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Control of Settlement and Uplift of Structures Using Short Aggregate Piers

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ABSTRACT: Two case histories are described in which short aggregate piers were used to control settlements and uplift movements. In one project, aggregate piers were effective in substantially reducing settlements of both individual footings and mat foundations. A method for estimating settlements of the aggregate pier-reinforced soil is described, with good correlation shown between predicted and actual settlements. Aggregate piers were used in a second project to provide uplift capacity for an airplane hangar susceptible to high uplift wind forces. The aggregate piers have performed well in winds up to 113 km/hr (70 mi./hr). A theoretical method for estimating the uplift capacity of aggregate piers is proposed.

INTRODUCTION

Frustrated by limitations posed by the overexcavation and replacement method of stabilizing poor or inadequate soils to support footings and control settlements, an effort was begun in 1984 to develop a more practicable, higher capacity method of providing soil reinforcement to support shallow foundations. The method developed was a short aggregate pier system involving the formation of a cavity by drilling or backhoe excavation, and building a highly densified, well-graded aggregate pier in lifts by impact ramming, while simultaneously causing buildup of lateral and vertical soil stresses. Projects utilizing this system since 1988 have included a variety of structures on widely differing soil conditions. In the past three years, with improved apparatuses and installation techniques, several thousand aggregate piers have been installed to support a wide variety of structures by providing settlement control, and in several projects, uplift and horizontal movement control. The system has been expanded to include stiffening and strengthening of good soils. The unique differences inherent within the aggregate pier system compared to other foundation types or ground improvement methods have resulted in the award of a U. S. patent (Fox and Lawton 1993), with international patents pending.

BACKGROUND

The aggregate pier method has been developed as an economical alternative to overexcavation/replacement and to deep foundations in many instances. Its primary use to date has been to control settlements beneath building footings and mats, while providing higher capacity, higher bearing pressure foundation elements. Projects

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completed range from single story structures to sixteen story towers, and from storage silos to airplane hangars. Soils improved with high capacity aggregate piers have included soft and loose sandy silts and silty sands, soft and firm silty clays and clayey silts, organic fills, debris fills, uncompacted or erratically compacted fills, very stiff silts, and medium dense sands. Groundwater conditions have varied from none to groundwater existing within the aggregate pier elements. Aggregate piers have been used on several projects to control uplift. Details of three projects in which aggregate pier foundations were used to control settlements have been described previously (Lawton and Fox 1994).

The major steps involved in creating an aggregate pier within an in-situ (matrix) soil are illustrated in Fig. 1 and summarized as follows: (a) A cylindrical or rectangularly prismatic cavity is formed in the soil using either an auger or a backhoe; (b) the soils at the bottom of the cavity are densified and prestressed by repeated impact from a specially designed tamper with a beveled head; (c) well-graded aggregate (normally highway base course stone) is placed loosely at the bottom of the cavity in a thin lift; (d) the aggregate is highly densified (typically to more than 100% modified Proctor maximum dry density) by repeated ramming from the tamper, which also prestresses the matrix soil laterally; and (e) compacted lifts are added until the desired height is achieved. Columnar aggregate piers have varied in diameter from 0.61 to 0.91 m (24 to 36 in.), while the rectangularly prismatic piers have varied in width from 0.46 to 9.76 m (18 to 30 in.). The height of an aggregate pier is generally between two to three times its diameter or width. The apparatuses used to densify the aggregate have included a small skid loader with a modified hydraulic impact source, a backhoe, and a hydraulic excavator. The diameters of the tamper heads have varied from 0.38 to 0.81 m (15 to 32 in.).

Case Histories

Expansion of Regional Hospital, Atlanta, Georgia

In a $312,000,000 expansion to a regional hospital in Atlanta, Georgia, two major structural towers were added to provide additional office space and hospital rooms. One tower was designed to add twelve stories on top of an existing four story building component, providing a total height of sixteen stories, while the other tower was to bear at the basement level and extend a full sixteen stories. Although each tower was designed with drilled pier foundations, there were distinct foundation installation problems associated with drilled piers for each case.

Subsurface and Construction Conditions, South Tower. The site is within the Piedmont geological province of Georgia. Subsoils in both tower areas, which are within 91 m (300 ft) of each other, were virgin residual soils consisting primarily of firm to very stiff fine sandy micaceous silt and loose to firm silty micaceous sands overlying dense silty micaceous sands and partially weathered rock. The total thickness of the soil strata overlying partially weathered rock and rock varied from about 3.0 to 9.1 m (10 to 30 ft). Standard penetration blowcounts (N) in the soils varied from as low as 4 to above 20, and typically ranged from 9 to 20. The underlying rock was classified as soft to hard gray and white biotitic gneiss. Groundwater at the time of drilling was found at elevations 0.9 m (3 ft) above finished floor elevation in one boring, and 1.2 to 4.6 m (4 to 15 ft) below finished floor elevations in other borings.

