Trunk Highway 67 Slope Repair, Yellow Medicine County, MN

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ABSTRACT: A 175 foot long section of Minnesota Trunk Highway 67 embankment experienced a slope stability failure in 2010; while there was existing distress at the site, it is likely that heavy September rainfall accelerated existing distress or promoted new adjacent instability. This highway section was along Hazel Creek, approximately 4 miles south of Granite Falls. A rapid response was needed to restore the slope and highway embankment prior to winter. Use of a column supported embankment (CSE) was considered the best option to provide strength and base support for a new roadway embankment. This approach was designed and specified as an "Aggregate Column Supported Embankment Stabilization System" (ACSESS). MnDOT also designed a Reinforced Soil Slope (RSS) with drainage and base riprap to complete the embankment repair and slope protection. Ground Improvement Engineering (GIE) designed the ACSESS system to increase the composite shear resistance and control settlement beneath the RSS. This was the first application of the Rammed Aggregate Pier® system by the Minnesota Department of Transportation (MnDOT).The paper describes the geotechnical investigations at the slide area, the stability issues, and the ACSESS and RSS repair designs as well as long term performance monitoring at the site.

1 INTRODUCTION

Problematic pavement distress along a short section of Minnesota trunk highway 67 was first brought to the attention of the Minnesota Department of Transportation's (MnDOT) Foundations Unit in June of 1998. A site investigation was conducted and an inclinometer installed. Minor movement was observed with limited impact to the roadway; moving forward, the location would be kept under study and routine patching would be conducted when needed. This continued until the fall of 2010 when heavy rains hit the region and major pavement cracking across the entire roadway occurred and, at a minimum, significant leveling and patching would be needed before, and perhaps during, the winter snowplowing season. The flooding likely contributed to ongoing embankment instability and promoted new, more widespread distress.

Soil conditions in the newly unstable area were found to be highly variable with some weak zones. This finding coupled with significant erosion at the highway embankment toe led to the initiation of a fast-track stabilization and remediation project with the intent of having the work designed and constructed between September and December of 2010. Design concepts were developed and MnDOT initiated an emergency contract and special letting for selected contractors experienced in ground improvement and stabilization. To reconstruct and support the roadway embankment, a two-component approach was used. An Aggregate Column Supported Embankment Stabilization System (ACSESS) would be installed below a Reinforced Soil Slope (RSS). The system would provide support for the uphill slope and the new roadway and provide protection from additional erosion. Ditch drainage, hydraulic protection, roadway base, pavement, and guardrail were additional work package components.

2 PROJECT BACKGROUND, SITE OBSERVATION, AND SITE INVESTIGATION

2.1 *Project Background*

In June of 1998, pavement distress was observed along a fifty foot section of roadway adjacent to a creek on Minnesota trunk highway 67, just southeast of Granite Falls, MN. The distress was characterized by pavement cracking and a roughly semi-circular depression extending into the center of the near driving lane. A soil boring, T-100, was drilled in the roadway shoulder adjacent to the distress and inclinometer casing was installed. A series of traversing-probe inclinometer readings were taken to determine if the pavement distress was caused by subsidence or slope instability. Early data suggested that the distress was the result of lateral deflection of the northern slope of the MN 67 embankment, adjacent to Hazel Creek. Site soils were highly mixed layers of platey, blocky, and fat clays occurring between layers of loamy sands and gravels. Mudstone was present about 40 feet below the roadway. The rate of movement appeared to be relatively slow following the initial readings, suggesting perhaps that after the initial pavement distress, the area had stabilized to a new equilibrium condition. A site visit in October of 1999, showed the site to be in good condition with no significant additional distress observed near the inclinometer. Periodic patching continued as a remediation whenever additional settlement and cracking was noticed by MnDOT Maintenance. The inclinometer casing remained in place; readings were taken intermittently, usually when new distress was observed. By January of 2007, movement had occurred and minor patching had been conducted by MnDOT maintenance crews. A drop in the pavement of about 2 inches was measured in the worst area of the northwest driving lane, shown in Figure 1 (left). The inclinometer readings showed relatively small movements at several elevations between 15 feet and 30 feet below the roadway.

