APPLICATION AND ANALYSIS OF ANCHORED GEOSYNTHETIC SYSTEMS FOR STABILIZATION OF ABANDONED MINE LAND SLOPES

by

Stanley J. Vitton, M. Frank Whitman, Wendell W. Harris, and Robert Y. Liang

Abstract. An anchored geosynthetic system (AGS) was used in the remediation of a landslide associated with an abandoned coal mine located near Hindman, Kentucky. In concept, AGS is a system that provides in-situ stabilization of soil slopes by combining a surface-deployed geosynthetic with an anchoring system of driven reinforcing rods similar to soil nailing. Installation of the system involves tensioning a geosynthetic over a slope's surface by driving anchors through the geosynthetic at a given spacing and distance. By tensioning the geosynthetic over the slope's surface, a compressive load is applied to the slope. Benefits of AGS are described to include the following: (1) increase soil strength due to soil compression including increased compressive loading on potential failure surfaces, (2) soil reinforcement through soil nailing, (3) halt of soil creep, (4) erosion control, and (5) long term soil consolidation. Following installation of the AGS and one year of monitoring, it was found that the anchored geosynthetic system only provided some of the reported benefits and in general did not function as an active stabilization system. This was due in part to the inability of the system to provide and maintain loading on the geosynthetic. The geosynthetic, however, did tension when slope movement occurred and prevented the slope from failing. Thus, the system functioned more as a passive restraint system and appeared to function well over the monitoring period.

Additional Key Words: slope stability, landslide remediation, geotextiles.

Introduction

A significant problem in abandoned mine lands is the stabilization of unstable slopes. According to Iannacchione et al., (1994), the Abandoned Mine Lands program, administered by the Office of Surface Mining (OSM), has spent approximately $64 million remediating 425 landslides between 1979 and 1992, at an average cost of $152,000 per landslide. In Kentucky alone, $45 million was spent during this period on 268 landslides.

In an attempt to reduce the cost of landslide remediation as well as to provide more efficient alternatives for landslide remediation, several new systems have been considered. One of the systems considered is an anchored geosynthetic system (AGS), which was proposed by Koerner (1984, 1985, 1986a) and Koerner and Robbins (1986c) in the mid-1980s. To evaluate this system an AGS was installed and monitored on a landslide associated with an abandoned mine site near Hindman located in Eastern Kentucky (Figure 1).

The basic function of an AGS is to provide active stabilization of the slope through tensioning a geosynthetic over a slope using ground anchors as illustrated in Figure 2. As the soil beneath the

[1]Paper presented at the 13th Annual National Meeting American Society for Surface Mining and Reclamation, Knoxville, Tennessee, May 18 to 23, 1996.

[2]Stanley J. Viiton is Assistant Professor of Civil & Environmental Engineering, Michigan Technoloical University, Houghton, MI 49931; M. Frank Whitman is an Engineer, Testing Incorporated, Decatur, AL 35602; Wendell W. Harris is Assistant Professor of Civil & Environmnetal Engineering, University of Alabama, 35487; Robert Y. Liang is Professor & Head of Civil & Environmental Engineering, University of Akron, OH 44325.

Proceedings America Society of Mining and Reclamation, 1996 pp 256-266
DOI: 10.21000/JASMR9601

https://doi.org/10.21000/JASMR96010256

geosynthetic deforms, membrane stresses develop in the tensioned geosynthetic and impart a compressive load onto the slope, which increases the stability of the slope. According to Koerner, anchorage of the geosynthetic is achieved with small diameter, ribbed steel rods (rebar) that are driven into the soil using hand held tools such as a vibropercussion hammer. The anchors are driven on a prescribed grid pattern through the geosynthetic, generally at right angles to the ground surface, to approximately 75 to 90% of their designed depth. The geosynthetic is then fastened to the anchor and the anchor is driven the remaining distance, thereby tensioning the geosynthetic and creating a curved geosynthetic-soil interface as the soil deforms the soil below the geosynthetic. This tension and curvature imparts compressive stress to the soil and an uplift loading on the anchor. According to Vitton (1991), this compressive stress, σ_{nr}, which is applied to the soil from the geosynthetic through membrane action, is directly related to the tension, N, in the geosynthetic (developed by the driving force of the anchor) and inversely related to the radius of curvature, r_c, of the geosynthetic-soil interface (from the deformation of the soil beneath the geosynthetic) as follows

$$\sigma_{nr} = \frac{N}{r_c} \qquad (1)$$

Koerner (1986a) lists the immediate and long term benefits of utilizing AGS for slope stabilization as follows:

1. Immediate slope stabilization due to bending and shearing resistance of the anchors.
2. Immediate halt to long term slope creep.
3. Immediate stabilization of the potential failure zone due to an increase in compressive stresses on the potential failure plane.
4. Immediate erosion control of the slope surface.
5. Increasing slope stability in the long term due to consolidation of cohesive soils and the densification of cohesionless soils from the compressive loading on the soil.

