Vertical and Horizontal Deformations of Foundations and Embankments
Volume 2

Proceedings of Settlement '94

Sponsored by the
Geotechnical Engineering Division of the
American Society of Civil Engineers

in cooperation with the
Federal Highway Administration
National Science Foundation

Hosted by the
Department of Civil Engineering
Texas A&M University

College Station, Texas
June 16-18, 1994

Geotechnical Special Publication No. 40

Edited by Albert T. Yeung and Guy Y. Félio

Published by the
American Society of Civil Engineers
345 East 47th Street
New York, New York 10017-2398

Settlement of Structures Supported on Marginal or Inadequate Soils Stiffened with Short Aggregate Piers

Evert C. Lawton,[1] Member, ASCE, and
Nathaniel S. Fox,[2] Associate Member, ASCE

Abstract: A short aggregate pier system, which was developed to provide an economical alternative to the overexcavation/replacement technique, has been used since 1988 to control settlement of structures located at sites with near-surface deposits consisting of marginal or inadequate soils. In this system, highly densified aggregate piers are incorporated within the marginal or inadequate soils, which results in a composite bearing material that is substantially stiffer than the unimproved soil, and on which shallow foundations can be supported with tolerable settlements. Three case histories are described in which the viability and effectiveness of the aggregate pier system in reducing settlements of shallow foundations bearing on the composite material are illustrated. Methods for analyzing and predicting settlements of footings supported on aggregate pier-reinforced soils are also discussed.

Introduction

A short aggregate pier system has been used since 1988 to control settlement of structures located at sites with near-surface deposits consisting of marginal or inadequate soils. In this system, highly densified aggregate piers are incorporated within the marginal or inadequate matrix soils, which results in a composite bearing material that is substantially stiffer than the unimproved soil, and on which shallow foundations can be supported with tolerable settlements. Buildings supported by

[1] Associate Professor, Department of Civil Engineering, University of Utah, 3220 Merrill Engineering Building, Salt Lake City, UT 84112.

[2] President, Geopier Foundation Company, Inc., 11421 East Aster Drive, Scottsdale, AZ 85259.

soils that were reinforced with aggregate piers have ranged from a large, single story, lightly loaded steel and glass greenhouse on deep, highly organic fills, to a 16 story tower on relatively strong and stiff residual soils. Column loads for the buildings have varied from 89 to 8,900 kN (20 to 2,000 kips), typically ranging from 222 to 4,450 kN (50 to 1,000 kips).

Background

This short aggregate pier system was developed to provide an economical alternative to the overexcavation/replacement technique that has been widely used throughout the world for many years to improve bearing soils beneath shallow foundations. The practical limitations of the overexcavation/replacement technique include the following: (1) large volumes of excavated soils and replacement aggregate are required; (2)excavations may collapse, necessitating the use of sloped or braced excavations deeper than 1.5 m (5 ft) in those cases where personnel are needed to compact the loosened bottom surface soils and/or place the aggregate in lifts; (3) support capacity may be limited unless the aggregate is compacted in thin lifts; (4) the bottom of the excavation may become softened by seeping groundwater or infiltrating rainwater; and (5) underpinning is needed if the project is a building addition or if another structure is located nearby.

Aggregate piers are constructed of well-graded aggregate, which typically consists of crushed stone as used for highway base course material. A pier is formed by creating a cavity within the matrix soil by augering or trenching, and densely compacting the aggregate within the cavity in thin lifts. The nominal shape of the pier may vary, but is generally either cylindrical or rectangularly prismatic. The matrix soil at the bottom of the cavity and the aggregate lifts are compacted using a special high energy, relatively high frequency, impact tamper with a 45° beveled foot. During these compaction processes, the soil at the bottom of cavity is prestrained and prestressed, and the aggregate displaces the matrix soil outward, resulting in a buildup of horizontal confining pressures prior to structural loading. The product is a very stiff, very dense, short pier element with an irregular (undulating) perimeter surface.

