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Case Study: Compacted Embankment Landslide in Grady County, Oklahoma

by A.B. Cerato[1], R.C. Oleski[2], C.C. Puklin[3]

Abstract

Landslides in the United Stated caused approximately $3.5 billion in damage in 2001 (USGS 2004). While the majority of the newsworthy landslides in the United States occur in California, Oregon, and Washington, landslides occur on a smaller scale in all 50 states. In fact, Oklahoma experiences approximately 20 landslides per year (J.B. Nevels, personal communication, September 15, 2005) that damage homes, roadways and infrastructure and quietly cost the taxpayers substantial amounts of money. A progressive landslide occurring initially 35 years ago in Sapulpa, Oklahoma, cost taxpayers approximately $4M to continually repair and eventually mitigate the slide. Many times, the slide recurs seasonally and roadways must be continually cleared of debris, or physically moved to avoid the sliding soil.

There is currently a lack of technical understanding about why certain soil deposits and road cuts fail at commonly used slope geometries. This paper discusses the construction background, geometry and physical/chemical soil composition of a compacted fill embankment that experienced progressive sliding in Chickasha, Oklahoma. The paper also discusses the hydrogeology and climate of the region in order to start to relate safe and stable constructed slope geometry to soil type and geologic setting. From the results of this study and others like it around the state of Oklahoma, engineers should be able to predict whether slopes of a particular geometry in a given location are likely to be stable. Finally, the paper discusses the Oklahoma Department of Transportation's recommendation for mitigating the slide based on the investigations performed in this study.

Introduction

In April of 2005, the Materials Division of the Oklahoma Department of Transportation received a letter from their Division 7 office stating that there was a soil sliding problem on the south side of an embankment along Route US 62. The slide recurred in January 2005. According to the division's records, the landslide had occurred at least three times since the embankment's construction in 1970. ODOT had previously attempted to correct the problem by benching the slope but apparently had not intercepted the slip plane. This recurring landslide was especially troublesome because there were four houses at the toe of the embankment and the current slide was encroaching upon the backyards of residents. The Division 7 engineers requested the assistance of the Geotechnical Branch of the Materials Division with an investigation and recommendation on potential treatments to correct the recurring landslide.

Background

The investigation took place in Chickasha, Oklahoma (Grady County) on the south side of Route US 62, approximately 600 feet NW of the Junction of the St. Louis and San Francisco Railroad (Figure 1) between May and September 2005. This compacted fill embankment was

[1] Assistant Professor, Department of Civil Engineering and Environmental Science, University of Oklahoma, Norman, OK 73019; acerato@ou.edu
[2] Undergraduate Research Assistant, REU Summer 2005, Lafayette College, Easton, PA 18042; oleskir@gmail.com
[3] Undergraduate Research Assistant, REU Summer 2005, Princeton University, Princeton, NJ 08544; cpuklin@princeton.edu

constructed in 1970 when the east bound 2-lane road was built, creating a 4-lane divided highway. The embankment slope was constrained by the existing Boles-Holms subdivision to the south, making it necessary to create a steep embankment of approximately 22.5° (2.5:1). The borrow pit for this compacted embankment is located approximately 0.5 mile NW on Route US 62 directly south of the intersection of Routes US 81 and US 62. The slope was vegetated with native grass and mowed monthly by the state.