Southwestern Minnesota and the Granite Falls area were subjected to 3 to 8 inches of heavy rain and local flooding on September 23, 2010. Following this event, larger and more significant roadway distress and down-slope embankment scarping developed along a larger section of roadway further southeast and adjacent to Hazel Creek. This new distress now impacted the entire roadway, for a linear distance of about 175 feet and was characterized by multiple areas of subsidence and cracks in several groups as shown in the right photo of Figure 1.

Figure 1. The roadway asphalt was patched sometime prior to January of 2007 (left). The 2010 distress, (right), propagated across the entire MN 67 roadway. The T-100 inclinometer installation [at left in the left photo] can be seen in the upper right of the right photo [yellow circle] taken from the other direction.

Settlement of more than 6 inches overall was seen in the pavement. Several large progressive failure scarps in the slope adjacent to the creek were observed through the brush covering the slope. The new and more significant scarps and roadway distress appeared to be promoted or triggered by the large rain event, although progressive loss-of-ground and streambank erosion at the outside of the bend in Hazel Creek can be seen in a historic aerial photograph, Figure 2.

Figure 2. The north side of the MN 67 highway embankment is adjacent to an outer stream bend of Hazel Creek. Progressive channel erosion on the outside bank of the stream appears to be a contributing factor in the movement observed on inclinometer T-100 [lower left, approximate station 208+75] and the long-term small-scale ongoing pavement distress seen from 1998 to 2010. Significant new movement and roadway distress occurred after a large rain and flood event in September of 2010.

2.2 *2010 Site Investigation*

When the asphalt settlement and pavement cracking were observed by maintenance personnel, it was clear that the 2010 distress was significantly larger and more severe than the ongoing settlement nearby. The Foundations Unit scheduled immediate drilling and CPTu investigation. Three lines of CPT soundings were advanced (C01 – C26) and three soil borings were drilled (T110, T111, and T112) to help characterize the site. The MnDOT Geology Unit also conducted a 70 meter 2D Electrical Resistivity geophysical survey along the east shoulder of the roadway starting at the approximate location of T-100 and heading southeast. The resistivity plot showing the generally problematic soils overlying the local bedrock is shown in Figure 3.

The exploration program revealed soils consisting predominantly of Granite Falls till (Wadena Lobe), a yellow to yellowish brown silt and clay rich till with abundant carbonate rocks. Underlying the till, bedrock varied between elevations of 850 and 870 feet. Above the bedrock, soil borings and CPTu soundings showed an unusual variety of highly mixed and layered materials including organic soils, crumbly clays, slickenside clays, greasy plastic silt loams, pebbles, plates of iron rich sediments, sands, gravels, residual soils, mudstone, and coal. Soil strengths, as determined from SPT blow counts, unconfined compression tests, direct shear test series were generally good but there were intermittent deposits with poor strength. While specimen behavior was consistent among test samples, in general, lab test data was only very locally representative.

The CPT was a particularly useful tool for characterizing the site as the variability was easy to distinguish on the CPT plots of tip resistance, friction ratio, and pore water pressure. Local pockets of strong and weak materials were easy to distinguish. Due to the large heterogeneity and variability in soil types, establishing meaningful stratigraphy and soil parameters to appropriately characterize and model the site was challenging, even with a relatively robust in-situ test program. The geotechnical work would eventually include 4 additional inclinometer borings: two down-slope borings outfitted with slotted casing for manual inclinometers and two up-slope borings which where ShapeAccelArray (SAA) slope monitoring systems would be installed. The initial inclinometer, T-100, needed to be removed to reconstruct the embankment.