Although appearing similar in some ways to stone columns formed by vibro-replacement methods, aggregate piers differ in a number of significant ways, including the following:

1. The piers are designed primarily to stiffen the subgrade soil. Although some strengthening of the subgrade soil and increased radial drainage within the subgrade soil may occur, these are secondary considerations.
2. Aggregate piers are short, typically only two to three times as tall as they are wide. The piers are normally not extended to stronger, deeper soil zones.
3. Construction of aggregate piers involves the formation of a cavity by removal of matrix soil, rather than by lateral or vertical soil displacement, thus preserving to a large extent the soil's natural cementation and fabric.
4. Radial drainage is not a primary factor in aggregate pier design, allowing the aggregate comprising the pier to be well-graded crushed stone that includes

a fine and medium sand fraction, and which can be highly densified during compaction.

5. Aggregate piers are constructed using impact densification methods with relatively high impact frequency, rather than vibratory methods.
6. Aggregate piers are densified in thin lifts, prestraining, prestressing, and densifying adjacent matrix soils and producing very dense and very stiff foundation elements, thereby reducing vertical displacements upon application of structural loads.

The unique differences inherent within the aggregate pier system compared to other foundation types or ground improvement methods have resulted in award of a U. S. patent, with international patents pending (Fox and Lawton 1993).

LOAD TRANSFER MECHANISMS IN AGGREGATE PIER-REINFORCED SOILS

The transfer of load from the footing to the aggregate pier-reinforced soil can be classified by three general cases: (1) the footing is supported by one pier with the same diameter or width as the footing; (2) the footing is supported by one pier with a smaller diameter or width than the footing; and (3) the footing is supported by two or more piers with smaller diameters or widths than the footing. Cases 2 and 3 are the most common. The percentage of the foundation load carried by the aggregate piers is primarily a function of two factors - the areal coverage of the piers within the footprint of the footing and the relative stiffness of the piers compared to the matrix soil.

In Case 1, the pier carries the entire load and the response of the pier to the applied load can be reliably modeled by conducting static load tests (ASTM D-1194) on the pier using plates with the same diameter or width as the pier. The results from selected static load tests on aggregate piers are shown in Fig. 1. The load-deflection response is a function of the density and type of aggregate within the pier, the lateral stiffness of the matrix soil, and the lateral confining stresses acting along the pier-matrix interface. The lateral pressures along the interface depend on the initial stresses (before construction of the piers), stress relief due to excavation of the cavity, the installation process, and the stiffness and type of matrix soil. The magnitude of lateral stress increase generated during the pier construction process is limited by the maximum passive resistance of the matrix soil. Comparison of results from static load tests on aggregate piers and unreinforced matrix soil using plates with the same diameters as the piers shows that the piers are 5 to 40 times as stiff as the matrix soil (Fig. 2).

In Cases 2 and 3, the areal coverage of the aggregate piers (area of piers divided by the total area of the footing) may vary from about 20 to 40%. Using the range of typical relative stiffness ratios given above (10 to 20) and assuming a rigid footing, the portion of the total load carried by the piers typically ranges from 71 to 93%. Numerical analyses using the finite grid method (Bowles 1988) show that the ratio of bearing stress applied to the piers to the bearing stress applied to the adjacent matrix soil is approximately equal to the relative stiffness ratio. In

