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EDITED BY
Magued Iskander, Ph.D., P.E.
Debra F. Laefer, Ph.D.
Mohamad H. Hussein, P.E.

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INSTRUMENTATION AND MONITORING OF MSE WALLS SUPPORTED ON THE RAMMED AGGREGATE PIER® SYSTEM

Terry Micnhimer, PE[1], Jorge R. Parra, PhD, PE[2], and Tommy Williamson[3]

[1]Senior Geotechnical Engineer, Tolunay Wong Engineers, 10710 S. Sam Houston Parkway West, Suite 100, Houston, TX 77031 tmicnhimer@tweinc.com
[2]Lead Engineer, Geopier Foundation Company, Inc., 150 Fairview Road, Suite 335, Mooresville, NC 28117, jparra@geopier.com
[3]President, Geopier Foundation Company-Houston, 17602 Sierra Creek Lane, Humble, TX 77346 tlw90@aol.com

ABSTRACT: As part of the upgrade plans for the U.S. Highway 90 and Highway 6 Interchange, the Texas Department of Transportation (TXDOT) proposed the construction of a series of bridges and ramps. The bridge abutments and access ramps required the construction of Mechanically Stabilized Earth (MSE) retaining walls with heights of up to 8.2-m (27-feet). The weak clay foundation soils along the wall alignment presented global instability, settlement, and bearing capacity challenges. As an alternative to massive over-excavation and replacement or preloading with a surcharging, the design team and the Texas Department of Transportation selected a *Rammed Aggregate Pier®* solution. The Rammed Aggregate Pier system (RAPs) increases the factors of safety for bearing capacity and global stability as a result of the high angle of internal friction achieved during ramming and reduces the magnitude and time of settlement by increasing the overall stiffness of the foundation soils and providing a drainage pathway for dissipation of excess pore water pressure. The performance of the Rammed Aggregate Pier-supported walls was monitored using a suite of geotechnical instrumentation consisting of vertical and horizontal inclinometers, vibrating wire piezometers, and Sondex vertical settlement gauges located near the critical sections. The instrumentation was monitored for approximately one year and indicated acceptable performance.

This paper discusses the results of the monitoring program of an MSE wall supported by the Rammed Aggregate Pier system. This work is of particular significance because it is the first MSE wall support application performed in Texas and the first instrumented wall project supported on Rammed Aggregate Piers funded by the Federal Highway Administration (FHWA).

INTRODUCTION

As the Texas Department of Transportation continues with its $40 million dollar, 3-year plan of infrastructure development and improvement, construction at the US-90A and State Highway 6 Interchange in Sugarland, Texas, resulted in the need for support of new MSE walls for ramps and bridge abutments.

The foundation soils along the wall alignments were predominantly soft to medium stiff clay in the upper 9-m (30 feet) representing a challenge for achieving the factors of safety for global stability and bearing capacity required by the design team.

The Rammed Aggregate Pier reinforcing elements improve the bearing capacity of the foundation soils by providing significant increases in the composite shear resistance (foundation soils + RAP) because of their high angle of internal friction developed during the ramming process. Additionally, the piers act as vertical drains reducing the potential for slope instability caused by excess pore water pressure in the slope.

Monitoring of the MSE walls at two ramps and two bridge abutments was scheduled to occur for a period of 24 months following RAP installations. Based on the positive performance and quick stabilization of the walls, and small amount of vertical and horizontal wall movement and quick dissipation of excess pore pressures measured with the different instrumentation, the monitoring period was reduced from 24 months to less than 12 months.

PROJECT DESCRIPTION AND SUBSURFACE CONDITIONS

Table 1 summarizes the scope of work for areas requiring ground improvement. The MSE wall selected for this project was a Tricona Retained Wall System consisting of segmental precast panels (4.2 square meters) connected to steel reinforcement. The design of the segmental wall followed AASHTO codes.

Table 1. MSE Wall Scope of Work using Rammed Aggregate Piers

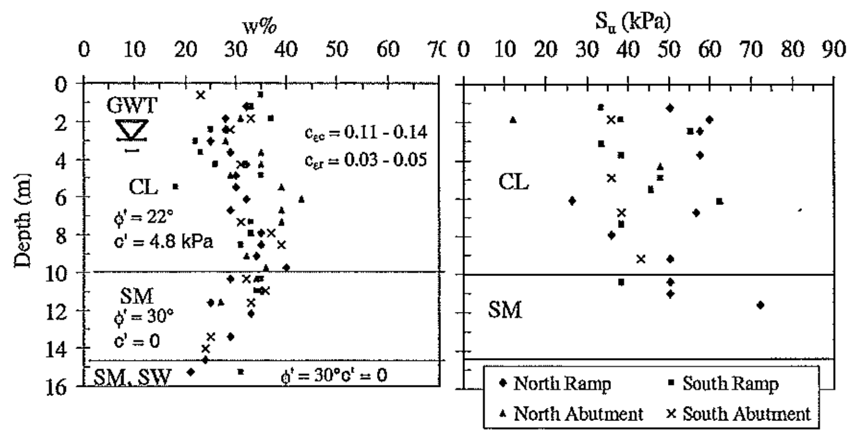
Location	Wall Length m (ft)	Maximum wall height (H_{wall}) m (ft)
North Abutment	79 (260)	7.3 (24)
South Abutment	69 (227)	8.2 (27)
North Ramp	108 (353)	7.3 (24)
South Ramp	108 (353)	6.7 (22)

