EMBANKMENT SUPPORT: A COMPARISON OF STONE COLUMN AND RAMMED AGGREGATE PIER SOIL REINFORCEMENT

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ABSTRACT
A highway expansion project in Des Moines, Iowa recently required soil improvement to support two adjacent embankment fills. Stone columns were installed to improve the compressibility and shearing resistance at one site and *Rammed Aggregate Piers*™ (*Geopier*™ soil reinforcement) were installed to reduce the magnitude and increase the time rate of settlement at the other site. The embankment sites consist of similar soils. Stone columns were chosen to produce larger diameter and longer elements, whereas *Geopier* elements were chosen to give a smaller diameter and shorter element, but with higher stiffness. Prior to placement of embankment fill soils, in situ tests and full-scale load tests were conducted to evaluate each soil improvement method. Further, settlement plates were installed to monitor compression of the reinforced embankment foundation soils. This paper is of particular significance because it represents the first direct comparison of stone columns and *Geopier* soil reinforcement.

RÉSUMÉ
A highway expansion project in Des Moines, Iowa recently required soil improvement to support two adjacent embankment fills. Stone columns were installed to improve the compressibility and shearing resistance at one site and *Rammed Aggregate Piers*™ (*Geopier*™ soil reinforcement) were installed to reduce the magnitude and increase the time rate of settlement at the other site. The embankment sites consist of similar soils. Stone columns were chosen to produce larger diameter and longer elements, whereas *Geopier* elements were chosen to give a smaller diameter and shorter element, but with higher stiffness. Prior to placement of embankment fill soils, in situ tests and full-scale load tests were conducted to evaluate each soil improvement method. Further, settlement plates were installed to monitor compression of the reinforced embankment foundation soils. This paper is of particular significance because it represents the first direct comparison of stone columns and *Geopier* soil reinforcement.

1. INTRODUCTION

This paper presents the results of geotechnical measurements obtained at two adjacent embankment sites where the foundation soils were improved with stone columns (SC) and *Geopier* (GP) soil reinforcing elements. Although the purposes of the installations are different, the installation of granular columnar elements at adjacent sites with similar foundation soil characteristics provided the opportunity to compare the behaviour and engineering properties of both systems.

While stone columns have been used in transportation applications for several years, *Geopier* soil reinforcement represents a relatively new method of soil improvement that has grown steadily over the last 12 years. In practice, *Geopier* elements are mainly used for settlement control of building foundations, uplift resistance, and slope reinforcement (Lawton and Fox 1994, Lawton et al. 1994, Fox and Cowell 1998, Wissmann and Fox 2000, Wissmann et al. 2000, Wissmann et al. 2001). Case histories show that some structures constructed on *Geopier* elements are performing better than predicted (Lawton and Fox 1994, Lawton et al. 1994,). As a result, research efforts have focused on better understanding the influence of lateral stress development and the complex interaction of the aggregate pier-soil matrix (see Handy et al. 2002). Recent transportation applications in Iowa using *Geopier* elements include (1) pavement subgrade reinforcement, (2) retaining wall support, (3) reinforcement of bridge approach embankment fill, (4) settlement control for a large box culvert, and (5) the embankment foundation reinforcement project discussed in this paper.

At the test site, stone columns were installed to depths ranging from 3 to 14 m to reduce settlement and increase the factor of safety for global instability prior to construction of a 9 m bridge approach embankment. On the adjacent test site, *Geopier* elements were installed around the abutment footprint to depths ranging from 4.5 to 6.5 m prior to construction of the 8 m fill embankment. The purpose of the *Geopier* elements was simply to reduce the magnitude and increase the time rate of settlement to facilitate rapid abutment construction.

Prior to placement of embankment fill, Standard Penetration Tests (SPT), Borehole Shear Tests (BST), Ko Stepped Blade, and load tests were conducted. SPT tests through production piers provide a measure of density of the compacted aggregate. BST friction angle measurements provide for the estimation of the in situ coefficient of lateral earth pressure prior to placement of the aggregate piers. Lateral stress was measured in the matrix soils surrounding both types of elements with the

Ko Stepped-Blade. Until recently, effects of lateral prestressing, induced by a variety of foundation systems, have been conservatively neglected largely due to lack of field data showing a contribution to the performance of the system. R. L. Handy (2001) describes a lateral stress theorem indicating that lateral stress induced from foundation systems such as displacement piles, tapered piles, *Geopier* elements and others can theoretically reduce settlement by creating a near-linear-elastic, stress-reinforced zone within the matrix soils.