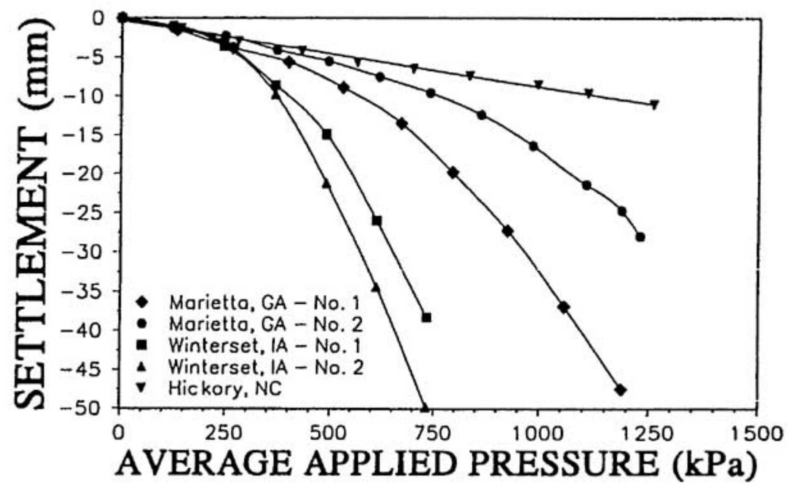

FIG. 1. Results from Selected Static Load Tests on Aggregate Piers

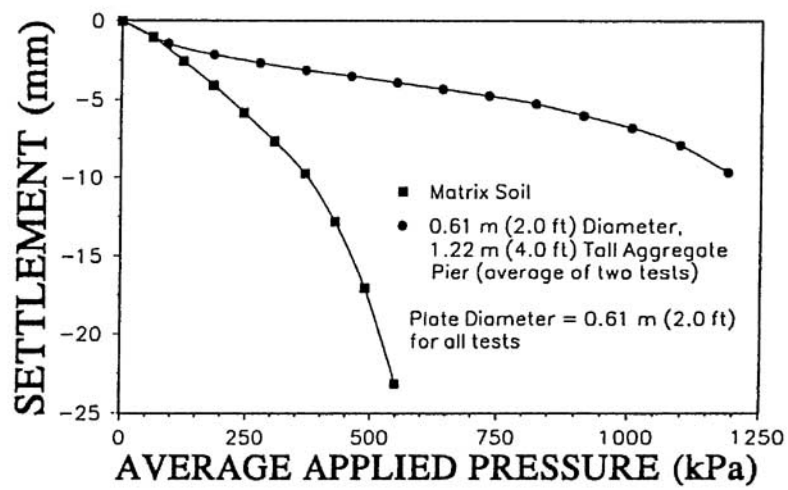

FIG. 2. Results from Static Load Tests on Unreinforced Matrix Soil and Aggregate Piers (Columbia, SC Project)

addition, even for fairly large footings that are relatively flexible, the bearing stress on each pier is roughly the same for a centric, vertical load.

At working loads, the stresses are transmitted through the aggregate piers in a manner similar to friction piles, but with two important differences. Most of the load is transmitted through the pier to the matrix soil by shear stresses along the pier-matrix interface, similar to the skin resistance that develops along the perimeter of friction piles. However, the aggregate piers also develop bearing resistance along the underside of the undulations at the perimeter of the pier, which transfers the potential shear surface out into the matrix soil rather than along the pier-soil interface. The result is greater shear resistance than that which would develop along a regular surface. A second important difference is that as settlement takes place, deformations occur near the top of the pier in the form of bulging, which displaces the matrix soil outward and increases the lateral pressures along the interface. The increased lateral pressure stiffens the pier similar to the stiffening that occurs in a strain-hardening material.

Settlement Analyses in Aggregate Pier-Reinforced Soils

The settlement of an aggregate pier-supported footing or mat is a complex soil-structure interaction problem consisting of interaction between footing and piers, footing and matrix soil, and matrix soil and piers. These complex interactions are not yet completely understood, and the preliminary methods described herein for estimating settlement provide rough estimates of settlement.

The reduction in settlement for an aggregate pier-reinforced soil compared to an unreinforced soil is related primarily to two factors: (1) The stiffness of the composite soil (matrix soil plus aggregate piers) within the reinforced zone is stiffer than the unreinforced soil; and (2) the stresses transmitted to the soil beneath the reinforced zone are lower than those transmitted through the comparable unreinforced zone. Therefore, settlement predictions are made separately for the upper and lower zones, and the two values are summed to yield the total predicted settlement.

Upper Zone Analyses

The upper zone (UZ), also called the aggregate pier influence zone, consists of the composite soil zone plus the soil beneath the composite zone that is densified and prestressed during the construction process. Densification and prestressing of this soil occurs by compacting the soil at the bottom of the excavated cavities with the tamper prior to placement of the aggregate lifts; some additional densification and prestressing also occurs during compaction of the first aggregate lift. During compaction of the aggregate lifts, the beveled foot of the tamper pushes some of the aggregate laterally against the adjacent matrix soil, resulting in densification of the matrix soil immediately surrounding the pier with a concomitant buildup of lateral stresses along the pier-matrix interface. The depth of this upper zone is considered to be equal to the height of the composite zone plus the width of the aggregate piers.

