Design of Short Aggregate Piers to Support Highway Embankments

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ABSTRACT

Construction of new embankments over soft soils and rapid widening of existing roadways and embankments can create stability and differential settlement problems. Past approaches to mitigate this problem include overexcavation and replacement, preloading, use of lightweight fill, driving piles and installing concrete caps, various geosynthetic reinforced soil—pile supported embankment systems, stone columns, lime/cement columns and deep soil mixing. Advantages and limitations of these approaches are well documented in the literature. An alternative approach to support highway embankments that is less well documented, but has seen increased use in recent years, is short aggregate piers. Although the technique has been used for support of shallow spread footings, design approaches and parameter values have not been widely available for embankment support applications and are not well known in the field. This paper summarizes the engineering properties and an approach used to design short aggregate piers for the support of highway embankments and describes mechanisms of load-settlement behavior. Techniques for evaluating overall stability, control and time rate of settlement, and bearing capacity are summarized. Performance measurements are referenced for embankment support projects recently constructed in Iowa. This paper provides a guide that may be used by engineers in the evaluation of an emerging technology for the support of highway embankments. *(207 words)*

Keywords: embankment support, soil improvement, aggregate piers, slope stability, settlement, bearing capacity

INTRODUCTION

A common occurrence on transportation rehabilitation and improvement projects is the addition of new traffic lanes, which requires the widening of existing embankments. One important problem for this type of construction is that the added loads due to the increase in cross-section of the embankment can cause additional and differential settlement below existing roadways and pavements, and below the newly added embankment sections. Embankment foundation treatments, such as overexcavation and replacement, preloading, use of lightweight fill material (*1*), driving piles and installing concrete caps, various geosynthetic reinforced soil—pile supported embankment systems (*2,3,4*), stone columns (*5*), lime/cement columns (*6*) and deep soil mixing (*7*), may offer effective and economical solutions to these problems. Some factors involved in selecting a suitable treatment method include:

- Additional construction costs,
- Safety and public relations,
- Future maintenance costs,
- Environmental considerations,
- Foundation stability during construction,
- Tolerable postconstruction total and differential settlements, and
- Construction time available (*8*)

Continued development of new methods of effective and economical foundation treatment systems is vitally needed to meet the needs of the expanding highway infrastructure. By providing alternative foundation treatment solutions, engineers and design-build contractors have the benefit of selecting the best treatment for project specific conditions in accordance with the aforementioned selection factors. An emerging soil improvement method that is increasingly being used in transportation construction is short aggregate piers.

Recently, short aggregate piers have been used to support a wide range of transportation structures in Iowa including: embankments; retaining walls; bridge approach fill; a box culvert; and an unstable pavement section (*9*). The increased use of this technology appears to follow a number of successful commercial projects over the last 10 years and the growing need of highway engineers to provide design solutions that result in rapid construction. This paper provides design guidelines used to reinforce embankment foundation soils with short aggregate piers.

CONSTRUCTION PROCESS

The piers are installed by drilling 0.6 m to 0.9 m diameter holes and ramming thin lifts of well-graded aggregate (GW) within the holes. For embankment support applications the drilled holes typically extend from about 2 m to 8 m below grade and center-to-center pier spacings range from about 1.8 m to 4.0 m. Below the groundwater table elevation, open-graded aggregate (GP) can be used to minimize fines contamination, and in unstable soils temporary casing and mud drilling may be used to provide sidewall stability.

The first lift of compacted aggregate forms a bulb below the bottom of a pier (estimated at a depth of about one pier diameter below drill depths). Compaction for the bottom bulb is achieved in 20 to 30 seconds using a beveled tamper, but depends on the soil stiffness and the aggregate compactibility. A hydraulic hammer striking with a frequency of about 5 Hz facilitates the compaction process.

After constructing the bottom bulb, subsequent lifts are nominal 300 mm in thickness and also rammed for 15 to 20 seconds. During ramming, the beveled tamper both compacts the aggregate and forces the aggregate laterally into the sidewalls of the hole. Others have shown that this action increases the lateral stress in surrounding soil (*10,11*). After the desired compaction time, field quality control and quality assurance (QC/QA) operations can be carried out using a dynamic cone penetrometer test (DPT) and the bottom stabilization test (BST). The DPT is used in general accordance with ASTM STP 399 (*12*) to verify densification near the top of the pier. A DPT measurement of ≥ 15 blows per 4.4 cm is generally considered adequate. The BST is used to monitor compaction near the bottom of the pier and is performed by subjecting a compacted lift to an addition 15 seconds of compaction time and measuring vertical compression of the lift. Acceptable BST results can be calibrated from a project load test pier and typically vary from about 2 to 4 cm. On average, a three-person crew consisting of a driller, hammer operator, and loader install 40 to 60 piers per day.

TEST RESULTS SHOWING EFFECTS OF CONSTRUCTION

The construction process results in the following engineering properties:

Pier Shear Strength Parameters

The shear strength of the placed and compacted aggregate has been measured using: (1) in-situ direct shear strength tests using a ring setup (*13*); and (2) laboratory triaxial tests conducted on a wide range of compacted aggregates (*14*). Test results are shown in Figure 1. Direct shear (i.e. plane-strain) results indicate friction angle, ϕ, varies from 49 to 52 degrees with no cohesion, c. For a wide range of natural and recycled aggregates, the triaxial compression (i.e. symmetric strain) tests yield average values of ϕ = 48 degrees and c = 30 kPa. The source of apparent cohesion is aggregate interlock. To simulate field conditions, triaxial test specimens were prepared using an impact compaction method that resulted in an average relative density of about 140 percent.

Increase in Lateral Earth Pressure

Field measurements show that the radial expansion that occurs during compaction induces lateral strain in the matrix soil surrounding the piers. Measured profiles of inclinometer tube deflections before and after pier installations for a 0.76 m diameter by 3 m long pier installed in soft clay (CL, undrained shear strength, s_u = 31 kPa measured from undrained unconsolidated triaxial compression tests) are presented in Figure 2. At a radial distance of 0.17 m from the edge of the pier and 0.7 m below the drill depth where the bottom bulb is formed, lateral displacement was about 8 mm.