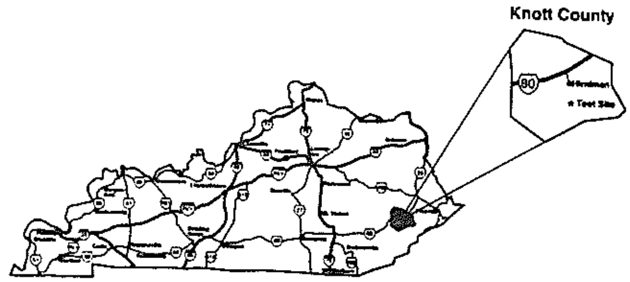

Figure 1. AGS project site location map.

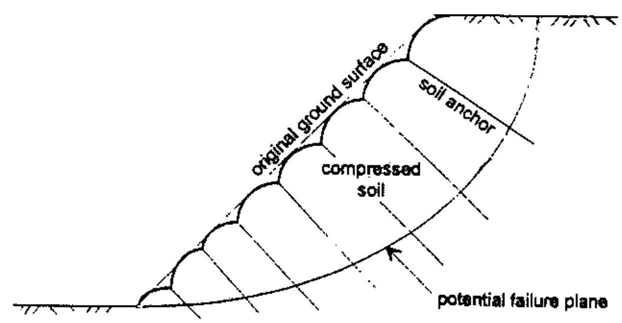

Figure 2. Cross-section of an anchored geosynthetic system.

However, as noted by Koerner, the system may require additional anchor redriving after the initial installation to maintain tension in the geosynthetic due to soil consolidation and stress relaxation in the geosynthetic. In addition to these benefits, AGS also eliminates the need for heavy construction equipment to repair the slope, resulting in substantial cost savings.

To date, only limited field research has been conducted on AGS. As previously stated, the original design concept was developed by Koerner (1984, 1985), who also performed the first theoretical AGS slope stability analysis (Koerner and Robins, 1986). Other theoretical work was done by Hryciw (1991, 1992), who calculated the optimum length and orientation of soil anchors in cohesionless soils, and by Greenwood (1985), who analyzed a soft clay slope that was remediated with a geogrid connected to duck-billed anchors.

Two field installations were completed by Koerner (1986) using a woven slit-film geotextile. This fabric was fitted with grommets on a 1.5 m (5 ft) triangular pattern and anchored to the slope using 13 mm (0.5 in) diameter steel rods. These rods were 1.2 m (4 ft) long and were coupled together to create total anchor lengths of 1.2 m (4 ft) to 7.3 m (24 ft). These lengths were varied so as to penetrate the potential failure plane by at least a meter. This design was used to stabilize a 10.7 m (35 ft) high, 60° slope of silty sand and a 4.5 m (15 ft) high, 50° slope of silty clay. Both slopes remained stable for one year, but no information has yet been published on the long-term effects of the installations.

Vitton's (1991) research, however, revealed that for cohesionless soil, the deformation of the geosynthetic-soil interface is limited to an area immediately around each anchor. This and the development of interface frictional forces on the geosynthetic limit the compressive load that can be applied to the slope. Vitton also noted that stress relaxation of the geosynthetic limits the amount of time that the system is in tension.

Research Site

The research site selected is located in Eastern Kentucky and is approximately 1.5 km south of Hindman, Kentucky in Knott County as shown in Figure 1. The landslide lies approximately 230 m (750 ft) downslope from the Fire Clay (Hazard Number Four) coal bed, which was mined in the early 1970's. A 3 ha (7 acre) landslide occurred in the mine spoil in May of 1984, sliding downslope approximately 275 meters (1000 ft) into the center of a hollow and onto a county road at the base of the hollow. The failure was remediated by the Kentucky Office of Surface Mining (OSM) as an Abandoned Mine Land (AML) project called the Madden Slide. Due to the volume of the spoil, some of it was kept at the site in an uncompacted head-of-hollow fill. In constructing the fill, part of an adjacent slope, which had not been disturbed by the original landslide, was cut back to allow for a drainage ditch to be constructed around the head-of-hollow fill. The undercut slope eventually failed in 1988 and was remediated in 1991. The slide failed again in 1993. The main cause of the instability was groundwater seepage, which was observed in at least three locations within the landslide. Due to the nature of this slide and size of the adjacent landslide, this landslide was selected for the installation and evaluation of the AGS. A cross-section of the slide is shown in Figure 3 while a plan view of the landslide area is shown in Figure 4.

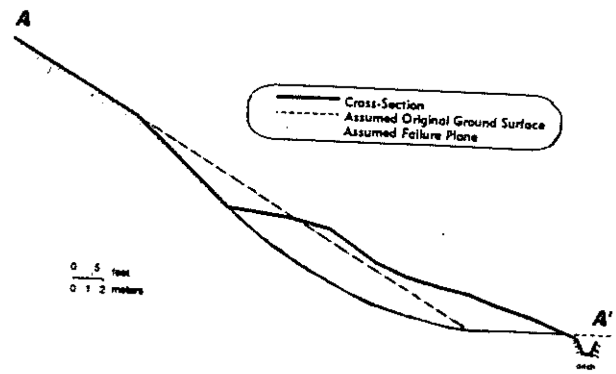

Figure 3. Cross-section of the landslide prior to remediation.

