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HYDROLOGIC MONITORING AND 2-D ELECTRICAL RESISTIVITY IMAGING FOR JOINT GEOPHYSICAL AND GEOTECHNICAL CHARACTERIZATION OF SHALLOW COLLUVIAL LANDSLIDES

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HYDROLOGIC MONITORING AND 2-D ELECTRICAL RESISTIVITY IMAGING FOR JOINT GEOPHYSICAL AND GEOTECHNICAL CHARACTERIZATION OF SHALLOW COLLUVIAL LANDSLIDES

DISERTATION

A dissertation submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy in the College of Arts and Sciences at the University of Kentucky

By

Matthew Michael Crawford

Lexington, Kentucky

Co-directors: Dr. L. Sebastian Bryson, Professor of Civil Engineering and Dr. Edward W. Woolery, Professor of Earth and Environmental Sciences

Lexington, Kentucky

2018

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ABSTRACT OF DISSERTATION

HYDROLOGIC MONITORING AND 2-D ELECTRICAL RESISTIVITY IMAGING FOR JOINT GEOPHYSICAL AND GEOTECHNICAL CHARACTERIZATION OF SHALLOW COLLUVIAL LANDSLIDES

Landslide characterization and hazard assessments require multidisciplinary approaches that connect geologic processes with geotechnical parameters. Field monitoring of hydrologic variables such as water content and water potential, coupled with geoelectrical measurements that can establish relationships used for geotechnical and landslide hazard investigations is deficient.

This study brings together different techniques to develop a methodology that connects geoelectrical measurements and shear strength. A field-based framework was established that includes (1) analysis of long-term soil moisture fluctuations within different landslides (2) establishment of constitutive and new equations that test the use of electrical conductivity to predict soil-water relationships and shear strength (3) using electrical resistivity tomography (ERT) to support and facilitate the prediction of shear strength in a slope.

Hydrologic conditions including volumetric water content, water potential, and electrical conductivity in the soil were measured at three active landslides in Kentucky. The in-situ electrical conductivity used within the framework is valid as a predictor of suction stress and shear strength. The ERT supports interpretations of landslide failure zones, landslide type, lithologic boundaries, and changes in moisture conditions, but also is able to utilize the methodology to calculate shear strength, and provide a spatial view of shear strength in the slope. The practical application of this framework is to support landslide hazard assessment and further understand the long-term influence of moisture conditions in hillslope soils. These parameters are pertinent to investigating the stability of landslides that are often triggered or reactivated by rainfall.
KEYWORDS: Landslides, Hydrologic Monitoring, Soil Mechanics, Soil Moisture, Shear Strength, Electrical Resistivity Tomography

Matthew M. Crawford

September 24, 2018
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CHAPTER 1

Introduction

1.1 Problem Statement

The societal and economic impacts of landslides are significant, and reported occurrences are underestimated globally down to the local level. Landslides, in general, occur along large mountain ranges and other areas with steep slopes, and the combination of climatic conditions, geology, and increasing development on hillslopes contributes to high occurrences (Petely, 2012; Lu and Godt, 2013, Highland and Bobrowsky, 2008). In the United States, landslides result in 25 to 50 fatalities annually and approximately $3 billion in damages (Highland and Bobrowsky, 2008; Lu and Godt, 2013). Landslides pose threats to roads, homes, utilities, rivers and streams. Although records are incomplete in the U.S., direct costs for repair, replacement, and maintenance of infrastructure are estimated to be 2 to 3 billion dollars annually (Highland and Bobrowsky, 2008). Indirect costs such as road closures, utility interruption, decreasing property values, and litigation expenses are difficult to quantify, but are certainly significant.

Landslides occur when the shear stresses imposed on a slope exceed its available shear strength, i.e., when the resisting forces such as friction and cohesion are overcome by the load. Bedrock geology, slope angle, slope morphology, groundwater dynamics, soil type, and slope modification are some of the factors that influence stress on a slope. These forces act over time and space at different scales, creating a localized phenomenon, which makes landslide hazard assessment challenging.

This study primarily focuses on the soil moisture conditions in shallow, active, colluvial landslides. Geology and geomorphology, soil strength, and seasonal hydrologic conditions are complex factors that affect moisture fluctuation, especially in the unsaturated zone. For example, clayey to silty colluvium develops on slopes underlain by shale and siltstone dominated bedrock that weathers rapidly, and because of the clay content, these soils have the capability to hold water and become susceptible to movement. Colluvial soil, that makes up most of the unsaturated zone, is typically poorly sorted with grain sizes that range from clay-size to large rock fragments, perhaps a meter or more long. Colluvium accumulates slowly to rapidly, forming veneers above bedrock of varying thickness across the slope. Colluvium transportation downslope and its velocity ranges from imperceptible (creep) to rapid. Landslide types that occur in colluvial soils are commonly thin (< 2 m) translational slides or thicker rotational slumps, but both types have the capability to morph into damaging debris flows or debris slides, especially on steep slopes (Turner, 1996; Fleming and Johnson, 1994).

The disturbance in the stress-strength equilibrium within the colluvium can occur at different time scales, but colluvial landslides are most commonly triggered by rainfall,
either long duration events or short intense downpours. Soil properties, slope morpholgy, and hydrology were examined in detail for this study, and used together along with electrical data in order to corroborate with shear strength. Long-term field monitoring of variables such as water content, water potential, and electrical conductivity that can establish relationships used for geotechnical and landslide hazard investigations is deficient, particularly in regard to the shallow unsaturated zone.

1.2 Hydrologic Conditions in the Unsaturated Zone

Landslide behavior and stability, especially for shallow colluvial landslides, are highly influenced by fluctuating water content and stresses in the unsaturated zone. These factors also contribute to subsequent landslides (Godt et al., 2009, 2012; Bittelli et al., 2012; Lu and Godt, 2013). Stresses in the unsaturated zone vary because of transient water flow, perched water, and various soil properties. Shear strength of the soil system is the mobilized shear stress along a failure plane at failure. In-situ soil systems are partially saturated and exhibit fluctuations in matric suction (water potential), which is the difference between the pore air pressure and the porewater pressure (i.e., $u_a - u_w$) and fluctuations in effective stress. Water potential and effective stress are often reduced when rainfall increases (Godt et al., 2009; Lu and Godt, 2013; Oh and Lu, 2015). Therefore, shear strength will also vary with moisture conditions.