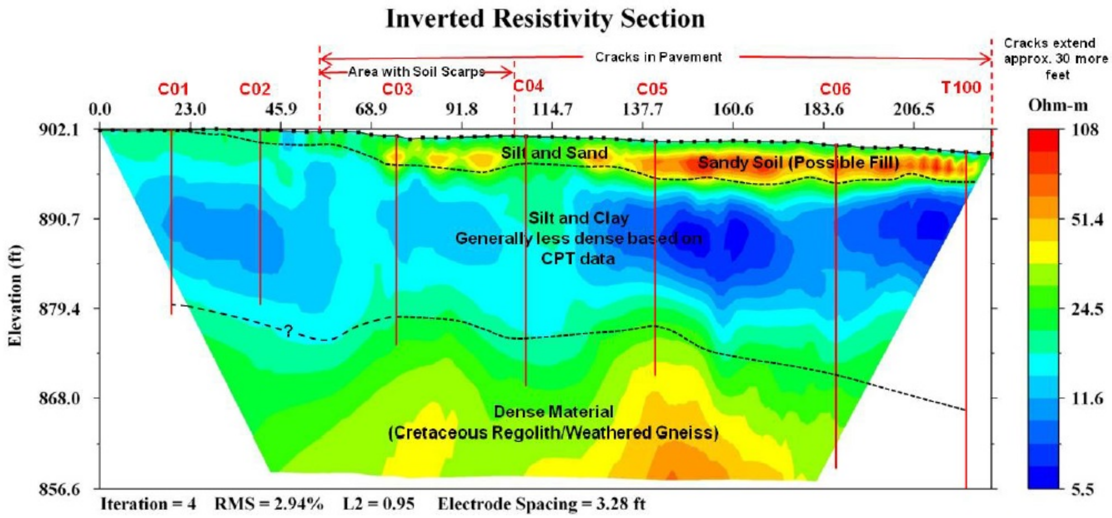

Figure 3. The resistivity survey showed weaker soils (in blue) below the roadway embankment.

2.3 *Site Information Review, Interpretation, and Recommendations*

Based on the new geotechnical investigation and historic inclinometer information, it was determined that the new region of instability was much larger than the area under study from 1998 to 2010 and this larger area was now marginally stable. More repair patching, without additional remediation or countermeasures, would likely accelerate the instability due to added driving force. While the site could possibly be re-paved to allow snow and ice plowing, it was uncertain if such a solution would last the entire winter season. It was unclear if additional distress was likely to be abrupt or a slower creep-type failure. While the previous roadway distress and the 2010 slide failure had been relatively "slow" in terms of landslide rate, some soils appeared to be potentially strain-softening which could lead to a sudden failure mechanism.

A foundation report was prepared and issued in early October of 2010 containing the site investigation information as well as an assessment of several potential design alternatives including: Expanded Polystyrene (EPS) Geofoam and Tire Derived Aggregate (TDA) [shredded tires] lightweight fill options, reinforced soil slope (RSS), and a column supported embankment (CSE) using aggregate, compacted aggregate, or stone, described in the following section. A recommendation was made to execute an emergency contract for a more substantial site repair.

3 SLOPE REPAIR OPTIONS AND CONTRACTING

3.1 *Site Considerations and Repair Options*

Several options to address the underlying embankment stability concerns were considered. Some were deemed unsuitable due to the site conditions, expense, or the length of time required for the design and construction. Reconstruction using bridge alternatives, permanent sheet piling, or complete soil remove/replace options were determined to be less viable than other remediation methods. A reinforced soil slope, lightweight fill (EPS Geofoam or shredded tires) embankment, and a [stone/aggregate] column supported embankment were considered as potential solutions. These options could be designed and constructed relatively quickly. Due to the severity of the failure extending across both driving lanes, the “Do Nothing Option” was not considered a practical alternative. Reestablishing the slope between the creek and placing hydraulic erosion control and mitigation measures were included in each option. These three alternatives were presented to the local MnDOT District during a teleconference on October 6, 2010.

Each option provided a solution to the embankment stability problem and possessed design and construction challenges and reliability and cost considerations. An abbreviated comparison matrix of options, costs, advantages, and disadvantages is included as Table 1.

Table 1. Options and Considerations for Proposed Site Solutions

Option	Cost Estimate	Advantages	Disadvantage
EPS Geofoam	$300,000 +sheeting	Significant weight reduction Specialty contractor not required Construction is not below streambed	Likely need for sheeting Compressible
Tire Derived Aggregate (TDA) [shredded tires]	$200,000 + sheeting	Reduces weight Less expensive than EPS Geofoam Specialized contractor not required	Likely need for sheeting Compressible
Reinforced Soil Slope (RSS)	$200,000	Specialized contractor not required Would provide internal stability Will buttress upslope area	No weight reduction Settlement may continue Deformation below RSS
Aggregate/Stone Column Supported Embankment (CSE)	$500,000	Reinforces area below streambed Provides bearing capacity/stiffness	Costs more than others Requires specialty contractor Does not improve slope.