In addition to evidence from inclinometer profiles, lateral displacements were measured at a different site by comparing the drill hole diameter and the constructed pier diameter. Drilled holes with an initial diameter of 0.76 m were observed to increase to about 0.84 m in soft clay (CL, s_u = 28 kPa measured from consolidated undrained triaxial compression tests). Handy and White (*10*), Lawton and Merry (*15*), and White *et al.* (*16*) have also recorded lateral stress measurements by using the K_o-stepped Blade Test and the Pressuremeter Test at project sites in Iowa (CH over CL), Utah (interbedded CL/CH/ML/SC), and Tennessee (CL). Maximum lateral stress measurements from several tests averaged about 120 kPa. Lateral stress increases have also been inferred from back-calculation performed on piers subject to uplift loads and show similar values (*17*). The measurements and back-calculations support the concept that at radial distances close to the piers, the lateral stress regime may be characterized by a passive pressure condition at shallow depths, underlain by a zone of compacted and remolded (densified) matrix soil.

Reduction in Matrix Soil Compressibility

Handy (*18*) suggests that an increase in lateral earth pressure results in a reduction in soil compressibility. The concept is that the lateral earth pressure increase effectively renders the matrix soil into an overconsolidated state having a vertical compressibility smaller than that for normally consolidated soils.

The discussion is presented for plane-strain conditions. Further research concentrating on axisymmetric conditions (*10*) suggests that the lateral stress regimes are more complex than originally thought. Tangential stress in the radial coordinate system plays an important, yet unresolved, role in the behavior of the soil and needs further investigation. For this reason, calculations performed to estimate the composite compressibility of the soil to the depth reinforced by piers do not generally include a reduction in matrix soil compressibility. However, matrix soil coupling to piers does play an important role in settlement calculations described below.

PIER BEHAVIOR

Load-Displacement

The construction process results in the formation of stiff pier elements. The stiffness of the piers and the behavioral modes associated with deflection can be measured using a full-scale load test. Two general modes of deformation have been observed: (1) bulging of the pier with little bottom movement and (2) plunging of the pier with mobilized skin friction and tip resistance. The mode of deformation depends mainly on the pier length and shear strength/confinement provided by the matrix soil.

Figure 3(a) shows the results of a load test for a 0.76 m diameter by 5.4 m long pier installed (floating) in soft clay (CL, s_u = 30 kPa measured from consolidated undrained triaxial compression tests) to support a highway embankment fill on Interstate-35 near Des Moines, Iowa. A telltale was installed at the bottom of the pier and indicates that pier bulging is the dominant mode of deformation. At top-of-pier stresses greater than about 300 kPa, the incremental slope of the top-of-pier deflection curve is steeper than for lower stress levels, which is an indicator of the onset of pier bulging. This behavior results in load transfer to the matrix soil by side friction along the shaft.

Figure 3(b) shows the results of a load test for a 0.76 m diameter by 2.7 m long pier installed (floating) in soft clay (CL, s_u = 10 to 20 kPa measured from CPT and DMT) on Hwy 191 near Neola, Iowa to support a large box culvert. At stresses greater than about 280 kPa, the top and bottom of the pier exhibit an increase in incremental deflection. This behavior indicates the onset of bulging, but also plunging of the pier. These results suggest that side friction along the shaft and tip stresses at the bottom of the pier are induced at applied stresses greater than 280 kPa. Detailed site conditions for both load tests are described in White *et al*. 2003 (*9*)

Stress Dissipation

Stress that is applied at the tops of the piers dissipates quickly with pier depth. Figure 4 presents instrumentation results of total stress cell measurements for cells installed at depth within a 0.76 m by 3.0 m long pier. The pier was installed to "float" in soft clay (s_u = 20 kPa) rather than extend to a stiff layer. Results show that 50 percent of the load is dissipated at a depth equal to 1 to 2 times the pier diameter (D) while about 80 percent of the load is dissipated at 3 times the pier diameter. Lawton and Merry (*15*) show similar trend for a 0.91 m diameter by 4.6 m long pier.

Stiffness

The load test plot is used to determine pier stiffness, which is used in settlement computations. The slope of the top-of-pier stress (q_p) versus top-of-pier deflection (δ) is defined as the pier stiffness (k_p):

$$k_p = \frac{q_p}{\delta} \qquad (1)$$

From a database of over 300 load tests in the U.S., correlations to soil type and consistency (i.e. SPT N-value, unconfined compressive strength) show that k_p values for design range from about 20 kPa/mm to 100 kPa/mm (*13*). Further, elastic modulus values have been estimated at about 90 MN/m^2 to 190 MN/m^2 (*19,20*). As a comparison, reported modulus values for other foundation systems are provided in Table 1.

Stress Concentration

Compared to the matrix soil, the high stiffness of the piers results in stress concentration to the tops of the piers during compression loading. For cases where a concrete load-transfer platform is placed over the piers and matrix soil, the settlement between the piers and matrix soil is the same and the stress concentration ratio, n, (ratio of average vertical stress applied on piers to the average vertical stress applied to the matrix soil between piers) can be assumed to be equal to the stiffness ratio, R_s (ratio of the pier element stiffness to the matrix soil stiffness). However, for cases where the piers are supporting embankment fill with no concrete load-transfer platform, the matrix soil may have a tendency to settle more than the stiff piers as depicted in Figure 5. This action mobilizes shear resistance in the fill soil above the piers and increases the stress on the piers and reduces stress on the matrix soil. This load transfer mechanism, referred to as soil arching, is well described in the literature (*24,25,26,27,28,29*). Stress concentration factors measured by Lawton and Merry (*15*) for piers supporting a concrete load-transfer platform are shown in Figure 6 and indicate an increasing amount of stress concentration from 10 to 45 with increasing top-of-pier stress. Stress concentration factors measured by Gaul (*29*) and Hoevelkamp (*20*) for piers supporting embankment fills are smaller at 2 to 8. Similar stress concentration values have been reported for embankment fill supported by lime columns, stone columns, soil cement columns, and concrete piles (*30*).

Coupling with the Matrix Soil

Similar to reported design approaches for stone columns (*5*) and deep soil mixing (*31*), short aggregate pier design calculations for settlement control include the assumption that the vertical deflection at the top of the piers is the same as the deflection of the adjacent matrix soils. This assumption is valid for rigid footings but can been questioned for embankment support applications. Figure 7 shows top-of-pier and matrix soil deflections for adjacent sites reinforced with stone columns and short aggregate piers. This test site compares stone columns fully penetrating the soft layer, whereas the short aggregate pier elements are floating (*9*). Settlement plate measurements (1 m x 1 m square plate) show little differential settlement between the matrix soil and the tops of the aggregate piers and the tops of the stone columns. At larger loads, differential settlement increases significantly between the tops of the stone columns and the adjacent matrix soils. These test results indicate positive coupling between the aggregate piers and matrix soil and support the design concept of uniform settlement for floating piers.

