechanisms

As shown in Table No. 1, an inflection point was reached in less than half of the modulus tests. For the tests exhibiting an inflection point at high stress levels, two tests showed bulging behavior, while three tests showed the development of tip stresses. The deformation behavior of the remaining two tests exhibiting an inflection point was not apparent because telltale data was not available.

For tests that showed the development of tip stresses, measured inflection point stresses were compared to the calculated inflection stresses using the procedures shown in Eqs. 3 and 4. Figure 5 presents a comparison of the estimated inflection stress and the observed inflection stress. The diagonal line on the figure indicates a 1:1 correlation between the estimated and the observed inflection stresses. The three shaded points represent tests that exhibited inflection points and showed the development of tip stresses. The open-square points represent tests where no inflection point was exhibited because the maximum applied stresses were less than the estimated inflection stress. The open-triangle points represent tests where no inflection point was exhibited even though the applied stress exceeded the estimated inflection stress. The comparison shows reasonable agreement between the estimated and the observed inflection stresses for tests that exhibit tip stresses. In many instances, the estimated inflection stress exceeded the maximum applied stress during testing and no inflection point was observed (open-square points). The lack of an observed inflection stress is indicative of a testing stress level that was not sufficiently high to exceed the shaft resistance or cause plastic deformation of the

TABLE 1: Summary of Modulus Test Results

Proj. No. (1)	Location (2)	ASTM Soil Classification (3)	Average $(N_1)_{60}$[a] (blows/0.3m) (4)	Estimated Matrix Soil Friction Angle (degrees) (5)	Aggregate Pier Diameter (mm) (4)	Shaft Length (m) (5)	Break in curve (Y/N) (6)	Deformation Behavior[c] (TS, B, U) (7)	Inflection Stress[d] (kPa) (8)	Stiffness Modulus (MN/m^3) (9)	Aggregate Pier Elastic Modulus (MPa) (10)
1	Tacoma, WA	Fill (SM)	16	33	760	4.1	Y	B	1124	151	621
2	Tacoma, WA	SM	46	41	610	1.8	N	-	1524	417	762
3	Bellevue, WA	Fill (SM)	22	35	610	2.9	N	-	1524	150	434
4	Manchester,WA	Fill (SM)/SM	6	30	760	2.4	N	-	938	60	147
5	Seattle, WA	Fill (SM)	27	37	610	2.4	N	-	1524	297	723
6	Kent, WA	ML/SM	4	29	760	4.0	N	-	487	73	290
7	Seattle, WA	Fill (SM)	9	31	760	4.4	Y	U	973	96	423
8	Vancouver, WA	Fill (SM/SP)	46	41	760	2.9	Y	TS	1274	257	745
9	Seattle, WA	Fill (SM/ML)	10	32	760	2.4	Y	B	919	82	201
10	Hillsboro, OR	SM/ML	12	32	840	2.7	Y	TS	763	98	268
11	Hillsboro, OR	SM/ML	12	32	840	1.8	Y	TS	551	72	131
12	Roseville, CA	SM/SC	18	34	610	2.1	Y	U	1032	239	510
13	Sherwood, OR	SM/SP-SM	15	33	760	2.7	N	-	957	140	383
14	San Diego, CA	Fill (SC)	50/0.15m	42	760	5.2	N	-	1340	403	2087
15	West Covina,CA	Fill (SC)	30	37	760	2.4	N	-	969	98	239
16	Palm Desert, CA	SP/SM	33[b]	40	760	2.4	N	-	1122	260	634
17	Salem, OR	Fill (ML/SM)	17	34	760	2.7	N	-	1178	157	430
18	Anaheim, CA	SM/SP	15	34	760	2.4	N	-	1178	238	580
19	Anaheim, CA	SM/SP	15	34	760	2.4	N	-	1178	211	514

[a] SPT $(N_1)_{60}$-value not corrected for fines content.
[b] Average SPT $(N_1)_{60}$-value obtained from correlations with CPT tip resistances (Robertson and Campanella 1989).
[c] TS: Tip stress, B: Bulging, U: Unavailable for lack of telltale data.
[d] Maximum recorded stress when inflection point not reached.

pier. An inflection point would have eventually been observed in all cases if a higher stress were applied during the testing. Additional data also indicate that applied stress levels exceeded the estimated inflection stress without reaching the inflection stress (open-triangle points). The inflection stress calculations for these tests were conservative, yielding inflection stress estimates that were exceeded in the field tests.

