Geosynthetics, J. Kuwano & J. Koseki (eds)

Landslide stabilization using an MSE buttress at a U.S. Superfund remediation site

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ABSTRACT: This paper describes the use of geosynthetics in the stabilization of an active landslide. The project site was used for many years for the open burning of waste, with burn residue described as solid waste residue (SWR). The site sits on a 50-m high sloping hillside underlain by colluvium and bedrock. A remedial design involving capping was developed by others and in 1997 remedial construction began. Only after the start of construction was it discovered that the slope was geologically unstable. The authors were retained to investigate the slope instability and re-design the remedial action plan for the site. The paper describes the site geotechnical evaluation, design of an MSE slope stabilization buttress, and design of remedial action components. During construction, a field geosynthetic survivability demonstration test was performed to evaluate the feasibility of using site soils with a larger than specified maximum particle size as backfill for the MSE buttress. Survey monuments were installed over the capped slope and these have been monitored on a quarterly basis for the past five years. These monitoring data are presented.

1 INTRODUCTION

The project is located in southeastern Ohio, USA, in the Appalachian Plateau Physiographic Province. The site was used from about 1940 to the late 1970's for the burning of uncontrolled waste. After the burn site was closed down in 1980, it remained contaminated with ash and debris containing heavy metals and polycyclic aromatic hydrocarbons (PAHs); this material was described by the U.S. Environmental Protection Agency (USEPA) as solid waste residue (SWR).

In 1986, USEPA placed the site on its Superfund National Priorities List, meaning the site was a priority for clean-up. The site sits on a 50-m high sloping hillside underlain by colluvium and bedrock. A remedial design involving large debris removal (i.e., cars, metal drums, etc.), reduction in the size of the SWR footprint by consolidation of material onto one portion of the slope, and final capping of the consolidated SWR, was developed by others and in 1997 remedial construction began. Only after the start of construction was it discovered that the slope was geologically unstable and the surficial SWR and colluvial soil was actively creeping downslope. Construction on the project was halted and the authors were retained to investigate the instability and develop a slope stabilization plan. The condition of the site when construction was halted is shown in Figure 1. The investigation and back analysis of existing conditions, approach to stabilization, construction, and post construction performance are presented.

Figure 1. Site conditions in 1997 showing slide scarps at multiple locations

2 INVESTIGATION AND ANALYSIS

An extensive geotechnical investigation was carried out that included soil borings and rock corings and the installation and monitoring of slope inclinometers, piezometers, and settlement points. Measured slope displacements with depth in one of the inclinometers are shown in Figure 2. The displaced shape of this inclinometer is typical of movements observed in other inclinometers at the site. The inclinometer results revealed that slope movements were limited to the SWR and colluvial soil that overlies interbedded sandstone and mudstone bedrock. The locus of slope movement was found to be in the colluvium near the bedrock interface.