*Costs were preliminary estimates of the ‘geotechnical’ stabilization and did not include sheeting/shorting, mobilization, streambank stabilization and other aspects of the work.

A desirable design would be cost effective, control total and differential roadway settlement, improve the embankment stability by providing additional shear strength, support the roadway and uphill slope, and improve drainage. Lightweight fill options possessed uncertainty in the ability to resist loading from the uphill slope area and would likely require temporary, and possibly permanent, sheeting and shoring. A reinforced soil slope (RSS) was considered a good alternative except that the failure was believed to exist outside part of the RSS area and new construction could be subject to movement near the face and below the base of the new RSS. The column supported embankment (CSE) was thought to potentially provide the largest factor of safety against additional movement resulting from continued erosion or settlement above the site bedrock. The columns would provide improved shear resistance and bearing support for the roadway embankment reconstruction. A risk using a tall CSE with a thin load transfer platform above the columns was limited confinement and erosion risk for columns near the creek.

3.2 *Preferred Concept Alternative*

The preferred solution evolved from the CSE and RSS options. A CSE system could support a continuously reinforced RSS which would in-turn, support the roadway and buttress up-slope areas. The hybrid system was detailed and specified as a non-proprietary “Aggregate Column Supported Embankment Stabilization System” (ACSESS). The design included additional elements such as a rear drainage blanket, ditch drainage system, roadway and pavement, guardrail, toe erosion protection, and topsoil and seeding for the slope face (Figure 4). To ensure the entire system acted as uniformly as possible, the primary reinforcing elements extended throughout the replaced embankment soils. MnDOT provided the design for the RSS.

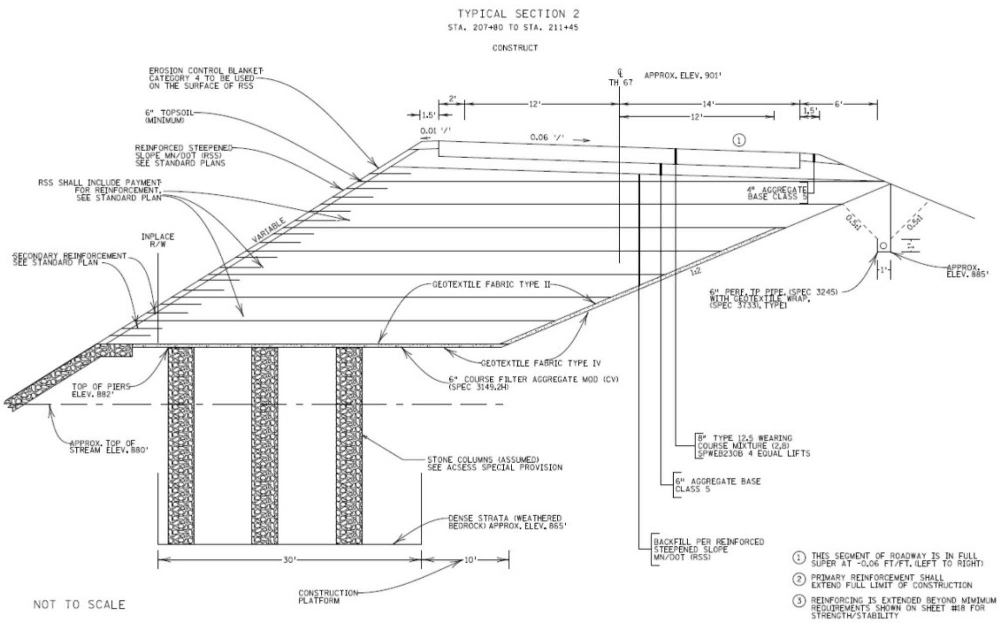

Figure 4. The repair detail showing support columns and reinforced earth area below the new roadway.