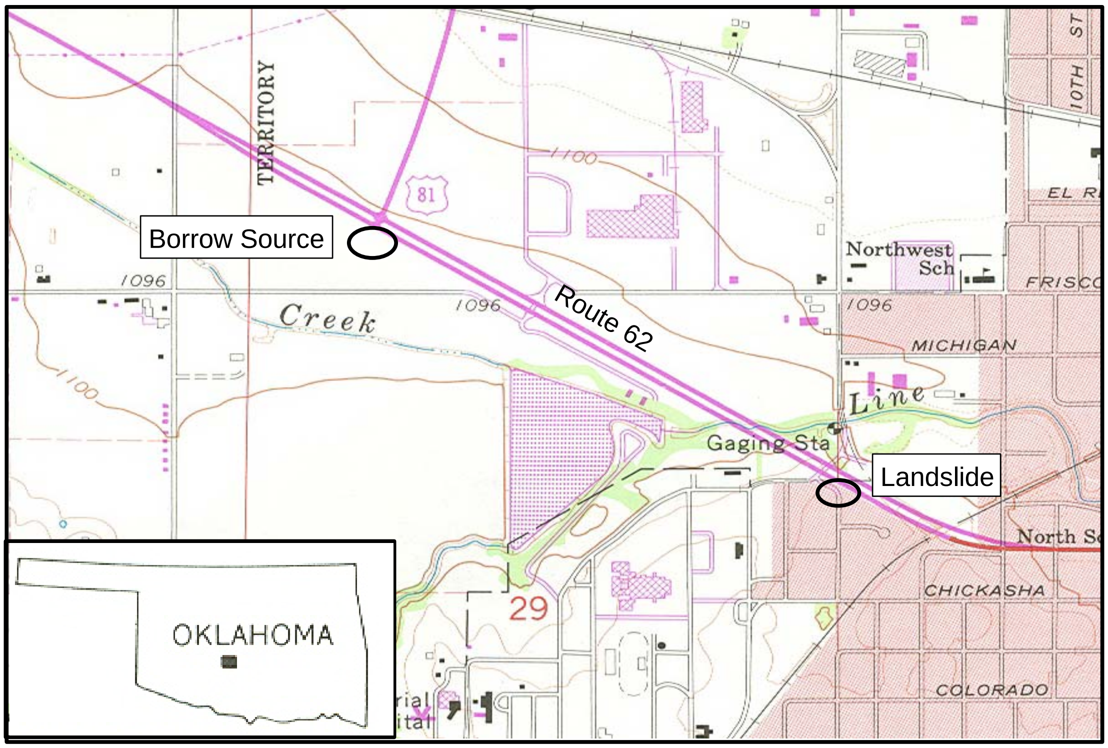


Figure 1. Location of Landslide and Borrow Source in Grady County, OK.

The geology of the region is of middle Permian age and is classified as the Dog Creek geologic unit, which is mostly red-brown silty shale with some fine-grained sandstone. The unit contains one or two layers of thin dolomite or gypsum in the lower part and the basal part grades southward into the Chickasha geologic formation. The thickness of this Dog Creek shale averages around 130 feet near Chickasha. The unit forms broad flat to gently rolling prairie topography. Above the Dog Creek shale lays Quaternary deposits of alluvium and terrace deposits consisting of sand, silt, clay and lenticular beds of gravel. The thickness ranges from a few feet to about 100 feet and contain major aquifers along the Cimarron, Canadian and North Canadian Rivers.

The climate in Chickasha, Oklahoma is moderate with yearly average high temperatures ranging from 52°F in January to 97°F in July and yearly average low temperatures ranging from 25°F in January to 70°F in July (Oklahoma Climatological Survey, OCS). The annual average rainfall is 35 inches with the wettest month occurring in May. The monthly rainfall data collected by Oklahoma Mesonet in Grady County, Oklahoma at elevation 1079 feet is shown for 3 years before the slide occurred for the third time in January 2005 (Figure 2). Years 2002 and 2004 were normal rainfall years with an average annual rainfall of approximately 34 inches. 2003 was

13 inches dry of average at 22 inches of rainfall. In October and November of 2004 there was a total rainfall of 10.75 inches. Then in December, 2004, the rainfall total was 0.35 inches. The landslide is assumed to have occurred in late December 2004 or early January 2005. The specific date is unknown.

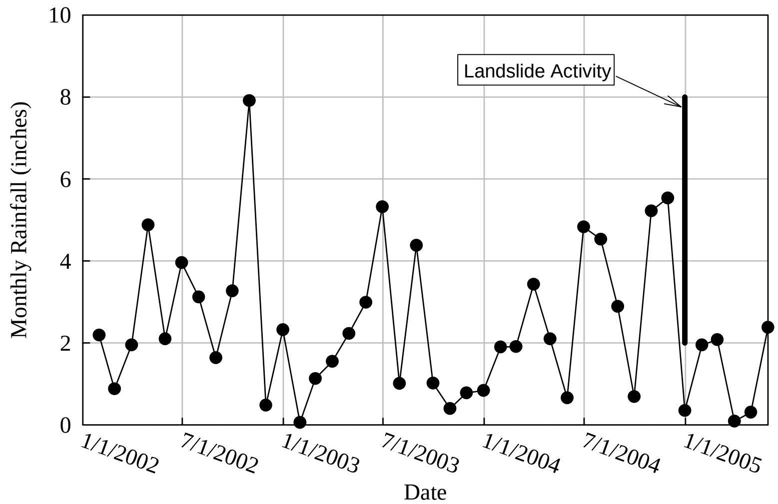


Figure 2: Monthly Rainfall for Grady County, Oklahoma.