There are competing conceptual models of soil mechanics and the initiation of landslides, particularly in shallow colluvial slides above the water table (1) the effective stress principle applied to saturated soils; i.e., sliding occurs at a failure zone that is saturated and has compressive pore-water pressures acting on it and (2) the state of stress in the soil is modified by discrete changes in infiltration and water potential, and these changes can lead to landslides without complete saturation (Lu and Godt, 2013). The stress state variables are to unsaturated soils what effective stress variables are to saturated soils. Stress state variables are independent of physical properties of a soil, and depend on the number of phases (air, water, air-water interface in voids). These main stress state variables are net normal stress ($\sigma - u_a$) and water potential ($u_a - u_w$). Shear strength of unsaturated soils is thus defined by these two stress state variables.

As in the colluvial soil, the moisture conditions in the unsaturated zone are anisotropic relative to changes in grain fabric and degree of saturation, thus making moisture condition an important factor to analyze in regard to slope movement (Lu and Likos, 2006). The relationship between effective degree of saturation and water potential is a form of a soil-water characteristic curve (SWCC). The SWCC describes the functional relationship between soil water content and water potential under equilibrium conditions. It is an important soil property related to pore space distribution (size, interconnectedness) that is critical for an understanding of landslide dynamics. A non-linear increase in strength as the soil de-saturates (dries) as a result of increase in water potential. Thus, shear strength of unsaturated soil should bear relationship to the SWCC. Unsaturated soil strength does not rely on “skeleton” particle on particle stress, but the available interaction energy within the soil similar to surface tension, i.e., a reduction in
capillary stress from a loss of water potential (as opposed to development of positive pore pressures) that can also trigger landslides. The use of the extended Mohr-Coulomb shear strength equation in this study does not require a coefficient of effective stress.

Another stress state variable important to the behavior of shear strength after the residual state is suction stress. Suction stress is the product of effective saturation and water potential, and can vary within the unsaturated zone depending on soil type, moisture conditions, and depth below the surface. As the soil becomes more saturated, suction stress is reduced and can contribute to triggering of landslides (Bittelli et al., 2012). In clayey soils, in which water potential has a large range, suction stress during infiltration could be reduced by as much as 500 kPa (Lu and Godt, 2013). Analyzing suction stress over time and correlating it with rainfall can be a proxy for changes in effective stress in a hillslope soil, during wetting and drying (Lu and Likos, 2004; Lu, 2008; Lu et al., 2010; Lu and Godt, 2013; Dong and Lu, 2017). This study focuses on using geoelectrical measurements to develop these soil-water relationships in the unsaturated zone, and thus be an indicator of colluvium shear strength.

1.3 Hydrologic Measurements and Electrical Resistivity Tomography

Geophysical methods such as electrical resistivity tomography (ERT) are commonly used in hydrologic and geotechnical investigations for subsurface characterization. The properties of electrical current flow through a soil system are affected by parameters such as soil type, pore structure, degree of saturation, stress history and state. These parameters also affect the strength and deformation behavior of a soil system. Thus, there is a high likelihood that electrical measurements in soils will provide a reliable means to evaluate and predict engineering behavior. Advantages of using electrical resistivity over other investigative tools include fast gathering of data, repeatability, and being a non-intrusive way to assess the geologic and hydrogeologic regimes. Having available equipment also makes it cost-effective and time-effective. Several experiments have attempted to correlate ER and soil behavior, but these were primarily conducted in a laboratory. Few researchers have used field ER measurements to obtain shear strength properties for shallow, heterogeneous landslides.

The factors that affect the physics of slope stability are also the factors that affect the physics of electrical current flow. Electrical resistivity variations in rock and soil are primarily caused by moisture content, conductivity of pore fluids, grain size, porosity, permeability, pore water temperature, and lithology (Sirles et al, 2012). For landslides, these variations may be detections of lateral continuity, slide planes, groundwater concentrations, or clays. However, it is not common practice to quantify slope stability based on electrical parameters because the non-uniqueness of geophysical surveys is difficult to link to mechanical properties of soils.

Geophysical and geotechnical data sets gathered for landslide investigations are commonly acquired independently in order to answer different questions. Field investigations of landslides that attempt to correlate geophysics and geotechnical
properties are conducted with a wide range of methodologies and rarely try to use electrical data to model soil-water relationships and predict shear strength. Current methods to obtain soil properties that address slope stability involve costly and lengthy geotechnical experiments; therefore a methodology that correlates in situ electrical data and surface electrical resistivity with geotechnical data, extrapolating data needed to improve slope characterization, would advance the ability to quantify and evaluate landslide hazards.

Field monitoring of hydrologic variables in a colluvial slope, combined with ERT surveys, is an effective joint framework that can assess a soil’s geotechnical properties. Variables such as water content, water potential, and geoelectrical measurements can establish the relationships needed for geotechnical and landslide hazard investigations, particularly concerning the shallow unsaturated zone.

1.4 Conceptual Overview

Assessment of landslides for geologic and geotechnical analyses requires four components; (i) definition of slope geometry with regard to probable shear surfaces and failure planes, (ii) definition of the hydrologic regime within the slide mass, (iii) determination of geologic materials comprising the slopes and estimates of shear strength and (iv) the detection of movement by or within the slide mass and characterization of such movements (McCann and Forster, 1990). It is posited, geophysical techniques supplemented with geotechnical laboratory data and information about the geologic conditions of a particular site can satisfy these components.

A field comparison of in-situ hydrologic parameters and electrical conductivity, along with surface electrical resistivity demonstrated in this study sets up a methodology to determine the shear strength of soils, ultimately showing that electrical data can be an indicator of shear strength. The hypothesis is that ER measured within shallow, colluvial landslide masses will correlate with the failure plane location and variations in moisture content, but can also be interpreted further to establish relationships with shear strength and be an effective and repetitive tool for slope stability assessment. Volumetric water content, water potential, and electrical conductivity were measured at shallow colluvial landslides in Kentucky and used in a framework to estimate unsaturated soil properties (soil-water characteristic curves) and suction stress. In order to model these hydrologic relationships, predictive curves were developed using two methods: (1) a basic linear regression equation and (2) a Logistic Power fitting equations. The linear regression equation uses the slope variables to model the volumetric water content versus electrical conductivity. The Logistic Power equations model soil-water relationships from wet to dry using geoelectrical data.

Relative constitutive equations were found to be valid for long-term soil measurements, and new equations developed from electrical data were determined to be useful to predict suction stress. The framework was then used with 2-D ERT measurements to predict shear strength. An unsaturated-soils shear-strength equation calculated shear strength
based on inverted ERT values. Shear strength changed at depth, indicating landslide failure zones, specific soil horizons, and areas of low resistivity, and provided a spatial view of shear strength.