3.3 *Project Cost and Timeline*

The embankment reconstruction was conducted as an emergency repair on a tight project timeline: restore MN 67 to a drivable, and plowable, condition, prior to the winter of 2010. The geotechnical investigation was completed in two weeks; design work was compressed into a one month timeframe. The design used some elements of design-build procurement: for the ACSESS design concept layout and performance specification was provided by MnDOT, leaving the final CSE design and construction means and methods to the contractor. A project timeline is included in Table 2. At time of bid, the project cost was $850,000.

Table 2. Project Timeline.

Date	Activity Description
Sep 23, 2010	Three to eight inches of rain falls in the region flooding occurs
Oct 5	CPT Soundings Advanced; Electrical Resistivity Survey: design concepts developed
Oct 12	Letter of Interest Solicited
Oct 26	Project Quotes Due; Design Revisions Developed
Nov 3	Notice of Scope Change based on Site Investigation
Nov 8	Re-Bid Submittals Due
Nov 12	Notice to Proceed for Emergency Repair
Nov 16;18	Brush Clearing; Road Closed; Equipment Moves In
Dec 1	Geopier System Installed
Dec 9	RSS complete; Inclinometers Installed*
Feb, 2011	Cable Guardrail Installed; Temporary Road Surfacing
Spring, 2011	Final Paving (project final completion October 2012)

4 “ACSESS” CONTRACTOR DESIGN

4.1 *Proposed Construction Methods*

Mn/DOT specified a generic ACSESS column supported embankment system. The prime contractor, KGM, partnered with Geopier [designed by Ground Improvement Engineering] to develop an approach using their Rammed Aggregate Pier (RAP) system for global reinforcement of the slope embankment beneath the RSS. The work limits were along the eastern right-of-way (30 feet wide) between stations 208+75 to 210+50. The RSS was constructed above the Rammed Aggregate Pier area along with installation of drainage elements and rip-rap at the base of the slope. The Geopier® "drill and fill" RAP method was selected, by Ground Improvement Engineering, for the stiffness it produced and the visual determination of soil types by observation of drill cuttings. The method increased the composite shear strength by installation of very dense aggregate pier elements exhibiting high angles of internal friction.

The RAP elements are constructed by first drilling a shaft and then ramming select aggregate in thin lifts into the shaft using a specially-designed beveled tamper. The vertical ramming action compacts the aggregate and pushes it laterally into the sidewalls, thereby increasing the horizontal stress in the surrounding "matrix" soils. The field equipment is shown in Figure 5; the buildup of lateral stress adjacent to the RAP elements is shown schematically in the inset.

4.2 *Design Properties*

Full-scale direct shear testing and laboratory triaxial tests were conducted to determine the RAP internal friction angle. Testing indicated the friction angle varies from 49° to 52° depending on the aggregate gradation (open-graded to well-graded) as shown in the graphs in Figure 6.

Figure 5. Several Geopier installation rigs were mobilized to perform the construction. The increase of lateral stress adjacent to installed RAP elements is shown schematically in the inset, at right.