Although several methods have been used to estimate the settlement of the upper zone, the method determined to be the most practicable is the finite grid method (Bowles 1988). Details of the method can be found in the reference. Values of subgrade modulus for the pier and the matrix soil are either (1) obtained from the results of static load tests conducted on aggregate piers and the specific soils, (2) estimated from previous static load tests on similar soils, or (3) estimated from other available information such as recommendations for allowable bearing pressure provided by the project geotechnical consultant. For the pier moduli, results from static load tests using steel plates of the same diameter as the pier are used, which is somewhat conservative because it neglects the beneficial effect of the additional confining pressure produced from the footing bearing stress acting on the matrix soil. Because the aggregate piers are so stiff compared to the matrix soil, the subgrade modulus used for the matrix soil generally has limited influence on the magnitude of predicted settlement.

Stress Distribution at Interface of Upper and Lower Zones

Since the upper zone is substantially stiffer than the lower zone (LZ), the use of Boussinesq-type equations for stress distribution is inappropriate because these equations are based on the assumption of a homogeneous semi-infinite material. The actual stress distribution at the UZ/LZ interface is quite complex because it involves a stiff composite zone of finite horizontal and vertical extent with less stiff soils below and surrounding this stiff zone. The authors are currently conducting some finite element analyses and are planning to conduct field tests to study the stress distribution along the UZ/LZ. Some pertinent information can be obtained from Burmister's (1958) analysis of circular loads founded on a stiff upper layer of infinite horizontal extent underlain by a less stiff lower layer of infinite horizontal extent and semi-infinite vertical extent. Although this work was conducted relative to wheel loads on highway pavement systems, it clearly shows that the presence of a stiff upper layer reduces the magnitude of vertical stress transmitted to the lower layer compared to a homogeneous soil with the stiffness of the lower layer, and that the reduction in vertical stress increases with increasing stiffness of the upper layer.

The general procedure presently used by the authors to estimate the vertical stress intensity at the UZ/LZ interface is a modification of the 2:1 method and involves engineering judgment. In this approach, the area over which the applied load is distributed at the UZ/LZ interface is a function of depth below the bearing level and the stiffness ratio. For example, for a stiffness ratio of 10, a slope of 1.67:1 (vertical to horizontal) is typically used by the authors. The actual values used for any particular foundation depends on the type of aggregate, pier installation equipment and procedures, matrix soil type and condition, and engineering judgment. The magnitude of vertical stress increase and area of stress application are used to compute settlement of the LZ.

Lower Zone Analyses

For prediction of settlement within the lower zone, a number of methods are used depending upon available soil information. For granular soil strata, typically only Standard Penetration Test (SPT) blow counts (N) are available, although sometimes Cone Penetration Test (CPT) point resistance (q_c) data are also available.

Laboratory one-dimensional consolidation tests are usually performed for cohesive soil strata, although more sophisticated testing techniques can be used but are rarely justified economically unless a cohesive stratum is very soft or unusually thick and close to the UZ/LZ interface.

Both Schmertmann's strain influence factor method (Schmertmann 1970; Schmertmann et al. 1978) and Bowles' modified elastic theory method (Bowles 1988) are used to estimate settlements within the LZ if all the relevant strata are granular. For both these methods, the soil must be divided into layers of assumed constant elastic modulus (E_s), and estimates of E_s made for each layer based on regional or published correlations based on SPT N or CPT q_c (e.g. Bowles 1988).

Bowles' method can also be used to predict settlements within subsoils containing cohesive layers, but it is not recommended unless the stiffness of the cohesive layer(s) is of the same order of magnitude as the other layers and reasonable estimates of drained E_s for cohesive layers can be obtained from laboratory or field testing. The settlement of cohesive layers is more commonly predicted from the results of one-dimensional consolidation tests using appropriate modifications for variations from one-dimensional loading conditions [e.g. Skempton and Bjerrum (1957) or Leonards (1976)]. Likewise, Schmertmann's method may be used with cohesive soils with the realization that greater predictive error may occur. For major projects, the settlement of cohesive soils may be better approximated using the stress path method (Lambe 1967).