This multidisciplinary approach connects geologic processes, geophysical surveys, and geotechnical parameters to assess landslides and determine parameters used to investigate slope stability. The practical application of this framework is to constraining long-term influence of moisture conditions in hillslope soils and demonstrate that surface electrical resistivity can be used to highlight strength throughout the slope.

1.5 Objectives

- Establish a long-term field-monitoring network to measure hydrologic parameters in three active landslides in Kentucky.
- Conduct ERT measurements, identifying landslide features, lithologic boundaries, and variable moisture regimes. Compare differences in multiple ERT surveys over time.
- Develop hydrologic relationships across the slope, and analyze specific parameters that influence how water moves through the slope, establishing field soil-water characteristic curves (SWCC) and suction-stress characteristic curves (SSCC).
- Develop a baseline, site-specific framework that uses field and laboratory techniques to correlates soil-water relationships and surface electrical tomography data to predict shear strength, predicting a spatial distribution of shear strength based on electrical data.

1.6 Contents of Dissertation

Chapter 1 – Introduction: This chapter consists of the Problem Statement, a background on unsaturated soil conditions and using electrical resistivity to investigate landslides, a Conceptual Overview of the research, project Objectives, and Contents of the Dissertation.

Chapters 2–5 consist of published papers and the contents is verbatim.

- Chapter 2 presents the geologic conditions, extent, and behavior of a rainfall-triggered landslide in eastern Kentucky. This portion of the study showed that landslide movement was correlated to rainfall and groundwater levels, and that electrical resistivity could be used as a tool to determine landslide stratigraphy, depth to the failure zone, and location of groundwater regimes. This chapter was published in the Environmental and Engineering Geoscience Journal in 2015.
Chapter 3 establishes a methodology to determine shear strength of soils from measured, in-situ, electrical conductivity and surface electrical resistivity tomography (ERT). The data were evaluated over multiple seasons to assess the effects of transient water fluctuations in shallow colluvial landslides. The results of this study were used to develop a framework of predictive stability models for slope systems. This chapter was published in Engineering Geology in 2018.


Chapter 4 presents the characterization of two landslides using ERT, a comparison of multiple ERT measurements over time, and implementation of a field-based methodology that uses long-term hydrologic monitoring techniques to establish a baseline framework designed to test non-unique electrical measurements and their capability of highlighting changes in shear strength within a slope. This chapter was published in the Journal of Applied Geophysics in 2018.


Chapter 5 presents long-term hydrologic monitoring in order to assess soil moisture fluctuations within the landslide and establish soil-water relationships across the slope, ultimately testing the effectiveness of constitutive models and new equations for predicting suction stress. The developed framework proves that the relative constitutive equations are valid for long-term soil hydrologic monitoring and that electrical data can be used as a predictor of suction stress. This paper has been submitted for publication to Engineering Geology.


Chapter 6 – Conclusions: This chapter summarizes the methods and findings of the research presented in the published papers, Chapters 2–5.
CHAPTER 2

Geologic, Geotechnical, and Geophysical Investigation of a Shallow Landslide, Eastern Kentucky

INTRODUCTION

Eastern Kentucky is located in the east-central Appalachian Plateau, part of the larger southern Appalachian Basin, and is affected by a wide range of landslide types and magnitudes. Landslides range from small slumps and translational slides along roadways to large earth and debris flows that can be hundreds of meters long. This physiographic region extends from Pennsylvania into parts of Ohio, West Virginia, Kentucky, Virginia, and Tennessee (Gray et al., 1979; Radbruch-Hall et al., 1982; and Outerbridge, 1987a) (Figure 2-1). The plateau is highly dissected with relief that ranges from approximately 120 to 300 m. Interbedded clastic sedimentary rocks of Paleozoic age dominate the region. Steep slopes have high incidences of landslides, and landslide susceptibility stems from particular bedrock lithologies and colluvial soils (Gray and Gardner, 1977; Outerbridge, 1987b). This region is prone to landslides, particularly during large precipitation events. For example, in 1998 storms produced 165 mm of rain in 72 hours over southeastern Ohio, causing six fatalities and millions of dollars in property and infrastructure damage (Shakoor and Smithmyer, 2005).

Landslides damage roadways, infrastructure, and residences, with mitigation costs exceeding $10 million per year in Kentucky, eastern Kentucky in particular (Crawford, 2014; Overfield et al., 2015). For example, in July of 1939, in Wolfe and Breathitt Counties, KY, 508 mm of rain fell during a thunderstorm over the course of 2 days, causing a reported four debris flows (Wieczorek and Morgan, 2008). Flash flooding in Virgie, KY, in May 1999, caused several damaging debris flows (Harley, 2011). Persistent rainfall totaling 381–457 mm across eastern Kentucky from late April to mid-May 2011 caused more than 60 landslides. A short, intense storm that dropped approximately 90 mm of rain in 3 hours over a very localized area caused a large damaging landslide in Powell County, KY (Crawford, 2012). The majority of landslides induced by heavy rain are shallow, colluvial mass wasting events. This type of landslide is common in Kentucky; however, there are few landslide characterization studies that include a combined geologic, geotechnical, and geophysical analysis. Transportation officials mitigate landslides along roadways, but very few other government agencies analyze landslide hazards, and if they do, their results are not made public or are difficult to access. Private geotechnical engineering firms conduct landslide investigations and provide mitigation services, but the landslide data in their reports are not typically accessible to the public.

This study investigated the Meadowview landslide in Boyd County, KY. The
landslide occurred in April 2011 and was caused by a combination of natural and anthropogenic factors, and triggered by heavy rainfall. Local geology, steep slope, house foundation excavation, vegetation removal, and fill placement contributed to the landslide. The purpose of this project was to assess the geologic conditions, geometry, and behavior of a rainfall-triggered landslide and to evaluate the use of electrical resistivity as a tool to characterize a shallow colluvial landslide. A variety of instruments, sensors, and laboratory testing were used to collect information on meteorological and hydrologic conditions and landslide movement. A slope inclinometer and total station monitored landslide movement. Piezometers and a rain gauge collected groundwater and rainfall data, respectively. Laboratory analyses provided material index and strength properties. These included Atterberg limits (ASTM D4318), grain size distribution (ASTM D422), and consolidated undrained triaxial shear tests. The shear test results were not used in a slope stability assessment. An eight-channel resistivity meter measured surface and borehole electrical resistivity.