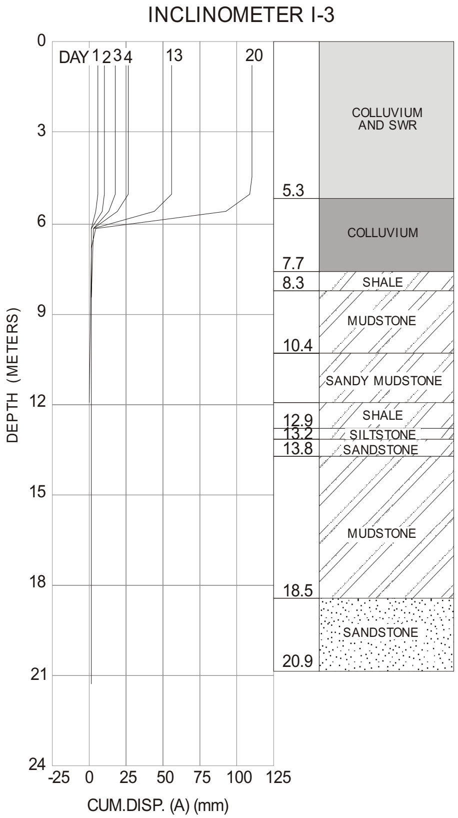

Figure 2. Typical slope inclinometer results

The investigation also included laboratory shear strength testing of the colluvium using a torsional ring shear device. This testing was performed to obtain residual shear strength parameters along the existing colluvium slip surfaces. Torsional ring shear testing results are described in detail in Morgan et al., 2000, and are summarized in Table 1. Based on the observed correlation in Table 1 between plasticity index (PI) and residual friction angle, coupled with numerous measurements of colluvium PI, an average colluvium residual friction angle of 16.5 degrees was established for purposes of slope stability analysis. A lower bound colluvium friction angle of 12 degrees was also established.

Table 1. Summary of index and torsional ring shear test results

Plasticity Index	Soil Classification	% Clay	Residual Shear Strength (degrees)
13	CL – Lean Clay	21.2	26.1
13	CL – Lean Clay	21.2	27.7
13	CL – Lean Clay	21.2	27.8
15	CL – Lean Clay	31.9	21.3
17	CL – Lean Clay	37.4	12.4
17	CL – Lean Clay	37.4	11.1
28	CH – Fat Clay	56.0	14.6
22	CL – Lean Clay	38.4	15.9
36	CH – Fat Clay	57.0	12.8

Piezometer data indicated a rapid response to rain events with the build up of water head in the SWR and colluvium above bedrock. Review of field instrumentation data revealed that slope inclinometer movement rates were temporally correlated to periods of rainfall. The measured water level in piezometers ranged from zero to 1,5 m above the colluvium/bedrock interface with maximum water levels occurring shortly after large precipitation events.

A back-analysis of existing slope conditions was performed using a 2-D limit equilibrium slope stability model, colluvium residual strengths obtained as described above, and varying pieziometric levels above the bedrock interface. Typical slip surfaces considered in the back-analyses are depicted in Figure 3.

The variation of specific water head above the colluvium/bedrock interface versus residual friction angle of the colluvium producing a calculated factor of safety of unity is plotted in Figure 4. Also shown on this figure is the average residual friction angle established from the laboratory testing program and the corresponding water head required for a calculated factor of safety of 1. The predicted water levels are within the range of water heads measured in the piezometers installed as part of the geotechnical investigation. The results of the field program, laboratory testing program, and back-analyses plotted in Figure 4 for the slip surfaces shown in Figure 3 led the authors to the conclusion that the slope was marginally stable under zero water head conditions, but continuing downslope movement of SWR and colluvium would occur in response to significant precipitation. This prediction is consistent with observations in the field. Based on this conclusion, any revised environmental remedy for the site would need to also stabilize the slope to prevent continued movement. The approach to stabilization is discussed in the next section.