The design requirements for the MSE wall related to ground improvement are summarized in Table 2.

Table 2. Wall Design Requirements

Design Criterion	Requirement
Global Stability	FS = 2.0 (drained and undrained conditions)
Bearing Capacity	FS = 1.3 (drained and undrained conditions)
Post-Construction Settlement	Less than 5-cm occurring within 6-months following end of construction

The geotechnical investigation was performed in 2001 by HVJ Associates, Houston, TX. The soil conditions consists of soft to medium stiff clay extending to depths of approximately 9-m to 12-m underlain by loose sandy silt to silty sand to depths of 15.2-m to 18.3-m, underlain by medium dense sand to silty sand to the maximum explored depth. Groundwater (GWT) was encountered at a depth of approximately 3-m. Figure 1a shows natural moisture content (w%) and effective stress shear strength parameter values with depth along with compressibility parameter values. Figure 1b shows undrained shear strength (S_u) with depth along with classifications.



a) b)

Fig. 1. Soil characteristics a) moisture content, compressibility and strength parameter values vs depth, b) Undrained shear strength vs depth.

RAP DESIGN APPROACH FOR MSE WALL SUPPORT

Rammed Aggregate Pier construction consists of a three step procedure as described in the literature by Fox and Cowell (Fox, N.S. and Cowell, M.J. (1998)) The design approach to address global stability and bearing capacity under MSE walls consists of increasing the composite shear strength parameter values within the Rammed Aggregate Pier-reinforced zones. The composite shear strength parameter values are estimated as a weighted average (FHWA 1999, Barksdale and Bachus 1983, Mitchell, 1981, Fitzpatrick and Wissmann, 2002). The reinforced zones with composite shear strength parameter values were modeled as rectangular areas directly beneath the MSE wall and backfill using the computer software GSlope version 4 by Mitre Software Corporation. The approach is based on two-dimensional limit equilibrium analyses using Bishop's method to determine factors of safety.

The design approach to estimate settlement within the RAP-reinforced zone (upper zone) and the area beneath the upper zone (lower zone) is based on a spring analogy and a two-layer approach as described by Lawton et. al. (1994), Lawton and Fox (1994). A modulus test is performed early during construction following general procedures for pile static compression tests (ASTM, 1994) to verify the assumed stiffness modulus value for the RAP element. The time for consolidation settlement is estimated taking into account radial drainage to the RAP elements and using Barron's approach (1948). Settlement of the soil below RAP-reinforced zone is estimated using conventional geotechnical approaches and includes an influence factor of unity.

The RAP design engineered for this project to satisfy the design requirements indicated previously in Table 2 consisted of closely-spaced RAPs along the wall face and within the reinforced zone to address bearing capacity, global stability and settlement control. The RAPs were installed to a maximum depth (H_s) of 7.6 m in most areas to reinforce through the soft clay and intersect any possible deep seated slip surface. Bearing capacity and global stability issues typically govern and dictate the spacing along the wall face and reinforced zone. These two design requirements normally do not govern the spacing required further back behind the main reinforced zone. For that reason, reinforcing elements at a wider spacing can be afforded for settlement control and consolidation time acceleration purposes only. Figure 2 summarizes the typical pier spacing and shaft lengths.

The project included a total of 1411 drilled RAPs for the four walls. PCI, the licensed installer, performed the installation with two working crews in approximately eight months. RAPs were installed during Phase I from July, 2005 through January, 2006. During Phase II, RAPs were installed from October, 2006 through January, 2007. Fill placement at Phase I started on January, 2006 through February, 2006.

