Stabilization of Failing Slopes Using Rammed Aggregate Pier Reinforcing Elements

Jorge R. Parra[1], Member, ASCE, J. Matthew Caskey[2], Member, ASCE, Eric Marx[3], Member, ASCE, Norman Dennis[4], Member Fellow, ASCE

[1]Lead Engineer, Geopier Foundation Company, Inc., 1997 South Main Street, Suite 703, Blacksburg, VA 24060, jparra@geopier.com
[2]Principal, GFC-MidSouth, 9160 Highway 64 Suite 12, No. 134, Lakeland, TN 38002 mcaskey@geopier.com
[3]Project Manager, Fugro Consultants LP, 4252 Rhoda Drive, Baton Rouge, LA 70816 EMarx@Fugro.com
[4]Professor, University of Arkansas, 4190 Bell Engineering Center Department of Civil Engineering University of Arkansas Fayetteville, AR 72701 ndennis@uark.edu

ABSTRACT

This paper presents two case histories describing the use of Rammed Aggregate Pier soil reinforcing elements to stabilize slope failures. The analytical methods used in the design solutions are presented along with a description of the construction sequence and post-slide repair, instrumentation, and performance. This paper is of significance because it describes how a simple and cost-effective solution may be effectively implemented to stabilize unstable slopes.

INTRODUCTION

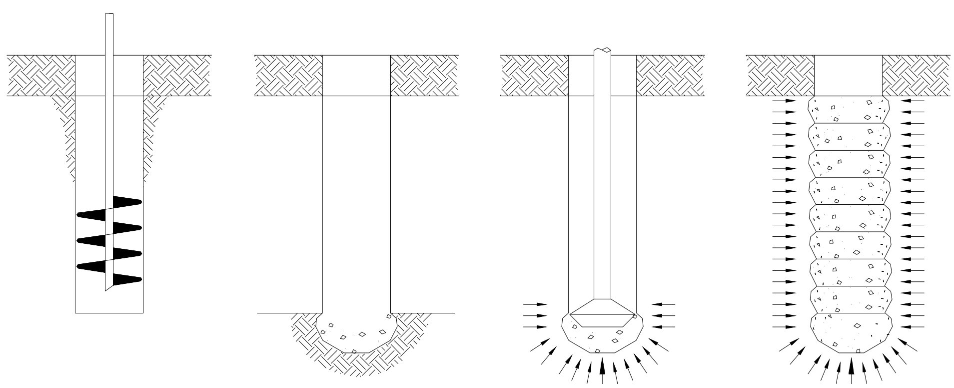
US Highway 71 near Krotz Springs, Louisiana and US Highway 167 near Batesville, Arkansas have one thing in common: both of these structures have experienced on-going landslides threatening the integrity of the roadway. For both structures, traditional slide repair solutions including secant pile walls, tie-backs, and other structural improvements were deemed to be too expensive by the state departments of transportation.

The US Highway 71 slide was caused by the construction of a new fill embankment placed adjacent to the Bayou Des Glaises. The placement of the fill embankment induced an initial deep-seated rotational slide seated in the soft Bayou clay soils. The first slide was noted prior to 2003. A second and bigger slide occurred in 2004 triggered by the placement of additional fill, affecting one lane of the roadway. At the US Highway 167 site, a similar recurring slope failure occurred. The slope failure not only affected the roadway but also endangered a nearby motel, including a swimming pool located just upslope from the head scarp of the slide.

After analyzing the slides at each site, the respective departments of transportation determined that the slides could be cost-effectively stabilized using Rammed Aggregate Pier^{TM} (RAP) reinforcing elements. The Rammed Aggregate Pier reinforcing elements improve the stability of the slope by providing significant increases in the composite shear resistance because of their high angle of internal friction of the aggregate (48 to 52 degrees). Additionally, the piers act as vertical drains reducing the potential for slope instability caused by excess pore water pressure in the slope in the slide area.