DESIGN

The first step in establishing a design is stability analysis (i.e. global slope stability, sliding wedge, and bearing capacity). If stability is shown to be adequate the next step is a settlement analysis. Unlike other systems such as driven pile or deep soil mixing that usually fully penetrate the weak layer; short aggregate piers are often designed as floating piers, thus necessitating unique design considerations (i.e. settlement below the piers). Approaches currently used for the design of short aggregate pier reinforced structures are generally based on those provided in the literature for stone columns (*5,32*) and modified to incorporate the aforementioned strength, stiffness and stress concentration characteristics. This section describes design approaches for embankment support applications.

Slope Stability Analyses

Composite Shear Strength – Floating Piers

Traditional global stability analyses using conventional software such as PCSTABL, UTEXAS, and SLOPE-W are implemented to determine the number of piers required for each application. End-of-construction factors of safety are typically 1.3 to 1.5. As shown in Figure 5, the shear strength of the reinforced zone is evaluated using composite shear strength parameter values for cohesion intercept ($c_{composite}$) and angle of internal friction ($\phi_{composite}$).

For piers that do not extend to a firm layer, the composite value for cohesion intercept is computed as the weighted average of the cohesion intercept of the pier aggregate (c_p) and the cohesion intercept of the matrix soil (c_m) (*5*):

$$c_{composite} = c_p\left(\frac{A_p}{A}\right) + c_m\left(\frac{A_m}{A}\right) \quad (2)$$

where A_p is the net cross-sectional area (plan view) of the pier elements in the reinforced zone, A_m is the net cross-sectional area of the matrix soil in the reinforced zone, and A is the gross cross-sectional area of the reinforced zone. Appropriate shear strength parameters (i.e. undrained – short-term vs. drained – long-term) should be selected consistent with the case under consideration. Recognizing that the true cohesion intercept of the pier aggregate is approximately zero and defining Area Ratio (R_a) as the ratio of the area of the pier elements to the gross area of the reinforced zone ($R_a = A_p/A$), Equation 2 reduces to:

$$c_{composite} = c_m(1 - R_a) \quad (3)$$

The composite value for angle of internal friction ($\phi_{composite}$) is computed using the weighted average of the tangent of the angle of internal friction for the pier aggregate ($\tan \phi_p$) and the tangent of the angle of internal friction for the matrix soil ($\tan \phi_m$):

$$\tan\phi_{composite} = R_a \tan\phi_p + (1 - R_a)\tan\phi_m \tag{4}$$

Composite Shear Strength – Piers Extending to a Firm Layer

In situations where the piers extend through weak embankment foundation soils to a firm bearing layer, the difference between the matrix soil stiffness and the pier stiffness can result in a concentration of stress to the bottom of the pier elements. This results in a significant further increase in the composite shear strength (*5,21*).

The composite shear strength of the reinforced zone is computed in a manner similar to that discussed above utilizing a weighted average approach as presented in Equations 2 through 4. However, the calculations to determine the composite friction angle and cohesion values incorporate additional terms to account for the stress concentration at the failure surface:

$$\tan\phi_{composite} = \left[\left(\frac{n}{R_a(n-1)+1}\right)R_a\tan\phi_p + \left(\frac{1}{R_a(n-1)+1}\right)(1-R_a)\tan\phi_m\right] \tag{5}$$

$$c_{comp} = \left[\left(\frac{1}{R_a(n-1)+1}\right)(1-R_a)c_m\right] \tag{6}$$

As described above, typical stiffness ratio values range from 10 to 45 when considering traditional foundation support applications (*15,33*) and lower design values from 2 to 8 for embankment fill applications (*20,29*). Design values for stress concentration must be verified and selected with care (usually ≤ 2), as $\tan\phi_{composite}$ is sensitive to small changes in stress concentration ratio.

Sliding Wedge Analysis

Embankment construction results in lateral earth pressure development that must be resisted by shear stresses acting at the base of the embankment (Figure 5). If the frictional forces between the pier elements and the embankment fill are insufficient to resist the applied shearing stress, lateral instability can occur. Calculations to determine the safety factor against sliding of the embankment on the piers is derived from a limit equilibrium approach as follows:

$$\frac{1}{2}K_a(\gamma H + q_o)H = \left[\mathrm{c}_{\mathrm{m,f}}(1-\mathrm{R_a}) + n\sigma_v \tan\phi_p R_a\right]L \tag{7}$$

where K_a is the active earth pressure coefficient, H and L are the height and length of the embankment, respectively, and $c_{m,f}$ is the cohesion developed at the matrix soil–embankment fill interface. Assuming: (1) the embankment fill is cohesionless, and (2) pier elements extend out from the crest of the embankment a distance equal to 1.5H (*6*), the required pier replacement area is determined from Equation 8.

$$R_a = \frac{K_a(\gamma H + q_o)}{3n\gamma H\left(1-\dfrac{0.75}{S}\right)\tan\phi_p} \tag{8}$$

where S is the slope factor (e.g. for 3h:1v, S = 3). Substituting n = 1 into Equation 8 results in the maximum replacement area, R_a, needed to achieve a safety factor against sliding equal to unity. The "true" safety factor against sliding is equal to the developed stress concentration. As described above, stress concentration factors from 2 to 8 have been measured for embankment fill applications. According to Holtz (*8*), using safety factors against sliding of at least 1.4 to 1.5 can reduce problems of excessive lateral movement.

Ultimate Bearing Capacity

Typically piers are designed so that they are long enough to inhibit the development of significant bottom-of-pier tip stresses (typically L/D > 3). Minimum required pier lengths are determined by comparing the top-of-pier load with the available side shearing resistance. These calculations, and the performance of load tests with telltales to demonstrate field behavior, provide assurance that shear failure below the tips of individual piers is precluded.