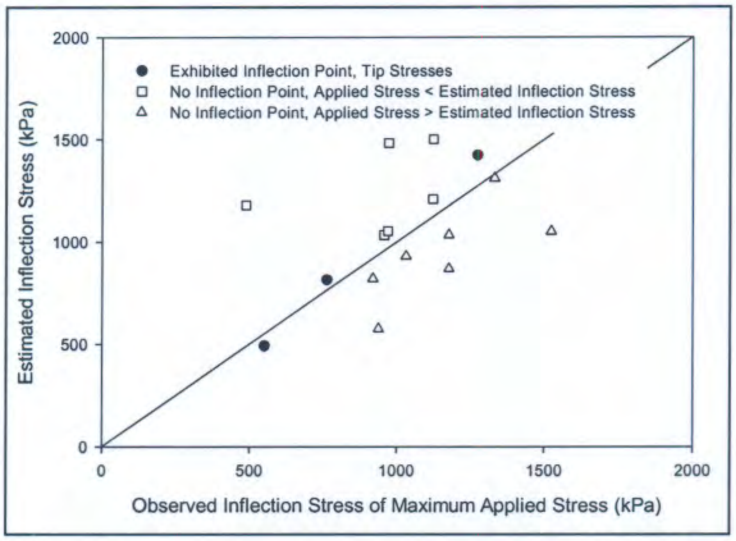

FIG. 5: Comparison of Estimated Inflection Stress and Observed Inflection Stress or Maximum Applied Stress

Figure 6 shows the relationship between the inflection stress and matrix soil friction angle in order to estimate the propensity for bulging. The two shaded points represent tests that exhibited inflection points and showed bulging. The open-square points represent tests that exhibited inflection points and showed the development of tip stresses. The open-triangle points represent tests where no inflection point was exhibited because the maximum applied stresses were not sufficiently large to induce bulging. The line that intersects the two points that represent tests exhibiting bulging deformations provides an empirical trend that relates propensity for bulging at different friction angles.

Figure 7 provides a tool that can be used to distinguish between piers that have the propensity for the development of tip stresses and piers that have the propensity for bulging. The near-vertical lines present contours of *predicted* inflection point stresses corresponding to the development of tip stresses as calculated using Eqs. 3 and 4. The single bold diagonal line represents the *empirical* relationship that describes the bulging propensity for different values of matrix soil friction angle from Figure 6. For a given matrix soil friction angle and shaft length, the propensity for tip stresses exists for a point located to the left of the bulging line, while a point located to the right of the bulging line is expected to bulge prior to exhibiting tip stresses. The plot suggests that the propensity for bulging increases with increasing shaft length and decreasing matrix soil friction angle.