The foundations for the South Tower (twelve story) had to be installed within an existing basement. The ceiling height of 4.9 m (16 ft) presented one limitation. Access to the site was limited to a wall opening about 2.4 m (8 ft) wide and 3.0 m (10 ft) high. Furthermore, access to the opening was by a steep ramp with a 1.3H:1.0V slope. Alternatively, equipment could be lifted down to the construction

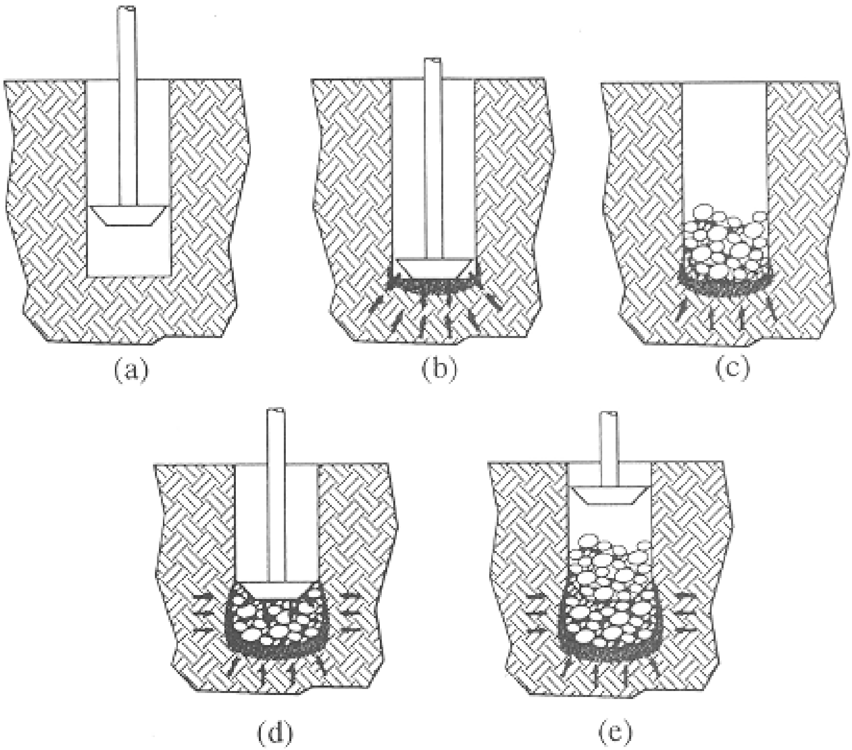

FIG 1. Steps in Construction of an Aggregate Pier

level by crane. Because of the equipment height and size limitations, hand-dug drilled piers were initially planned.

The initial plan to install 0.91 m (36 in.) diameter columnar piers was modified when it was learned that the lifting crane could not handle the drill rig. Rectangularly prismatic (linear) aggregate piers with widths of 0.61 m (24 in.) were therefore substituted for the columnar piers. The areal coverage of the aggregate piers was about one-third of the total footing area. The aggregate piers were installed by excavating a trench about 1.83 m (6 ft) below the bearing elevation and compressing the soils at the bottom of the trench to a depth of about 0.15 m (6 in.), resulting in a total height of the piers of about 1.98 m (6.5 ft).

Subsurface and Construction Conditions, North Tower. The foundations for the North Tower (sixteen stories) had to be installed within a 6.1 m (20 ft) deep basement excavation braced with a tieback retention system. Access to the basement was provided by an access ramp with about the same slope as for the South Tower. Drilling equipment for the drilled piers was to be lifted by crane onto and off of the site.

The geology and subsoil conditions were essentially the same as for the South Tower with the following exceptions: (1) Groundwater existed 0.37 m (18 in.) below the ground surface of the basement excavation; (2) the depth to rock was greater; and (3) results of dynamic penetration tests on the subsoils showed that the lower consistency sandy silts and silty sands extended only approximately 1.5 m (5 ft) in depth and that the underlying subsoils were stiff to very stiff and firm to medium dense below this depth.

The 24 hour time period immediately prior to installation of the aggregate piers produced a record rainfall of 102 mm (4.0 in.). The site, being a deep excavation adjacent to the existing hospital, offered no drainage except seepage into the subsoils. As a result, the upper subsoils were saturated to varying depths, resulting in "pumping" of both wheeled and tracked construction equipment. To stabilize these soils to support wheeled skid loader tamping and front loading equipment, and to provide protection against trench cave-ins resulting from wheel loads near the edge of the pier excavations, the authors elected to install geogrid on the surface with 102 mm (4 in.) of #57 stone on top. This performed well, stopping the subgrade pumping and preventing cave-ins during installation.