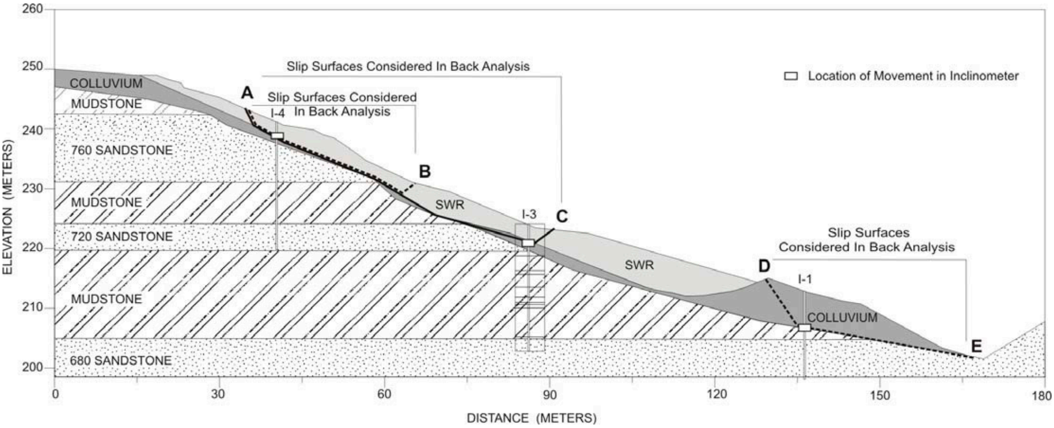

Figure 3. Slip surfaces considered in back analysis

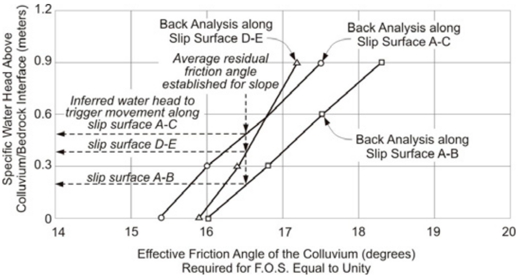

Figure 4. Relationship between water head and effective friction angle for a factor of safety of unity

3 APPROACH TO STABILIZATION

A slope stabilization plan was developed incorporating a geosynthetic-reinforced, mechanically stabilized earth and crushed rock buttress (MSE/CR buttress) keyed into bedrock near the toe of slope. A cross-section drawing of the MSE/CR buttress is presented in Figure 5. Compacted crushed rock having a high angle of internal friction comprises the lower portion of the buttress. The crushed rock portion of the buttress disrupts and intersects the zone of critical potential slip surfaces within the colluvium. The overlying MSE portion of the buttress was constructed on top of the crushed rock to both increase the normal stress in, and hence frictional strength of, the crushed rock and also to retain the SWR and colluvium consolidated from other areas of the site.

In addition to the MSE/CR buttress, the revised remedial design for the site included internal drainage features and a final cover system over the SWR retained by the MSE/CR buttress. This final cover system is consistent with that incorporated into the original design for remediation of the site. Details of the final cover system are shown in Figure 6. A gravel drain is incorporated between the MSE buttress and the retained impacted material. The function of the drain is to remove liquids that collect behind the buttress and limit the buildup of hydraulic head in these materials. The liquids from the impacted fill may contain hazardous constituents leached from the SWR. For this reason, the pipe at the base of the drain discharges to a sump from which the liquids are collected and removed from the site for treatment.

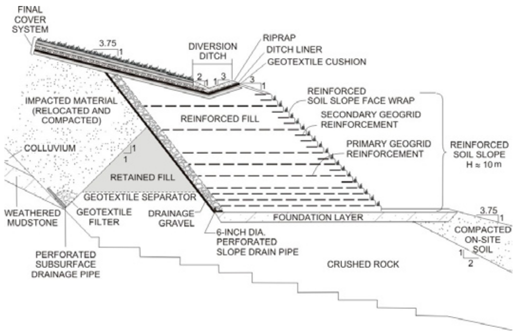

Figure 5. MSE/CR buttress cross-section

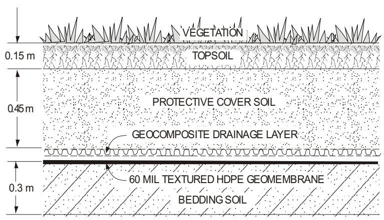

Figure 6. Final cover cross-section

Detailed stability analyses were conducted as part of the design of the MSE/CR buttress. Global stability analyses of the MSE/CR buttress and retained material were conducted using 2-D limit equilibrium analyses. Analyses were performed to calculate global factors of safety along both existing slip surfaces at or above the colluvium/bedrock interface and deeper potential slip surfaces through the fractured bedrock.