As described by Barksdale and Bachus (*5*) and Mitchell (*21*), the resistance to bulging of an aggregate column depends on both the limiting radial stress of the matrix soil adjacent to the pier ($\sigma_{r,lim}$) and the friction angle of the pier (ϕ_p). Equation 9 describes this relationship:

$$q_{ult,p} = \sigma_{r,lim} \tan^2\left(45 + \frac{\phi_p}{2}\right) \qquad (9)$$

Hughes and Withers (*34*) developed the well known approach to evaluate the limiting radial stress ($\sigma_{r,lim}$) based on cavity expansion theory as shown in Equation 10:

$$\sigma_{r,lim} = \sigma_{r,o} + c\left[1 + \ln\frac{E}{2c(1+\mu)}\right] \tag{10}$$

The limiting radial stress depends on the total radial stress after the installation of the pier and prior to the application of the foundation load ($\sigma_{r,o}$), the matrix soil undrained shear strength (c), the undrained modulus of the matrix soil (E), and Poisson's ratio of the matrix soil (μ). Using the simplifying assumptions: (1) the matrix soil lateral pressure is increased to the plane-strain Rankine passive earth pressure value after construction (using ϕ = 20 degrees), and (2) the ratio of the undrained modulus to the undrained cohesion is about 200, and (3) Poisson's ratio is 0.5, Equation 10 may be simplified as shown in Equation 11:

$$\sigma_{r,\lim} = 2\sigma_v^{'} + 5.2c \tag{11}$$

where σ'_v is the vertical effective stress at the anticipated depth of bulging, z_b:

$$z_b = D\tan\left(45 + \frac{\phi_p}{2}\right) \tag{12}$$

The ultimate pressure that may be applied to the top of an aggregate pier is calculated by combining Equations 9 and 11 to arrive at the following simplified equation assuming a pier friction angle of 48 degrees:

$$q_{ult,p} = 14\sigma_v^{'} + 35c \tag{13}$$

Load tests equipped with telltales have been used to verify the design calculations for piers installed in soft cohesive soils (*35*). Barksdale and Bachus (*5*) and Hoevelkamp (*20*) indicate that groups of piers are less susceptible to bulging than individual piers, because of the confinement offered by the adjacent elements. Therefore the single pier bearing capacity analysis is sufficient for most designs. Typical safety factors against bearing capacity failure are from 1.2 to 1.4.

Settlement Analyses

Once stability calculations satisfy the minimum required safety factors, settlement calculations are performed. Calculations for settlement control are based on a two-layer settlement analysis described by Lawton *et al.* (*35*), Lawton and Fox (*33*), and Fox and Cowell (*13*). Settlements within the "upper zone" (zone of soil that is reinforced) are computed using a weighted stiffness method that accounts for the stiffness of the pier elements, the stiffness of the matrix soil, and the area coverage of pier elements below the embankment. Settlements within the "lower zone" (zone of soils beneath the upper zone) are computed using conventional settlement methods. Stone columns, deep soil mixing, and driven pile design methods typically result in full penetration of the compressible soil layer. This is to avoid settlements below the tips of the elements. Because aggregate piers are typically shorter than other systems, design methods should include lower zone settlements.

Upper Zone Settlement

The settlement in the reinforced zone (upper zone) is based on the unit cell equilibrium method of analysis (*21*). The pier elements are considered stiff springs with a specific stiffness (k_p) and the matrix soils are considered soft springs with a specific matrix soil stiffness (k_m). For the case of floating piers, it is assumed that the settlements of the tops of the aggregate piers and adjacent matrix soil are the same.

The settlement in the upper zone may be computed by dividing the top-of-pier stress, q_p, by the pier stiffness, k_p. Using the assumption of uniform settlement and equations of static equilibrium, it may be shown that the top-of-pier stress depends on the ratio of the area coverage of the pier elements to the total foundation area (R_a), the stiffness ratio between the pier elements and the matrix soil (R_s), and the average bearing pressure applied to the foundation (q). This relationship is shown in Equation 14:

$$q_p = q\left[\frac{R_s}{R_a(R_s - 1) + 1}\right] \tag{14}$$

The design stiffness of the pier elements can be estimated based on a database of load test results (*13*) and later confirmed in the field by performing a load test. For the case of piers extending to a rigid layer, differential settlement between the matrix soil and piers requires the substitution of R_s in Equation 14 with the stress concentration factor, n. In this case, lower zone settlements are neglected.

Lower Zone Settlement

For floating piers the zone of soil below the bottom of the piers is considered the "lower zone." Settlements in this zone are computed using conventional analysis approaches that include estimating the induced stress and the soil compressibility. Settlements are determined using elastic theory or conventional consolidation approaches. For projects in organic clays and peats, secondary settlement may be a significant portion of the total settlement (*21*).

Stress influence factors are typically computed using Boussinesq or Westergaard stress influence charts and assuming that the influence factor commences at the plane in which the load is applied (i.e. bottom of embankment). This is because of the coupling of the piers to the matrix soil, and the demonstrated rapid stress dissipation with depth within the piers.

Lateral Spreading

In addition to vertical settlements, lateral spreading of the embankment foundation soils should be given consideration, especially for sensitive structures such as culverts and utilities where lateral spreading effects can be a serious problem (*8*). During embankment fill placement, stress applied to the foundation soil can result in exceeding the preconsolidation pressure in the foundation soils. When this occurs, continued fill placement results in undrained shear distortion under constant effective vertical stress (*36*). Case histories show that lateral displacement ($\Delta\delta_L$) is approximately equal to the vertical settlement ($\Delta\delta_V$) for undrained loading conditions. For long-term drained conditions, however, lateral displacement has been observed to be a smaller fraction of the vertical settlement (*37*). Assuming: (1) aggregate pier installation renders the matrix soil into an overconsolidated state as described by Handy (*18*); (2) stress concentrates on the pier elements; and (3) piers act as a drainage pathway to facilitate a drained response, Equation 15 can be used as a simplified empirical solution to estimate lateral spreading.

$$\Delta\delta_L = 0.2\Delta\delta_V = 0.2\frac{q_p}{k_p} \qquad (15)$$

Rate of Settlement

Design methods for radial drainage are based on approaches outlined in Han and Ye (*38*). Short aggregate piers may be installed to provide reductions in the settlement duration of embankments in the following ways:

1. When constructed using open-graded stone, the piers act as drainage elements. The incorporation of radial drainage usually reduces the drainage path lengths, thereby reducing the settlement duration.