Lastly, full-scale load tests on isolated elements were conducted and settlement plates were installed and monitored for a period of one year. The settlement plates were installed to monitor and compare settlements on individual pier elements and on the surrounding matrix soils. This research represents the first reported comparison of stone columns and *Geopier* soil reinforcing elements used to reduce settlements below bridge approach embankments.

2. PROJECT AND SITE DESCRIPTIONS

2.1 Stone Column Site

Figure 1 shows the location of a bridge abutment site near Des Moines, Iowa where the Iowa Department of Transportation (Iowa DOT) planned the construction of the 9 m embankment fill. The site is underlain by 2 to 13 m of compressible clay and silt overlying highly weathered shale, dipping approximately 11 degrees. Cone Penetration Test (CPT) results indicate that tip resistances (qt) in the clay and silt generally range between about 650 to 1000 kPa and CPT friction ratio (Rf) values range between about 2 to 3. Slope stability calculations performed prior to construction revealed inadequate factors of safety against global instability along the sloping weathered shale interface. Stability concerns led to the specification of stone columns for shear reinforcement. A friction angle equal to 38 degrees was used in design stability calculations.

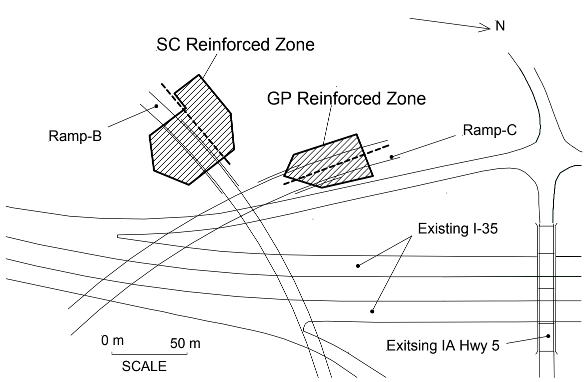

Figure 1. Iowa Highway 5/Interstate 35 research sites.

Stone columns were installed in an equilateral triangular pattern 1.8 m on-center to depths of 3 to 14 m below grade using the dry bottom feed technique (vibro-displacement). Stone column installations are facilitated with a horizontal oscillating vibroflot well described in the literature (Jebe and Bartels 1983, Munfakh et al. 1987, Elias et al. 2000). The crane-mounted, vibroflot probe penetrated the ground under static weight with the assistance of vibration and air. After reaching the design elevation, the vibroflot was withdrawn while aggregate was deposited out through the probe. Aggregate was placed in approximately 1.5 m lifts and compacted by raising and lowering the probe. A photograph of the installations is shown in Figure 2. Aggregate gradation characteristics are shown in Figure 3. The installation of stone columns at this site was advantageous because large diameter and long elements were needed.

(a)

(b)

Figure 2. Installation equipment for (a) SC Site - electric generator, crane to suspend vibroflot and extension tubes, bucket loader for aggregate transport from stockpile to hopper and (b) GP Site - track mounted drill, track

mounted hydraulic rammer and small track mounted bucket for aggregate deposition in drilled cavity.

2.2 Geopier Site

Figure 1 also shows the location of a site adjacent to the stone column site where Iowa DOT planned the construction of an 8 m bridge approach embankment fill. This site is underlain by 5 to 6 m of compressible clay overlying alluvial sand and highly weathered shale. CPT results indicate tip resistances (qt) in the clay layer generally range between about 400 to 950 kPa. CPT friction ratio (Rf) values range from about 4 to 7 within the clay layer.

Settlement calculations, based on laboratory odometer testing performed prior to construction, revealed excessive settlement magnitudes (about 34 cm) and inadequate time rates as a result of fill placement. *Geopier* elements were specified to reduce the total settlement magnitude (< 8 cm) and time of consolidation. Since the granular *Geopier* elements have a higher permeability than the matrix soil, increased consolidation was expected. Aggregate gradation characteristics are the same as used for stone columns, shown in Figure 3.