Global stability analyses were performed for a conservative end-of-construction condition wherein all stabilization and remediation components were constructed, but drainage of water in the SWR and colluvium had not yet occurred for the condition immediately after the installation of the final cover system. For this condition, it was assumed that prior precipitation had resulted in a build up of water head during construction above the colluvium/bedrock interface similar to that observed during the geotechnical investigation, but this water head had not yet drained out. This condition is conservative because construction of the final cover system essentially eliminates precipitation infiltration and water already in the SWR will drain rapidly.

The soil and rock shear strength parameters used in the global stability analyses are summarized in Table 2. The analyses were performed with two shear strength parameters for the colluvium, which represent average and lower bound values based on the results of the geotechnical investigation program. A minimum calculated factor of safety of 1.5 was required for the average colluvium shear strength parameters and 1.3 for the lower bound shear strength parameters.

Table 2. Soil shear strength properties used in slope stability analyses

Soil or Rock Description	Soil Friction Angle, Degrees	Cohesion, KPa
SWR	30	0
Colluvium (Average)	16.5	0
Colluvium (Lower Bound)	12	0
Relocated and Compacted SWR	35	0
MSE Fill	30	0
Crushed Rock	40	0
Mudstone and Shale	30	24

Internal stability of the geosynthetic reinforced portion of the MSE buttress was analyzed using a limit equilibrium slope stability methodology presented in Elias and Christopher, 1996. The reinforcement is represented by a concentrated force in the soil mass that intersects the potential slip surface. The analysis results provide the required geosynthetic spacing, tensile properties, and lengths. Table 3 presents the specified geosynthetic tensile properties. A geogrid manufactured of high tenacity polyester multifilament yarns, woven in tension, and finished with a latex coating was used for the primary geosynthetic reinforcement layers, and extruded biaxially oriented polypropylene geogrid for the secondary geosynthetic reinforcement layers (for the slope face).

Table 3. Required geogrid strength properties

Geogrid	Allowable Tension, kN/m	Serviceability state strain limit	Method
Primary	31	10	Part 6, Task Force 27 Report
Secondary	3	10	Part 6, Task Force 27 Report

4 CONSTRUCTION

Construction of the buttress required excavation at the toe of the quasi-stable slope. To maintain stability during construction, excavation was conducted incrementally along the toe of slope, in sections approximately 15 m wide. Each section was excavated down to intact bedrock and back-filled with crushed rock prior to commencing with excavation and backfilling of another section. The back slope of the excavation was monitored for movement before and during excavation and during backfilling. Once the crushed rock component of the buttress was in place, the MSE portion of the buttress was constructed. Placement of impacted material behind the buttress occurred concurrently with buttress construction (Figure 7). As construction progressed, additional impacted material was discovered in other areas of the site. When this occurred, the feasibility of increasing the MSE buttress height to accommodate this additional material was evaluated. The results of stability analyses showed that it was possible to increase the height of the MSE portion of the buttress, using an additional layer of geosynthetic reinforcement and additional lifts of compacted backfill, to accommodate the additional material. The completed buttress is shown in Figure 8. The site after installation of the geomembrane component of the final cover system is shown in Figure 9.

The specified maximum allowable particle size for MSE buttress fill was based on construction survivability test results provided by the geogrid manufacturer. Based on these tests, the specifications prepared by the authors required MSE buttress fill (which is different from the crushed rock used for the lower portion of the buttress) to be screened to a 20 mm maximum particle size. To expedite construction, a request was made by the contractor to allow for a larger maximum particle size. A field geosynthetic survivability demonstration test was performed to evaluate the feasibility of the proposed larger fill. In the demonstration test, a full design length of geogrid was installed in accordance with the specifications using the proposed larger fill particles. The length of geogrid was then exhumed and visually checked for damage. The exhumed geogrid was observed to be in good condition and samples were shipped to a laboratory for tensile testing of single ribs. A control sample of geogrid from the site stockpile was also shipped to the laboratory. Laboratory test results are summarized in Table 4. The test results did not reveal any significant difference in average geogrid rib strengths between the exhumed and control samples. Based on these results, the contractor was allowed to increase the maximum particle size for MSE fill to 50 mm, thereby avoiding the need for screening.