MEADOWVIEW LANDSLIDE

The Meadowview landslide is located in Boyd County, eastern Kentucky (Figure 2-1). The bedrock in the area consists of interbedded shale, underclay, sandstones, and coals. Dobrovolny et al. (1963) stated that plastic and semiplastic shales and underclays are highly impermeable and are the least competent rocks in the area. Most landslides occur along the underclays, where hillsides are steep. Many small landslides have occurred along these beds in hillside excavations for houses. These rocks develop sandy to clayey colluvial soils on the slopes and residual soils on the ridgetops. The landslide material consists of colluvium with added disturbed material from foundation excavation. Colluvium ranges in thickness from 1 to 3 m. During the excavation of the house foundation, material was pushed down into a naturally concave part of the slope. The concavity was accentuated near the toe by additional excavation for a power line that leads from the base of the slope toward the crown of the slide. The colluvium and excavated material observed at the surface are light brown and clayey to silty, with abundant shale and sandstone fragments. The soft clay soil is mottled gray, and the silty shale fragments are micaceous. During bulldozing, an outcrop of gray, soft clay was exposed near the toe of the slide that correlates to the “clayey shale” described in the boring logs. Large sandstone slabs are also present in the slide material.
Figure 2-1. Locations of the Appalachian Plateau, eastern Kentucky, and project area in Boyd County.

The Meadowview landslide occurred in late April 2011 as approximately 203 mm of rain fell over the month and triggered the failure (Community Collaborative Rain, Hail, and Snow Network, 2013; Kentucky Mesonet, 2013). The slope containing the slide ranges from approximately 13° near the ridgetop, above the crown, and steepens to 16.7° near the toe of the slide. The landslide occurred in a naturally concave part of the slope that is forested, with the exception of the trees and shrubs that were removed for the house excavation. The landslide is active, containing multiple scarps, seeps, and small localized flows. Rotational movement occurred in the upper-most part of the landslide, and closer to the toe the slide material morphed into a translational flow. The slide measures approximately 44 m long down the axis and 40 m wide near the middle (Figure 2-2). The main scarp height ranges from a few centimeters at the flanks to approximately 1.5 m near the middle. The volume of displaced material (after the landslide) was calculated as approximately 2,517 m³, assuming a half-ellipsoid shape and using a maximum depth of rupture (approximately 2.7 m) (Working Party on Worldwide Landslide Inventory, 1990; Cruden and Varnes, 1996). A prominent secondary scarp is present approximately 10 m downslope from the head scarp. Small tension cracks occur on the flanks of the
upper slide area. High concentrations of water occur at the toe of the landslide. Identifying slope geomorphology is an important part of assessing landslide susceptibility. Natural colluvial soils accumulate in concave parts of slopes and often have high landslide incidences. There is evidence of pre-existing landslide activity along the ridge, adjacent to the main slide area, including old (historic?) scarps, hummocky topography, and bent tree trunks.

Figure 2-2. Aerial image of the Meadowview landslide. The main landslide area is within the dashed outline. Axes show dimensions of the slide. Borehole locations showing instrumentation types also depicted.

GEOTECHNICAL INVESTIGATION

Boreholes and Material Properties

Six boreholes were drilled into the Meadowview landslide (Figure 2-2) on March 13 and 14, 2013. The borehole locations were chosen to obtain data near the downslope axis of the landslide and near the main scarp and toe. A 3.25-in. (8.25-cm) hollow-stem auger was used to core all boreholes. Continuous sampling was performed with a Standard Penetration Test split spoon (18 in.; 45.7 cm) to obtain moisture content through two of the boreholes. A summary of the material properties is contained in Table 2-1. Two boreholes (B1 and B3) were constructed with inclinometer casing,
two boreholes (B2 and B4) were converted to open standpipe piezometers, and two boreholes (B5 and B6) were cased with slotted polyvinyl chloride (PVC) and used for borehole electrical-resistivity measurements. B5 and B6 were located such that they lined up with the inclinometer boreholes. Lithologic units in boreholes B1 and B3 were logged, and stratigraphy was interpreted.

Borehole B1 was drilled into bedrock to a total depth of 6.5 m and well below the assumed failure surface. The uppermost soil consisted of 2.7 m of disturbed colluvium, and water was encountered at a depth of 1.2 m. The disturbed colluvium was divided into two types: 1.2 m of sandy, lean clay with gravel that overlies 1.5 m of sandy, fat clay. The boundary between the two colluvial types may explain a difference in the disturbed material that came from excavation of the house foundation above the landslide and natural hillslope colluvium. Below the disturbed colluvium are three layers: 0.7 m of stiff to hard, fat clay; 0.7 m of weathered claystone; and 2.4 m of clayey shale. The boring was terminated at 6.5 m in weathered clayey shale. Soil density increased significantly at the contact between the two colluvium types and also between the native fat clay and weathered claystone. The field N-values increase from 4 to 43 and from 18 to 50, respectively.

Borehole B3 was drilled to a total depth of 4.7 m. The uppermost soil consisted of 0.6 m of disturbed colluvium, and groundwater was encountered near the surface. Below the fill is 1.2 m of lean clay and 2.7 m of clayey shale. Drilling was terminated when carbonaceous, laminated, weathered shale was encountered at a depth of about 4.7 m. The field N-values increased at the lean clay–clayey shale contact, indicating an increase in density.
Table 2-1. Summary of the material properties from borehole samples and boring logs of the Meadowview landslide.

<table>
<thead>
<tr>
<th>Borehole B1</th>
<th>Depth (m)</th>
<th>Field Description</th>
<th>% Gravel</th>
<th>% Sand</th>
<th>% Silt</th>
<th>% Clay</th>
<th>Plasticity Index</th>
<th>Field N-value</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 – 1.2</td>
<td>sandy lean clay with gravel (CL) - fill</td>
<td>4.3</td>
<td>45.5</td>
<td>23.8</td>
<td>26.3</td>
<td>16</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>1.2 – 1.5</td>
<td>sandy fat clay (CH) - fill</td>
<td>4.2</td>
<td>28.6</td>
<td>23.1</td>
<td>44.1</td>
<td>N/A</td>
<td>43</td>
<td></td>
</tr>
<tr>
<td>1.5 – 2.7</td>
<td>sandy fat clay (CH) - fill</td>
<td>9.1</td>
<td>41.4</td>
<td>19.4</td>
<td>30.1</td>
<td>N/A</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>2.7 – 3.4</td>
<td>fat clay (CH)</td>
<td>very stiff to hard, residual soil structure</td>
<td>16</td>
<td>18</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.4 – 4.1</td>
<td>claystone</td>
<td>severely weathered, very soft</td>
<td></td>
<td>50</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.1 – 6.5</td>
<td>clayey shale</td>
<td>thinly laminated, weathered, very soft, minor interbedded sandy shale</td>
<td>N/A</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Borehole B3