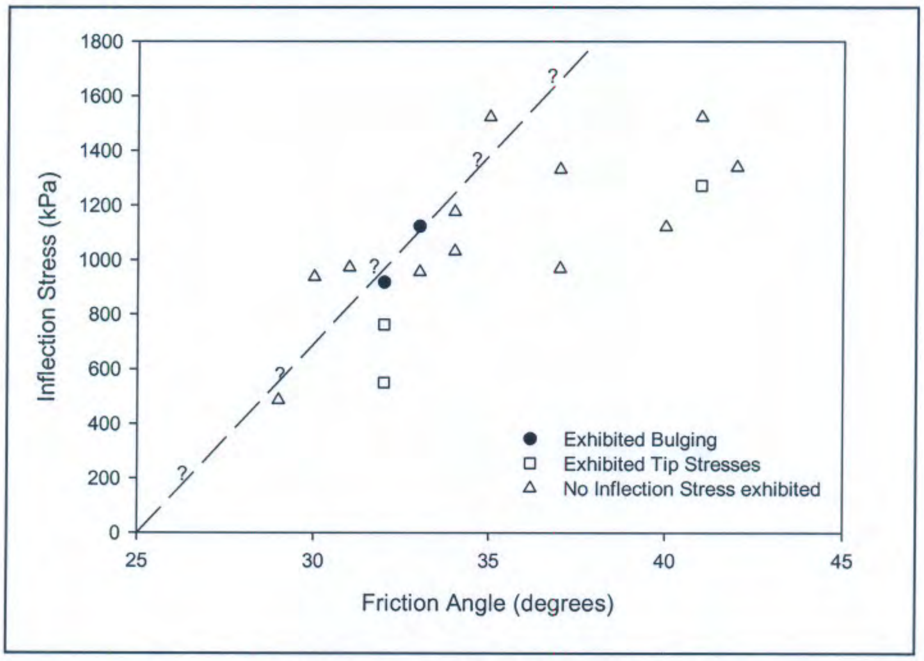

FIG. 6: Bulging Propensity in Granular Soils

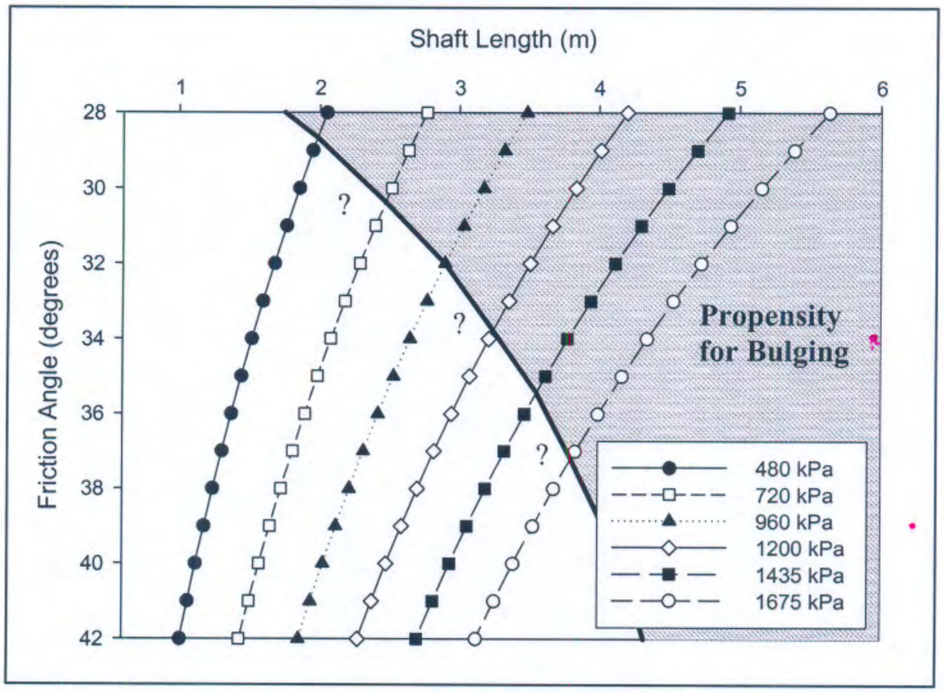

FIG. 7: Inflection Stress Prediction Chart

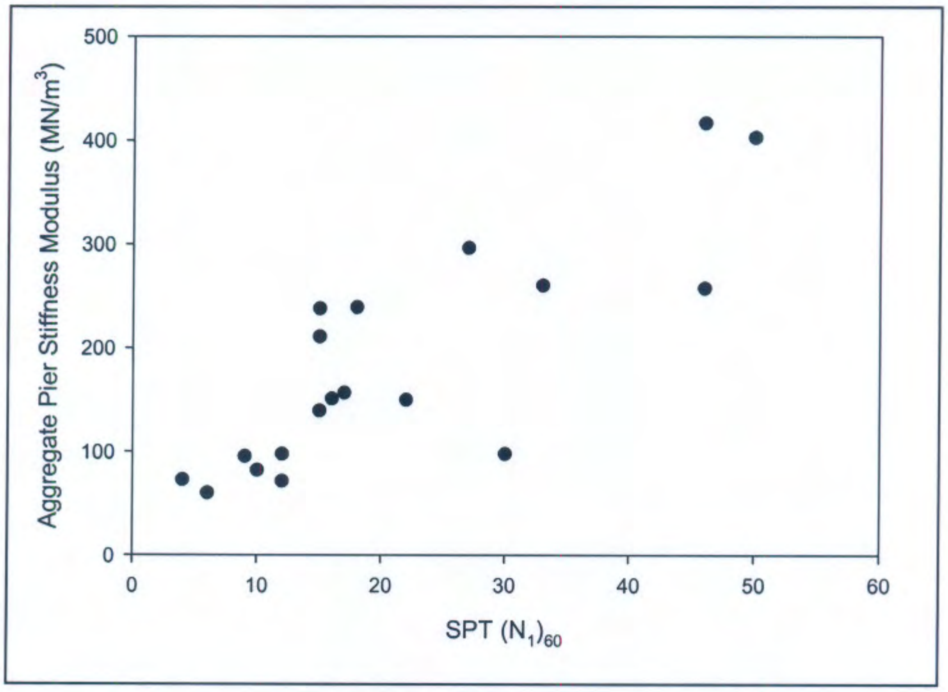

FIG. 8: Aggregate Pier Stiffness Modulus versus SPT $(N_1)_{60}$

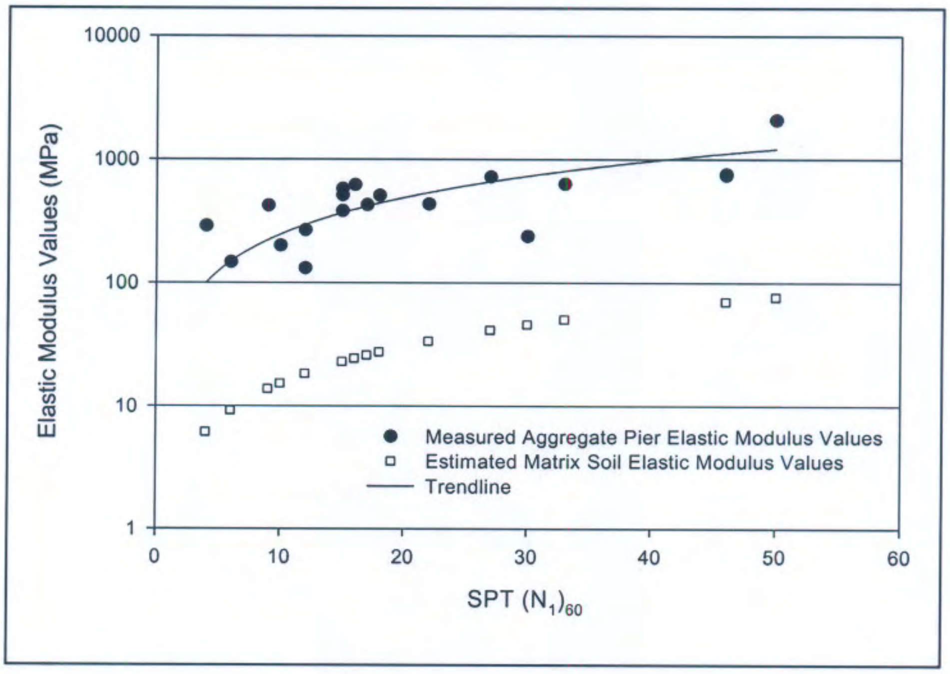

FIG. 9: Elastic Modulus Values versus SPT $(N_1)_{60}$

Aggregate Pier Stiffness

The plot shown in Figure 8 describes the relationship between the aggregate pier stiffness modulus and the measured SPT $(N_1)_{60}$-value of the unreinforced matrix soil prior to pier installation. The data clearly show that the pier stiffness modulus increases with the matrix soil $(N_1)_{60}$-value.

Figure 9 provides a comparison of the *measured* elastic modulus values of the rammed aggregate piers with the *estimated* matrix soil elastic modulus values. The elastic modulus value of the aggregate pier is conservatively estimated as the product of the stiffness modulus and the shaft length. The matrix soil elastic modulus value is calculated based on correlations between SPT N-values, CPT tip resistances, and elastic modulus values as reported by Robertson and Campanella (1989). The ratio between the measured aggregate pier elastic modulus values and the estimated matrix soil elastic modulus values ranges from 5 to 45. The largest ratios occur at sites exhibiting the lowest matrix soil SPT N-values.