USE OF RAMMED AGGREGATE PIERS FOR SLOPE STABILIZATION

Rammed Aggregate Pier construction is shown in Figure 1. The piers are constructed by drilling 760 mm (30 inch) diameter holes to depths ranging between 2.0 and 7.6 m (7 to 25 feet) below grade, placing controlled lifts of aggregate stone within the cavities, and compacting the aggregate using a specially designed high-energy beveled impact tamper. The first lift consists of clean stone and is rammed into the soil to form a bottom bulb below the excavated shaft. The bottom bulb effectively extends the design length of the aggregate pier element by one pier diameter. The piers are completed by placing additional 0.3 m (1 foot) thick lifts of aggregate over the bottom bulb and densifying the aggregate with the beveled tamper. During densification, the beveled shape of the tamper forces stone laterally into the sidewall of the excavated cavity. This action increases the lateral stress in the matrix soil, thus providing additional stiffening and increased normal stress perpendicular to the perimeter shearing surface.



A. Make cavity.
B. Place stone at bottom of cavity.
C. Ram stone to form bottom bulb.
D. Place and ram thin lifts to form undulated-sided shaft.

Figure 1- Rammed Aggregate Pier Reinforcement Installation Procedure (Fox, N.S. and Cowell, M.J. (1998))

The design methodology for slope stabilization consists of increasing the composite shear strength parameter values within the Rammed Aggregate Pier-reinforced zones. The composite shear strength parameter values are estimated using the following equations (Barksdale and Bachus 1983, Mitchell, 1981, Fitzpatrick and Wissmann 2002):

$$\phi_{comp} = \arctan\left[R_a \tan\phi_g + (1 - R_a)\tan\phi_m\right] \quad (1)$$

$$c_{comp} = (1 - R_a)c_m \quad (2)$$

Where R_a is the area replacement ratio, ϕ_g is the friction angle of the Rammed Aggregate Pier (48 to 52 degrees), ϕ_m is the friction angle of the matrix soil, and c_m is the matrix soil cohesion.

US-71 SLOPE STABILIZATION

The southwest embankment of the southeast approach bridge ramp of US Highway 71 over Bayou Des Glaises experienced a recurring slope failure after placement of a new 0.9 m to 1.5 m (3 to 5 foot) high fill embankment as part of the construction of a new replacement bridge. The slope failure produced a 1.2 m (4 foot) high scarp and the failed mass displaced out approximately 3 m to 4.5 m (10 to 15 feet) into the Bayou Des Glaises as shown in Figure 2. The slide caused the Louisiana Department of Transportation to shut the southeast bound lane of the road because of the potential of progressive sloughing towards the centerline of the road.

Figure 2- Slope Failure at US-71, Krotz Springs, LA

A geotechnical investigation was performed and pre-stabilization instrumentation was installed by Fugro Consultants LP at the site after the slope failure occurred. The geotechnical investigation included twelve borings and one test pit as shown in Figure 3. Inclinometers were installed at the B-1 and B-2 locations. Fill material consisting of high plasticity clay with silt and organics extending to depths of up to 6 m (20 feet) over soft to medium stiff, high plasticity clay (Bayou Des Glaises) were encountered within the failed mass area (Figure 4). Static groundwater levels coincided with the slope toe elevation. The post-sliding inclinometer data indicated the failure surface extended into the Bayou clay at depths of 3 to 6 m (10 to 20 feet) below the road level.