2. The stress concentration to the stiff aggregate piers reduces the amount of pressure on the matrix soil. The reduction of pressure results in reduced settlement between the piers (*38*). A modified coefficient of radial consolidation may be incorporated in the radial drainage calculations to account for the influence of stress concentration on the rate of drainage. Han and Ye (*38*) developed the following equation to estimate the modified radial coefficient of consolidation based on research performed on stone columns:

$$c_r^{'} = c_r\left[1 + n_s \frac{1}{N^2 - 1}\right] \tag{16}$$

where c_r is the coefficient of consolidation in the radial direction, n_s is the steady stress concentration ratio, and N is the diameter ratio. Gaul (*29*) and Hoevelkamp (*20*) reported good correspondence between measured time rates of settlement for short aggregate pier supported structures and time rates of settlement predicted using the methods presented above.

Actual Settlement Results

In general, the two-layer settlement methodology is found to overestimate predicted settlements when compared to field measurements (*20,33,36,39*). The improved performance of the system, in comparison to field measurements, is thought to stem from conservative factors inherent in the design method and the inclusion of the lower zone in design calculations. To improve settlement predictions, development of methods to more accurately measure shear strength and stiffness parameters and more complex analyses (i.e. finite elements analysis) to model the behavior of floating pier systems are encouraged.

SUMMARY AND CONCLUSIONS

This paper provides needed design guides that may be used by engineers in the evaluation of short aggregate pier for the support of highway embankments. Recent experience on transportation projects in Iowa *(9)* demonstrates that short aggregate piers can be an effective foundation treatment and that the proposed design guides are suitable. The principal conclusions drawn from this paper are:

- The construction process of short aggregate piers for embankment applications results in the following engineering properties: ϕ = 48 to 52 degrees, c = 0, n = 2 to 8, k_g = 20 to 100 kPa/mm, and E = 90 to 190 MN/m^2.
- Slope stability analysis for embankment foundations reinforced with short aggregate piers is based on composite shear strength parameters that can include the influence of stress concentration for piers extending to a rigid layer.
- Sliding wedge analysis ensures an adequate number of pier elements are placed under the slope portion of the embankment to resist lateral loads and is especially important for sensitive structures (i.e. utilities).
- Two general modes of load-displacement behavior of short aggregate piers have been observed — bulging of the pier with little bottom movement and plunging of the pier with mobilized skin friction and tip resistance. Typically piers should be designed to limit the amount of tip stress to prevent bearing capacity problems.
- Stress cell measurements indicate that about 80 percent of the applied stress is dissipated at a depth of three times the pier diameter for floating piers in soft clay.

- Settlement is based on a two-layer approach—upper (reinforced) zone and lower (unreinforced) zone. For the upper zone, settlement is calculated as the applied stress at the top-of-piers divided by the pier stiffness. Settlements in the lower zone are computed based on elastic or consolidation parameters and an assumed stress influence factor.
- The time rate of settlements for the upper zone of the reinforced embankment foundation is enhanced by the stiff piers, which can concentrate stress and act as drainage pathways.

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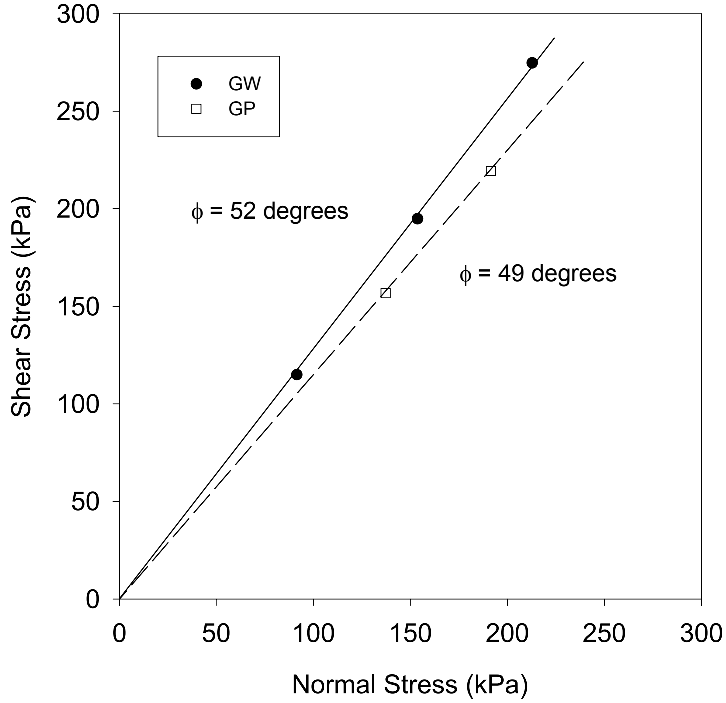
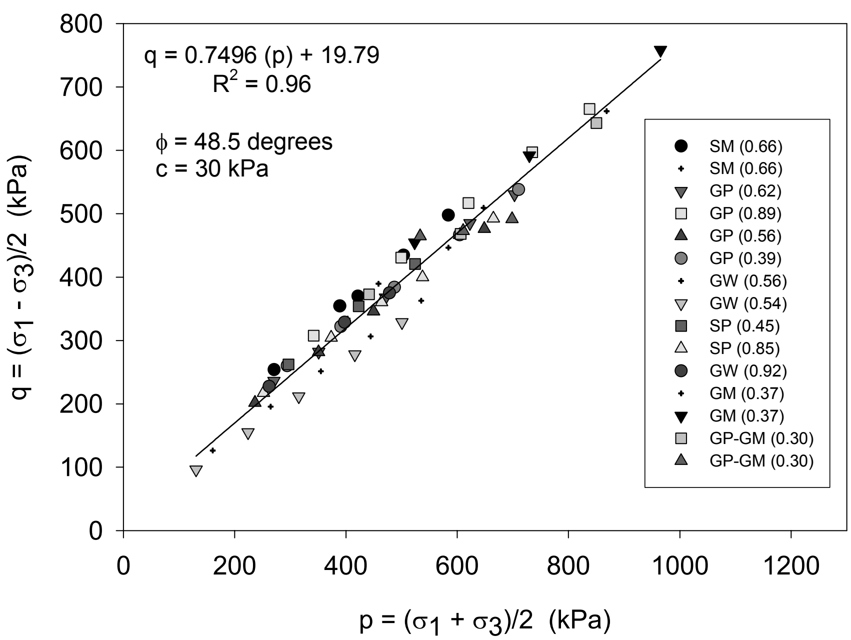
TABLE 1. Engineering properties of various foundation elements

Foundation Type	Cohesion (kPa)	Friction Angle, ϕ (degrees)	Elastic Modulus (MPa)	Reference
Short Aggregate Piers	0	48 to 52	96 to 190	(*9*)
Stone Columns	0	35 to 45	30 to 70	(*5,21*)
Lime/Cement Columns	210	44	90 to 239	(*22*)
Deep Soil/Cement Mixing	250 to 2,250	—	60 to 65	(*7*)
Reinforced Concrete Pile	~14,000	—	~30,000	(*23*)



(a)



(b)

FIGURE 1. Average results of shear strength tests (a) In situ direct shear tests (*13*) and (b) Triaxial compression tests on 100 mm by 200 mm high samples (*14*). Legend indicates Unified soil classification and compactibility values.

