Landslides in Shale-Derived Glacial Till

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ABSTRACT

An area in south-east Iowa is notorious for landslides in both cut slopes and embankments. Most of the failed slopes have been modified to lower slope angles as attempted remediation; however in many instances the failure reoccurred. The dominant soil in the cuts and used for borrow material is Kansan age glacial till, while the underlying bedrock is Pennsylvanian shale of the Des Moines series. The till contains significant amounts of the underlying bedrock. In an effort to gain insight into the cause of these failures, an embankment with extensive failures in Monroe County was studied. One slide selected for intensive study was at a 2V:1H slope and 8 meters high. The slide was 16 meters in length. The failure zone was essentially parallel to the original surface at a depth of 0.75 meters. Average strength parameters used in stability analyses produced factors of safety that indicate that failure should not have occurred. X-ray diffraction tests indicate a slightly broader montmorillonite peak in surface soils compared to interior embankment soils, which may indicate the presence of intracrystalline moisture and perhaps a different exchangeable cation in the failure zone soil. This subtle difference could account for a lower shear strength in the soil at the surface of the embankment. It is interpreted that the most likely cause of failure is the reduction of cohesion near the embankment surface after construction. If the reduction of strength is due to weathering and saturation leading to the subsequent swelling of the montmorillonite is the cause of failure, then remediation such as flattening the slope would be ineffective. The use of other soil strengthening techniques such as the use of geosynthetics, minipiles, or chemical stabilization would provide better, long term stability.

INTRODUCTION

U.S. Highway 34 extends 267 miles across southern Iowa from the Missouri River to the Mississippi River; however, a segment of the road approximately 30 miles long in western Monroe and eastern Wapello counties has been affected by an unusually large number of landslides in both cut slopes and embankments. The geology of the region consists of fairly shallow loess over glacial till with the underlying bedrock consisting of Pennsylvanian shale. The till contains significant amounts of the underlying shale. The soil association that coincides with the area of slope failures is the Pershing-Gosport-Lindley in which the topography is gently to steeply sloping. The soils are moderately well drained and formed from loess glacial till, and residuum of acid shale on the uplands.

In many cases, slope failures that have been repaired by loading the toes or decreasing slope angles have reoccurred. In an attempt to identify the cause of these slope failures, an intensive investigation of one slide was undertaken. The landslide studied was on the foreslope of an embankment located on Highway 34 about 1.6 miles (2.7 km) southeast of Albia, Monroe County, Iowa. The landslide proximity to the highway shoulder posed a risk to public safety. The Gosport soil series was the borrow material for the embankment and is weathered from loess, glacial till and a residuum of acid shales. The soil is subject to severe shrink-swell behavior, and in general, has a low strength. (Oelmann 1984).

DESCRIPTION OF THE SLIDE

The foreslope, shown in Fig, 1, is 24 ft (8 m) high at a 2:1 slope and on the north side of the embankment. The slide is classified as a translational slide in which a few blocks of fine-grained soil have failed along a shallow failure plane approximately 2.5 feet (0.75 m) deep. The failure plane is approximately parallel to the slope of the existing embankment. During fieldwork in April 1999, it was observed that the soil in the failure zone appeared wetter and with an aggregated structure that differed from the soil samples collected from the borings and the soil on the surface adjacent to the slide. A similar landslide existed on the south side of the roadway but was not studied in detail.

SOIL SAMPLING

Because visual inspection indicated a wetter and aggregated soil in the slide, disturbed grab samples were collected along with undisturbed Shelby tube samples from two boreholes located along the roadway shoulder in the embankment fill. The grab samples and Shelby tube samples were subjected to engineering index tests. The Shelby tube samples were tested to determine the shear strength parameters and unit weight of the soil. Subsurface logs for the borings can be found in Karnik (2001).