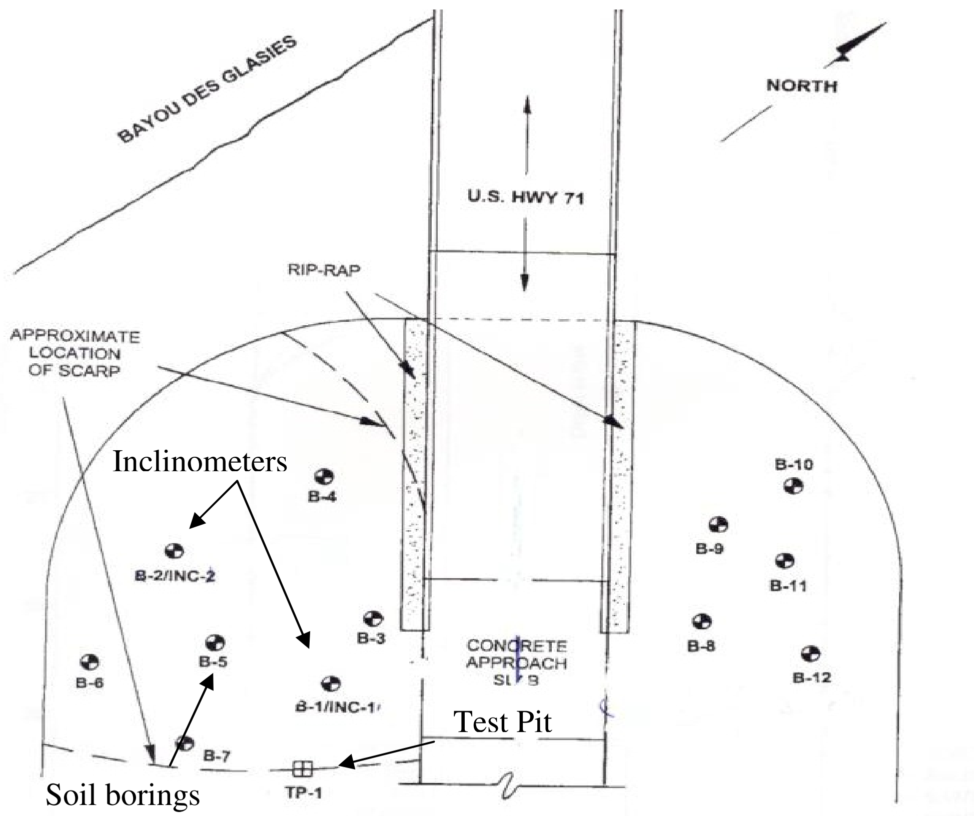


Figure 3- Plan View of Slope Failure and Boring Location/Instrumentation Plan

The RAP stabilization scheme was established on the basis of back analysis of the slope to determine the soil shear strength parameter values and pore pressure conditions that led to slope failure (Factor of Safety = 1.0). Results of the back-analyzed, long-term shear strength parameter values are presented in Table 1 below. Because of the heavy rains that occurred prior to the failure, the pore pressure condition used in the back analyses simulated a fully saturated condition throughout the slope.

Table 1- Summary of Back Analyses

Soil Description	ϕ' (degrees)	c' (psf)
Embankment Fill	18	100
Upper Foundation Clay	21	0
Lower Foundation Clay	21	0

After establishing soil parameter values from the back analyses, additional slope stability analyses were performed to determine the RAP area replacement ratio required to increase the factor of safety to the design requirements (FS=1.3 or greater). The results of the additional slope stability analyses indicated the area replacement ratio to achieve the design criteria was 42% within a 6-m (20-foot) wide area of the middle third portion of the slope as shown in Figure 4, resulting in a pier spacing of 1.1 m on-center (3.5 feet on-center). The slope face was graded in the middle portion of the slope by preparing a flat working platform at Elevation 100 m (328 feet) to allow installation of the piers to a depth of 6 meters (20 feet) to completely penetrate through the failed soil mass and critical surface to penetrate into the underlying firm clay. The piers were constructed using clean, open-graded stone to promote drainage and minimize excess pore pressure generation within the surrounding clay matrix soil. Installation of nearly 340 piers occurred in less than 2 weeks without interfering with local traffic. After pier installation the tops of the piers were capped with high plasticity clay to prevent water from infiltrating into the piers, the slope face was regarded, and the road shoulder lost after the slope failure was repaved.