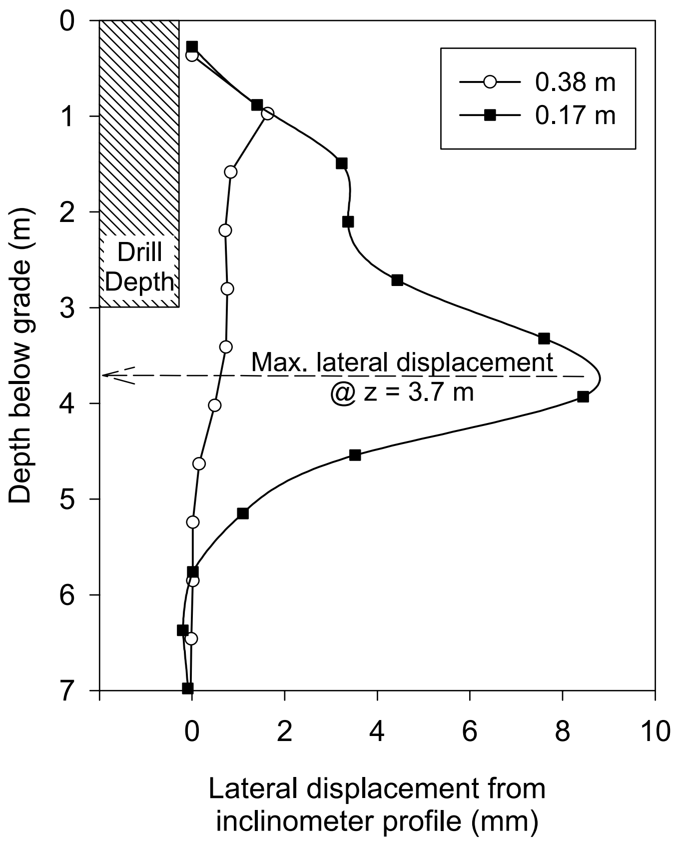


FIGURE 2. Inclinometer profile showing lateral straining from construction of 0.76 m diameter by 2.91 m long pier. Inclinometer casings were positioned at 0.17 m and 0.38 m from edge of pier.

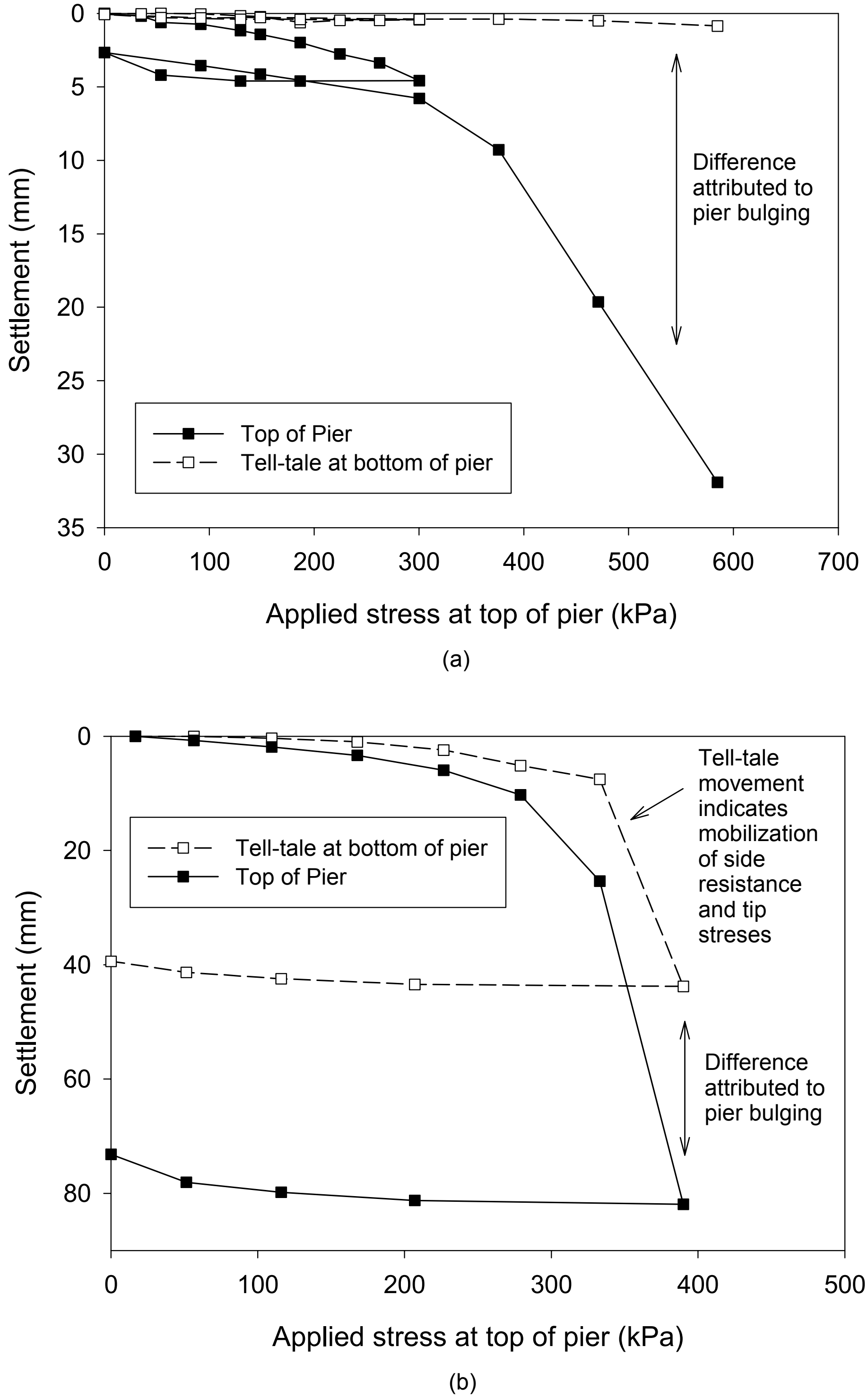


FIGURE 3. Modulus test results (a) for 0.76 m diameter by 5.4 m long pier in soft clay (CPT tip resistance values = 650 to 1000 kPa) and (b) for 0.76 m diameter by 2.8 m long pier in soft clay (CPT tip resistance values = 400 kPa).