CASE HISTORIES

In the six years since the first short aggregate pier system was installed, thousands of aggregate piers supporting over one thousand footings, mats, and grade beams, have been constructed in six states. Although the initial idea of this system was to improve on the overexcavation/replacement method, the most common use has been to replace the need for deep foundations. Aggregate piers have also been used to improve soils that otherwise would require larger footings, resulting in the use of more economical smaller footings as well as reducing settlements.

General project information, foundation data, and predicted and actual values for settlement are summarized in Table 1 for ten projects in which shallow foundations have been supported on aggregate pier-improved soils. Settlements for the pier-reinforced soils were estimated using the methods described previously. Predicted settlements for the unreinforced soils were conducted using the same methods as for the lower zone analyses of the pier-reinforced soils, and are for the actual average bearing pressure and foundation dimensions listed in the table.

Comparison of the actual settlement values with the predicted values shows that the inclusion of aggregate piers within the matrix soil was successful in substantially reducing settlements for a variety of loading and matrix soil conditions. It also appears that the method used by the authors for predicted settlement of pier-reinforced soils provides a reasonable estimate of settlement in these cases.

TABLE 1. Predicted and Actual Settlements from Ten Aggregate Pier Projects

Project Description	Typical Foundation Description	Load kN (kips)	Bearing Pressure kPa (ksf)	Settlement, mm (in.) Predicted		
				Unreinforced Matrix Soil	Reinforced with Piers	Actual
5 story office bldg. Columbia, SC	3.66 m (12 ft) square footing	3,560 (800)	266 (5.6)	33 to 102 (1.3 to 4.0)	18 (0.7)	< 1.5 (< 0.06)
12 m (40 ft) tall milk silo Atlanta, GA	4.57 m (15 ft) square footing	3,010 (675)	144 (3.0)	48 to 104 (1.9 to 4.1)	13 (0.5)	< 1.8 (< 0.07)
46 x 91 m (150 x 300 ft) greenhouse, Atlanta, GA	0.91 m (3 ft) diameter footing	160 (35)	244 (5.0)	58 to 79 (2.3 to 3.1)	5 (0.2)	< 6 (< 0.25)
Industrial warehouse Winterset, IA	1.52 m (5 ft) square footing	445 (100)	193 (4.0)	150 to 230 (6 to 9)	23 (0.9)	< 19 (< 0.75)
Office addition Orangeburg, SC	1.07 x 2.13 m (3.5 x 7 ft) footing	801 (180)	352 (7.3)	41 to 112 (1.6 to 4.4)	13 (0.5)	< 13 (< 0.50)
Hospital addition Hickory, NC	12.2 m (40 ft) square mat	47,150 (5,300)	317 (3.3)	61 to 109 (2.4 to 4.3)	10 (0.4)	3.3 (0.13)
Hospital addition Hickory, NC	2.74 m (9 ft) square footing	1,824 (410)	242 (5.1)	30 to 104 (1.2 to 4.1)	13 (0.5)	< 6 (< 0.25)
16 story tower Atlanta, GA	15.2 x 30.5 m (50 x 100 ft) mat	66,720 (15,000)	144 (3.0)	20 to 89 (0.8 to 3.5)	10 (0.4)	3 to 8, avg. 6 (0.1 to 0.6, avg. 0.25)
12 story tower Atlanta, GA	3.66 m (12 ft) square footing	4,448 (1,000)	332 (6.9)	61 to 66 (2.4 to 2.6)	10 (0.4)	< 6 (< 0.25)
7 story parking deck Marietta, GA	4.27 m (14 ft) square footing	5,782 (1,300)	318 (6.6)	124 to 188 (4.9 to 7.4)	38 (1.5)	20 to 33 (0.8 to 1.3)

Case I: Five Story Office Building, Columbia, South Carolina

This 91 m by 49 m (300 ft by 160 ft), five story steel-framed structure with full basement, was constructed in Columbia, South Carolina in 1991. The site was within the Piedmont geological province, and all soils below the foundation bearing elevations were virgin residual soils. The site was within a seismic 2 zone, which by building code required horizontal support between pile or pier caps in the form of tie-beams to control lateral displacements if deep foundations were used. Column loads varied from 222 to 3,560 kN (50 to 800 kips), with wall loads of 58 to 102 kN/m (4 to 7 kips/ft). The predominant soil type within the upper 9 to 12 m (30 to 40 ft) was a very loose to firm silty fine to medium sand (SM). SPT blow counts varied from 2 to 20 with an average of 8. The geotechnical consultant determined that the soil could not satisfactorily support shallow isolated spread footings. Underlying this unsuitable stratum was a zone of stiffer clayey fine to coarse sand (SC) with N varying from 12 to 37, averaging 20. No groundwater was encountered to the maximum drilling depth of 15 m (50 ft). As is often the case in the southeastern U. S., no laboratory testing of soils was performed.