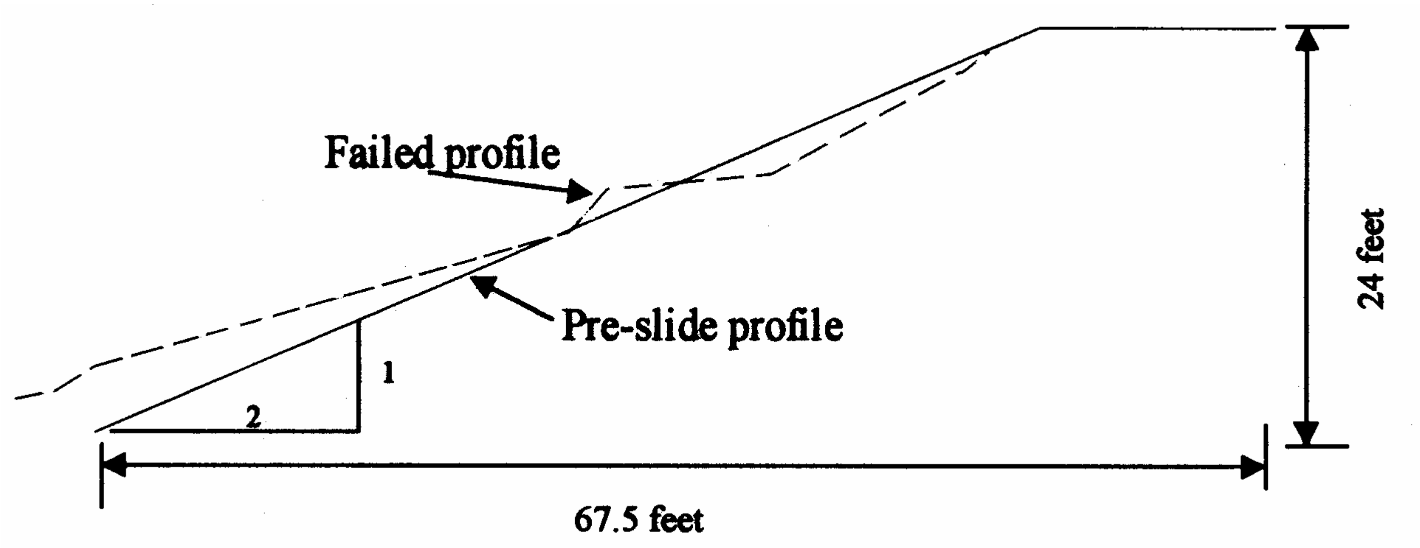

Fig. 1. Typical cross section of U.S. Highway 34 slide, Monroe County, Iowa.

GEOTECHNICAL PROPERTIES OF THE EMBANKMENT SOILS

Atterberg limit tests and mechanical analyses were conducted on grab samples from the slide and the Shelby tube samples from the interior of the embankment. The mean values of liquid limit and plastic limit for the grab samples are 43.7% and 22.4%, respectively, while the mean liquid limit and plastic limit of the Shelby tube samples are 41.6% and 20.6% respectively. Both sets of samples have more than 82% of the soil particles passing the No. 200 sieve (0.075 mm). Based on the particle size distribution and the Atterberg limits, the soil samples classify as CL or CH by the Unified Classification System. According to AASHTO, these soils classify as A-6 and A-7-6 and fall into the category of cohesive embankment soils according to Bergeson et al. (1998). Engineering index properties and engineering classification of the surface grab samples and samples from the interior of the embankment are essentially the same. This was verified by statistical analysis that used the "t test".

Standard Proctor tests conducted on composite samples determined a maximum dry unit weight of 105 pcf (16.5 kN/m^3) at an optimum moisture content of 19%. The unit weights of compacted embankment soil, determined from 37 Shelby tube samples, were compared with present day recommended specifications of a minimum compacted soil density of 95% of γ_{dmax} and a moisture content of ± 2% of OMC. The dry unit weights of the compacted samples range from 87.8 pcf (13.8 kN/m^3) to 115 pcf (18.1 kN/m^3) with field moisture contents from 14% to 38.5%. Nearly 42 % of the samples from the interior of the embankment are below 95% relative compaction that is recommended (Bergeson et al. 1998); however only 13% have unit weights below 90% maximum dry unit weight. The field moisture contents of 81% of the samples are higher than the optimum, suggesting that the soil imbibed moisture subsequent to construction, as can be seen in Fig. 2.

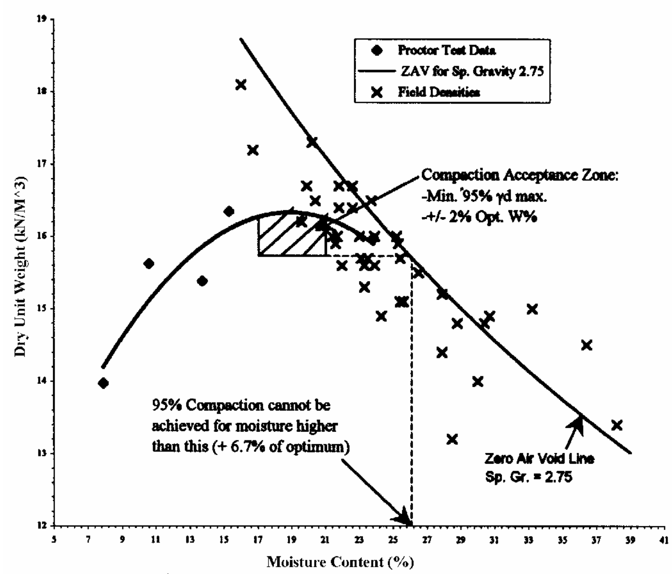

Fig. 2. Standard Proctor density and field density results for embankments soils.