Field Investigation

The embankment was surveyed in five cross sections and was found to have an average slope of 2.5:1 (22.5°). The landslide extent was found to be approximately 74 feet from the top of the scarp to the toe and 152 feet across (Figure 3). The field investigation revealed a significant amount of tension cracks in the slide mass; some of which were found to be filled with water. Numerous hand auger borings were found to have perched water levels as well. The landslide was recognized as a rotational slide from the existence of a vertical scarp and base debris. The failure surface of the landslide was determined using the friction circle method. This method is useful for homogenous soils with $\phi_u > 0$ and may be used when both cohesive and frictional components for shear strength have to be considered in the calculations. Once the failure surface was determined, the ordinary method of slices was employed to determine a backcalculated friction angle of the slide assuming a factor of safety (FOS) of 1 and cohesion equal to zero. A further assumption is made that since the slide has been recurring the operating shear strength along the failure surface approaches a value at or near the residual shear strength (ϕ_r, c=0). Rotational slides in soils generally exhibit a ratio of depth of the surface or rupture to length of the surface of rupture (d/L) between 0.15 and 0.33 (Skempton and Hutchinson 1969). This slide had a d/L ratio of 0.28 (21 ft./74 ft.).

After the embankment was surveyed and the landslide extents delineated, a thorough investigation of the subsurface of the site was performed using split spoon tests, numerous hand auger holes along each survey profile (A, B, C, D and E), thirteen piezocone soundings across the base of the embankment and fifteen borehole shear tests (BST) performed in the hand auger holes along cross section C. Although numerous soil samples were collected across the site, this paper will discuss only the soil samples from cross section C which runs through the middle of the landslide.

Soil samples were collected using a hand auger in five locations approximately every 0.5 foot in depth along cross section C (Figure 3). Borehole shear tests (ASTM D4917-02) were performed in each of the auger holes just above, at and just below the failure surface of the slide to determine the drained peak friction angle of the soil. The locations of the BSTs are shown as points along the hand-augered holes (B2-B6) in Figure 4 and the depths and results of the tests are presented in Table 1. The borings were then continued until refusal. Thirteen electronic friction cone and piezocone soundings (ASTM D5778-00) were also performed across the embankment, however only the closest three piezocone tests to the landslide are shown. The embankment was too steep for the cone truck to drive across and maintain a level working platform, therefore there are no piezocone soundings on the landslide.

There was one continuously sampled Split Spoon Test (SPT) boring completed at the top of the embankment to a depth of 49.7 feet. From this SPT boring, and from the piezocone tests performed at the base of the site, it is estimated that the Dog Creek geologic unit is located at an elevation of 1075.53 feet (Figure 4). The water table was determined from the level in the SPT hole at the top of the embankment in late May 2005. The water table was found to be located at EL 1088.78 feet. In late May 2005, after approximately six inches of rain in six months, the water level coincided with the progressive slip plane of the landslide. Before the slide recurred in January 2005, the site received 10.75 inches of rain in two months creating a higher water table elevation which intercepted much more of the existing slip plane and weakened the soil, causing the embankment to slide.