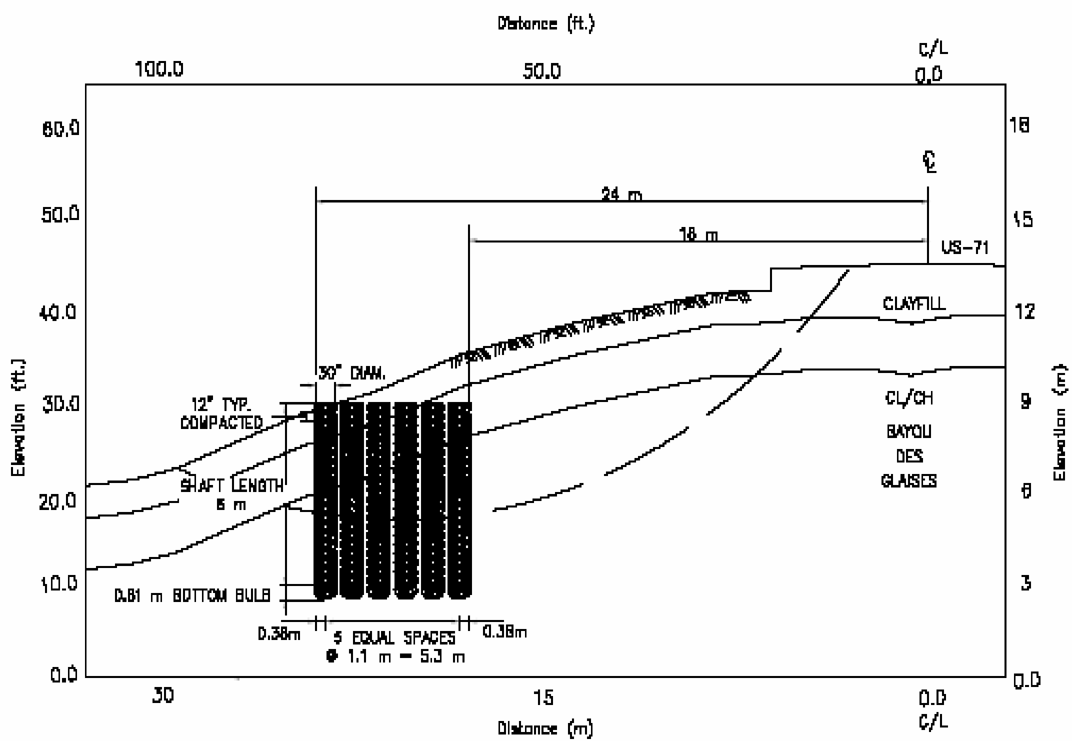
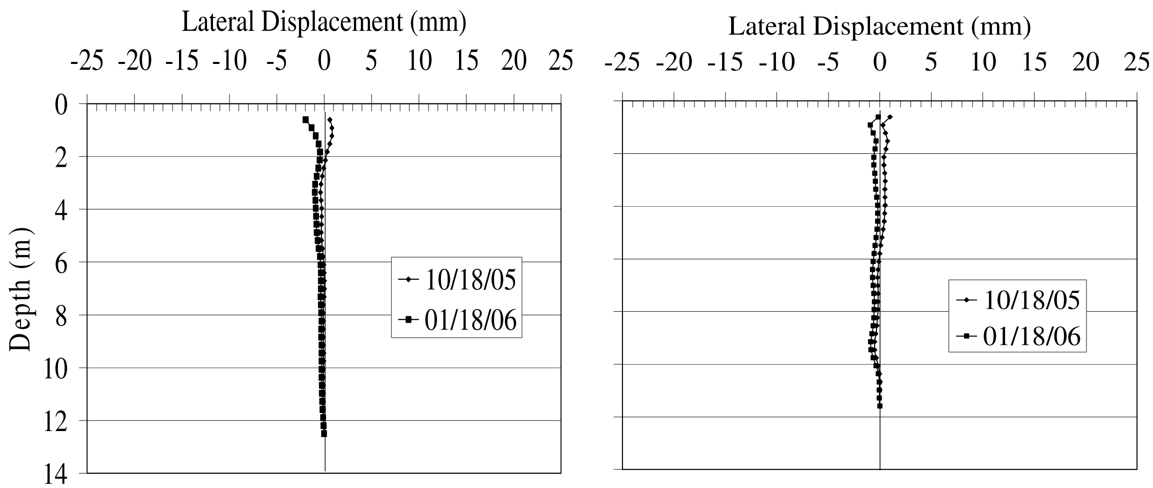


Figure 4- RAP Reinforcement and Stabilization at US-71 Slope Failure.

INSTRUMENTATIONS AND POST-STABILIZATION PERFORMANCE

Post-installation lateral displacement monitoring was conducted during a 15-month period after RAP installation. Two vertical inclinometers were installed to a depth of 13.7 m (45 feet). One inclinometer was installed near the top of the slope (uphill) and one inclinometer was installed up the slope from the RAPs area (downhill). The lateral displacement measured in the inclinometers was less than 5 mm (0.2 inches) as shown in Figure 5.



a) downhill **b) uphill**

Figure 5- US-71 Slope Repair Lateral Displacement Monitoring Results.