SUMMARY AND CONCLUSIONS

The use of rammed aggregate pier elements to provide cost-effective support of shallow foundations has gained wide-spread acceptance over the past few years. Modulus tests are used to evaluate the inflection stress, measure the stiffness modulus, and identify the deformation behavior of rammed aggregate piers installed in granular soils. This paper presents a summary of 19 modulus test results for rammed aggregate pier elements in granular soils from project sites on the West Coast of the United States. The test results and interpretation indicate the following:

1. Aggregate piers subjected to high stresses deform either by developing tip stresses or by bulging radially.

2. A comparison of the estimated inflection stress and the observed inflection stress is shown in Figure 5. The data indicate the inflection stress may be estimated using Eqs. 3 and 4. This approach to estimate the inflection stress consistently predicts the development of tip stresses at high applied loads as indicated by the strong correlation between the estimated and observed inflection stresses.

3. Theoretical approaches and site-specific modulus test results confirm the inflection stress values for piers not subject to bulging increase with increasing shaft length and with increasing matrix soil SPT $(N_1)_{60}$-values.

4. An empirical trend that describes the bulging propensity for aggregate piers is shown in Figure 6. The trend suggests that the propensity for bulging decreased with increasing matrix soil friction angle for a given shaft length.

5. The stiffness modulus of rammed aggregate piers is related in large degree to the matrix soil SPT $(N_1)_{60}$-value.

6. Measured elastic modulus values for rammed aggregate piers are on the order of 5 to 45 times greater than estimated elastic modulus values for matrix soils. The installation of the significantly stiffer aggregate piers provides a greater degree of settlement control and predictability.

ACKNOWLEDGMENTS

Modulus test data and soil information used for this research was made available by Geopier Foundation Company, Inc., Geopier Foundation Company – Northwest and Geopier Foundation Company – West.

REFERENCES

Fox, N.S., and Cowell, M.J. (1998). Geopier Foundation and Soil Reinforcement Manual. Geopier Foundation Company, Inc., Scottsdale, AZ.

Handy, R.L. (2001). "Does Lateral Stress Really Influence Settlement?" ASCE Journal of Geotechnical and GeoEnvironmental Engineering. Vol. 127 No. 7, July.

Lawton, E. C., and N. S. Fox (1994). "Settlement of structures supported on marginal or inadequate soils stiffened with short aggregate piers." *Geotechnical Specialty Publication No. 40: Vertical and Horizontal Deformations of Foundations and Embankments*, ASCE, 2, 962-974.

Lawton, E. C., N. S. Fox, and R. L. Handy (1994). "Control of settlement and uplift of structures using short aggregate piers." *In-Situ Deep Soil Improvement,* Proc. ASCE National Convention, Atlanta, Georgia, 121-132.

Lawton, E.C. and Merry, S.M. (2000). "Performance of Geopier® Supported Foundations During Simulated Seismic Tests on Northbound Interstate 15 Bridge Over South Temple, Salt Lake City." Final Report No. UUCVEEN 00-03. University of Utah. December.

Minks A.G., Wissmann, K.J., Caskey, J.M., and Pando, M.A. (2001). "Distribution of Stresses and Settlements Below Floor Slabs Supported by Rammed Aggregate Piers." Proceedings, 54th Canadian Geotechnical Conference, Calgary, Alberta. Sept. 16-19.

Robertson, P.K. and Campanella, R.G. (1989). "Guidelines for Geotechnical Design Using the Cone Penetrometer Test and CPT with Pore Pressure Measurement." Hogentogler and Com., Gaithersburg, MD 20877, Fourth Edition, pp. 193.

Terzaghi, K., Peck, R.B., Mesri, G. (1996). Soil Mechanics in Engineering Practice. John Wiley & Sons, Inc., New York, NY. p.151.

White, D.J., Suleiman, M.T., Pham, H.T., and Bigelow, J. (2002). "Shear Strength Envelopes for Aggregate used in Geopier® Foundation Construction." Final Report. Iowa State University. September.

Wissmann, K.J., N.S. Fox, and J.P. Martin (2000). "Rammed Aggregate Piers Defeat 75-foot Long Driven Piles." *Performance Confirmation of Constructed Geotechnical Facilities*. ASCE Geotechnical Special Publication No. 94. April 9-12. Amherst Massachusetts.