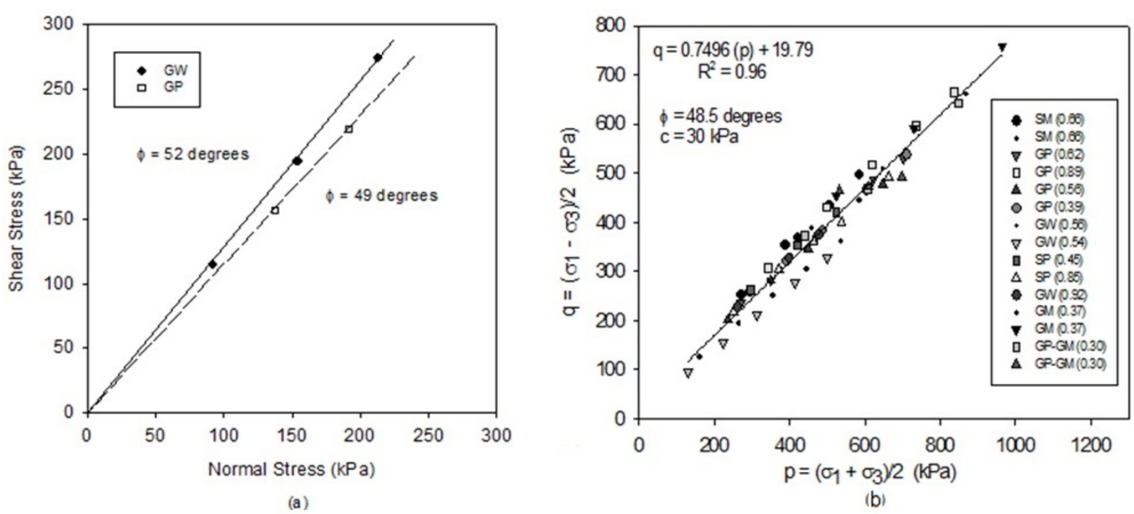

Figure 6. Rammed Aggregate Pier field shear testing results (a, left) and lab testing results (b, right); results are not from this site.

The installation of Rammed Aggregate Piers (RAP) increases the composite shear strength parameter values within the aggregate pier-reinforced zones. The composite shear strength parameter values are estimated using the following equations (Barksdale and Bachus 1983, Mitchell et al. 1981, FitzPatrick and Wissmann 2002):

Equation 1: $\varphi_{comp} = \arctan [\, R_a \tan \varphi_g + (1 - R_a) \tan \varphi_m]$

Equation 2: $c_{comp} = [\, (1- R_a\,)\, c_m\,]$

Where R_a is the area replacement ratio, φ_g is the Geopier friction angle, φ_m is the matrix soil friction angle, and c_m is the matrix soil cohesion. The Geopier RAP-reinforced zone was designed to intersect the critical shearing surfaces located beneath the slope. Within the reinforced zone, the composite friction angle values and cohesion (Equations 1 and 2) represent the composite shear strength of the soil zones reinforced by the aggregate elements. Analyses were performed on a trial and error basis; the area coverage (R_a) of the Geopier RAPs was varied until the required factor of safety (FS) was achieved. The strength properties of the in-place soils were estimated based on back-calculations assuming a FS = 1.0 for drained conditions. Analyses were performed using Slope/W software.

4.3 *Analysis*

An angle of internal friction of 42° was conservatively used for the aggregate pier elements. The ratio of the soil below the RSS to the area replaced by the RAP, R_a, was determined using a Geopier spacing of 4.5 ft. on-center and a diameter of 30 in. for each element. The RAP spacing was determined on a trial basis as necessary to achieve a factor of safety against instability of 1.3. This analysis was for failure surfaces that intersected the RAP-reinforced zone. A plot of the FS = 1.0 failure condition and the proposed remediation with FS = 1.3 is shown in Figure 7.

Equation 3: $R_a = \{(1 \text{ element}) * [\pi\, (\text{diameter})^2/4]\} / (\text{spacing})^2$

$R_a = 0.242$

The design required that the RAP elements penetrate the native soils and terminate in either dense sand or refusal in weathered rock. The grade was established at elevation 882 ft.; the explorations encountered the termination strata between approximately elevations 876 ft. to 861ft.

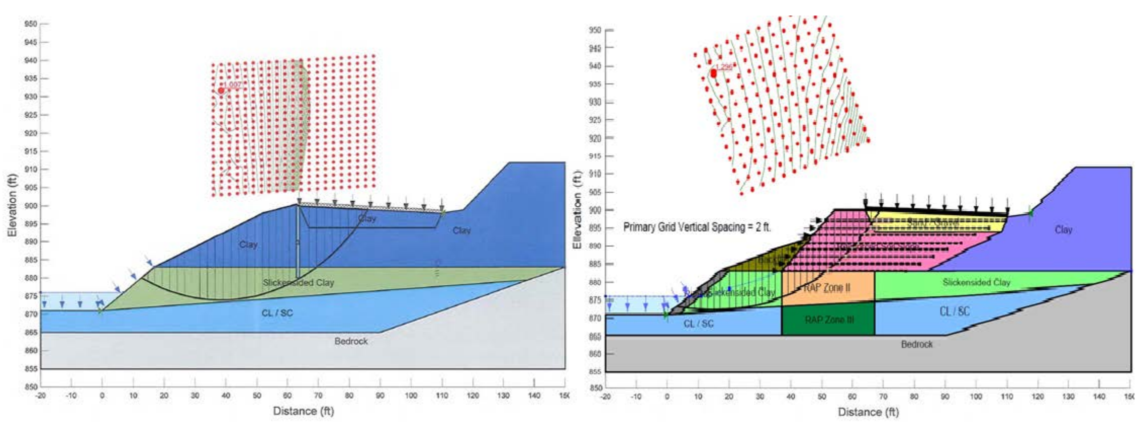

Figure 7. Plots of the Slope/W FS calculations for both back-calculation of the slope properties at imminent failure; FS = 1.0 (left) and the proposed RAP elements and the RSS above (right); FS = 1.3.