US-167 SLOPE STABILIZATION

In March of 2003 the Arkansas State Highway and Transportation Department requested the Civil Engineering Department at the University of Arkansas to monitor movement in an approximately 91 m long (300 feet) active slide that was encroaching into the south bound lanes of Highway 167 just South of Batesville, endangering an adjacent motel facility. The soil profile in the slope consisted of sand fill and soft sandy clay over soft to hard shale. Remnants from previous repair attempts and numerous boulder-size rock fragments were encountered at random depths within the overburden soils. Perched groundwater was encountered at depths ranging from 1 to 4 meters (4 to 12 feet) within the stratum. The slide was monitored in two locations using both inclinometers and Total Deformation Reflectometer (TDR) cables as illustrated in Figure 6. The upper inclinometer casing was installed to a depth of 12.5 m (41 feet) on the right edge of the slope crest along the traveled way. The lower casing was installed to a depth of 10 m (33 feet) about mid-way down the slope. In addition to inclinometer casings, TDR cables were grouted into adjacent 0.1 m (0.3 feet) diameter boreholes drilled to the same depth as the inclinometer casings. Both inclinometers and the TDR cables were monitored on a monthly basis for a period of six months to determine the location of the failure surface.

During the monitoring period both the inclinometers and the TDR cables indicated movement. Figures 7 and 8 illustrate that movement. Incremental movement reported in Figures 7 and 8 illustrate that a distinct failure surface developed at approximately 7 m (23 feet) in the upper inclinometer casing. That failure surface was also detected by the TDR cable at a depth of 7.5 m (25 feet) as indicated by the downward "spike" of the waveforms in the circled region in Figure 8. Significant deformation was experienced in the failed mass, causing the lower inclinometer casing to be sheared four months into the monitoring program impeding further inclinometer readings. The TDR cable, however, continued to report movement until the end to the monitoring program. The shear surface was identified passing through the soft shale.

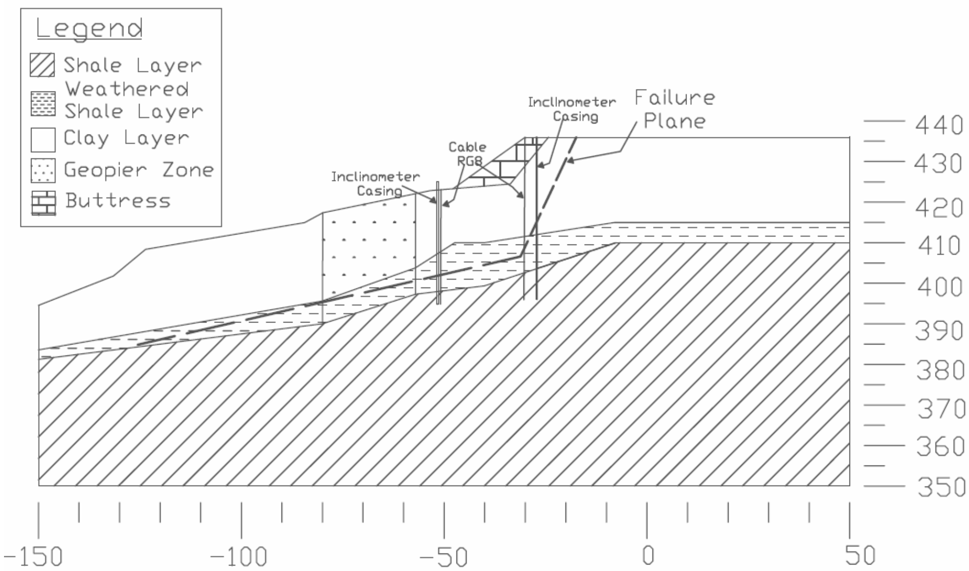
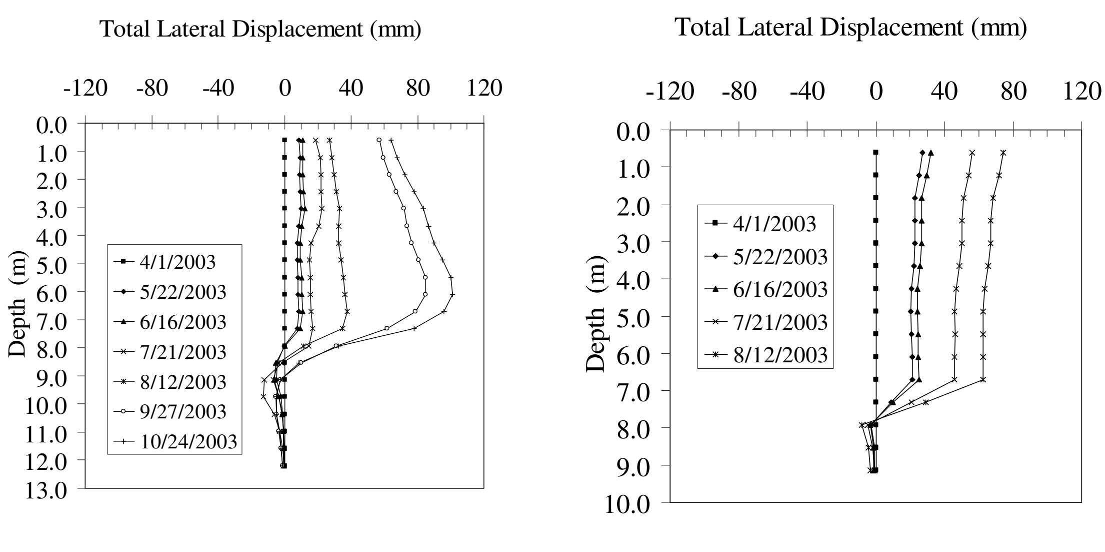