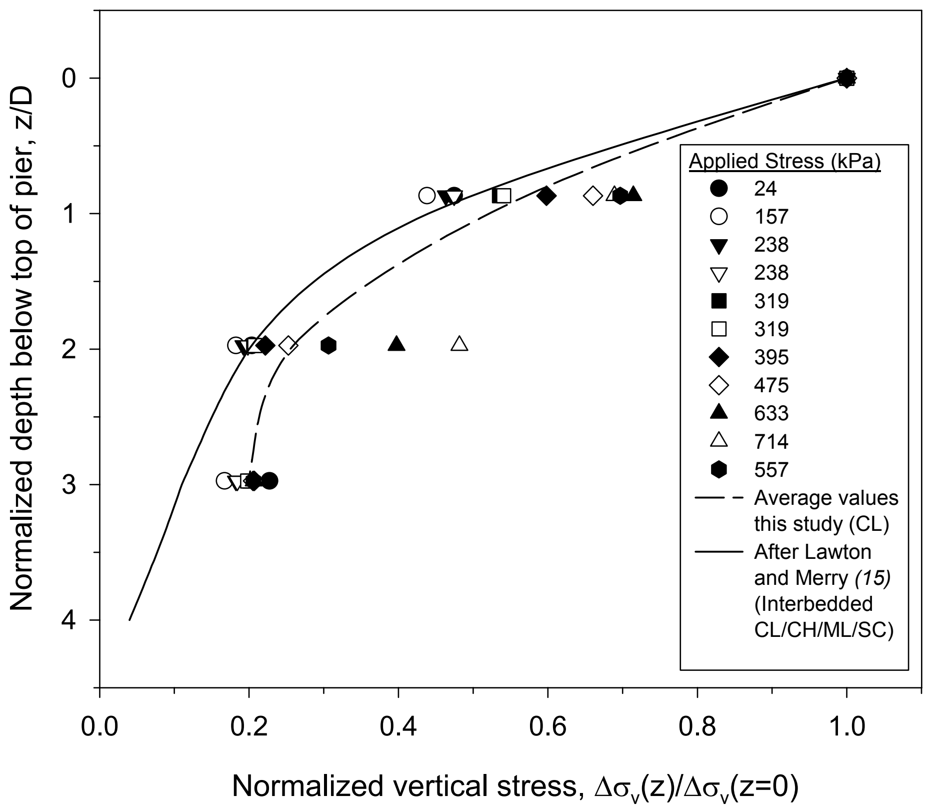


FIGURE 4. Stress dissipation with depth for 0.76 m diameter by 3 m long piers.

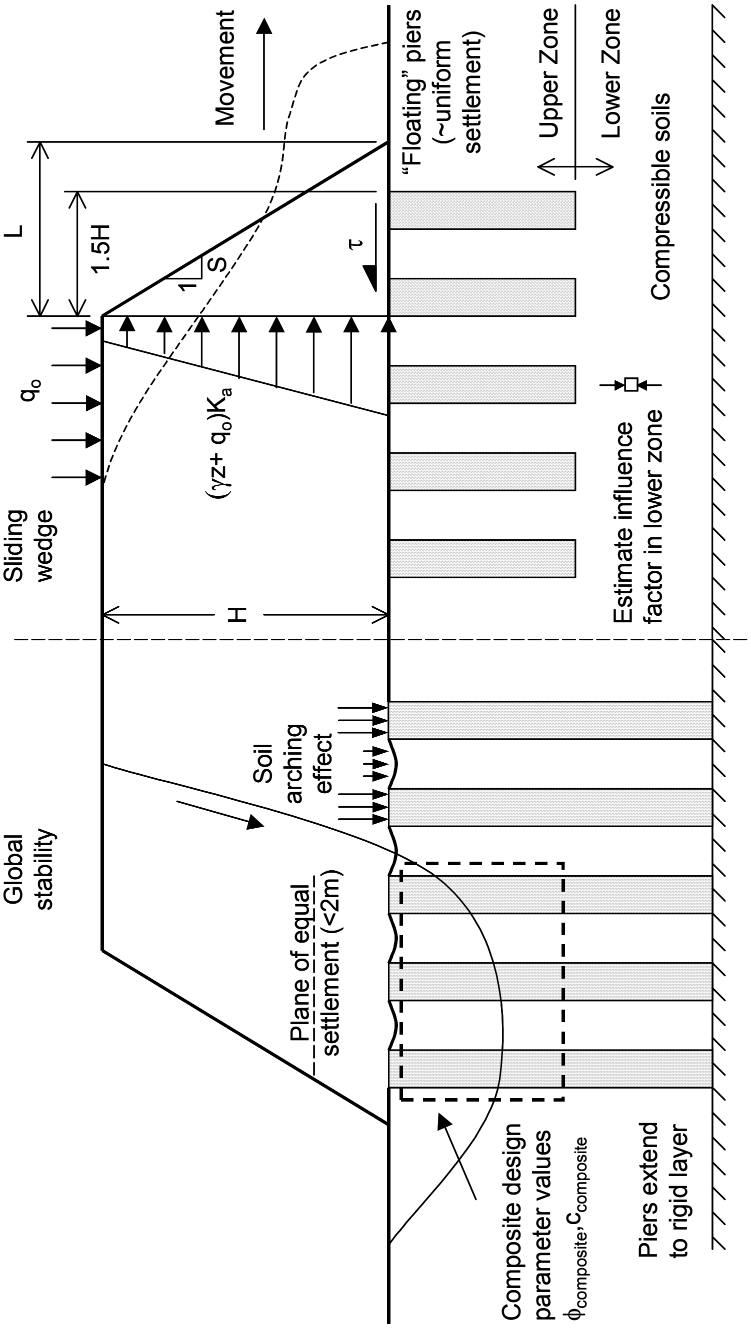


FIGURE 5. Aggregate pier under a highway embankment to control stability and settlement.