The geotechnical consultant recommended using either (1) a mat foundation with compacted subgrade, (2) pressure injected footings, or (3) drilled piers. Driven piles were eliminated from consideration because the structure was adjacent to church and office facilities, and the noise and vibrations accompanying driven piles would have been objectionable. Cast-in-place concrete piles were not mentioned as a foundation solution in the geotechnical consultant's report. The bid documents, however, did ultimately specify auger-cast piles to an estimated depth of 18 to 21 m (60 to 70 ft). The low bid exceeded the owner's budget by about \$1.5 million, so the owner chose the lowest two bidders and had them each provide a Value Engineering study to get the project within budget. Aggregate piers were approved during the Value Engineering study, and all piles were eliminated along with 85% of the tie-beams, at a total cost savings estimated at \$250,000.

Two static load tests were performed on 0.61 m (2.0 ft) diameter, 1.2 m (4 ft) deep aggregate piers (Fig. 2), and the results from the two tests were nearly identical. The pressure-settlement plot was fairly linear up to a pressure of 1,000 kPa (21 ksf), and the calculated subgrade modulus of 149 MN/m^3 (550 pci) was nearly twice the initial estimate of 76 MN/m^3 (280 pci). The final design bearing pressure of 287 kPa (6.0 ksf) for the aggregate pier system was four times the 72 kPa (1.5 ksf) allowable bearing pressure estimated by the geotechnical consultant for the soil without piers. The estimated settlement for the pier-reinforced soil was 18 mm (0.7 in.), somewhat less than the maximum tolerable settlement of 25 mm (1.0 in.) specified in the contract.

Nominal dimensions of the installed aggregate piers were 0.76 m and 0.91 m (2.5 ft and 3.0 ft) in diameter and 1.5 m and 1.8 m (5 ft and 6 ft) in height, respectively. Compaction of the matrix soil at the bottom of the holes prior to placement of the first aggregate lift increased the depth of the cavity by 100 to 180 mm (4 to 7 in.), averaging about 130 mm (5 in.). The quality of the compacted aggregate in the piers was determined by performing dynamic penetration tests on selected lifts. Comparative tests indicated that 15 blows from the dynamic

penetrometer correlated to modified Proctor maximum dry density (100% modified Proctor relative compaction). Except for the top lift on the piers, blow counts for the compacted aggregate ranged from 18 to 46, indicating that densities greater than 100% relative compaction were achieved. Blow counts as low as 8 were measured within the upper 150 mm (6 in.) of the piers, primarily as a result of the aggregate being loosened during subsequent footing excavations to the top of the piers. After light surface compaction with a hand-held mechanical compactor, the top of each pier was densified to the same condition as the rest of the pier.

Six months after the building was completed, the general contractor performed a settlement survey on twelve instrumented columns, with a representative of the authors as an observer. The maximum settlement of any footing was 1.6 mm (0.06 in.), and most registered zero. The survey was repeated three times. The Project Manager stated that he had "never before seen shallow foundations behave this well."

Case II: Forty Foot High Milk Silo, Atlanta, Georgia

This project is the smallest aggregate pier project performed to date, but is also one of the most interesting. No preconstruction load test was performed, but the project itself constituted a full-scale footing load test on aggregate-pier reinforced soil since a single footing supported a known vertical load.