5 ACSESS CONSTRUCTION AND QUALITY CONTROL

The contractor design used 287 RAP elements in seven rows between roadway stations 208+75 and 210+50. The elements were constructed between November 22 and December 1, 2010. The prime contractor graded a 40 foot wide level work area for equipment access. The installation grade was established at elevation 882 feet. Each RAP element was 30 inches in diameter and extended to the termination strata (weathered bedrock) between elevations 867 and 861 feet. The aggregate consisted of ¾-inch to 2-inch diameter clean crushed rock for the entire shaft. The RAP element layout is shown in Figure 8.

Bottom Stabilization Tests (BST) were performed, first on the initial production piers and then on at least 10% of the RAP elements during production. The purpose of the BST was to verify that the installation of the RAP element had achieved general stabilization in the bottom portion of the shaft. The test also provided a method for comparing the production piers to the first successfully installed piers. A BST was done on the initial three production piers and at the beginning of each production day to provide quantitative performance information.

After ramming a lift, BSTs were performed by placing a reference bar over the cavity, marking the tamper shaft, applying energy to the tamper for an additional 15 seconds, and measuring the downward deflection of the tamper shaft compared to the initial reference line. If the measured vertical movement exceeded 150% of the BST value measured during the initial successful production piers, tamper energy was further applied to increase the compaction of the bottom bulb. The BST test procedure was then repeated. If the movement still exceeded 150% another lift was rammed into place and the BST measured. If the BST values in the lower 2/3 of the pier shaft remained above 150% of the initial production piers then the element was re-drilled and re-installed. A full-time quality control technician was on site to document the RAP construction, including element identification number, drill diameter, soil types encountered, install date, bottom and top elevation, number and type of aggregate lifts, and BST results.

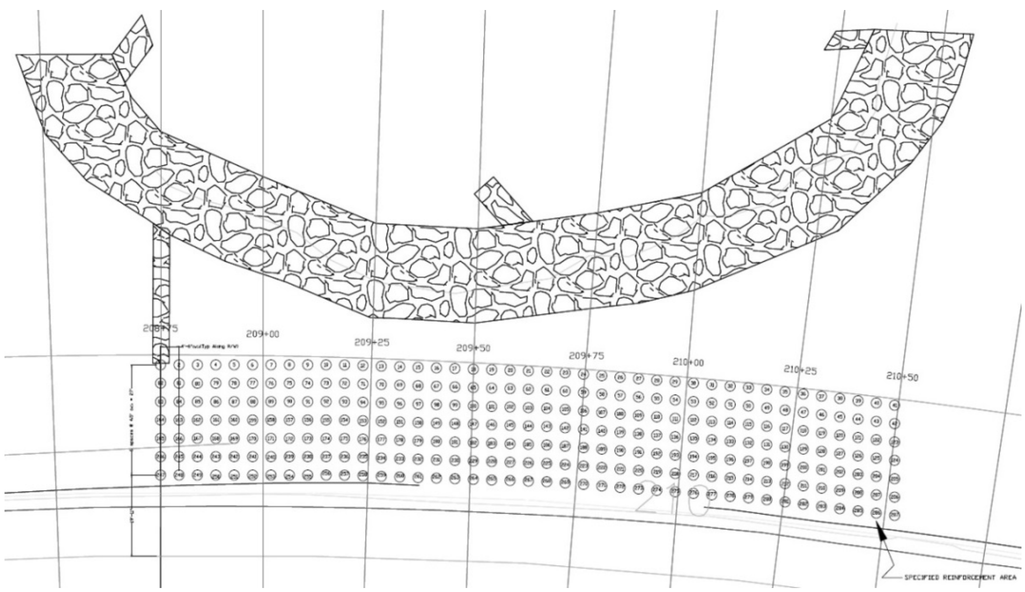

Figure 8. The Geopier layout is shown relative to the stream and the protective slope riprap.