Consolidated undrained (CU) triaxial tests were carried out on eight Shelby tube samples obtained from the interior of the embankment to obtain strength parameters for use in slope stability analyses. The samples were backpressure saturated before testing and pore pressures measured. The results were used to plot the effective deviator stress versus strain curves and maximum values from the stress-strain curves were interpreted as the shear strength of each sample. These data were used to plot the effective stress K_f line and intercept, a, and angle, α, values were obtained by linear regression with a correlation coefficient (R^2) value of 0.74. The a and α values were converted to a cohesion, c′, of 313 psf (15kPa) and an angle of internal friction, ϕ', of 19 degrees.

It has been observed that if embankment compaction is less than 95%, the soil shear strength is likely to be lower (Bergeson et al. 1998). Of the eight samples tested, four samples had dry unit weights less than 95% the maximum dry unit weight. The effective stress-strain data from these samples were separated from the group to obtain the shear strength parameters for lower density soils. The regression for the low density soil, with a (R^2) of 0.85, resulted in a c′ of 25psf (1.20kPa) and ϕ′ of 25 degrees.

A third regression was carried out on the remaining four samples with dry unit weights greater than 95% maximum dry unit weight. The regression showed the soil to have a c′ of 587psf (28kPa) and a ϕ′ of 12 degrees with a R^2 of 0.80.

Unconsolidated, undrained (UU) triaxial tests with backpressure saturation were conducted on eight other Shelby tube samples. These tests resulted in average undrained strength of 835 psf (40 kN/m^3).

SLOPE STABILITY ANALYSES

To determine the probable cause of the slope failure, a series of slope stability analyses were conducted. Bromhead (1992) noted that for very shallow, translational slides, the infinite slope method is a useful technique for slope stability analysis. Calculations were carried out using infinite slope analyses, with the failure plane parallel to the surface and 2.5 feet (0.75m) deep. The analyses were conducted for both a moist slope and a saturated slope with seepage parallel to the failure plane. Total unit weight was used for the analysis without seepage, while the saturated unit weight was used for the analysis with seepage. In addition to the different analyses, the strength parameters were varied using the composite, high strength, and low strength values. The high strength values resulted in safety factors in excess of 4.5 for both moisture conditions. The safety factors from the analyses for both seepage conditions and composite and low strength strengths are shown in Table 1.

The safety factors using the effective strength parameters obtained from the composite data and the infinite slope analyses with and without seepage are 2.6 and 2.9, respectively, indicative of a slope that should not fail. If the low strength values are used, the safety factors for seepage and moist slope are 0.67 and 1.13, indicative of a slope that would be unstable, or at best marginally stable.

To evaluate the unlikely condition of a saturated rapid drawdown adjacent to the embankment, the UU strength and saturated unit weight were used in Taylor's analysis for $\phi = 0$. This analysis resulted in a safety factor of 6.3 and reinforced the presumption these conditions are highly unrealistic for this material.

Table 1. Factors of safety using infinite slope analysis.

Type of analysis	Composite shear strength: $c'=15$ kPa, $\phi' = 19^o$	Low shear strength $c'=1.2$ kPa, $\phi' = 25^o$
Infinite slope analysis-moist slope	2.9	1.13
Infinite slope with seepage	2.6	0.67

The stability of the overall slope was assessed using the computer program XSTABL (Interactive Software Designs, Inc. 1994). Analyses were conducted using the Modified Janbu method. Because the slide is translational, non-circular failure surfaces were assumed. The Modified Janbu analyses were carried out for a dry slope, a slope with water table elevation at toe and at mid height of slope and a fully submerged slope, using the composite and low strength parameters. The safety factors for four conditions using the high strength soil parameters were all greater than 2.3. The failure surfaces for the various XSTABL analyses are given in Karnik (2001). The safety factors of the eight conditions for the composite and low strengths are tabulated in Table 2.

Table 2. Factors of safety for Modified Janbu analyses for composite and low shear strength assumptions and various groundwater conditions.