Because of strict schedule constraints and the threat of additional hard rains, the installation of the aggregate piers was continued non-stop using two 12-hour shifts. The total project was completed within a 48 hour time frame. Several hours after completion, the predicted heavy rainfall occurred. The highly densified aggregate piers were not degraded by the action of this rain.

Settlement Analyses. Idealized geologic profiles for the South Tower footings and the North Tower mat are shown in Fig. 2, along with pertinent engineering properties of the strata. In the South Tower, structural loads for the columns supported on aggregate pier-reinforced soils varied from 1.69 to 4.40 MN (380 to 990 kips). The aggregate pier system was designed to provide a maximum design bearing pressure of 5.0 ksf, with resultant footing sizes ranging from 2.74 m (9 ft) square to 3.66 by 5.03 m (12 by 16.5 ft). The foundation system for the North Tower was originally designed as isolated columns and grade beams supported by drilled piers. This system was replaced by an aggregate-pier supported mat with an average bearing pressure of about 144 kPa (3.0 ksf), and a maximum bearing pressure within the heavier loaded portion of about 215 kPa (4.5 ksf).

To estimate the settlement of a shallow foundation bearing on an aggregate pier-reinforced soil, the subgrade is divided into an upper zone (UZ) and a lower zone (LZ). The upper zone is assumed to consist of the composite soil comprised of the aggregate piers and matrix soil, plus the zone of appreciable densification and prestressing immediately underlying the pier, which is estimated to be equal to the width of one pier. For this case, the piers were 1.98 m (6.5 ft) high and 0.61 m (24 in.) wide, with the height of the upper zone equal to 2.59 m (8.5 ft). The lower zone consists of all strata beneath the upper zone. Settlements are calculated individually for the UZ and LZ, with the two values combined to yield an estimate of the total settlement. Using the analyses to be discussed subsequently, predicted settlements were calculated for both the smallest and largest footings in the South Tower. The mat for the North Tower was 15.2 by 30.5 m (50 ft by 100 ft), with about 75% of the mat loaded to 144 kPa (3.0 ksf) and about 25% loaded to 215 kPa (4.5 ksf). Settlement of the mat was calculated separately for the two distinct zones with different applied loads. Since rock was so close to the surface (ratio of soil thickness to width of the loaded area was small), varying the assumed size of the different zones of the mat had little influence on the predicted settlements. Therefore, settlements of the lighter loaded zone were calculated based on the full

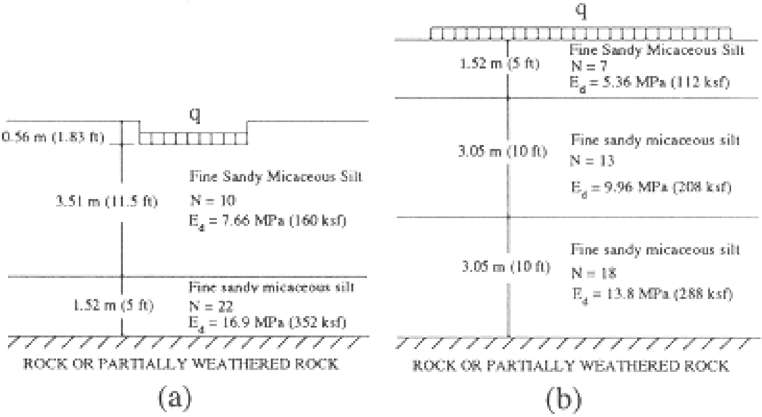

FIG. 2. Idealized Geologic Profiles for Hospital Project: (a) South Tower; (b) North Tower

dimensions of the mat, and settlements of the heavier loaded zone were calculated based on dimensions of 7.6 by 15.2 m (25 by 50 ft).

Over the past six years, the authors have conducted numerous settlement analyses for the UZ using the finite grid method (Bowles 1988), which have shown that little error is introduced in the settlement calculations by assuming that the footing is perfectly rigid. Using this assumption and a subgrade modulus approach, the following equations apply:

q_p = bearing stress applied to aggregate piers = $q \cdot R_s / (R_a \cdot R_s - R_a + 1)$ .... (1)
q_m = bearing stress applied to matrix soil = q_p / R_s .... (2)
S_{uz} = settlement of the UZ = $q_p / k_p = q_m / k_m$ .... (3)

where q = average design bearing pressure = Q / A; R_s = subgrade modulus ratio = k_p / k_m; R_a = area ratio = A_p / A; Q = vertical design load at the bearing level; A = total area of footing; A_p = total area of aggregate piers supporting footing; k_m = subgrade modulus for matrix soil; and k_p = subgrade modulus for aggregate piers.