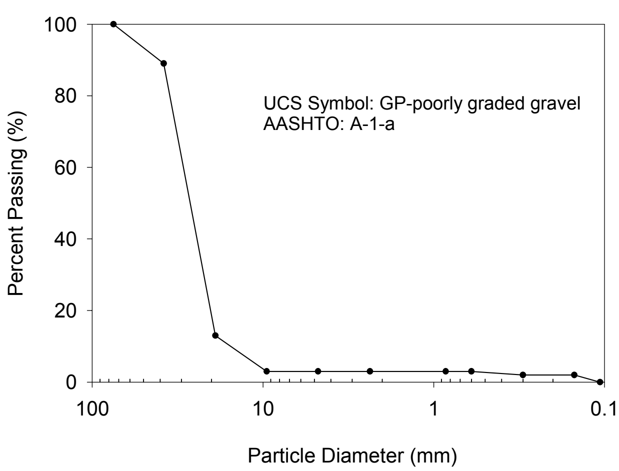

Figure 3. Grain-size distribution of aggregate used in construction of stone columns and *Geopier* elements.

The *Geopier* elements were constructed in a square pattern also 1.8 m on-center. Depths ranged from 4.5 to 6.5 m. Each element, 0.76 m in diameter, is constructed by building successive layers (0.3 m thick) of densely compacted aggregate (Fig. 2). The aggregate is laterally rammed into the surrounding matrix soil with a beveled tamper. It is estimated that each aggregate lift is subjected to 0.8 MNm of high-energy impact ramming action (Handy et al. 1999), which reportedly generates about 2500 kPa of lateral stress in the surrounding matrix soils (White et al. 2000). While comparable in cost to stone columns at this site, *Geopier* elements were chosen because it provided the opportunity to compare a new technology with stone columns in a full-scale application.

2.3 Comparison of Site Characteristics

Although the test sites are very close together, some differences in site conditions were observed. Table 1 presents a comparison of characteristics for the stone column and *Geopier* sites. The ratio of the CPT tip resistances for the stone column site to the *Geopier* site is approximately 1.2. This ratio suggests that the clay and silt material at the stone column site is slightly stiffer than the materials at the *Geopier* site. CPT friction ratio values at the stone column site are lower than those at the *Geopier* site. Lower friction ratio values are generally interpreted to suggest a less cohesive response for the tested soils (Douglas and Olsen 1981).

Element spacings for both sites are similar; however, the end bearing materials are different. The stone columns were designed to extend much deeper and to a minimum of 0.6 m into the underlying highly weathered shale; whereas, the *Geopier* elements only extend to the underlying sand layer. The *Geopier* elements were designed to penetrate the underlying sand layer to complete the drainage path out of the clay layer thus facilitating consolidation. The smaller diameter of the *Geopier* elements results in a smaller replacement ratio (ratio of the cross-sectional area of an element to the area of each unit “cell” reinforced by the element) than the area replacement ratio for the stone column site.

Table 1. Comparison of Site Characteristics

Characteristic	SC Site	GP Site
Depth to bearing layer (m)	3 to 13	4 to 6
CPT tip resistance (kPa)	650 to 1000	400 to 950
CPT friction ratio (%)	1.7 to 2.9	3.8 to 6.7
Element installation depth (m)	3.0 to 14.0	4.5 to 6.5
Element diameter (m)	0.91	0.76
Element spacing (m)	1.8 (equilateral triangle)	1.8 (square)
Area replacement ratio (%)	23	14
Embankment Fill height (m)	9	8
Number of elements	871	234

3. GEOTECHNICAL MEASUREMENTS

In order to characterize engineering properties of the stone column and *Geopier* elements the following insitu tests were performed:

- Borehole Shear Tests (BSTs) within the matrix foundation soils.
- Standard Penetration Tests (SPTs) within production stone column and *Geopier* elements.
- Ko Stepped-Blade Tests within the matrix soils surrounding production stone column and *Geopier* elements.
- Full-scale load tests on individual stone column and *Geopier* elements.

3.1 Borehole Shear Test Results

BSTs were performed prior to installation of *Geopier* elements as a rapid and direct means to measure soil cohesion (c) and friction angle (ϕ) on a drained or effective stress basis. The test procedure is described in detail by Handy and Fox (1967), and consists of expanding diametrically opposed contact plates into a borehole under constant normal stress, then allowing the soil to consolidate, and finally by pulling and measuring the shear stress. Points are generated on the Mohr-Coulomb shear envelope by measuring the maximum shear resistance at successively higher increments of applied normal stress.

The results of the BST measurements, shown in Table 2, indicate that the effective stress friction angle of the clay soils at the *Geopier* site varies between 11 and 32 degrees; the effective stress cohesion intercept varies between 3 kPa and 36 kPa. It is the authors' opinion that the variability in the measured shear strength parameter values is related to the alluvial nature of the soil.