Assumed in-situ conditions	Composite shear strength $c=15$ kPa, $\phi = 19^o$	Low shear strength $c=1.2$ kPa, $\phi=25^o$
Natural moisture	2.2	1.4
Water table at toe of slope	2.2	1.4
Water table at mid height	1.8	1.6
Fully submerged slope	1.6	0.8

As can be seen from the results in Table 2, the minimum factors of safety obtained from Modified Janbu analyses using the composite shear strength values range from 1.6 to 2.2 for the fully submerged to natural moisture cases. The critical failure surfaces obtained in the analyses were very deep and did not approximate the observed, shallow failure surface.

Analyses carried out using the effective stress strength parameters obtained from the samples having dry unit weights less than 95% maximum dry unit weight, the low shear strength case, resulted in minimum safety factors from 0.8 to 1.4 for the fully submerged to natural moisture cases. The failure surfaces obtained in these analyses were shallow and approximate the in-situ failure surface.

OBSERVATIONS AND DISCUSSION

Of the 20 stability analyses, only one with seepage parallel to the slope and one with a fully submerged slope give minimum factors of safety of 1.0 or less. These analyses indicate that the slope could have failed due to the combined effects of low shear strength and the slope fully submerged or with seepage parallel to the slope. Given the topography of the site, a fully submerged slope is an unrealistic condition. A parallel seepage condition could occur under adverse precipitation conditions in which the upper portion of the slopes' soils becomes saturated due to infiltration, creating a perched water table condition. The low shear strength infinite slope analysis yielded a factor of safety of 1.13, indicating that the slope would likely be only marginally stable.

The above analyses indicate that the most likely cause of failure is the reduction of cohesion of the soils near the embankment surface after construction, coupled with adverse water conditions in the near surface soils most likely resulting from saturation due to infiltration of precipitation into the soils. If it is assumed that the embankment fill was reasonably compacted at the time of construction, then weathering of the soil or some other mechanism such as freeze-thaw effects near the surface could have led to the reduction in cohesion. The engineering index properties indicate no major difference in the soil near the surface and the soil within the embankment. To further investigate the weathering potential of the soils, x-ray diffraction tests were conducted on soils from the failure zone and from the interior of the embankment

SOIL CHARACTERIZATION BY X-RAY DIFFRACTION

X-ray diffraction tests were carried out on both the grab samples from the failure zone and Shelby tube samples from the interior of the embankment to determine their mineral composition. Fig. 3 shows an overlay of the X-ray diffractograms from the grab samples from the surface and Shelby tube samples. The composite diffractograms show that both soils are similar in mineralogy and composed of clay minerals, quartz and feldspar. The clay minerals were dominated by montmorillonite (Schlorholtz 2000). This supports the description from the soil survey report (Oelmann 1984) of the Gosport soil series having high shrink-swell potential on drying or wetting. The diffractogram from the grab sample shows a slightly broader montmorillonite peak, which may indicate the presence of intracrystalline moisture and perhaps a different exchangeable cation in the failure zone soil. This subtle difference could account for a lower shear strength in the soil at the surface of the embankment. The surface clay will swell on coming in contact with water and may undergo a strength reduction. Other studies have shown that shear strength of clays and shales varies with adsorbed cations on montmorillonite (Steward and Cripps, 1983; Moore, 1991, and Mitchell, 1993) and it is possible that weathering of the surface soil could cause replacement of the exchangeable cations.

POST SCRIPT

The reduction in shear strength hypothesis was included in a research report submitted in March, 2001 (Lohnes et al. 2001). During the summer of 2002, the north and south landslides were repaired by reducing the slopes of the embankment from 2:1 to 3:1. Subsequent to that repair, the south slope failed again and field inspection in July, 2003 suggested that the north slope was in an incipient failure condition. It is of interest to note that an infinite slope analysis of a 3:1 slope using the low shear strength parameters yields a factor of safety of 1.7 for the moist condition and of 0.98 for the seepage condition. This region of Iowa was subject to heavy rainfall in the spring of 2003, potentially providing the moisture for swelling of the montmorillonitic based shales. Additionally, the precipitation may have resulted in a seepage condition in the near surface soils.