Figure 6- Pre-Stabilization Instrumentation Plan (units in Feet)



a) Upper **b) Lower**

Upper

Figure 7- Summary of the Pre-repair Inclinometer Data at Highway 167 Near Batesville, AR.

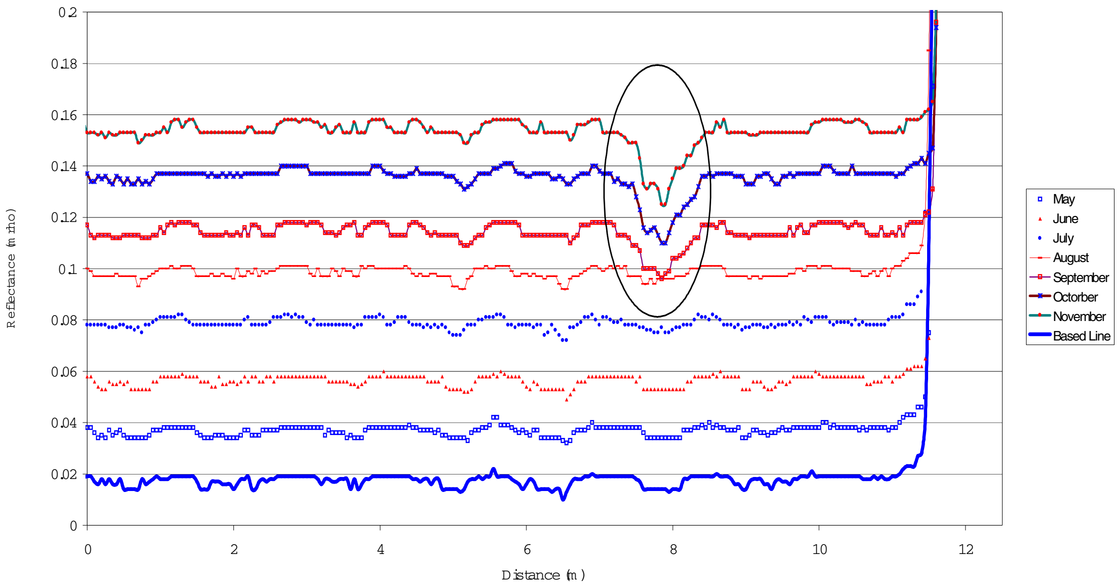


Figure 8- Comparison of Waveforms for the RG8 Cable in the Upper Borehole on Highway 167 near Batesville, AR (May – November 2003)

To obtain parameter values for the design of the stability system, a series of limit equilibrium stability analyses were performed. Results of the back-analyzed, long-term shear strength parameter values that resulted in a factor of safety of 1.0 are presented in Table 2 below. The pore pressure condition used in the back analyses simulated a full saturated condition throughout the slope.

Table 2- Summary of Back Analyses

Soil Description	ϕ' (degrees)	c' (psf)
Sand Fill and sandy clay	28	0
Soft Shale	10	0
Weathered Shale	18	0

The RAP mitigation solution for obtaining a minimum factor of safety of 1.3 resulted in an area replacement ratio of 38% within the middle third portion of the slope (identified as Geopier zone) as shown in Figure 6. This resulted in a pier spacing of 1.1 m on-center (3.5 feet on-center), and shaft lengths of 7 to 7.5 m (23 to 25 feet) to completely penetrate through the soft shale and embed the piers 0.6 m (2 feet) into the underlying firm shale. Drilling complications related to the buried boulders and groundwater infiltration into the drilled holes led to the construction of surficial bleeder ditches. The ditches were excavated to depths of about 2.4 m (8 feet) and were approximately 1.2 m (4 feet) wide. The cavities were backfilled with crushed limestone, 5 to 10 cm (2 to 4 inch) nominal particle diameter. After pier installation the slope face was regraded.