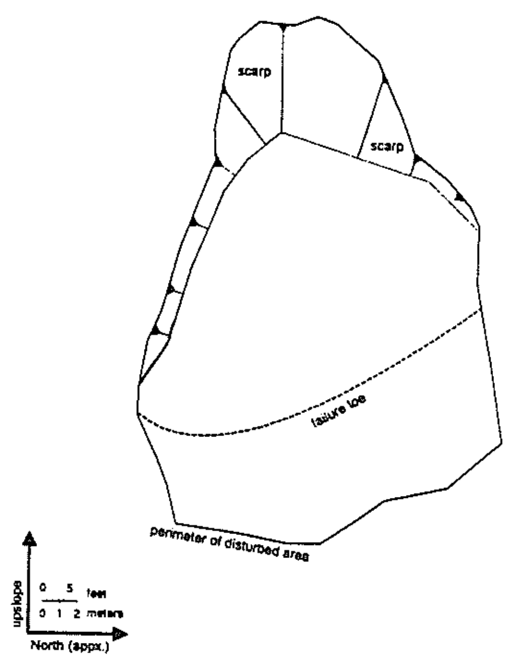

Figure 4. Plan view of landslide area.

The slope in which the slide occurred consists of colluvial soils formed from the overlying rock formations and containing significant sandstone fragments. The USCS classification for the soil is ML, a sandy silt. However, the Atterberg limits did plot very close to the hatched zone of the plasticity chart indicating a silt-clay mixture. The liquid limit was found to vary between 25 and 35 with an average value of approximately 28. The plastic limit varied between 17 and 25 with an average value of approximately 22. The corresponding plasticity index was approximately 6, indicating a material of low plasticity. As expected the moisture content of the surface soils varied by season. Average moisture contents in the spring were typically around 30%, while in the fall the moisture content decreased to an average value of 16%. These water content values indicate a liquidity index of greater than one during the spring and less than zero during the fall. The softer response in the spring coincides with the increase of water content when a slope failure occurred.

Consolidation tests were conducted since soil consolidation will result when the compressive load from the geotextile is applied to the slope's surface. It was found that the coefficient of consolidation, c_v, averaged 15 m^2/yr while the virgin compression index, C_c, averaged 0.132 and the recompression index averaged 0.025. The preconsolidation pressure, which was difficult to determine, was estimated to be approximately 100 kPa (2100 psf) for soil samples from an undisturbed area adjacent to the slide at a depth of 0.6 m (2 ft). This indicates that the colluvial soils were overconsolidated. Although the past maximum pressure is given with some reservation due primarily to the disturbance during sampling, it does indicate that the soils are overconsolidated probably due to the wetting and drying cycles that develop the desiccated crust.

To estimate the shear strength of the colluvial soil, a series of consolidated-undrained shearing triaxial compression tests and unconsolidated-undrained compression tests were performed on shelby tube samples taken from the site. The drained strength parameters were found to be as follows; the effective angle of internal friction, ϕ', was approximately 33°, while the effective cohesion, c', was approximately 3.5 kPa. The undrained shear strength, S_u, was approximately 23 kPa (1100 psf). This would indicate a soil of medium consistency. However, the undrained shear strength would be expected to be significantly less during the wetter spring months when the natural water content was higher.

System Design and Installation

The general design of the AGS installed followed the original system proposed by Koerner (1984) with modifications as suggested by Vitton (1991). The as-constructed system, however, was based primarily on site conditions and dimensions and on anchor driving tests. Unexpected developments in the field and accumulated experience necessitated changes in the original AGS design during actual installation. Due to delays, the installation of the system was completed over a period of four months, although the total installation time was only approximately two and half weeks using an average of four people. The components of the AGS consisted of (1) geotextile, (2) anchors, and (3) a geotextile-anchor connection. The following section provides the selection and design of each these components and the installation procedure used for the AGS.

Geotextile

Since a high to moderate strength geotextile is required for an AGS, only woven geotextiles were considered for this project. Other fabric properties of concern were stress relaxation, ultraviolet (UV) radiation stability, fabric construction, and stress-strain characteristics. The fabric selected for installation was Linq's (formally Exxon) GTF-1000T, a woven polyester geotextile. This fabric has a tensile strength of 160 kN/m (925 lb/in) in the warp direction and 140 kN/m (800 lb/in) in the fill direction. The fabric strength selected was based on the pullout resistances of the ground anchors in field tests at the site in Kentucky. According to Van Zaten, (1986), if the tension in a polyester geotextile is kept below 60% of its ultimate strength the problem of creep and stress relaxation in the geotextile is minimized. Therefore, to minimize creep and stress relaxation, the strength of the geotextile selected was such that when the loading from the ground anchor approached 60% of the strength of the geotextile the anchor pullout capacity would be exceeded and the anchors would start to pullout. In addition, this allows for a continuous load on the soil in the case of slope movement, as noted by Vitton (1991).

It was decided that one 15 m (50 ft) by 24 m (80 ft) sheet of geotextile plus two cut sections of one 3.8 m (12.6 ft) by 30 m (100 ft) roll would be sufficient to stabilize the site. The polyester geotextile was sensitive to UV radiation, so to reduce the exposure of the geotextile to the sun, a thin sheet of a nonwoven Linq's Typar was placed over the entire installation and secured to the tops of the anchors and simply acted as a sacrificial material.