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Field Description</th>
<th>% Gravel</th>
<th>% Sand</th>
<th>% Silt</th>
<th>% Clay</th>
<th>Plasticity Index</th>
<th>Field N-value</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 – 0.6</td>
<td>sandy lean clay with gravel (CL)</td>
<td>moderately stiff, micaceous, sandstone fragments</td>
<td>8</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.6 – 1.8</td>
<td>lean clay (CL)</td>
<td>6.6</td>
<td>37.6</td>
<td>21.8</td>
<td>34</td>
<td>8</td>
<td>11</td>
</tr>
<tr>
<td>1.8 – 4.6</td>
<td>clay shale</td>
<td>thinly laminated, weathered, very soft</td>
<td>9</td>
<td>24</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.6 – 4.8</td>
<td>shale</td>
<td>carbonaceous, fissile, weathered, soft</td>
<td>N/A</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Surface and Subsurface Water Observations

Elevated groundwater levels cause landslides, and precipitation that elevates these levels to an instability threshold can often be the triggering mechanism. Field reconnaissance at the Meadowview landslide prior to drilling revealed the main landslide area to be very wet, especially near the toe. Several seepage zones existed throughout the landslide. Based on our hydrostratigraphic model for the site, we inferred that shale beds were causing perched water along the slope. Water runs along low-permeability clay shales and seeps out where these beds intersect the surface.

**Rainfall**

Rainfall data were collected by a RainWise tipping-bucket rain gauge. The rain gauge consists of a standalone collector and recording system. The recorder has the
ability to accumulate 1 year of rainfall with 1-minute resolution. The tipping bucket was set with a 0.25 mm/tip threshold. We installed the rain gauge on March 19, 2013. Total rainfall accumulation at the Meadowview landslide from the installation date through May 20, 2014, was 1,227.2 mm (48.3 in.) (Figure 2-3). Average annual precipitation from 1981 to 2010 in nearby Ashland, KY, was 1,122.6 mm (44.2 in.) (National Climatic Data Center, National Oceanic Atmospheric Administration, [NOAA], 2014). Considering the average annual precipitation in the area, the monitoring of the Meadowview landslide occurred during a dry year.

Figure 2-3. Daily rainfall measured at the Meadowview landslide from March 2013 to May 2014.

**Piezometer Data**

Boreholes B2 and B4 were converted to open standpipe piezometers and were used to measure groundwater levels within the landslide mass (Figure 2-3). We recorded depth-to-water using a water-level meter that consisted of an electronic probe and a cable reel. The cable measured depth from the surface (at borehole tip) to the water. The initial depth readings in B2 and B4 (both 3 m in total depth) were taken on March 19, 2013. We measured water depth once a week for the first 2 months and then recorded it monthly after that, because water levels did not fluctuate extensively.

Beginning on April 12, 2013, we also used a wireless, battery-powered Telog PR-38 Pressure Recorder to measure the groundwater levels in piezometer B2 (below the assumed failure zone). The recorder contains a pressure sensor that is placed at the bottom of the piezometer, measuring water level above the sensor. The sensor samples the frequency of water levels at user-defined intervals. We correlated groundwater fluctuations (measured in the piezometers) with rainfall. The largest pulses of rainfall caused an increase in groundwater level in the piezometers. A graph from late June to mid-September 2013 correlates with increases in ground-
water level above the bottom of the borehole with rainfall pulses (Figure 2-4). In B2, groundwater level change above the sensor, after rainfall pulses, varied from 80 mm in the spring of 2013 to 122 mm in the spring of 2014. The time frame for the groundwater increase ranged from 1 to 3 days following a rainfall pulse. The clayey colluvial fill stores a lot of water, which is perched on the low-permeability clay layers, controlling a smaller groundwater-level response to rainfall.

Figure 2-4. Maximum daily groundwater levels measured from the bottom of B2 from the pressure recorder compared with daily rainfall from June 25 through September 23, 2013. Note the slight increase in groundwater level after the rainfall events.

Landslide Movement

Inclinometer

Inclinometer measurements were used to determine the magnitude, rate, direction, and depth of movement at boreholes B1 and B3. We used a Slope Indicator Digitilt Inclinometer System, including a biaxial probe that contains two perpendicular accelerometers, in effect monitoring the displacement normal to the axis of the borehole casing. The baseline inclinometer reading was conducted on March 25, 2013. Readings were taken once a week for the first 2 months and once a month after that. Cumulative horizontal displacement in B1 in the head of the landslide through May 20, 2014, was approximately 2 cm. Cumulative displacement in B3 near the toe of the landslide through May 20, 2014, was approximately 5 cm. The greatest average velocity in B1 (0.05 mm/d) occurred from June 11 to July 2, 2013. This interval corresponded with 78.7 mm of rainfall and had the second highest daily event during monitoring, 36.8 mm on June 26. The two greatest average velocity increases in B3 were 0.16 mm/d from April 19, 2013, to May 8, 2013, and 0.5 mm/d from April 19, 2014, to May 20, 2014. These intervals corresponded with 46.9 and 130.7 mm of rainfall, respectively. Although the inclinometer measured little
movement, a correlation was made between landslide movement and rainfall events (Figure 2-5).

Generally, the increase in movement in B3 in the spring of 2013 and 2014 correlated with the obvious pulses of rainfall. The summer months contained pulses of rain that triggered most of the movement in B1. April and May 2014 showed significant increase in movement, backed up by more rainfall in these months (166.5 mm) than in 2013 (92.2 mm). To fully observe seasonal patterns in movement, monitoring should extend beyond the 14 months of data presented here.

Figure 2-5. Inclinometer displacement versus time in B1 and B3 plotted with rainfall. June 11 to July 2, 2013, and April 19 to May 20, 2014, contained a high frequency of rainfall events that corresponded with the highest average velocity displacement in B1 and B3.