Values of subgrade moduli for the aggregate piers are determined either by static load tests on individual piers or by estimation from previously performed static load tests within similar soil conditions and similar aggregate pier materials and installation methods. This is considered conservative since the static load tests do not consider the beneficial effect of confining pressures produced from the loaded footing acting on the matrix soil. Subgrade moduli for the matrix soils are either determined from static load tests or estimated from boring data and allowable bearing pressures provided by geotechnical consultants. Because the aggregate piers are typically 10 to 20 times as stiff as the matrix soil, the value of subgrade modulus for the matrix soil generally has only a minor influence on the estimated settlement within the UZ.

For this project, no load tests were conducted on either the aggregate piers or the matrix soil, so values of subgrade moduli for both materials were estimated. From the results of over thirty static load tests on aggregate piers installed within matrix soils from the Piedmont geological region, values of k_p for aggregate piers installed

Table 1. Predicted Upper Zone Settlements

Location and Foundation Type	Plan Dimensions of Foundation	q kPa (ksf)	R_a	q_p kPa (ksf)	q_m kPa (ksf)	S_{uz} mm (in.)
South Tower square footing	2.74 m (9 ft)	225 (4.7)	0.35	563 (11.8)	42 (0.88)	7.4 (0.29)
South Tower rectangular footing	3.66 x 5.03 m (12 x 16.5 ft)	239 (5.0)	0.33	630 (13.1)	47 (0.99)	8.3 (0.33)
North Tower mat	15.2 x 30.5 m (50 x 100 ft)	144 (3.0)	0.29	418 (8.7)	31 (0.66)	5.5 0.22
North Tower mat	7.6 x 15.2 m (25 x 50 ft)	215 (4.5)	0.29	628 (13.1)	47 (0.98)	8.3 (0.33)

Note: For all cases, k_p = 76 MN/m^3 (280 pci), k_m = 5.8 MN/m^3 (21 pci), and R_s = 13.3.

in similar types of subsoils have varied from 49 to 190 MN/m^3 (180 to 700 pci). Based on load tests from sites with similar soils and blowcounts, the k_p for this project was estimated at 76 MN/m^3 (280 pci). Using an allowable bearing pressure of 144 kPa (3.0 ksf) based on a tolerable settlement of 25 mm (1 in.), k_m was estimated as 5.8 MN/m^3 (21 pci). Using R_s = 13.3 and these values for k_p and k_m, as well as the values for q and R_a shown in Table 1, values for q_p, q_m, and S_{uz} for both footings were calculated from Eqs. 1, 2, and 3 and are summarized in Table 1.

An estimate of the applied stresses transmitted to the interface between the UZ and the LZ is needed so that predicted settlements in the LZ can be calculated. Burmister's (1958) work on two-layered elastic strata of infinite horizontal extent clearly showed that the presence of a stiffer upper layer substantially reduces the applied stresses transmitted to the lower, more compressible layer compared to the case of a homogeneous soil. For example, for a uniform circular load, $E_1 / E_2 = 10$, and a thickness of the upper layer equal to the radius of the loaded area, the vertical normal stress beneath the centerline of the loaded area at the interface between the two materials is about 30% of the applied stress, compared to about 65% for a homogeneous soil (Boussinesq-type analysis) at the same depth.

The procedure used by the authors to estimate vertical stress increase at the UZ-LZ interface is a modification of the 2:1 method, and involves the use of engineering judgment. Use of this type of method is readily applicable to settlement calculations of the LZ because it provides an estimate of uniform vertical stress increase at the UZ-LZ interface. For estimates of the lower zone settlements (S_{lz}) for this project, a stress dissipation slope through the UZ of 1.67:1 is used.

To estimate settlements of the lower zone, and to estimate settlements for comparable footings without aggregate piers (S_{un}), both Schmertmann's (1970; 1978) strain distribution method and Bowles' (1988) modified elastic theory method are used. Based on comparisons of SPT blowcounts in Piedmont residual soils versus Young's modulus values backcalculated from actual settlements and estimated from plate load and other in-situ tests, the authors have found that most values for drained Young's modulus fall within the following range: E_d (kPa) = (383 to 1149)·N [E_d (ksf) = (8 to 24)·N], where N is the field blowcount (not corrected for overburden pressure). Although there is considerable scatter in the data, the best straight-line fit to the data is E_d (kPa) = 766·N [E_d (ksf) = 16·N], and this relationship has proved satisfactory for most settlement estimates in Piedmont residual soils and is used herein. The results of these calculations are summarized in