Figure 7. Placing impacted material behind MSE/CR buttress

Figure 8. Front of MSE/CR buttress

Figure 9. Site after installation of geomembrane

Table 4. Geogrid survivability test results

Sample	No. of Ribs Tested	Average Rib Strength, kg	Standard Deviation, kg
Control	10	211.7	15.1
Exhumed	30	211.5	19.5

5 POST CONSTRUCTION PERFORMANCE

Construction of the remedy was completed in summer 2000 and the site has been monitored for the past five years. The monitoring results are summarized below.

Drain monitoring indicates that there has been no flow from behind the MSE portion of the buttress during the past five years. This observation confirms that the geomembrane/GCL cap has been effective as a hydraulic barrier in isolating retained impacted material from precipitation.

Nine survey monuments were installed on top of the final cover system at the end of construction. Three monuments were installed across the upper section of the slope, three across the middle of the slope and three across the lower slope near the back edge of the MSE portion of the buttress. The monuments on the upper and middle slope have shown essentially no movement over the past five years. The monuments directly behind the MSE portion of the buttress have shown on the order of 60 to 75 mm of downward vertical movement with essentially no horizontal movement. The observed vertical movement is attributed to fill settlement as demonstrated in Table 5. Information on the total thickness of natural soil, relocated SWR, and final cover soils over bedrock at each monument location is summarized in the table. The corresponding settlement from 19 September 2000 to 23 September 2004 is also presented as is the percent vertical compression defined as measured settlement divided by total soil thickness. The vertical compression calculated at each location is relatively consistent. The observed vertical compression throughout the slope is attributed to compression of the soil and SWR under self weight. The observed settlement in the vicinity of the buttress is greater because the fill is thicker.

A site visit was conducted in November 2005 as part of scheduled monitoring. The MSE/CR buttress (Figure 10) and surface of final cover system were observed to be in good condition (Figure 11).

Table 5. Measured vertical movement and corresponding vertical compression

General slope Location	Thickness of soil and impacted material over bedrock, m	Measured settlement, mm	Vertical Compression, %
Upper	2.9	15	0.52%
Upper	3.7	18	0.49%
Upper	2.5	12	0.49%
Middle	5.6	15	0.27%
Middle	7.5	24	0.33%
Middle	5.2	27	0.53%
Lower	10.3	61	0.59%
Lower	13.0	76	0.59%
Lower	11.4	58	0.51%

Figure 10. MSE/CR buttress in November 2005

6 CONCLUSIONS

At a U.S. Superfund site, colluvial soils on a 50 m high slope experienced downslope movements. The slope therefore required stabilization prior to construction of the final environmental remedy for the site. A composite MSE/CR buttress was used to both stabilize the slope and provide capacity for the consolidation of SWR material from other areas of the site into a confined area behind the buttress. Once the SWR was consolidated, the area

Figure 11. Top of MSE/CR buttress and storm water controls (left side) and RCRA final cover system (right side) five years after construction

was capped with a multi-component soil and geosynthetic cover system. Based on five years of post-construction monitoring the MSE/CR buttress has stabilized the slope and the final cover system is isolating SWR from precipitation.

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REFERENCES

American Association of State Highway and Transportation Officials – Associated General Contractors of America – American Road and Transportation Builders Association (AASHTO-AGC-ARTBA) Joint Committee, Subcommittee on New Materials, Task Force 27 Report, "In-Situ Soil Improvement Techniques," August 1990.

Elias, V. and Christopher, B.R., "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines," FHWA Report No. FHWA-SA-96-071, 1996.

Morgan III, R.A., Beech, J.F., Monteleone, M., "Case History: Characterization of Physical Conditions at the Vandale Junkyard Superfund Site", *Proceedings, 53rd Canadian Geotechnical Conference,* Montreal, Canada, October 2000.