CONCLUSIONS

A stretch of highway slopes in southern Iowa has been subject to numerous slope failures. An intensive investigation of one of the failures has revealed that the presence of montmorillonite in the parent soils. Analysis of the slope failure indicates that a low shear strength must be present for failure to occur. It is suggested that weathering of the shale in the soils may be the cause of the strength reduction. The recurrence of slope failures in these materials so soon after repair lends credibility to the interpretation of shale weathering as the fundamental cause of slope instability in this region and suggests that remediation by methods other than slope flattening should be considered. If the reduction of strength is due to weathering and saturation leading to the subsequent swelling of the montmorillonite is the cause of failure, then remediation such as flattening the slope would be ineffective in the long term. The use of other soil strengthening measures such as geosynthetics, minipiles, or chemical stabilization would provide better, long -term stability.

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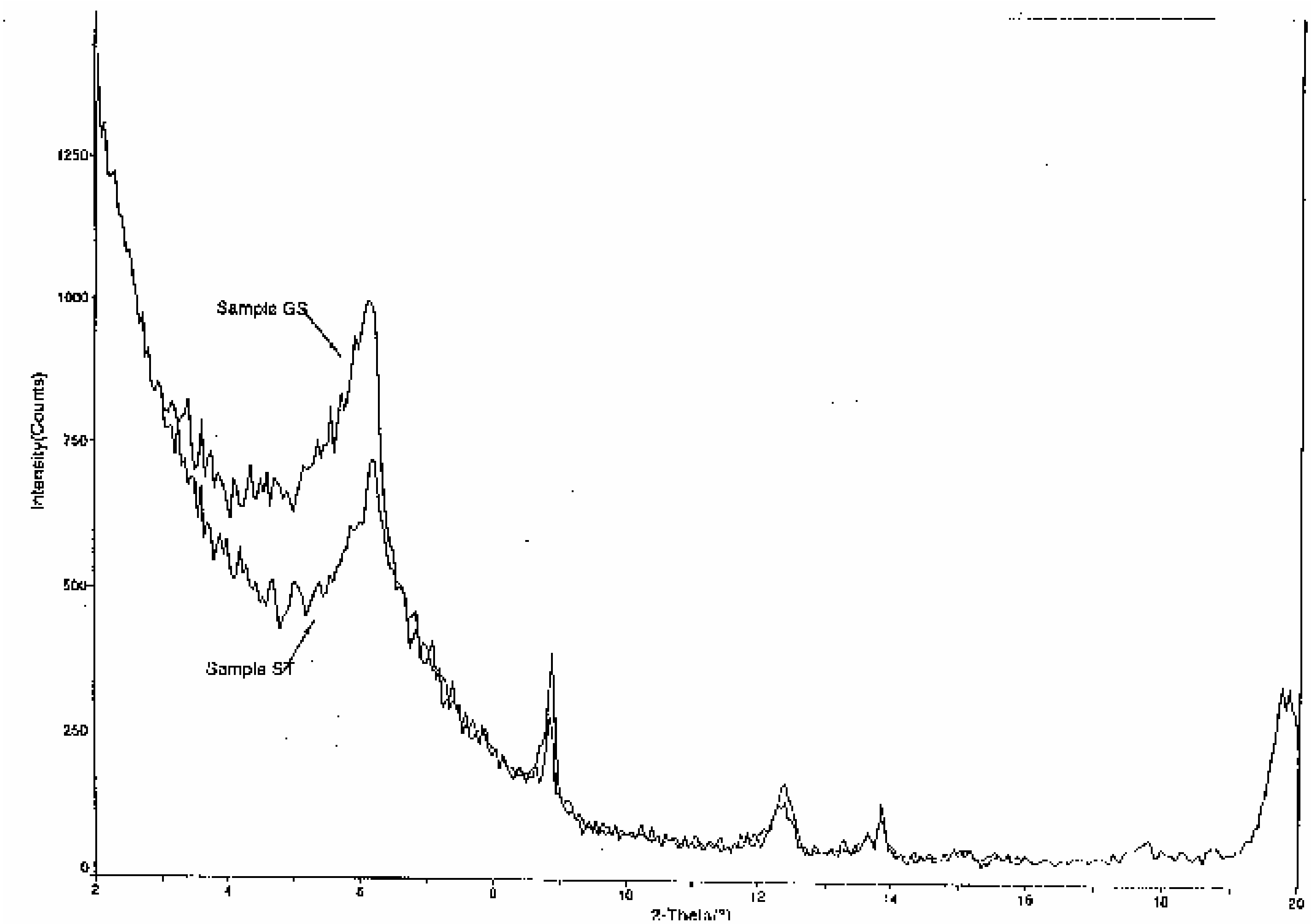

Fig. 3. X-ray diffractograms for soil from failure surface and from interior of embankment.