TABLE 2. Borehole Shear Test (BST) shear strength parameters at *Geopier* test site.

Depth (m)	Cohesion, c' (kPa)	Friction Angle, ϕ' (degrees)
1.20	38	13
1.80	36	19
2.59*	9*	24*
2.70	22	25
2.80	9	23
3.40	16	24
4.11	20	18
4.60	15	30
5.20	16	11
6.00	3	32

* Note: Consolidated drained triaxial test

3.2 SPT Results

The results of 42 SPT N-values taken within stone columns and 6 SPT N-values taken within *Geopier* elements are shown in Figure 4. An average N-value of approximately 11 was achieved for the stone columns; an average N-value of approximately 17 was achieved for the *Geopier* elements. The ratio of the average N-value for the *Geopier* elements to the stone columns is about 1.5. Reportedly, N-value is proportional to friction angle (Shioi and Fukui 1982). In the literature stone column friction angle varies from 35 to 45 degrees (Greenwood 1970, Rathgeb and Kutzner 1975, Goughnour and Barksdale 1984), whereas, *Geopier* friction angle measurements are reported at 49 to 52 degrees (Fox and Cowell 1998).

The ratio of shear strength of the *Geopier* elements to the stone columns can be calculated by assuming equal normal stress and taking the ratio of the average coefficient of friction angle of the *Geopier* elements (tan 50°) to the average coefficient of friction angle of the stone columns (tan 40°). The ratio of the tangents of the friction angle values for *Geopier* elements and stone columns is about 1.4, a value similar to the ratio of the tested N-values for the elements. The difference in friction angle must be attributed to increased compaction.

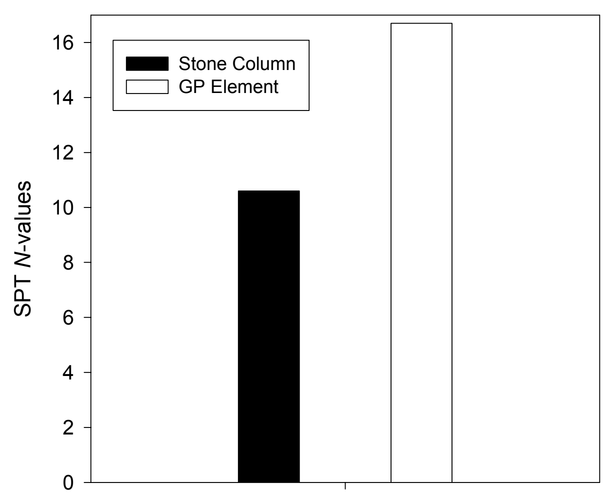

Figure 4. Comparative SPT N-values through production stone columns and *Geopier* elements.

3.3 Ko Stepped-Blade Results

The results of Ko Stepped-Blade Test measurements are presented in Figure 5. The Ko Stepped-Blade is a device developed at Iowa State University and uses lateral stress measurements taken at pressure cells embedded in the blade with variable thickness to determine insitu (zero blade width) lateral stress (Handy et al. 1982). Measurements adjacent to the stone columns were made in a tangential orientation (perpendicular to lines extending outward from the center of the element) at a radial distance of 70 cm from the edge of the stone column. Measurements adjacent to the *Geopier* elements were also made in a tangential orientation at a slightly larger distance of 85 cm from the edge of the pier. As shown in Figure 5, test measurements are normalized by the estimated insitu vertical effective stress at the test depth, and thus may be interpreted to be the effective horizontal earth pressure coefficient (k) after pier installation. Also shown in Figure 5 is the estimated insitu coefficient of lateral earth pressure at rest, using the well-know expression for normally consolidated soils (1-sinϕ). Estimated values are made using the BST test results summarized in Table 2.