Table 2. Predicted Settlements for Lower Zones and Unreinforced Matrix Soils

	Predicted Settlement, mm (in.)			
	Lower Zone, S_{lz}		Unreinforced, S_{un}	
Location and Foundation	Schmert.	Bowles	Schmert.	Bowles
South Tower square ftg.	0.8 (0.03)	4.6 (0.18)	43.2 (1.70)	40.4 (1.59)
South Tower rect. ftg.	1.5 (0.06	7.9 (0.31)	51.8 (2.04)	58.2 (2.29)
North Tower large mat	7.4 (0.29)	32.5 (1.28)	28.7 (1.13)	70.6 (2.78)
North Tower small mat	15.7 (0.62)	39.6 (1.56)	61.5 (2.42)	98.3 (3.87)

Table 3. Predicted and Actual Settlements

	Settlement, mm (in.)		
	Predicted		
Location and Foundation	Unrein. Matrix Soil	with Aggr. Piers	Actual
South Tower sq. ftg.	40 - 43 (1.6 - 1.7)	8 - 12 (0.3 - 0.5)	< 6 (< 0.25)
South Tower rect. ftg.	52 - 58 (2.0 - 2.3)	10 - 16 (0.4 - 0.6)	< 10 (< 0.4)
North Tower large mat	29 - 71 (1.1 - 2.8)	13 - 38 (0.5 - 1.5)	< 10 (< 0.4)
North Tower small mat	62 - 98 (2.4 - 3.9)	24 - 48 (0.9 - 1.9)	< 20 (< 0.75)

Table 2. Both methods gave comparable values for unreinforced settlements of the South Tower footings. In the other six cases, Bowles' method yielded predicted settlement values significantly higher than Schmertmann's method; values calculated for the mats using Bowles' method seem especially high.

The total predicted settlements, with and without aggregate piers, are shown in Table 3 along with the actual settlements. In all four cases, the minimum value of predicted settlement with aggregate piers (using Schmertmann's value for the LZ) is slightly larger than or equal to the maximum value of actual settlement, suggesting that the settlement method used by the authors gives reasonable estimates. Values of predicted settlement with and without aggregate piers indicate that the aggregate piers were effective in reducing both total and differential settlements.

Mississippi Air National Guard Hangar, Meridian, Mississippi

A state-of-the-art hangar with massive doors that fold up, much like venetian blinds, was built at an Air National Guard field in Meridian, Mississippi. Owing to the open door space, design uplift forces from wind loads were as great as 1,156 kN (260 kips) per column. Helical screw anchors were considered and initially bid, but an alternative anchoring system was sought because the helical anchors presented two problems: (1) Difficulty in locating the anchor shafts within specified tolerances; and (2) a significantly higher cost than was previously budgeted.

Since December 1991, the authors have successfully used aggregate piers as hold-down anchors during compressive static load tests on aggregate piers, with two significant characteristics observed during tests conducted in silty sands and sandy silts: (1) The uplift capacity of an aggregate pier was significant; and (2) in 31 of 32 piers where uplift deflections were measured, the rebound upon removal of the load was 100%. These results suggest that the uplift loads were transferred primarily as shear stresses along the aggregate pier-matrix soil interface, and that the stresses were within the elastic range for the interfacial materials. The maximum uplift forces per aggregate pier in these tests were typically between 200 to 214 kN (45 to 48 kips), with measured uplift deflections mostly less than 25 mm (1.0 in.) and always less than 51 mm (2.0 in.), and in no cases did failure occur. The results from uplift tests in sandy clays have shown less than 100% rebound, indicating some plastic soil behavior.

Geological Background. The Key Field Hangar site is located within the Coastal Plain geologic region, and identified within the Wilcox formation. The sedimentary soils in this region typically consist of complexly interbedded clays and sands, and are usually underlain by sandstone or limestone. Subsoils within the hangar site included an upper zone of well-compacted, well-graded sand fill extending from the graded surface to depths of about 0.9 to 1.8 m (3 to 6 ft). Available test data indicated that the density of the sand fill was approximately 98% of standard Proctor maximum, with $N \approx 15$. Underlying the recent sand fill was a zone of primarily loose clayey sand varying from 1.2 to 2.7 m (4 to 9 ft) thick. Unconfined compressive strengths in the clayey sand varied from 48 to 192 kPa (1.0 to 4.0 tsf), and more typically from 72 to 96 kPa (1.5 to 2.0 ksf). Beneath this was a stratum consisting of stiff fine sandy silt and medium dense silty fine sand extending to a depth of 7.6 m (25 ft). Groundwater was at depths of about 1.5 to 2.1 m (5 to 7 ft), and near or within the bottoms of the installed piers.