A 12 m (40 ft) high, 3.7 m (12 ft) diameter steel silo was designed to store milk at a dairy products company in Atlanta, Georgia. The foundation was a 4.57 m (15 ft) square, 1.22 m (4.0 ft) thick footing, which fit tightly into an L shaped area with two sides of the footing less than 50 mm (2 in.) from an existing two story brick and masonry block building. The subsoils were identified by two SPT borings performed in close proximity on the tiny site. Each boring showed consistently low blow counts (average N = 3) to a depth of 8 m (25 ft), below which blow counts increased to 8 and 10. The soils classified as soft, fine sandy highly micaceous silts (ML). The site was within the Piedmont geological region, and the bearing strata were virgin residual soils. An indication of their high compressibility and low strength was observed in the 20 year old adjacent structure, in which numerous cracks had occurred from excessive settlements.

Three rectangularly prismatic piers, each 0.61 m (2.0 ft) wide and 1.5 m (5.0 ft) deep, were constructed diagonally across the footprint of the footing to the edges of the existing wall footings, exposing limited sections of the footings. Vibrations were minimized by use of a small installation apparatus, which coupled with the relatively high impact frequency of 500 cycles per minute, resulted in low intensity vibrations. Existing cracks in the adjacent building were monitored and none showed any signs of worsening or widening. Dynamic penetration blow counts for the pier aggregate varied from 20 to 42 blows, indicating that densification of aggregate exceeded 100% modified Proctor relative compaction by a considerable margin.

The bearing pressure producing settlement (144 kPa = 3.0 ksf) was accurately calculated based on the weights of the silo and the milk stored within it when filled. The predicted settlement based on this bearing pressure was 13 mm

(0.5 in.). Elevation readings were taken on the footing prior to construction of the silo. Three months after the silo had been filled, a final settlement survey was performed. The three readings varied from 1.5 to 1.8 mm (0.06 to 0.07 in.). Observations of relative displacements of the footing and adjacent concrete pavement also indicated that less than 2.5 mm (0.1 in.) settlement had occurred.

Case III: Industrial Manufacturing Building, Winterset, Iowa

This project represents one of the poorest soil conditions within which aggregate piers have been installed to date. The structure was a large, one story, steel framed manufacturing building with column loads varying from 180 to 800 kN (40 to 180 kips). The soils were soft aeolian silts (loess) that classified as either CL-ML, CL, or CH, overlying stiffer glacial till (CH), with the groundwater table located between 0.08 m and 0.9 m (6 in. and 3 ft) below the ground surface. The loesses were moderately to highly plastic, with plasticity indices as great as 45. The foundation was designed as shallow continuous footings for walls, and isolated footings for columns, and was based on 96 kPa (2.0 ksf) allowable bearing pressure. In addition, overexcavation of footings and backfilling with compacted aggregate had been recommended by the project geotechnical consultant.

When excavation for the footings began, problems occurred almost immediately as walls of the excavations collapsed. The bottoms of the excavations extended below groundwater in many cases, which combined with low strength soils, caused the collapses. Construction was halted while a solution was sought. The two alternatives considered were piles and short aggregate piers. Aggregate piers were selected because of lower cost, shorter time of construction, and lesser time required to redesign the footings.

Two static load tests were performed on 0.61 m (2.0 ft) diameter aggregate piers. In the first test, well-graded stone with a fine to medium sand fraction was used in the first lift before groundwater began to seep into the cavity. None of the overlying lifts were affected by the groundwater. For the second load test, it was decided to make the cavity and pier construction represent a potential worst case scenario. After drilling the cavity, water was poured into the hole to a depth of 0.5 m (18 in.), and left in place for four hours. Water was then pumped out, with a residual level of 80 mm (3 in.) of water remaining. The first aggregate lift consisted of an open graded stone without fine or medium graded sand, which was not sensitive to water. The remaining lifts were well-graded stone including the fine to medium sand fraction that is sensitive to seepage and water content during compaction. Unintentionally, some excess water saturated the lower portions of the second lift during placement. Somewhat surprising was the relatively close results from the two load tests (Fig. 1). The second test was poorer, but the difference was not as great as anticipated. Actual production pier construction proved to be closer in quality to the first test, and no piers were installed under conditions as severe as the second. This was accomplished by leaving each cavity open for short time periods only, and, where necessary, pumping water out of the cavity before placing the aggregate.