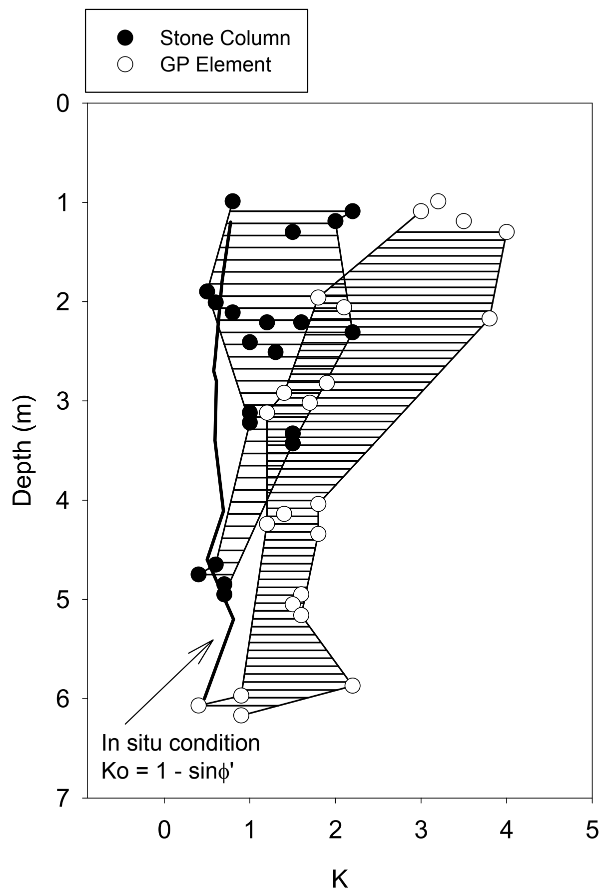

Figure 5. Ko Stepped-Blade measurements conducted 70 cm from stone column and 85 cm from *Geopier* element. All tests were oriented to measure radial stress.

Table 3. Results of lateral stress measurements.

Lateral Earth Pressure Coefficient Condition	SC Site (70 cm radial distance)	GP Site (85 cm radial distance)
Range of data	0.4 to 2.2	0.4 to 4.0
Average of data	1.2	2.1
Ratio of average of data to Ko average	1.8	3.3
Ratio of average of data to Rankine Kp average	0.5	0.9

Results of the measurements shown in Figure 5 indicate that the post-installation values for coefficient of lateral earth pressure at the stone column site range between 0.4 and 2.2 with an average of 1.2. At the *Geopier* site, the coefficient of lateral earth pressure ranges between 0.4 and 4.0 with an average of 2.1, which is very close to the calculated Rankine coefficient of passive earth pressure, 2.3. Test results are summarized in Table 3. Lateral stress measurements at other *Geopier* sites have also shown passive stress development in the foundation soils (Handy et al. 2002, White et al. 2000).

Test results indicate that greater post-installation lateral earth pressures are measured in the soil surrounding the *Geopier* elements than in the soil surrounding the stone columns, despite the measurements for the stone column being 15 cm closer to the edge of the element than are the measurements for the *Geopier* test element. During stone column installation, ground heave (0.8 to 1.0 m) and radial cracking were observed at the surface, whereas no ground heave and minimal radial cracking were observed at the *Geopier* site. Furthermore, field measurements indicate that cavity expansion during stone column construction averaged about 30 percent; whereas, the *Geopier* element installations resulted in about 10 percent cavity expansion. In the authors' opinion the soil fabric at the stone column site was highly disturbed due to excessive cavity expansion, subsequent ground heave and radial cracking. Thus, the soil shear strength may have been reduced to residual strength and therefore, did not retain high lateral stresses.

3.4 Load Test Results

Load tests were performed on a production stone column (7 days after installation) and a *Geopier* element (3 days after installation). The tested stone column was 91 cm in diameter and installed to a depth of 5.0 m. The tested *Geopier* element was 76 cm in diameter and installed at a depth of 5.4 m. To measure deflection near the bottom of the *Geopier* element a telltale was installed at a depth of 4.9 m. To compensate for the effects of the greater diameter of the stone column element the load test results are presented as applied stress versus settlement in Figure 6.

Test results for the stone column suggest bi-linear stress-deformation behaviour as increasing stress is applied. A steeper stress-deformation response is noted at applied stresses greater than about 70 kPa. Test results for the *Geopier* element also suggests a bi-linear response with a steeper stress-deformation response noted at applied stresses greater than about 300 kPa. The *Geopier* telltale installed at the base of the pier indicates essentially no movement for the full range of applied stresses. The authors interpret this response as initiation of pier bulging at stresses greater than about 300 kPa.

The ratio of stresses, at which a steepened stress-deformation response is noted, for the Geopier and stone column elements is about 4. This ratio could be interpreted to represent the ratio of the elastic compressive behaviour of the two elements prior to plastic deformation (bulging). Initiation of bulging type deflection for granular columnar elements is a function of the friction angle of the aggregate and the soil limiting radial stress (Hughes and Withers 1974). Pier bulging is not necessarily undesirable, as it should increase load transfer to the matrix soils.