6 ADDITIONAL PERFORMANCE MONOITORING

Following the installation of the CSE, as the RSS was being constructed, and just before a snowstorm was predicted, on December 9, 2010, MnDOT advanced two borings downslope and slightly outside on either end of the RSS area and installed inclinometer casing. Traversing-probe inclinometer readings were taken periodically. Limited movement was seen below the RSS. Deep lateral deflections were near zero in the months immediately following CSE/RSS installation and less than one-half inch five years after construction. Larger movements of up to 4 inches in the near surface are believed to be related to frost or shifting of the near-surface channel rip-rap. Inclinometer plots are shown in Figure 9. In 2011, some distress above the roadway was noticed and two ShapeAccelArray sensors were installed in the upper hillside. No deepseated movements have been observed; some movement is occurring in the upper hillside, potentially from disturbance and stress relief from project excavation or dewatering.

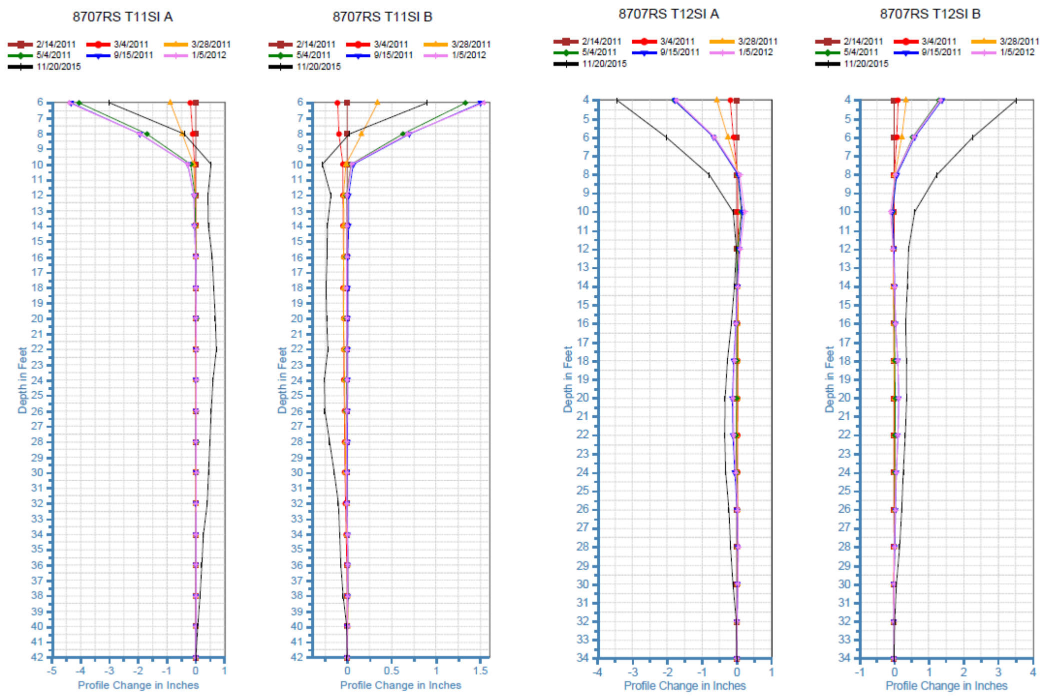

Figure 9. Inclinometer plots showed movement below the RSS was limited to less than one half inch. The movement seen at the top of the plots is related to tilting of the above-ground protective casing.

7 CONCLUSIONS

A significantly distressed 175 foot embankment on MN highway 67, was successfully reconstructed and stabilized in less than three months, meeting the goal of having the slope repaired prior to winter maintenance season. A blend of contract requirements incorporating both performance based specifications and means and methods specifications were used to construct a MnDOT designed, reinforced soil slope above a, contractor designed, column supported embankment. Rammed Aggregate Piers® were used to increase the composite shear resistance and control settlement beneath the RSS. Based on visual observations and inclinometer monitoring, the $850k embankment repair and stabilization solution is performing well.

8 REFERENCES

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