The subsoil profile at the location of the uplift load test consisted of 1.4 m (4.7 ft) of medium dense, well-graded sand fill overlying a zone of loose, very clayey sand. The groundwater was 0.3 m (1 ft) above the bottom of the pier excavation, at a depth of about 1.8 m (6 ft). Consolidated-drained strength parameters determined from borehole shear tests performed within the aggregate pier test zone prior to excavating the pier cavity were $\phi = 45^{\circ}$ and $c = 0$ for the compacted sand fill, and $\phi = 20^{\circ}$ and $c = 0$ for the clayey sand.

Uplift Load Test. Rectangularly prismatic aggregate piers were used instead of columnar piers because of subsoil conditions and anticipation of some limited cave-in situations. It was also felt that if the uplift capacity were less than anticipated, greater aggregate pier coverage and depth would be more readily available with backhoe excavation than drilled hole excavation.

The test aggregate pier was 1.8 m (6 ft) high, 0.61 m (24 in.) wide, and 1.5 m (5 ft) long, with the top of the pier at a depth of 0.3 m (1 ft). The uplift loads were transferred to the bottom of the pier by steel tension rods located along the perimeter of the pier, which were attached to a continuous steel plate at the bottom of the pier.

The load test was performed essentially in accordance with ASTM D1194. A total of eight loading increments were applied, with an average increment of 67 kPa (15 kips) each. The time between loading increments was 15 minutes; for each increment the deformation rate after 15 minutes was less than 0.25 mm (0.01 in.) per minute. The maximum load of 267 kN (60 kips) was held for five hours. As can be seen in Fig. 3, the deflection at the maximum load was 23 mm (0.91 in.), and the load-deflection curve is fairly linear. In addition, 100% rebound was measured upon release of the load, indicating that the soil behaved elastically within the range of stresses applied in the test. From the results from this load test, a design capacity of 178 kN (40 kips) per aggregate pier was approved.

Theoretical Uplift Capacity. The purpose of the following theoretical analysis is to provide a plausible explanation for the unusually high pullout strength and elastic behavior of the load test aggregate pier. Prior to performing this theoretical analysis, it is necessary to consider the changes in stress which occur in the matrix soil and the aggregate pier during the construction process. The in-situ soil is initially in an at-rest condition and if the ground surface is fairly flat, it is reasonable to assume that the major principal stresses are horizontal and vertical. After the cavity is excavated, the horizontal stress reduces to zero (with capillary suction keeping the hole open), while the vertical stress remains approximately constant, and it is reasonable to assume that the vertical face of the cavity is a principal plane because there is no applied shear stress. During construction of the aggregate pier, horizontal stresses are established along the aggregate pier-matrix soil interface.

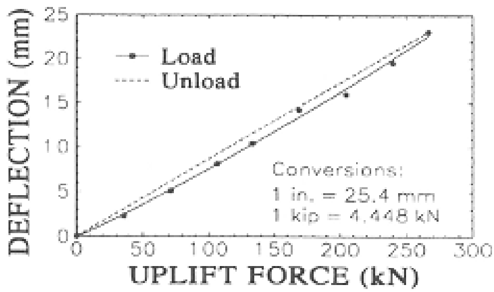

FIG. 3. Uplift Load Test Results from Meridian, Mississippi Hangar Project

Results from K_o-blade tests at other sites have indicated that full passive pressure can be achieved in the matrix soil (Lawton and Fox 1994). If it is assumed that no shear stresses are developed along the interface (so that the principal planes remain vertical and horizontal), and that full passive pressure is developed in the matrix soil, the interfacial horizontal stress can be estimated as $\sigma_h = \sigma_v \cdot K_p$, where $K_p = \tan^2 (45° + \phi / 2)$ (assuming c = 0); and ϕ = friction angle of the matrix soil. The stress state within the interfacial matrix soil is represented by Mohr's circle "a" in Fig. 4. As uplift force is applied to the aggregate pier, shear stresses develop along the vertical plane so that a rotation in the principal planes occurs and a major principal stress arch (Handy 1985) develops within the matrix soil as shown in Fig. 4. A similar stress arch would develop within the aggregate pier. Assuming that failure will occur along the interface and that the interfacial horizontal stress remains constant up to failure, the state of stress at failure would be represented by circle "b," and the interfacial shear stress at failure can be found from $\tau_{ff} = \sigma_h \cdot \tan \phi$. For this scenario to be correct, the interfacial vertical stress would have to increase from σ_{va} to σ_{vb}. Although it is reasonable to expect that some increase in interfacial vertical stress will occur as the vertical shear stress develops, it is not known if σ_{vb} is sustainable. If a lesser value of vertical stress is sustainable (σ_{vc}), a concomitant reduction in horizontal stress (σ_{hc}) would also occur, and τ_{ff} would be less than for the case of constant horizontal stress. Unpublished results from K_o-blade tests conducted by the third author adjacent to a model pile for the Talmadge Memorial Bridge in Savannah, Georgia may support the possibility of a reduction in horizontal stress during loading. The model pipe pile was driven open-ended, and was cleaned out and filled with concrete. The soil was at full passive pressure, apparently as a result of expansion when hit by sea water. As the pile was loaded in compression, it failed prematurely by plunging. As the load was increased, the horizontal stress decreased, explaining the 76 mm (3 in.) plunge to relieve the load. However, additional research and testing must be conducted to determine if the interfacial horizontal stress decreases when an aggregate pier is subjected to an uplift force, and what is the magnitude of the decrease (if any).