Anchors

Informal driving tests at the University of Alabama demonstrated that anchors could be driven sufficiently by a Hilte TE804 electric demolition hammer powered by a 4000 watt generator. The demolition hammer was used to strike a custom-made driver head threaded over the protruding end of the anchor as shown in Figure 5. As a contingency, extra driver heads were made to fit a 13 N (60 lb) pneumatic jackhammer that was rented, along with a compressor, if necessary.

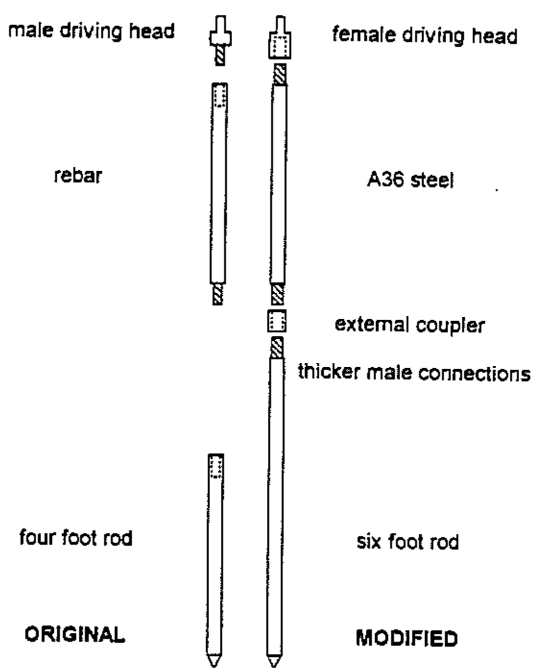

Figure 5. Anchor pattern used in the AGS.

Two anchor materials were considered: rebar and an A36 cold-rolled steel. Driving tests revealed that the rebar was too brittle for dynamic driving. The extreme vibrations caused by the impacts of the demolition hammer often caused the rods to twist apart or break. The cold-rolled steel rods, on the other hand, were more ductile due to the higher quality of steel. Therefore the rolled steel, despite being more expensive, was chosen over the rebar as the anchor material.

Since it is important to have the AGS anchors driven deep enough to penetrate the potential failure plane of a slope, a total anchor length of 3.9 m (13 ft) was chosen for the site based on an estimated maximum depth to the original failure plane of about two meters (7 ft). The total anchor assembly length consisted of four rods: (3) 1.2 m (4 ft) lengths and (1) .3 m (1 ft) length. The 0.3 m section was the last rod to be driven, and was threaded along its entire length to facilitate the assembly and tightening of the anchor-geotextile connection.

Early in the anchor testing process, the test rod lengths were manufactured with threaded male and female ends as illustrated in the left-hand portion of Figure 5. These rods could simply be twisted together without the need for any type of fastener or coupler. Driving tests showed that these unreinforced connections were vulnerable to shearing failure during driving. The new test rods were made with both ends turned down and threaded. These rods were connected by an external coupler as shown in the right-hand portion of Figure 5. This design proved sturdy, however, the jackhammer's dynamic driving action, which resulted in considerable vibrational movement of the top portion of the anchor, caused the anchor hole to enlarge at the surface. While this allowed room for the coupler, which had a somewhat larger diameter, to be driven into the soil, it also minimized the pullout resistance of the anchor since it prevented all but the first rod of the anchor assembly from having good contact with the soil. To minimize this problem, the first rod used in driving was increased from 1.2 m to 1.8 m (6 ft) in length as shown in Figure 5. The last section in the assembly was machined to a length of 0.6 m (2 ft) to maintain an overall anchor length of 3.9 m.

Rod diameters of 13 mm (0.5 in.), 19 mm (0.75 in.), and 25 mm (1 in.) were tested at the University of Alabama to determine the relative pullout resistances of the three different sizes. These tests were conducted in a field on campus slated for construction, so good geotechnical information was available. The anchors were tested next to a Standard Penetration Test (SPT) hole that had N values ranging from 20 near the surface to 70 at resistance in a gravel layer at a depth of about 4.5 m (15 ft). The soil was a stiff clay with gravel lenses. Anchors of each size were driven to resistance with the Hilte TE804, then pulled out of the ground using a hydraulic lift or a forklift.

The 13 mm diameter rods tended to wander during driving, creating a hole that was not straight. This added to their pullout resistance, but the larger diameter rods had higher overall pullout resistance due to the increased soil-interface area. The demolition hammer was unable to effectively drive the 25 mm diameter anchor as deeply as the other anchors and were not considered further.

The 13 mm and 19 mm diameter rods were then field-tested at the research site in Kentucky using the Hilte TE804. Pullout results showed a significant increase in resistance for the 19 mm diameter anchors over the 13 mm rods and therefore 19 mm diameter rods were used in the installation.

For the remediation, the anchors would be driven 1.4 m (4.6 ft) apart in a hexagonal pattern as in Figure 6. Anchor driving was to begin in the center of the slide and proceed outward as shown in Figure 7.