Figure 7 presents the relationship between stiffness and applied stress for both the stone column and *Geopier* elements. Stiffness is defined as the slope of the stress deformation curve shown in Figure 6. Stiffness values of the stone column decrease from about 80 MN/m3 at low levels of applied stress to less than 10 MN/m3 at stresses of about 200 kPa. Stiffness values of the *Geopier* element decrease from about 190 MN/m3 at low levels of applied stress to about 80 MN/m3 at an applied stress of 600 kPa. Table 4 presents ratios of stiffness values for the stone column and *Geopier* elements. The ratio of *Geopier* to stone column stiffness values increase from approximately 2 to 9 with increasing applied stress.

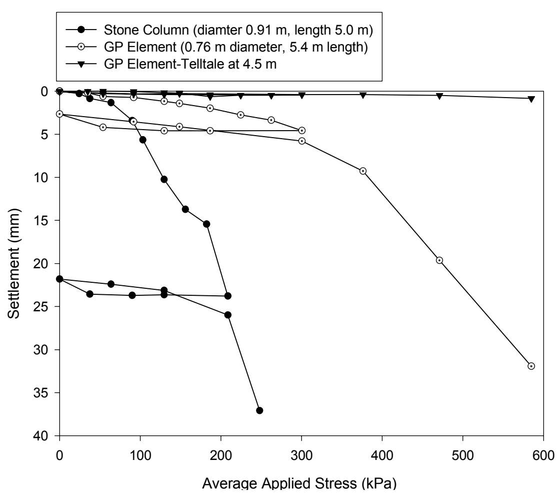

Figure 6. Comparative stress-deformation plot for stone column and *Geopier* elements.

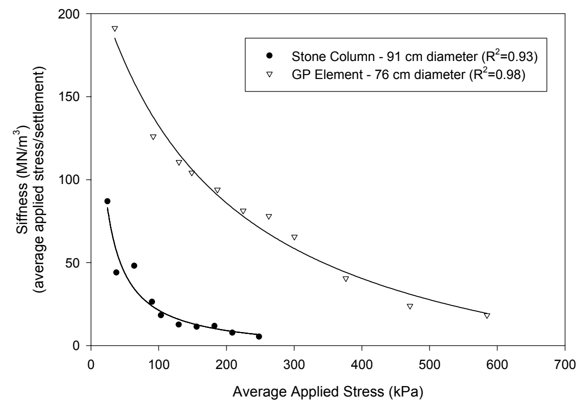

Figure 7. Stiffness versus applied stress for stone column and *Geopier* elements. Trend lines are best-fit hyperbolic decay functions.

Table 4. Comparison of stiffness values derived from load test results

Applied Stress (kPa)	SC Stiffness (MPa)	GP Stiffness (MPa)	GP to SC Stiffness Ratio
25	81	196	2.4
50	44	171	3.9
100	21	132	6.3
200	9	86	9.6
400	—*	40	—
600	—	18	—

* Note: Data not available

4. SETTLEMENT MEASUREMENTS

During the placement of fill soils, settlement surveys were made at the stone column and *Geopier* sites. Measurements at both sites were made using 0.9 m square settlement plates placed on individual aggregate elements and on the matrix soils between the elements. The results of the settlement measurements are presented in Figure 8 and summarized in Table 5. The settlement measurements indicate the following:

- After the placement of 6 m of fill, the stone column matrix soils settled about 19.5 cm. The *Geopier* matrix soils settled about 5.4 cm under this same fill pressure.
- The ratio of the settlement of the stone columns (4.8 cm) to the settlement of *Geopier* elements (1.5 cm) is approximately 3.2 at a fill height of 6 m.
- The differential settlement between the stone columns and adjacent soil is significantly larger than the differential settlement between the *Geopier* elements and the adjacent matrix soil.
- The ratio of the settlement of the stone column matrix soils to the settlement of the *Geopier* matrix soils is about 3.6 at a fill height of 6 m.

One explanation for the stone columns settling more than the *Geopier* elements is that the remoulded stone column matrix soils did not restrain the columns and the columns expanded (see Mckenn et al. 1975). This theory is supported by the Ko Stepped-Blade lateral stress measurements, which indicate lower lateral stress development at the stone column site compared to the *Geopier* site. The magnitude of lateral stress surrounding aggregate piers and other foundation systems is a phenomenon of considerable significance and should be studied more extensively.