**Total Station**

Surface displacement at various locations on the landslide was monitored using a Leica TC(R) 403 total station to supplement subsurface displacement information from the inclinometer. Eight survey stakes were leveled and secured with concrete approximately 0.45 m into the ground. The stakes were distributed along the landslide’s longitudinal axis from near the main scarp down to the landslide toe (Figure 2-6). The inherent accuracy of total station surveying allows small amounts of movement to be detected even before cracking or tension scarps are apparent (Keaton and DeGraff, 1996).
A relative coordinate system was created using the stakes and two known reference base points outside the slide area that were assumed to be stable. Two locations above the headscarp (denoted as pole and garage) were used as the reference points to calculate temporal movements of the eight stakes. Measurements were calculated once a month starting May 1, 2013, and ending November 13, 2013. Displacements were measured using the differences in easting, northing, and height from the initial starting date measurement. This allowed displacement of each stake to be monitored over time; it also allows measurement of the overall average stake displacement over time. The general direction of movement of the eight stakes is to the northeast, which corresponds to the general slope direction and movement of material (Figure 2-7). Stakes S3, S5, S6, and S8 moved in the expected direction, trending generally northeast. With the exception of S8, these stakes moved horizontally a total of 5.8 cm. S8 had horizontal displacement of approximately 3.74 cm in the northeast direction. S8 is at the toe, where the landslide flows, and more subsurface displacement was measured here.

Not all stakes moved in the expected direction, and several had little downslope movement, which was not discernable from the error threshold of the total station (approximately 5 mm). However, several points appeared to move upslope, located...
on the slump block, or were located at a hinge and showed no movement. S7, for example, showed backward movement and movement over time that generally trended in the southeast direction. This is reasonable, because S7 lies near the flank of the landslide that faces southeast and may have experienced rotational movement on the steep flank of the landslide. The stakes that moved downslope were all in the lower part of the landslide, below the secondary scarp, where the translational flow is occurring. The relatively small horizontal movement of the stakes agrees with the small subsurface horizontal offset measured by the inclinometer.

Figure 2-7. Coordinate system showing surface displacement of all eight stakes in the Meadowview landslide. The general trend of movement is downslope, toward the northeast. Stakes in the area indicated by the dashed circle show approximate area of little discernable movement or movement backward from rotation.

ELECTRICAL RESISTIVITY

The technique of two-dimensional electrical-resistivity tomography (ERT) has been
applied successfully for imaging many different types of landslides in order to detect failure surfaces, lithologic interfaces, and moisture regimes (Brooke, 1973; Bogoslovsky and Ogilvy, 1977; McCann and Forster, 1990; Godio and Bottino, 2001; Bichler et al., 2004; Lapenna et al., 2005; Drahor et al., 2006; Sastry et al., 2006; Jongmans and Garambois, 2007; Perrone et al., 2008; Sass et al., 2008; Schrott and Sass, 2008; de Bari et al., 2011; Travelletti et al., 2012; and Van Dam, 2012). We conducted six surface electrical-resistivity survey measurements and two borehole resistivity measurements (Figure 2-8). The borehole and surface measurements were each conducted initially on separate dates on June 14 and July 26, 2013, and both were repeated on November 13, 2013. The surface measurements were set up as two arrays perpendicular to the slope direction and one array parallel to the slope direction, down the axis of the landslide. An Advanced Geosciences Supersting eight-channel resistivity meter was used to make the measurements. The surface arrays utilized a dipole-dipole electrode configuration with 1.5-m electrode spacing. Short spacing allows for higher resolution and is optimal for landslides anticipated to be shallow (<10 m). The dipole-dipole array has been proven to be successful for obtaining higher-resolution data and for determining shallow inter- faces in landslides (Lapenna et al., 2005; Schrott and Sass, 2008). To account for topographic changes, a total station was used to survey points along the arrays. Using those points, a terrain file containing horizontal distance and elevation was created for use in generating the inverted resistivity images.

Figure 2-8. Electrical-resistivity array locations (arrows and yellow circles) in the Meadowview landslide outlined by dashed line.

The borehole measurements were made in B5 and B6, the slotted PVC boreholes,
and utilized a cross-hole method that measured voltage between electrodes. We used borehole electrodes at 0.5-m intervals. The boreholes were spaced 7.1 m apart and were 5 m deep, resulting in an aspect ratio (depth of hole/ distance between holes) close to 1.5, to maximize resolution (Advanced Geosciences, Inc., 2003). The cables hung in the two open boreholes. The electrodes must be in direct contact with the soil (as with the surface arrays), so the boreholes were filled with water to transmit the current to the soil. The boreholes were aligned with surface array MVS1, which is parallel to the downslope direction of the slide. This allowed comparison with the surface ERT images of MVS1 and MVS2, which were arranged perpendicular and parallel to the downslope direction.

Resistivity Results

Layering and clear resistivity contrasts show that high and low resistivity zones were present in the inverted images and reflect the shallow landslide geometry and both rotational and translational styles of movement. Interpreted surfaces coincide with sharp drops in resistivity, indicating high water content (perched water) and/or possibly higher clay content. With saturated soil (disturbed colluvium) encountered at a depth of just 1.2 m, we considered water to be the influential factor in the low resistivity near the surface and were most concerned with identification of the failure zone. These resistivity zones, including the failure surface, correlate with lithologies observed in the boreholes and landslide depth determined from the two inclinometers. The surface and borehole arrays show ranges of electrical-resistivity values that are generally the same with all profiles, and the ranges do not vary significantly between the two different measurement dates. Very little precipitation had fallen in the 2 days leading up to all the measurements, and little groundwater fluctuation occurred in piezometer B2. Overall precipitation amounts were less in the fall than in the summer, which may account for slight differences in the inverted imagery.

Inverted Resistivity Sections

MVS1—7/26/2013: Parallel to the landslide axis in the downslope direction

MVS1 spans 45.7 m and extends downslope from the crown of the slide to the toe (Figure 2-9). The inverted resistivity section shows that distinct layering and contrasts in resistivity are evident near the headscarp of the slide. A semi-continuous high-resistivity layer (oranges to reds) is present near the surface, ranging between approximately 50 and 600 Ohm-m. An identifiable break in the high-resistivity layer occurs at the surface at the headscarp displacement. A thin, lower-resistivity zone (greens) appears below the high-resistivity layer, ranging from 30 to 50 Ohm-m. Perched water on the underlying clay shales creates the lower resistivity (higher conductivity) values. This zone continues downslope, occurring near the surface, where water intersects the surface seeps near the toe of
the landslide. A patchy low-resistivity zone (blues) occurs below the high-
resistivity zone, approximately 2.7 m below the surface in the head of the landslide.
This low-resistivity zone ranges from approximately 8 to 19 Ohm-m. Starting at the
headscarp, this low-resistivity zone extends downslope for about 22 m and has an
undulating, arculate shape. It becomes shallower farther downslope and ends
abruptly. We interpreted this zone as the failure surface; this was confirmed by
inclinometer data that indicated displacement depth at B1 to be about 2.7 m. Below
the low-resistivity zone, resistivity increased to a range of approximately 30–50
Ohm-m (greens) down to the bottom of the section.