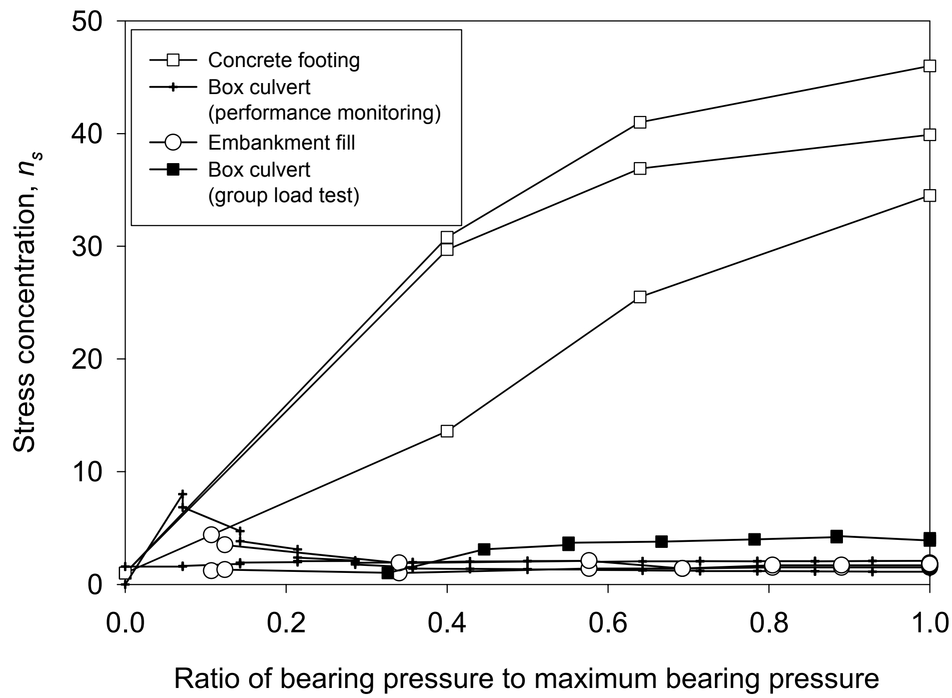
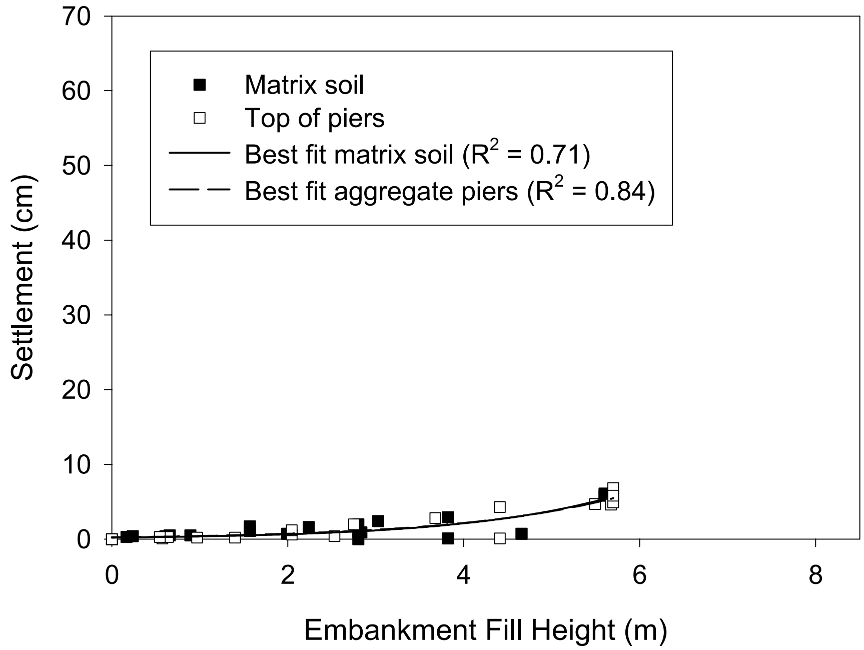
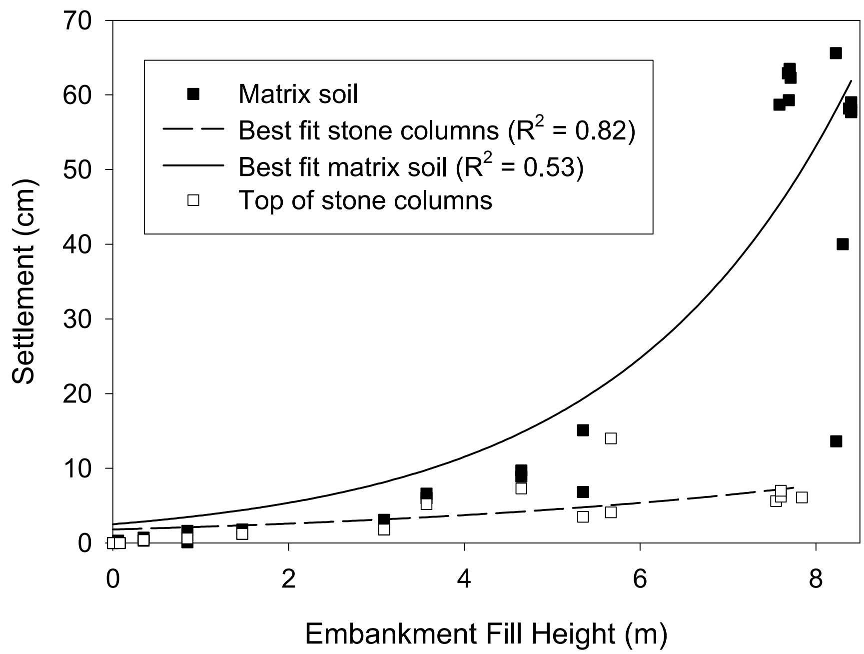


FIGURE 6. Stress concentration factors measured for instrumented concrete footing (*15*), box culvert project (*20*), and embankment support project (*30*).



(a)



(b)

FIGURE 7. Settlement plate monitoring at adjacent embankment support projects for (a) short aggregate piers and (b) stone columns.