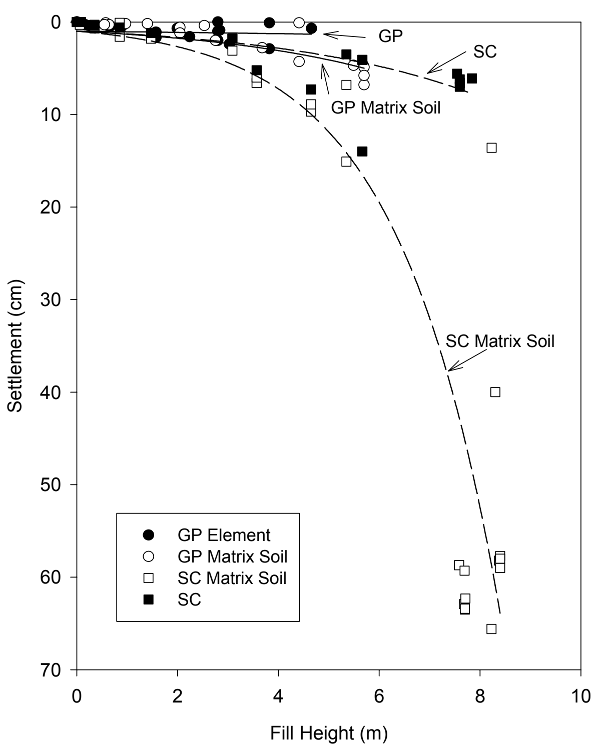

Figure 8. Settlement versus fill height from 0.9 x 0.9 m settlement plates installed immediately after pier installations. Settlement plates were monitored for one year.

Table 5. Results of settlement survey measurements

Settlement (cm)	Embankment Fill Height (m)			
	2	4	6	8
SC Element	1.7	2.8	4.8	8.1
SC Matrix Soil	2.7	7.2	19.5	52.4
Ratio of SC Matrix Soil to SC Element	1.6	2.6	4.1	6.5
GP Element	1.1	1.3	1.5	—*
GP Matrix Soil	1.8	3.1	5.4	—
Ratio of GP Matrix Soil to GP Element	1.6	2.4	3.6	—

* Note: Fill height at test location did not exceed 6 m

5. SUMMARY AND CONCLUSIONS

Geotechnical measurements were taken at two adjacent embankment foundation sites improved with stone columns and *Geopier* elements. A summary of the measurements is as follows:

- The subsurface conditions at the stone column site were slightly stiffer and less cohesive than the subsurface conditions at the *Geopier* site, based on interpretation of CPT data.
- Element spacings at both sites were 1.8 m on-center. The greater diameters of the stone column elements and application of a triangular spacing pattern result in a greater area replacement ratio.
- SPT results for tests performed within the elements indicate an average N-value of 11 for the stone columns and an average N-value of 17 for the *Geopier* elements.
- The ratio of post-installation matrix soil lateral stress for the *Geopier* elements to the post-installation matrix soil lateral stress for the stone columns is about 2.
- Load test results indicate that the ratio of pre-bulging compressive strength for the *Geopier* element to the pre-bulging compressive strength for the stone column is about 4.
- Load test results indicate that the ratio of *Geopier* stiffness to stone column stiffness ranges from about 2 to 9 as a function of applied stress.
- Settlement of matrix soils surrounding the stone columns was about 3 times as large as the settlement of matrix soil surrounding the *Geopier* elements.

The stone column site has performed its intended function for global slope reinforcement. This is evidenced by the fact that the embankment has not failed. The *Geopier* installations also have performed as intended by reducing settlement and the construction delay between embankment completion and abutment construction from the original 120 days to just 30 days. In short, advantages of the stone columns at this site include larger diameter and shaft length, whereas the *Geopier* elements were smaller but stiffer. Future comparative investigations are highly encouraged with emphasis on documenting the influence of lateral stress on the load-settlement behaviour.

6. ACKNOWLEDGEMENTS

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7. REFERENCES

Elias,V., Welsh, J., Warren, J., and R. Lukas. 2000. *Ground Improvement Technical Summaries*. Publication No. FHWA-SA-98-068.