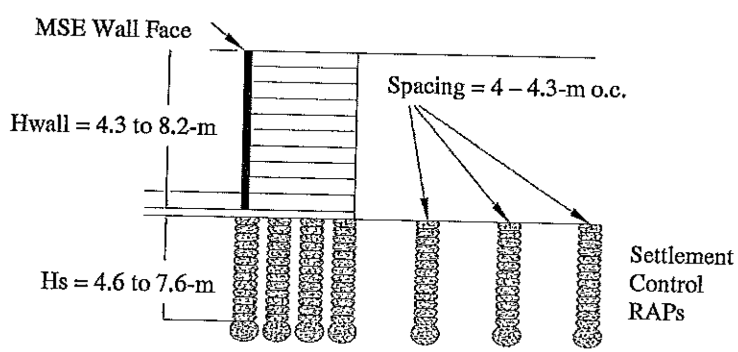


Fig. 2. Rammed Aggregate Pier design layout

The stiffness of the RAP elements was measured in the field to verify the initial spring stiffness assumed for design. A modulus test was performed at every wall location as shown in Figure 3. The design was based on the assumption of a RAP stiffness modulus value of 27 MN/m^3 (100 pci) at the design pressure of 862 kPa (18000 psf). The stiffness modulus values measured during the performance of the modulus tests exceeded the initially assumed value by at least 50 percent. The tests consistently indicated that the point of maximum curvature occurred at the design stress or at slightly higher stress.

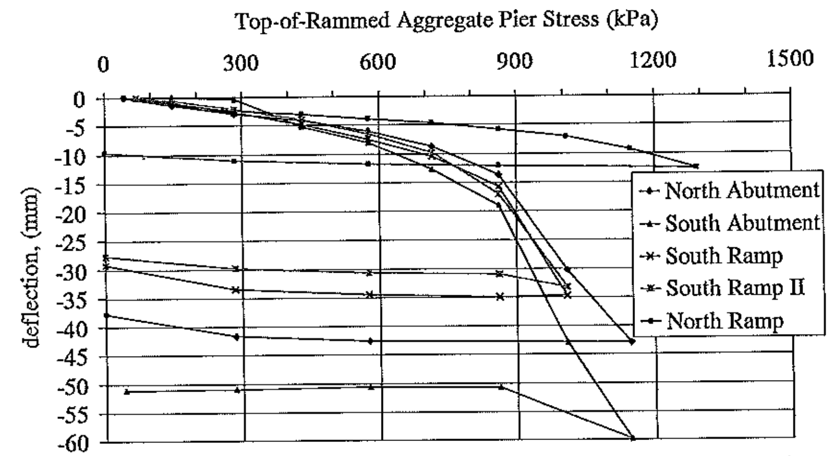


Fig. 3. Modulus Test Results

INSTRUMENTATION AND WALL MONITORING RESULTS

FHWA funded the geotechnical instrumentation to monitor the tallest sections at each ramp and bridge embankment. The instrumentation was installed, monitored and plotted by Tolunay Wong Engineers, Houston, TX. Figure 4 shows the instrumentation location for the south bridge abutment. The instrumentation consisted of horizontal inclinometers located perpendicular to the center line of the embankment extending over the subgrade and over the tops of some of the piers from one wall face to the other. Also, vertical inclinometers and Sondex settlement gauges were installed outside and adjacent to the wall face. Vibrating wire piezometers were installed at the middle of the embankment in grouted vertical nests. The vertical inclinometers and Sondex gages were both installed to a depth of 15.2 m (50 ft).

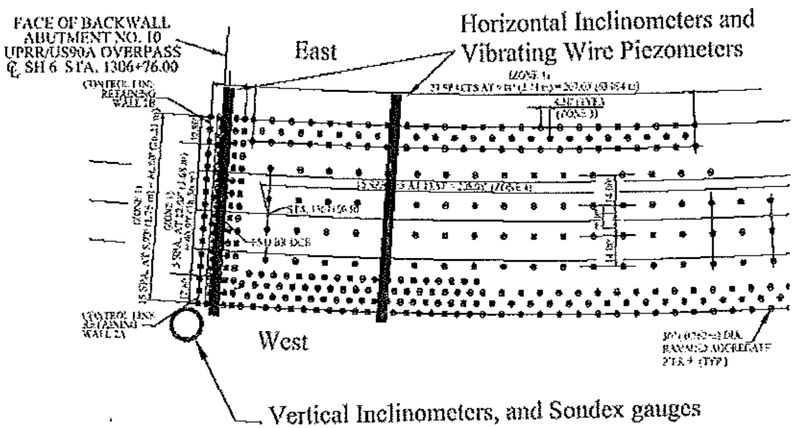


Fig. 4. Geotechnical Instrumentation Layout

The piezometer readings at the South Ramp are representative of the trend in excess pore pressures at all four structures and are shown in Figure 5. The piezometric readings registered minimal change in excess pore pressure during earth fill placement because of the radial drainage component to the closely spaced RAP elements. Pile driving operations in the bridge abutment early during construction explain the highest peak on the curve on December, 2005. Regional rise in groundwater elevation due to heavy rain activity was responsible for the gradual rise in piezometric readings observed after June, 2006.