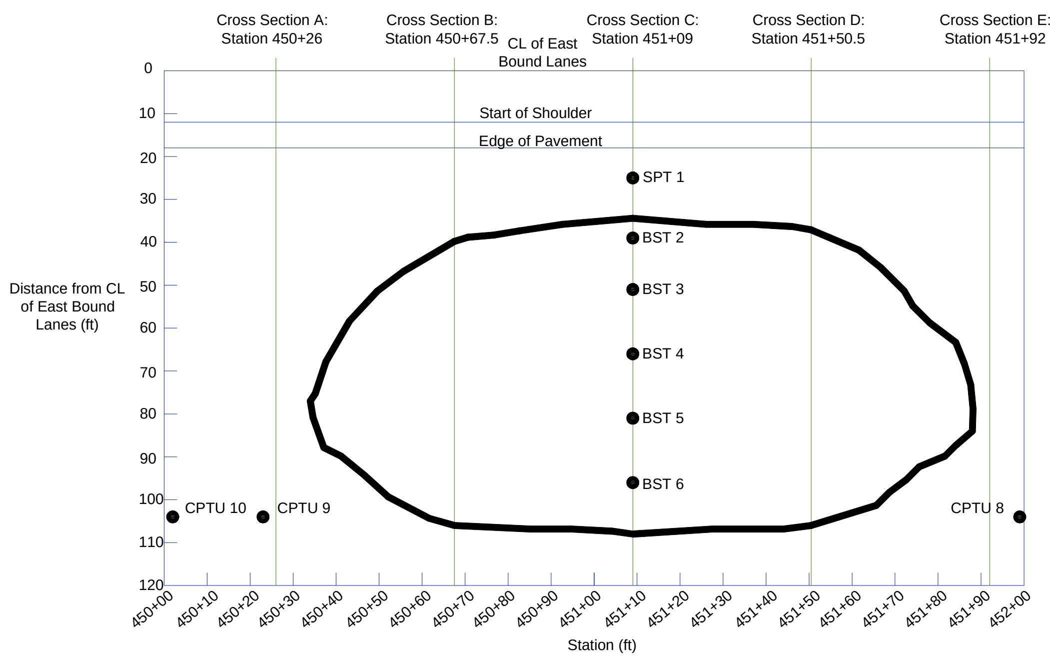


Figure 3: Plan View of Chickasha Landslide, Grady County, OK.

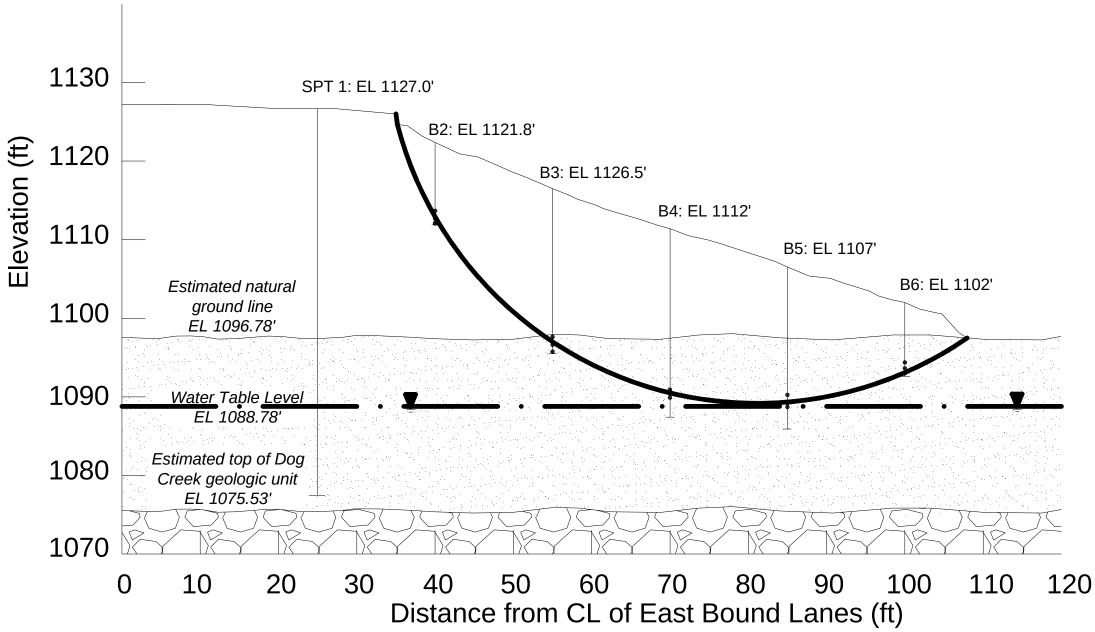


Figure 4. Profile of Cross Section C showing the landslide extent, depth of hand-augered holes and location of Borehole Shear Tests (BST).