<u>Anchor-Geotextile Connections</u>

The entire anchor geotextile connection assembly used in the installation is shown in Figure 8. The anchor geosynthetic connection cup was pressed out of one foot diameter, 14-gage steel circles to form bowls approximately 130 mm (5 in.) in maximum diameter and 50 mm (2 in.) deep. A 22 mm (0.875 in.) hole in the center of each of the connections allowed them to be placed on the .3 m, threaded section of the anchors. A flat 50 mm diameter area at the bases provided a contact surface for belleville springs. These springs were placed on the anchors between the anchor geotextile connections and tightening bolts in an attempt to maintain tension

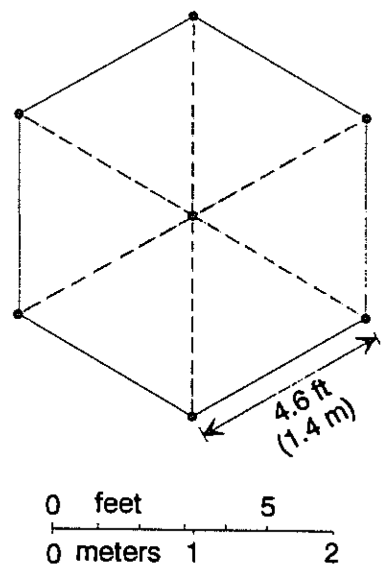

Figure 6. Hexagonal anchor arrangement.

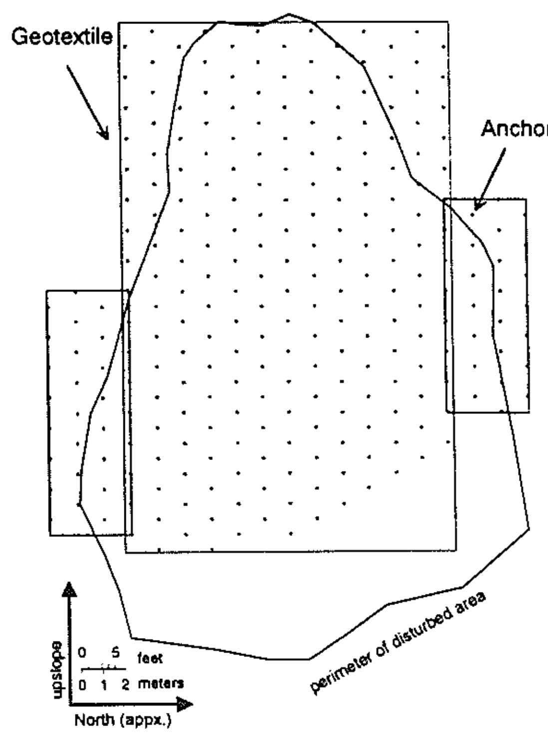

Figure 7. Anchor installation plan.

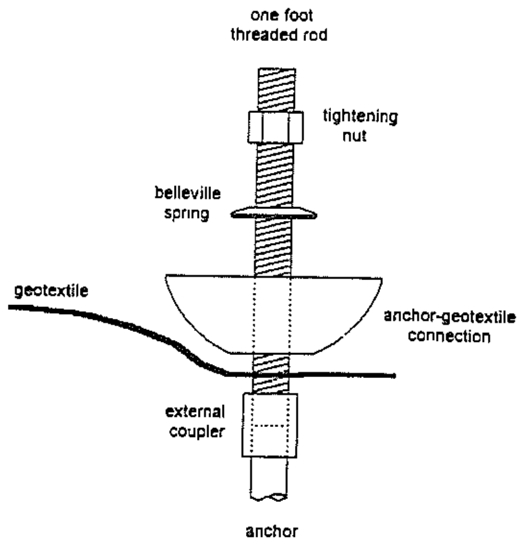

Figure 8. Anchor-geotextile connection.

on the geotextile in the event of geotextile creep or anchor movement. The tightening bolts were used to tension the AGS after anchor driving was completed and at later dates.

Installation

Installation of the AGS was started on December 1, 1993. A bulldozer was first used to placed the failed soil back up slope and to provide a more level surface for placement of the geotextile. Since some soil had been removed from the landslide in 1991, the bulldozer was unable to match the original volume and grade of the slope but did manage to adequately place the remaining material back up slope. It was very difficult in the upper portions of the slide due to very wet ground conditions, which at times were very soft. However, a relatively level surface was created in which to apply the AGS. Due to the smaller than expected area to be remediated, only the 15 m by 24 m geotextile was needed to secure the slope. Access to the landslide was limited, especially at the top of the slope, so it was necessary to manually place the fabric. Four people were able to lift the geotextile over a ditch, unroll it up the slope, and drag it into final position, though this was accomplished with difficulty.

As anchor driving began, all three available driver heads for the Hilte demolition hammer failed. At this time only seven anchors had been driven. A pneumatic jackhammer and associated driver heads were available, but considerable rain had begun to fall and the forecast for the next several days was poor. The installation was temporarily abandoned. Possible reasons for the unforeseen increase in driving resistance that caused the failure of the driving heads are the use of couplers, the unexpectedly shallow depth of soft fill over the stiffer, unfailed layer, and possibly the colder temperatures encountered in December as opposed to the temperatures in June when the field tests were conducted.