Unpublished data from field tests conducted on soils located near the two static load tests by Handy et al. (1993) using the K_o-stepped blade show a substantial increase in soil lateral stress after installation of the aggregate pier down to and several feet below the depth of the bottom of the pier. The lateral stress decreased radially with distance outward from the surface of the aggregate pier, where an extrapolation of the data indicated about a 48 kPa (7 psi) increase in stress at the top of the pier along the interface with the matrix soil, and a total lateral stress of 90 kPa (13 psi), the additional lateral stress being attributed to the expansive nature of the clay.

Borehole shear tests conducted in undisturbed loess at the site gave a drained friction angle of 37° and a cohesion intercept of 10 kPa (200 psf). These data, combined with a unit weight of 14.3 kN/m^3 (91 pcf), indicate that at a depth of 0.91 m (3 ft, the depth to the top of the aggregate pier), the limiting passive pressure was 90 kPa (13 psi), the same as was determined from the stepped blade tests. That limiting passive pressure was induced in the adjacent matrix soil during installation of the aggregate piers also was suggested by observations of radial tensile cracks in the surficial soil adjacent to the piers.

The final design bearing pressure of 192 kPa (4.0 ksf) with an estimated settlement of 23 mm (0.9 in.) was twice the design bearing pressure without aggregate piers. 0.61 m (2.0 ft) and 0.76 m (2.5 ft) diameter aggregate piers were installed, with the nominal height of the piers equal to twice the diameter. Compaction of the soft soil at the bottoms of the cavities typically increased the height of the cavity by about 0.2 m (8 in.). This was accomplished by placing open-graded stone as a first lift and tamping the stone downward, forming an aggregate bulb at the bottom of the pier. The project was completed in nine days. Data provided by the general contractor indicated that the maximum settlement was 19 mm (0.75 in.) six months after completion.

SUMMARY AND CONCLUSIONS

Short aggregate piers, with height to width ratios of 2:1 to 3:1, can be incorporated within existing soils to control settlements under various subsoil and structural loading conditions. In some instances, this solution can be used either as an alternative to overexcavation/replacement or in place of deep foundations. Future plans include

(1) conducting a full-scale load test on a footing supported by three aggregate piers to determine how a group of aggregate piers behaves in comparison to a single pier;

(2) undertaking field tests and numerical analyses to determine the stress distribution characteristics along the interface between the upper reinforced zone and the lower unreinforced zone;

(3) performing field tests and numerical analyses to better define the load-transfer characteristics between the aggregate piers and the adjacent matrix soil; and

(4) conducting additional field tests to better understand the magnitude of lateral stress increase along the pier-matrix interface during installation of the piers,

and how these increased lateral stresses improve the stiffness of the pier-reinforced soil.

ACKNOWLEDGMENT

The assistance of Dr. Richard L. Handy in reviewing the manuscript and providing constructive comments is greatly appreciated.

APPENDIX - REFERENCES

Bowles, J. E. (1988). *Foundation Analysis and Design*, 4th Edition, McGraw-Hill, New York, New York.

Burmister, D. M. (1958). "Evaluation of pavement systems of the WASHO road test by layered system methods." *Highway Research Board Bulletin 177*, 26-54.

Fox, N. S., and Lawton, E. C. (1993). "Short aggregate piers and method and apparatus for producing same," U. S. Patent No. 5,249,892.

Handy, R. L. (1993). *Personal Communication*, September 22.

Lambe, T. W. (1967). "Stress path method." *J. Soil Mech. Found. Div.*, Proc. ASCE, 93(SM6), 309-331.

Leonards, G. A. (1976). "Estimating consolidation settlements of shallow foundations on overconsolidated clays." *Special Report 163*, Transportation Research Board, Washington, D. C., 13-16.

Schmertmann, J. H. (1970). "Static cone to compute static settlement over sand." *J. Soil Mech. Found. Div.*, Proc. ASCE, 96(SM3), 1011-1043.

Schmertmann, J. H., Hartman, J. P., and Brown, P. R. (1978). "Improved strain influence factor diagrams." *J. Geotech. Eng. Div.*, ASCE, 104(GT8), 1131-1135.

Skempton, A. W., and Bjerrum, L. (1957). "A contribution to the settlement analysis of foundations on clay." *Géotechnique*, 7(4), 168-178.