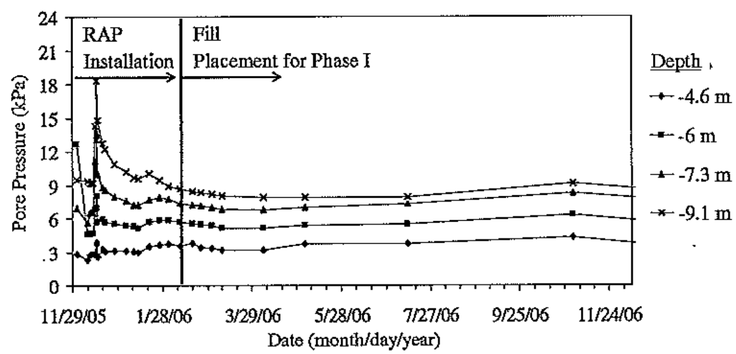


Fig. 5. Piezometric Readings at the South Bridge Abutment.

Figure 6 shows vertical a deformation profile across the bottom of the south ramp wall. Overall vertical settlement for all four structures was on the order of 5-cm (2-

inches) or less, mostly occurring during construction. Post-construction settlement was less than 1.5-cm (0.5-inch). Disturbance at ground surface from construction equipment passing near the wall face produced the apparent excessive deformation shown on the left hand side of the curve.

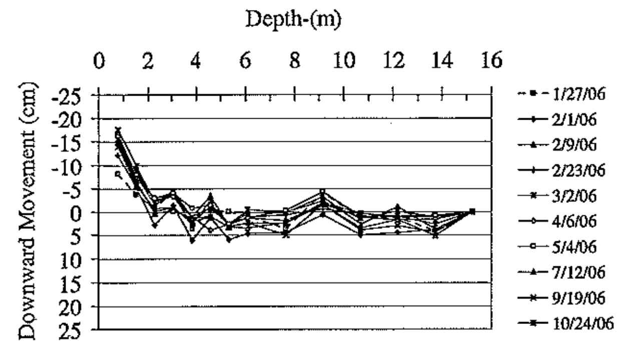
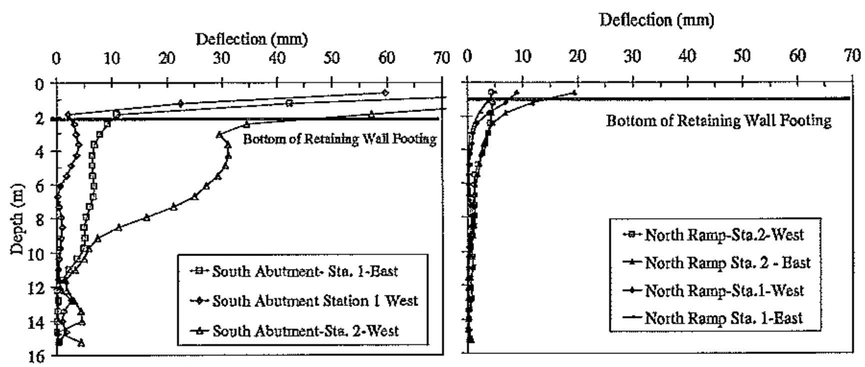


Fig. 6. Sondex settlement reading for the South Ramp.

Figures 7a and 7b show the maximum and minimum horizontal soil displacements measured near the wall face for all four structures, respectively, taken from February, 2006 through June, 2007.



a) b)

Fig. 7. Inclinometer Readings. a) Lateral displacement at South Abutment, b) Lateral displacement at North Ramp.

The lateral soil displacement for the majority of the walls was consistent with the readings at the North Ramp and generally on the order of 1.25-cm (0.5-in). The maximum lateral soil displacement was approximately 3.2-cm (1.25-in) and occurred in front of the South Abutment wall which had the maximum wall height and softer soil near ground surface. The apparent excessive deformation measured near bottom of wall footing corresponds to disturbance caused by construction equipment operating near the monitoring station.

SUMMARY AND CONCLUSIONS

Rammed Aggregate Pier soil reinforcing elements were selected to increase the factors of safety against bearing capacity, global stability and settlement control under MSE walls in compressible and unstable foundation soils. Modulus tests along with a monitoring program testified that the performance of the MSE reinforced by Rammed Aggregate Piers satisfied the design criteria established by the design team. The post-construction settlement following wall construction was significantly less than the original design. This is explained by the stiffening and improvement of the foundation soils and because of the practically immediate excess pore pressure dissipation afforded by radial drainage to the RAP elements allowing both shear strength increase, reduction in compressibility of the matrix soil and faster settlement as the wall was constructed.

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