The first attempt to complete the AGS installation took place in March of 1994. This followed a very wet and snowy winter in which a portion of the slope beneath the geotextile, which had been placed in December, failed. Fortunately, however, it appeared that the seven anchors driven in December prevented this portion of the slope from completely failing downslope.

To complete the AGS installation a 13 N (60 lb) pneumatic jackhammer and diesel compressor was used in place of the electric demolition hammer and generator. The jackhammer was generally adequate for driving the ground anchors. The extensive sandstone fragments in the colluvium, however, caused difficulty in driving the anchors. Frequently, when the anchors encountered the sandstone fragments the anchor would either force the fragment out of the way or would penetrate through the fragment, which was indicated by difficult anchor driving followed by relatively easy anchor driving, i.e., the anchor would break through the fragment. In two cases, the anchors met resistance and could not be driven further. It was also observed that when an anchor hit a significant sandstone fragment, excessive vibrations would develop in the anchor causing additional enlargement to the anchor hole and reducing the anchor's pullout capacity.

The soil conditions below the geotextile ranged from firm at the base of the slide to extremely wet near the top of the slide. In some areas, even walking on the geotextile caused significant deformation of the geotextile. However, even in the very wet areas driving of the anchors resulted in little to no general deformation of the soil between the anchors. Only very local deformation in the vicinity of the anchor occurred, which was generally within 30 cm or less of the anchor.

A total of four people completed the installation. One person worked ahead of the jackhammer laying out the anchor pattern using a fabricated triangular template which was 1.4 m (4.6 ft) on each side. By using this template the hexagonal anchor pattern could be properly positioned. This person would slide the anchor through the fabric making sure that the geotextile was not ruptured and then drive the first rod section as deeply as possible using a simple post-hole driver.

Two people were required to lift and operate the jackhammer due to the difficulties of working on a steep slope with a heavy jackhammer. A third person assisted by screwing on the driver head, guiding the jackhammer onto the head when it was lifted, and holding the driving head in place during driving since the head would often vibrate loose and damaging the rod threads if constant tightening wasn't maintained.

Because the depth of relatively soft, failed material was less than anticipated, anchor refusal during jackhammer driving occurred at depths less than the designed anchor length of 3.9 m. Most anchors in the completed system could only be driven 2.7 m (9 ft), while some anchors in the upper third of the AGS could only be driven 2.1 m (7 ft). The upper right corner of the AGS was unsecured because ground resistance in that area prevented the 1.8 m pointed rods from being driven completely, so the anchored geotextile connection could not be attached.

All but two of the anchors penetrated the shallow failure plane of the slope, though not by the 1.5 m (5 ft) suggested by Koerner (1990). Due to the difficulties in driving and the smaller area to be remediated, only 172 anchors were used to secure the slope as shown in Figure 9 then the planned 225 anchors.

Field Performance

Monitoring of the AGS included the use of anchor load cells, soil pressure gages, a rain gage, and a temperature probe. The purpose of these instruments was to record the responses of the AGS and the soil to the slope remediation process and changes in the weather. The site was also surveyed on a regular basis to detect any slope movement, and test anchors were driven at the base of the site to quantify any improvements in anchor pullout resistance over time. A Campbell Scientific CR10 datalogger was placed at the site in May of 1994 to monitor the instrumentation of the system. However, vandalism and power failures destroyed some of the data which was collected over several periods and totaled about three months throughout the monitoring period.

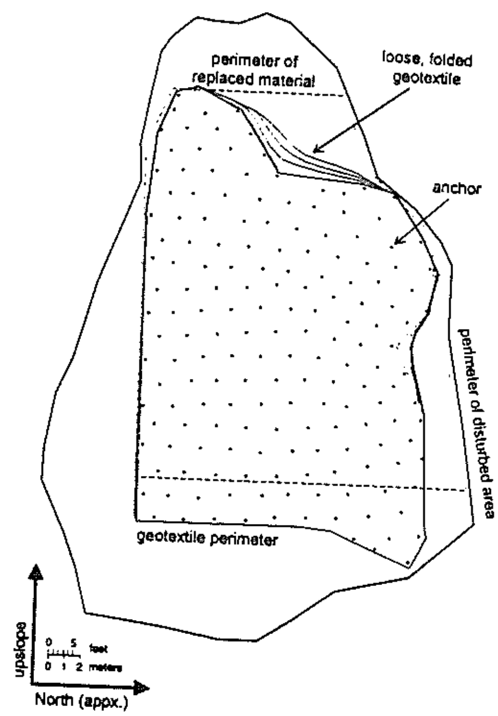

Figure 9. Completed AGS anchor layout.

A significant concern on anchor installation was that since the soils consisted of fine-grain materials, the initial pullout capacity of the anchor would be limited due to possible excess pore pressures build up during anchor driving, which would have to dissipate to achieve expected pullout capacity. Therefore, tensioning of the system was designed to be accomplished by the tightening bolt on each anchor sometime after the initial driving and not by the initial driving as suggested by Koerner (1984).

Nine load cells were installed to measure the tension in the anchors. This was done to determine the load that the anchors placed on the geosynthetic and whether or not this tensile load could be maintained over time. The load cells were fabricated from .3 m (1 ft) lengths of 19 mm diameter threaded steel as illustrated in Figure 10. After installation of the load cells, the assembly was tightened to tension the anchor and geosynthetic. The force in the anchor was recorded during and immediately after the tightening as well as for long-term changes.