Figure 2-9. Inverted electrical-resistivity array MVS1. Dashed lines represent multiple
failure surfaces. Note locations of boreholes, the headscarp, and secondary scarp.

To get a closer look at the resistivity data, we extracted resistivity and depth (x, y,
and z) from the raw inverted resistivity data at the location of borehole B1. These
data show a resistivity profile through the high- and low-resistivity layers near the
headscarp (Figure 2-10). A sharp peak of a resistivity increase at about 128 Ohm-
m correlates to the lithologic change in the disturbed colluvial fill. This material
grades from a sandy lean clay into a moderately stiff sandy fat clay. There was
also a big jump in density at this interface, as shown by the blow counts in the
boring logs. Water was encountered during drilling at this interval, at about 1.2 m.
Resistivity then decreased (moisture content increased) to approximately 19
Ohm-m. This interval and the low-resistivity peak correlate with the contact
between high-moisture conditions at the colluvial fill and very stiff, fat clay-shale,
which is also the inferred failure surface. Below the inferred failure surface, the
resistivity increased slightly as the moisture content decreased.
Midslope, approximately 17.3 m downslope from the headscarp, resistivity ranged between 14 and 19 Ohm-m in the low-resistivity zone that is the interpreted failure surface. Below the failure surface, resistivity increased toward two distinct high-resistivity zones. One is a continuous arcuate zone that continues downslope; the other deeper zone is lenticular shaped. These may be the deeper, drier (?) clay-shale layers (less conductive). These high-resistivity zones range between approximately 80 and 160 Ohm-m. No borehole was drilled midslope, but the interpreted failure surface (low-resistivity peak) from the resistivity profile from MVS1 correlates with the failure surface determined from the inclinometer data (Figure 2-11).

Figure 2-11. Vertical resistivity profile taken midslope from section MVS1. The low-resistivity peak correlates with the failure surface depth measured with the inclinometer. Depth starts at the first point, toward the top of the curve, which is at the surface. Vertical axis values on inclinometer reading are depth in feet. Horizontal axis is displacement in inches.
Toward the toe (Figure 2-9), the distinct resistivity zones became more complex. Extracted resistivity and depth data (x, y, and z) from the raw inverted resistivity profiles at the location of borehole B3 showed a high-resistivity peak of 79 Ohm-m just below the surface. At B3, the colluvial fill was only 0.6 m deep, supporting the shallow flow type of slope movement at the toe. The failure surface is difficult to identify in the inverted resistivity section’s correlation to the borehole data. The inclinometer data from borehole B3 indicated that the failure surface was 1.2 to 1.5 m below the surface. The underlying high-resistivity layer (curved yellows and orange layer that start midslope) was approximately 90–130 Ohm-m and correlates to the lean clay–clay shale contact, where a stiff, structured lean clay transitions to a very soft, weathered clay shale. A distinct low-resistivity peak of approximately 50 Ohm-m occurred about 4.3 m below the surface, which correlates with the clayey shale–shale contact and a decreasing moisture content, as described in the borehole. A high-resolution, lenticular zone was present at the end of the MVS1 array. This zone was approximately 2 m in length and showed significantly higher resistivity values than did the continuous high-resistivity zone that started midslope and curved toward the toe. This feature could be a large sandstone boulder that was dislodged during excavation of the house foundation. Large boulders of that size were identified in the field, at the toe of the slide.

MVS2—7/26/2013: Perpendicular to the downslope direction, upper slope

Electrical-resistivity array MVS2 spanned 36.6 m perpendicular to the downslope direction along the upper part of the slide. This array crosses borehole B1 (Figure 2-12). There was a clear contrast between a higher-resistivity zone and an underlying low-resistivity zone present. We interpreted this boundary to be the failure surface, which corresponds with the colluvial fill and fat clay bedrock contact, and the landslide depth indicated in the inclinometer data from borehole B1. Two lenticular-shaped, high-resistivity zones (possibly connected) occupied the right side of the inverted section above the failure surface. The right side of the section (toward the end) runs northwest, leading toward the headscarp. A moderately thick sandstone layer crops out behind the headscarp and MVS2 may be intersecting this high-resistivity layer. Resistivity at this location and along the identified failure surface ranged between approximately 20 and 30 Ohm-m. Similarly, to MVS1, a high-resistivity peak from x, y, and z data extracted at the B1 location correlates to the contact among colluvial fill types, sandy lean clay, and sandy fat clay. The highest moisture content was measured at a low-resistivity peak, supporting the location of the failure surface.
MVS3—7/26/2013: Perpendicular to the downslope direction, toeslope

Electrical-resistivity array MVS3 spanned 24.4 m in a transverse direction across the toe of the slide. The inverted section shows a complex pattern of resistivity zones (Figure 2-13). An undulating low-resistivity zone is present near the surface. This zone ranged from approximately 24 to 50 Ohm-m. This low-resistivity zone transitioned to a high-resistivity zone with lenticular regions. The undulating boundary between the low- and high-resistivity zones for MVS3 was shallow, about 0.6 m deep, and correlates to the contact between sandy lean clay with gravel fill and stiff, residual, lean clay. The inclinometer measurements from borehole B3 indicate the failure surface is below the colluvial fill–lean clay contact; therefore, the failure zone at the toe may also include the lean clay unit.

November Results

On November 13, 2013, these same arrays were laid out and the electrical resistivity was measured. In general, the resistivity contrasts, interpreted features, and correlations to stratigraphic boundaries were similar to those measured in July. One change in MVS1 was the presence of a low-resistivity zone (8–26 Ohm-m) that extended down vertically below the inferred failure surface, just in front of the headscarp. This zone accentuated the rotational movement in the head. Water may have infiltrated this zone, causing the low resistivity. For MVS3 (November measurement), the measurements from the high-resistivity zones (24–50 Ohm-m) were larger and were spaced differently than the measurements from the July inverted section. Approximately 104 mm less rainfall was measured in the month preceding the November resistivity measurements. This could account for the increased area of higher resistivity in MVS3.
Borehole Resistivity

We also measured electrical resistivity at the slide using downhole electrode cables in the slotted PVC boreholes. A cross-hole method was used to measure the resistivity. Similar to the surface dipole-dipole array, this method is designed to measure the voltage between all electrodes that hung down in the boreholes. In the center of the inverted section, Figure 2-14 shows a change in resistivity that correlates with a change in material type in borehole B1 (black dashed line). B1 is between the slotted PVC holes, which are 7.1 m apart. There was no significant difference between the June 14 and November 11 measurements and resulting inverted profiles. Figure 2-14 also shows the resistivity data at depth taken from the middle of the borehole profile. There is a slight decrease in resistivity that correlates to the failure surface depth.