Using the previous discussion as the basis, the theoretical uplift capacity of the load test aggregate pier can calculated using the information shown in Fig. 5 and the following equation:

$$T_{max} = 0.5p[(\sigma_{h1} + \sigma_{h2}) \cdot H_1 \cdot \tan \phi_1 + (\sigma_{h3} + \sigma_{h4}) \cdot H_2 \cdot \tan \phi_2] + W \quad \text{......................} (4)$$

where p = length along horizontal perimeter of aggregate pier; σ_{h1} = interfacial horizontal stress at top of aggregate pier; σ_{h2} = interfacial horizontal stress in sand fill along boundary with clayey sand; σ_{h3} = interfacial horizontal stress in the clayey

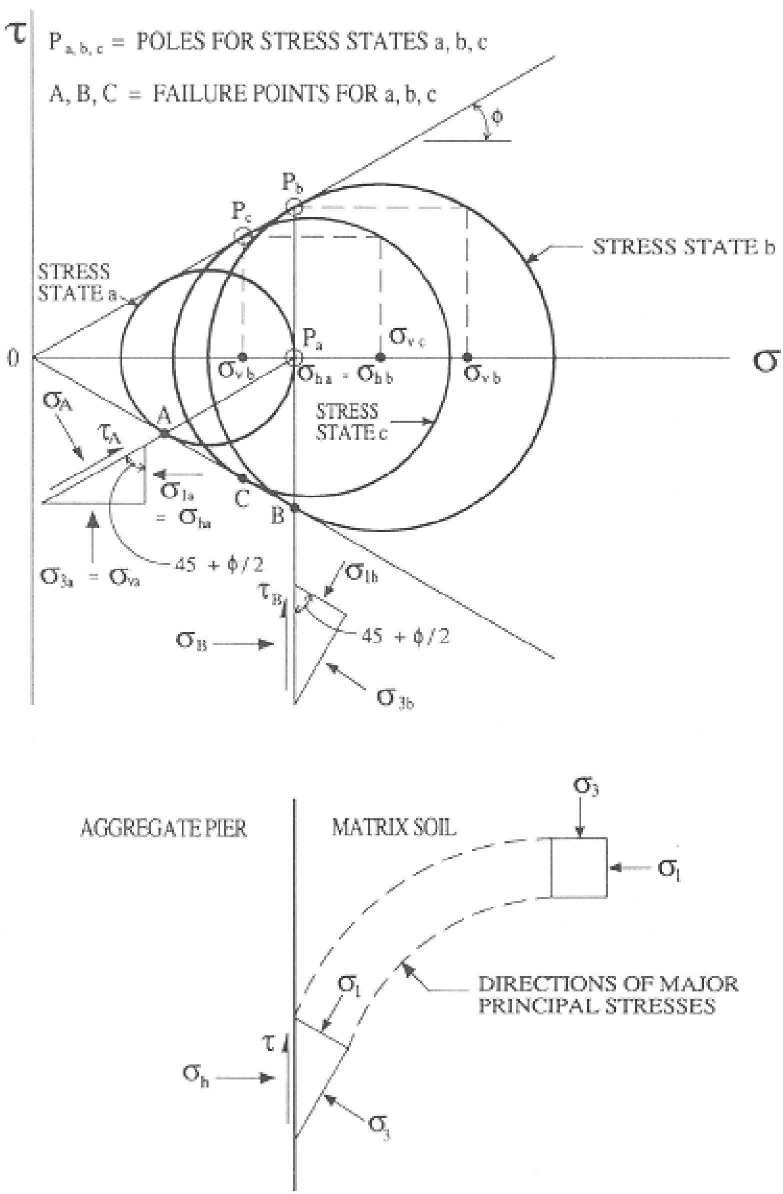

FIG. 4. Theoretical Stress States During Uplift of an Aggregate Pier

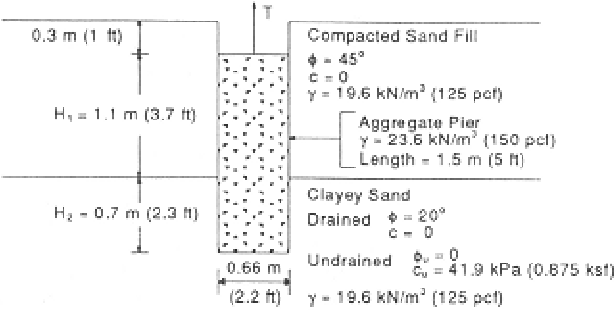

FIG. 5. Schematic Diagram of Uplift Load Test on Aggregate Pier

sand along boundary with sand fill; σ_{h4} = interfacial horizontal stress at bottom of aggregate pier; and W = weight of aggregate pier.

Assuming that full passive pressure was developed in the matrix soil during construction of the aggregate pier, and that the horizontal stress remained constant during the uplift, T_{max} is calculated as 663 kN (149 kips) short-term and 614 kN (138 kips) long-term. Both the short-term and long-term values are substantially greater than the maximum uplift force of 267 kN (60 kips) applied during the load test, and would be consistent with the small, essentially elastic deflections measured during the test. Whether these values overestimate or underestimate the actual maximum uplift capacity is not known since the uplift load test was not taken to failure. As discussed previously, it is possible that a reduction in horizontal stress may have occurred during the uplift process, resulting in a lower uplift capacity than predicted. However, two other factors tend to result in an underestimation of the uplift capacity: (1) The friction angles used in the analysis were obtained in the in-situ soils prior to installation of the aggregate piers. Borehole shear tests conducted on other aggregate pier projects have shown that installing an aggregate pier densifies the adjacent matrix soil, with a concomitant increase in shear strength; and (2) the actual dimensions of the aggregate pier were greater than nominal because this densification.