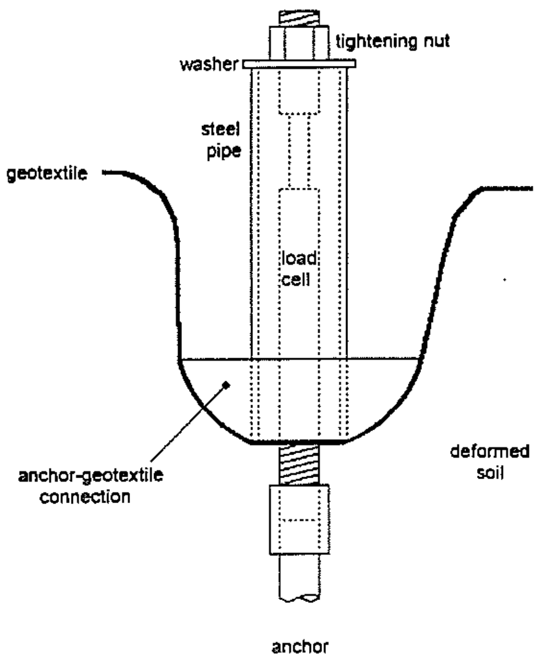

Figure 10. Anchor load cell design.

All load cells recorded an anchor load of between 110 N (500 lb) and 340 N (1500 lb) immediately after tightening. This load decayed to zero in ten to twenty days. An example of this is shown in Figure 11, where the loss in load occurs within 12 days. The load loss was believed to be due to geotextile relaxation or to local soil displacement, which resulted from soil consolidation in close proximity to the anchors, as opposed to pullout of the anchors, since anchor displacement measurements were made during tensioning that indicated little to no displacement of the anchor occurred at the time of loading. However, since the slope consisted of fine-grained soils it was also possible that soil creep along the length of the anchor could be responsible for the load loss.

After the AGS was completely installed, each anchor was tagged and numbered and surveyed using a laser theodolite. The site was first surveyed on July 11, 1994 and on an approximate two month intervals through July of 1995. The surveying showed negligible to no anchor movement for the entire monitoring period.

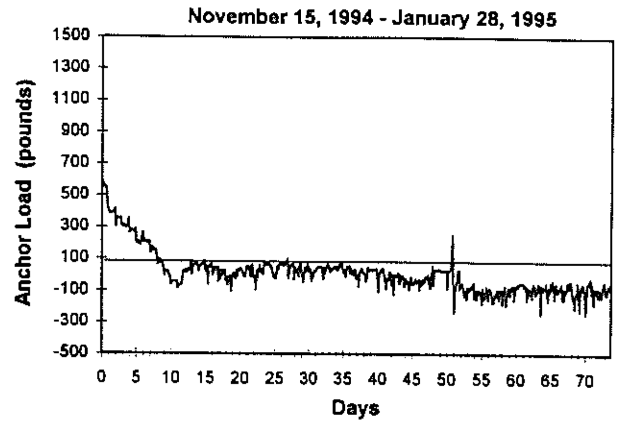

Figure 11. Anchor load versus time plot.

Anchor load changes in response to wet or dry periods were of interest as this could indicate potential slope movement in wet seasons or a shrink/swell phenomena, but no such changes were observed. However, increases of up to several hundred pounds were recorded during particularly cold weather. These spikes occurred in response to a sudden drop in air temperature to about -15° C. Closer inspection revealed trends of higher loads during colder weather and lesser loads during warmer spells. These changes are probably due to the thermal expansion and contraction of the imbedded steel anchors rather than any changes in the soil itself. The thermal changes in the steel also probably cause the daily fluctuations in load cell readings. In general, the loads reach a maximum overnight and quickly drop in the morning. This is consistent with the suspected thermal expansion and contraction of the anchors.

Before the placement of the geotextile, five soil pressure gages were installed approximately 0.6 m (2 ft) below the ground surface at various points along the slope. Three gages were located beneath the geotextile and two gages were located below the geotextile perimeter and were not influenced by the AGS. All anchors were driven (the system was completely installed) before the datalogger was available to read the soil pressure gages. Therefore, it is not known if any pressure increases occurred during the tensioning of the geotextile over the slope. This is an important deficiency, as soil pressure increases would indicate possible soil consolidation, a major purported benefit of an AGS. However, two gages were relatively near load cell positions, where anchors were tightened on September and November of 1994. They recorded no apparent increase in soil pressure due to anchor tightening. This is consistent with the lack of deformation between the anchors, which is required to transfer the tension in the geotextile to a compressive load on the soil. While tension may exists in the geotextile, if no curvature develops through soil deformation then little to no compressive loading of the soil will result. For all of the anchors driven, only local deformation developed around the anchor with little to no deformation developing between the anchors. Therefore, there was insufficient curvature, r_c, or long term tension, N, in the geotextile to develop a compressive loading to the soil. While, all gages did show some increase in pressure due to rainfall events, the increases were small (typically less than 35 kPa (5 psi)), and were usually caused only with significant rainfall. It was concluded from the measurements made during this research, therefore, that the active stabilization of a slope by tensioning of the geotextile over the soil slope was not successful. However, based on the AGS's field performance, it is believed that other functions of the AGS did improve slope stabilization such as the effect of soil nailing from the anchors, halt of slope creep movement, and erosion control.