INSTRUMENTATION AND POST-STABILIZATION PERFORMANCE

Following the slide repair, new inclinometer casings and TDR cables were installed in the vicinity of the pre repair monitoring locations. In addition to monitoring movement of the slope, the Arkansas Department of Transportation also wanted to monitor water levels in the slide area. For that purpose, the inclinometer casings were converted into pseudo-piezometers by perforating the area between the grooves with 6.5 mm (0.25 inch) holes for the lowest 3 m (10 feet) of each casing. The casings were again installed in 200 mm (8 inches) diameter boreholes. However, rather than grouting the full length of the casing like the pre-repair inclinometers, the lowest 5 m (16 feet) of each borehole was backfilled with pea gravel before grouting to the surface with a weak bentonite-cement grout. TDR cables were grouted in adjacent 100 mm (4 inches) diameter boreholes using a bentonite-cement grout having an unconfined compressive strength of 1,725 kPa (250 psi). During the one year monitoring period following the repair, no significant movement was observed at any of the monitoring stations. Figure 9 provides a summary of the movement reported by the lower inclinometer. Water levels in the inclinometer casings were observed at every visit during the post-repair monitoring program. The free water surface in the upper casing ranged from 5.5 to 6.5 m (18 to 21 feet) below the ground surface while levels in the lower casing ranged from 2.6 to 4 m (8.5 to 13 feet) below the ground surface. The trend of the free water surface in both casings was downward during the monitoring period. By comparing these water levels to the stratigraphy portrayed in Figure 6, one can see the free water surface was nearly horizontal across the failure site and was located approximately 1 to 2 meters (3 to 6.5 feet) above the weathered shale.

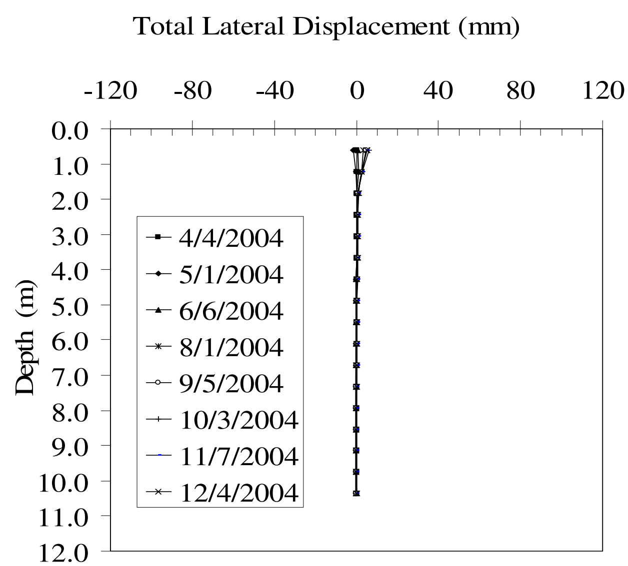


Figure 9- Post-Slope Repair Lateral Displacement Monitoring Results.

SUMMARY AND CONCLUSIONS

Rammed Aggregate Pier soil reinforcing elements were selected to effectively stabilize two slope failures along the shoulder of two major Highways. Traditional limit equilibrium methods were employed to determine the soil strength parameter values and pore water pressure conditions leading to slope failure at each slide site, and also to determine the area replacement ratio required to increase the factor of safety against slope instability to meet the design requirements. The results of the post-slope repair monitoring program indicate that installation of Rammed Aggregate Pier reinforcing elements stopped progressive lateral displacement at both failed slope sites and met the intent of the design.

ACKNOWLEDGMENTS

The authors recognize the efforts of the Rammed Aggregate Pier installer, PCI Inc., of Reinbeck, Iowa, and geotechnical consultant, Fugro Consultants of Baton Rouge, Louisiana. The authors are indebted to the Louisiana Department of Transportation and Arkansas Highway and Transportation Department for their data sharing, careful design review, comments and helpful suggestions.

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