Laboratory Investigation

The soil samples from boring 4 in cross section C were tested using a number of different index tests including; moisture content, Atterberg LL and PL (ASTM D4318-95a), shrinkage limit (ASTM D427-93), linear shrinkage (BS 1377:1975), total specific surface area (SSA) using the EGME Method (Cerato and Lutenegger 2002), external SSA using BET N_2 Adsorption (Brunauer et al. 1938), carbonate content using the Chittick Apparatus to measure the amount of calcite and dolomite (Dreimanis 1962), and hydrometer analysis to measure the clay fraction (ASTM D422-63). The results of the laboratory testing program were analyzed with depth and used with the results of the in situ testing program to determine the approximate location and physical/chemical properties of the failure zone.

Results

The results of the laboratory investigation of the soils in boring 4 along cross section C show that the site generally consists of lean clays (CL) with some layers of fat clay (CH). A summary of the grain-size characteristics is presented on the ternary diagram shown in Figure 5.

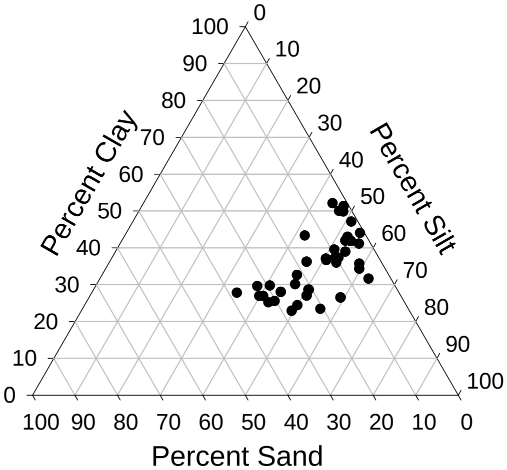


Figure 5. Ternary Diagram of soils in embankment.

As can be seen, there is a tight distribution of individual grain-size characteristics and the majority of the samples classify as "fine-grained' with more than 50% passing the No. 200 sieve. The clay size fraction ranged from 23 to 52%. Figure 6 shows the location of the samples on a Casagrande Plasticity Chart.

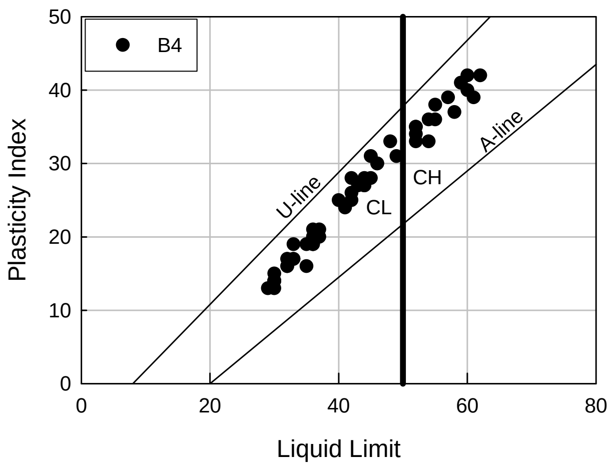
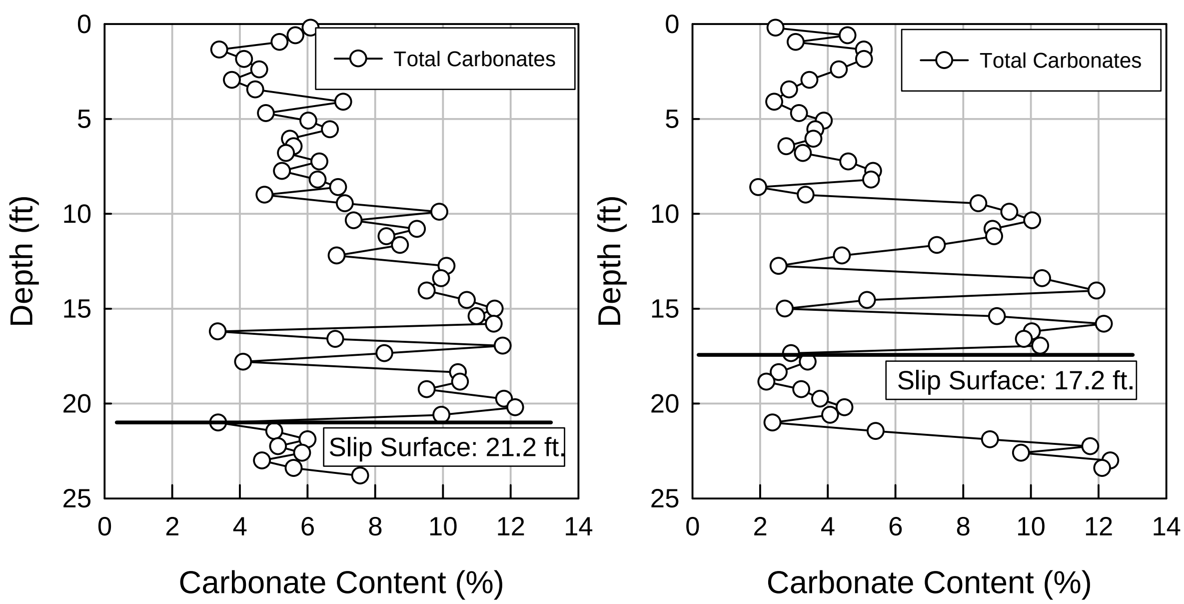