Conclusions and Comments

An abandoned mine land landslide located in Eastern Kentucky was remediated using an anchored geosynthetic system. Installation of the system took approximately two and half weeks utilizing four installation personnel although equipment failure and poor weather extended the installation to a period of four months. A high strength geotextile was used as the membrane and was anchored using steel ground anchors. The driving of the anchors was accomplished using a 13 N (60 lb) pneumatic jackhammer. The AGS was monitored using load cells, soil pressure cells, along with temperature and precipitation measurements over a period of about one year. Conclusions from this installation are as follows:

1. Although portions of the slope were very wet and soft, anchor driving was unable to deform the soil below the geotextile and thus minimal compressive loading was applied to the slope since limited curvature of the geotextile-soil interface developed.
2. Tensioning of the geotextile by the ground anchors was lost within ten to twenty days of application. The loss of load was believed to be a combination of soil consolidation immediately below the anchor-geotextile connection where most of the deformation occurred, and stress relaxation in the geotextile. Consequently, even if sufficient deformation developed, constant retensioning of the geotextile would be required.
3. While the AGS was not capable of functioning as an active remediation system, it did appear to function well as a passive remediation system, i.e., it prevented the slope from failing, which was in a relatively unstable state especially during wet periods, although the monitoring period was not sufficient to confirm that the system would function well into the future.
4. Driving of the anchors in the colluvial soil was difficult at times due to the presence of sandstone fragments. It was also observed that in the driving process the anchor hole would be enlarged due to the vibrations induced in the anchor. Therefore, the top portion of the anchor was not in contact with the soil. This effect was further enhanced when sandstone fragments were encountered at depth and the anchor vibrations significantly increased as the anchor attempted to break through the rock

fragment. An additional consideration on anchor driving is the build up of excess pore pressure, which reduces the pullout capacity of the anchor and makes tensioning the system difficult during anchor driving.

5. The affect of the ground anchors acting as soil nails appears to be effective but was not quantified in this research. Also, the geotextile as expected provided erosion control for the slide area.

While the results of this research are mixed, it is believed that AGS may provide an effective remediation system for certain types of slopes prone to creep. Further research, however, is needed to determine the ability of AGS to halt progressive slope failure for slopes consisting of heavily overconsolidated soils that exhibit strain-softening behavior leading to creep failure. Since colluvial soils commonly consists of these types of soil, it is believed that anchored geosynthetic systems may provide a possible remediation system for these types of slopes.

Acknowledgment

This research was sponsored by the United States Bureau of Mines under the Abandoned Mine Lands Program and administered through Contract No. 1432-J0230005 titled "The Application of Anchored Geosynthetic Systems for Abandoned Mined Land Landslide Remediation." In particular, the authors would like to thank Mr. Tony Iannacchione, the technical project officer for his support, encouragement, and timely advise throughout the project. The financial and moral support are gratefully acknowledged.

Literature Cited

Greenwood, J. R. 1985. Geogrids and Anchors for Slope Stabilization - Simple Method of Analysis. Proceedings of the 11th International Conference on Soil Mechanics and Foundation Engineering, Discussion, Vol. 5, pp. 2770-2771.

Koerner, R. M. 1984. Slope Stabilization Using Anchored Geotextiles: Anchored Spider Netting. Proceedings, Spec. Engineering for Roads and Bridges Conference, PennDot, Harrisburgh, PA.

Koerner, R. M. 1985. Load Transfer Considerations Using Geotextiles. Geotechnical Fabrics Report, IFAI, Vol. 3, No. 2, pp. 11-14.

Koerner, R. M. 1986a. Designing with Geosynthetics. Prentice-Hall, Englewood-Cliffs, NJ, pp 118-123.

Koerner, R. M. 1986b. A New Method of Soil Slope Stabilization Using Geosynthetics. Geotechnical Fabrics Report, IFAI, Vol. 4, No. 1, pp. 18-25.

Koerner, R. M. and Robbins, J.C. 1986c. In-situ Stabilization of Slopes Using Nailed Geosynthetics. Proceedings, Third International Conference on Geotextiles, Vienna, Austria, April 7-11, pp. 395-400.

Iannacchione, A. T., Bhatt, S. K., and J. Sefton 1994. Geotechnical Properties of Kentucky's AML Landslide and Slope Failure Evaluation. Proceedings, 16th Annual Conference of the Association of Abandoned Mine Land Programs, Park City, Utah, September 18-21, 1994.

Van Zaten, R. V. 1986. Geotextiles and Geomembranes in Civil Engineering. Wiley & Sons, New York, USA.

Vitton, S. J. (1991), "An Experimental Investigation into the Load Transfer Mechanism in Anchored Geosynthteic Systems, Thesis presented to The University of Michigan, at Ann Arbor, MI, in partial fulfillment of the requirement for the degree of Doctor of Philosophy.