Douglas, B., and R. Olsen. 1981. Soil Classification Using Cone Penetrometer. *Symposium on Cone Penetration Testing and Experience, Geotechnical Engineering Division, ASCE*, St. Louis, pp. 209-227.

Fox, N.S. and M.J. Cowell. 1998. *Geopier Foundation and Soil Reinforcement Manual*, Geopier Foundation Company, Inc. Scottsdale, Arizona.

Goughnour, R., and R. Barksdale. 1984. Performance of a Stone Column Supported Embankment. *International Conference on Case Histories in Geotechnical Engineering*, Vol. 2, St Louis, pp. 735-742.

Greewood, D. A. 1970. Mechanical Improvement of Soils Below Ground Surface. *Ground Engineering*, pp.11-22.

Handy, R.L. 2001. Does Lateral Stress Really Influence Settlement. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, Vol. 127, No. 7, pp. 623-626.

Handy, R.L., White, D.J., and K.J. Wissmann. 2002. Concentric Stress Zones near Rammed Aggregate Piers. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE* (under review)

Handy, R.L., and N.S. Fox. 1967. A Soil Borehole Direct Shear Test Device. *National Acad. Sci. Highway Research Board, Highway Research News*, No. 27, pp. 41-45.

Handy, R.L., Fox, N.S., and K.J. Wissmann. 1999. Short Aggregate Piers Reinforce Soils Near Tunnels. *Geotechnical Special Publication No. 90: Geo-Engineering for Underground Facilities, ASCE*, pp. 1039-1047.

Handy, R.L., Remmes, B., Moldt, S., Lutenegger, A., and G. Trott. 1982. In Situ Stress Determination by Iowa Stepped Blade. *Journal of the Geotechnical Engineering Division, Proceedings of ASCE*, Vol. 108, GT11, Nov., pp. 1405-1422.

Hughes, J.M.O. and N.J. Withers. 1974. Reinforcing Soft Cohesive Soil with Stone Columns. *Ground Engineering*, May, pp. 42-49.

Jebe, W., and K. Bartels. 1983. The Development of Compaction Methods with Vibrators from 1976 to 1982. *8th European Conference on Soil Mechanics and Foundation Engineering*, A.A. Balkema, Helsinki.

Lawton, E.C, and N.S. Fox. 1994. Settlement of Structures Supported on Marginal or Inadequate Soils Stiffened with Short Aggregate Piers. *Geotechnical Special Publication No. 40: Vertical and Horizontal Deformations of Foundations and Embankments, ASCE*, Vol. 2, pp. 962-974.

Lawton, E.C., Fox, N.S, and R.L. Handy. 1994. Control of Settlement and Uplift of Structures Using Short Aggregate Piers. *Geotechnical Special Publication No. 45: In-Situ Deep Soil Improvement, ASCE*, pp. 121-132.

McKenn, J.M., Eyre, W.A., and D.R. Wolstenholme. 1975. Performance of an Embankment Supported by Stone Columns in Soft Ground. *Geotechnique*, Vol. 25, No. 1, pp. 51-59.

Munfakh, G., Abramson, L., Barksdale, R., and M. Juran. 1987. In-Situ Ground Improvement. Soil Improvement, *Geotechnical Special Publication No. 12, ASCE*, New York, pp. 1-17.

Rathgeb, E., and C. Kutzner. 1975. Some Application of the Vibro-Replacement Process. *Geotechnique*, Vol. 25, No. 1, pp. 45-50.

Shioi, Y., and J. Fukui. 1982. Application of N-value to Design of Foundations," *JACI*, Vol. 81, No. 5, Sept-Oct., pp. 469-475.

White, D.J., Lawton, E.C., and J.M. Pitt. 2000. Lateral Earth Pressure Induced from Rammed Aggregate Piers. *Proceedings of the 53rd Canadian Geotechnical Conference*, Montreal, Vol. 2, pp. 871-876.

Wissmann, K.J., Moser, K., and M.A. Pando. 2001. Reducing Settlement Risks in Residual Piedmont Soils Using Rammed Aggregate Pier Elements," *ASCE Geotechnical Special Publication No. 113*, Foundations and Ground Improvement, Blacksburg, Virginia, June 9-13, pp. 943-957.

Wissmann, K.J. and N.S. Fox. 2000. Design and Analysis of Short Aggregate Piers Used to Reinforce Soils for Foundation Support. *Proceedings: Geotechnical Colloquium*, Technical University Darmstadt, Germany, March.

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