Figure 6. Plasticity Chart.

LL values ranged from 29 to 62% and PI ranged from 13 to 42%. Total SSA ranged from 25 to 188 m^2/g. Skempton's activity, A = PI/CF of the soil ranged from 0.4 to 1. The Skempton (1953) activity values for kaolinite, illite and Na-montmorillonite are 0.38, 0.9 and 7.2 respectively. Most natural soils, which are typically composed of mixed layer minerals, would fall somewhere between those values. From the A results of this study, the predominant mineralogy should be illite, however, a clearer method in delineating mineralogy is to use SSA, a more fundamental soil property than PI. The surface area activity, S_c = SSA/CF ranged from 0.9 to 3.9. The average S_c for the soil deposit is 3. For a given CF, SSA increases according to mineralogy in the following order: kaolinite < illite < montmorillonite. Cerato and Lutenegger (2005) presented S_c values of pure clays showing that kaolinite has a S_c around 0.4, illite has a S_c of approximately 3.3 and montmorillonite has S_c values above 5. Therefore, the SSA results conclude that the predominant mineralogy of the site is illite.

It was anticipated that the carbonate content of the soil would be relatively high due to the composition of the parent rock (Dog Creek shale) and it can be seen that the carbonate content at the surface of the embankment is around 6% and increases to 12% at 20 feet. Around 21 feet, where the failure plane is located, the carbonate content decreases immediately to 3.5% and then slowly increases to 7.5% at 24 feet where Boring 4C was terminated due to auger refusal (Figure 7). It is thought that the increase in carbonate content with depth is due to the leaching of the carbonates through the soil profile with rainfall infiltration. The depth of maximum carbonate content in boring 4C is where the slip surface was backcalculated to have occurred. In order to determine if this phenomenon held true throughout the slide, Boring 5C, just downslope of 4C, was tested for carbonate content with depth. Again, it was found that the slip surface could easily be seen in the profile at approximately 17 feet where the carbonate content decreased rapidly from 10% to 3% and then gradually increased above the bedrock.



(a) (b)

Figure 7: a.) Boring 4 Carbonate Content and b.) Boring 5 Carbonate Content with depth.

Carbonates act as a cementing agent in soils which increases the shear strength as long as they remain unsaturated. However, when carbonates are inundated with water, they weaken and may cause sudden settlement, collapse or sliding of the soil deposit. The 10.75 inches of rain in the 2 months prior to the slide could have initiated the slide along the existing shear plane by elevating the water table which is located close to the failure plane and in the existing soil material. The progressive slide surface weakens with the inundation of water and slips along the circular path, therefore the slip plane can be delineated by the sudden decrease in carbonate content along the length of the slide. As can be seen from Figure 4, the slide recurs through the compacted fill and into the existing native soils.