As a hypothetical comparison, a similar theoretical analysis can be conducted to show that the uplift capacity of an aggregate pier should be substantially greater than the uplift capacity of a poured concrete anchor with the same dimensions. The key factor is the difference in horizontal stress developed along the pier-matrix interface during installation. Assuming the concrete is a viscous liquid, the horizontal pressure applied to the matrix soil is $\sigma_h = \gamma_{conc} \cdot z$, where γ_{conc} = unit weight of the wet concrete; and z = depth of concrete above the point being analyzed This corresponds to a lateral coefficient for the concrete of K = 1 (based on γ_{conc}), compared to K_p = 5.83 for the sand fill and K_p = 2.04 for the clayey sand. If a stiff concrete mix is used, voids and irregularities along the interface with the matrix soil may result in lower horizontal stresses than predicted using this analysis. Using the same assumptions as for the aggregate pier and K = 1, the maximum uplift capacity for a poured concrete anchor with the same dimensions is 236 kN (53 kips) short-term (assuming an adhesion factor of 1.0 in the clayey sand) and 147 kN (33 kips) long-term. Thus, the theoretical uplift capacity of the aggregate pier is 2.8 times (short-term) and 4.2 times (long-term) that for the poured concrete anchor. Future full-scale uplift tests on aggregate piers and poured concrete anchors for the same subsoil conditions are planned to determine the validity of this theoretical procedure.

Performance of Aggregate Piers. Windstorms within the area of the site have been recorded to be as high as 97 to 113 km/hr (60 to 70 mi./hr) since the hangar structure was erected. No measurable uplift displacements have been recorded. Footing settlements were surveyed after erection of the structural steel and prior to the roof and door construction. No settlements were measurable with surveying instruments accurate to 0.25 mm (0.01 in.).

SUMMARY AND CONCLUSIONS

Two case histories in which short aggregate piers were used have been described. In the first project, an expansion of an existing hospital, aggregate pier-reinforced soils supporting both individual footings and mat foundations were effective in substantially reducing settlements. A method for estimating settlements of the aggregate pier-reinforced soils was described, with good correlation shown between predicted and actual settlements. Aggregate piers were used in the second project to provide uplift capacity for an airplane hangar susceptible to high uplift forces resulting from wind loads. The aggregate piers have performed well in winds up to 113 km/hr (70 mi./hr). Using a proposed theory, it was shown that the uplift capacity of aggregate piers is substantially greater than that of poured concrete anchors with the same dimensions. Future research and full-scale field testing is planned to establish the validity of the theoretical procedure.

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