Friction cone and piezocone tests were performed in general accordance with ASTM D 5778-00 and showed a silty clay to clay profile for 9 feet in CPT 8 (east side of landslide), 31 and 33 feet in CPT 9 and CPT 10 which are located on the west side of the landslide. The tip resistances (q_t) and sleeve friction (f_s) were found to be 0 to 853 psi and 7.1 to 28.4 psi respectively. At 9 feet in CPT 8, the tip resistance jumped to 2,559 psi, indicating a silty sand to sandy silt. The profile was terminated at 12 feet. On the west slide of the landslide, the piezocone hit a rock layer at 31 feet in CPT 9 and 33 feet in CPT 10 and the profile was terminated.

The shear strength of the slide was backcalculated using the ordinary method of slices assuming a factor of safety of 1 and cohesion equal to 0. The friction angle determined from this analysis was approximately 13°. The borehole shear tests (BST) were performed three times in each boring along cross section C shown previously in Figure 4. The test was performed above, at and below the failure plane delineated by the friction circle method, to determine the drained peak friction angles. The results are presented in Table 1.

Table 1: BST Results

Location	Depth (ft)	ϕ (deg)	c (psf)
B2	8.0	11.0	0
	9.0	28.0	20.9
	9.6	17.4	20.9
B3	19.0	12.5	41.8
	20.0	15.6	158.7
	21.0	10.0	167.1
B4	20.5	17.5	292.4
	21.0	22.0	198.4
	21.5	31.0	41.8
B5	16.3	23.0	229.7
	17.3	33.0	229.7
	17.8	33.0	0
B6	7.8	35.0	62.7
	8.5	33.0	20.9
	9.0	28.0	83.5

The friction angles are lower at the top of the slide in B2 and gradually increase toward the toe of the slide (B6). It is unclear if the BST results are completely drained tests or if the results represent undrained shear strength of the soil mass, but they serve to provide a reference shear strength for similar embankment materials if the proximity of the water table in the foundation soils and steep embankment slope had not influenced the development of a progressive slope failure. The BST's were performed in unsaturated soil, therefore the friction angles will be higher than tests performed on saturated soils. The slide occurred three times since its construction in 1970 along the same slip plane; therefore the slide seems to be controlled by the residual shear strength in the predefined shear plane, which is substantially lower than the strength of the undisturbed material.

Conclusions

A progressive, rotational landslide in a compacted embankment was studied near Chickasha, Oklahoma in order to determine the most appropriate mitigation technique. The embankment was built during the Route US 62 expansion to four lanes in 1970. Because of land constraints due to a housing subdivision, the embankment was constructed with a slope of 2.5:1 (22.5°). The soil used in the embankment construction was from a local borrow source and using numerous laboratory and in situ tests was determined to consist of silty clay to clay soils, predominantly illitic in nature. The carbonate content of the soil deposit increased with depth from 6% to 12 % above the slip surface due to leaching and decreased rapidly directly below the failure plane. The significance of the carbonate reduction at the failure plane in relation to the overall slope stability is that the carbonates weakened with an increase in saturation due to rainfall infiltration and rising water table and the cement bonding broke creating a slippery surface for the soil mass to slide along.

The rotational failure surface was located using the landslide survey and the friction circle method assuming a FOS of 1 and cohesion value of zero. Since the slide is a progressive slide, the residual friction angle of the slip plane governs the strength of the soil mass. This friction angle was backcalculated using the ordinary method of slices and was found to be 13°. The friction angle determined using the Borehole Shear Test along the slip plane ranged from 15° to 33°, which are typical values for unsaturated soils. The climate of central Oklahoma is such that a majority of soils in embankments remain unsaturated for most of the year. In instances of large rainfall events, soils become saturated decreasing the shear strength. If an unsaturated compacted soil embankment is constructed with too high of a slope, it may fail under these conditions. In the case of the progressive Chickasha landslide, the natural water table is located in the existing native soil directly below the slip surface. Any significant amount of rain will raise the water table and saturate the soil, weaken the carbonate bonds and cause the embankment to slide.

The Oklahoma Department of Transportation Department's (ODOT) Materials Division recommended that a z-section sheet pile wall be driven across the length of the embankment to refusal to ensure the slide does not encroach on the Boles-Holms subdivision residents. The sheet pile will intercept the rotational slip plane and stop the recurring landslide from sliding again. The completed grade of the sheet pile wall will be raised to such a point that the embankment slope can be flattened to a 3:1 slope. The sheet pile will be curbed to improve the aesthetics and protect the sheet pile.

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