![Borehole resistivity results from June 14, 2013. The middle of the inverted section shows a contrast in resistivity that correlates to the colluvial fill–fat clay stratigraphic boundary.](image)

DISCUSSION

For discontinuous, variable bedrock lithologies and heterogeneous soils, drilling boreholes may not provide the data needed to interpret the landslide type and failure surface. Geophysical investigations, specifically electrical-resistivity investigations, can expand landslide hazard research by providing an overall view of the subsurface that can supplement drilling by not only identifying failure planes and moisture regimes but also by relating the electrical resistivity values to mechanical properties. Quality subsurface data, including detailed lithologic logs, an idea of groundwater flow, and the applicable laboratory data, are imperative to using electrical resistivity...
as a tool for characterizing landslide behavior. The challenge, and possible future work, involves taking a non-unique solution of resistivity measurements in the subsurface and linking it to landslide behavioral properties, such as moisture content, matric suction, clay content, and porosity. Long-term studies on more than one landslide in a region and additional materials testing could improve the use of electrical resistivity as a tool for landslide assessment. Although not addressed in this study, shallow, colluvial landslide investigations that aim to correlate electrical resistivity with factors needed to calculate shear strength, ultimately providing a tool for repetitive, effective slope-stability assessments, would be beneficial.

CONCLUSIONS

The Meadowview landslide movement corresponded to periods of greatest rainfall. This study showed that increases in groundwater levels corresponded with particular precipitation events. During the study, total displacement observed from the inclinometer in B1 was 2 cm; it measured 5 cm in borehole B2 at the toe. The highest average velocity at borehole B1 occurred between mid-June and early July 2013. During this interval, 78.7 mm of rain fell, and the second greatest daily event during monitoring, 36.8 mm, occurred on June 26. The highest average velocity at borehole B3 occurred from July 2 to July 18, 2013, during which 91.4 mm of rain fell. The rainfall at the site during the year was approximately 127 mm less than the average annual rainfall in the region, which may explain why there was only minor movement over the course of the year. The total station measurements of surface movement supplemented the subsurface inclinometer measurements. An intense or long-duration rainfall has the capability to trigger future movement.

This study also showed that the surface and borehole electrical-resistivity measurements across the Meadowview landslide resulted in inverted resistivity sections with distinct resistivity contrasts that correlate to borehole stratigraphy, failure surface depth, and groundwater conditions. Low-resistivity zones were indicators of high moisture content (along with high clay content) and correlated to the failure surface of the landslide. The inverted resistivity profiles confirmed the curviplanar and undulating nature and shallow depth of the failure surface indicated by the inclinometer data.
CHAPTER 3

Assessment of Active Landslides Using Field Electrical Measurements

1. Introduction

Landslides pose serious threats to highways and transportation infrastructure, homes, industrial structures, and utilities. The U.S. Geological Survey estimates that landslides cause in excess of $1 billion in damage and about 25 to 50 deaths each year in the United States, and worldwide they are responsible for thousands of fatalities and hundreds of billions of dollars in damage. In Kentucky, direct costs resulting from landslide mitigation along roadways and requests for Kentucky Emergency Management Hazard Mitigation grants for landslide-damaged homes are estimated to exceed $10 million per year (Crawford, 2014; Overfield et al., 2015). Assessment of mechanisms leading to failure greatly increases the capacity to model and predict future occurrences of these hazards.

Geophysical methods such as electrical resistivity (ER) are commonly used in hydrologic and geotechnical investigations for subsurface characterization. The geophysical properties of a soil system are affected by parameters such as soil type, pore structure, degree of saturation, stress state, and history. These parameters also affect the strength and deformation behavior of a soil system. Thus, there is a high likelihood that geophysical measurements in soil systems will provide a reliable means to evaluate and predict engineering behavior. In addition, geophysics-based monitoring systems can be field-deployed at costs less than that of traditional geotechnical monitoring systems. Several researchers (Lapenna et al., 2005; Mahmut et al., 2006; Perrone et al., 2008; de Bari et al., 2011; Travelletti et al., 2012; Perrone et al., 2014; Crawford et al., 2015) have used geophysical techniques such as electrical resistivity tomography (ERT) to define landslide morphology, depth-to-slide plane, lithologic interfaces, and moisture regimes. ERT is a two- or three-dimensional image of spatially distributed ER data. The advantage of ERT data is that they allow variations in moisture content and geologic materials to be determined over a large volume directly involved with the landslide, rather than at a single discrete point. Clearly, correlating ER data with strength data would be a significant benefit.

Using geotechnical data alone for landslide assessment will provide detailed information at only discrete locations. Natural geologic formations are typically highly variable spatially, however. Geophysical data can provide bulk spatial data for a site, but most geophysical data do not provide detailed information regarding the shear strength or the engineering behavior of the soil. The optimal solution to this dilemma is to couple the geophysical and geotechnical data using laboratory-based models that account for the geologic conditions at a particular site and that directly and indirectly relate geophysical measures to geotechnical parameters and behavior. The purpose of this study was to develop a framework that uses field and laboratory techniques to correlate in-situ hydrologic data and surface ER data to predict shear strength. An assessment of the
hydrologic behavior in two shallow colluvial landslides supports the use of ER as a tool to characterize landslide structure and soil shear strength.

2. Background

Landslide behavior and stability, especially for shallow colluvial landslides, are highly influenced by fluctuating water content and stresses in the unsaturated zone. These factors also contribute to subsequent landslides (Godt et al., 2009, 2012; Bittelli et al., 2012; Lu and Godt, 2013). Stresses in the unsaturated zone vary because of transient water flow, perched water, and soil characteristics. Shear strength of the soil system is the mobilized shear stress along a failure plane at failure. In-situ soil systems are partially saturated and exhibit fluctuations in matric suction (water potential), which is the difference between the pore air pressure and the porewater pressure (i.e., $u_a - u_w$) and fluctuations in effective stress. Matric suction and effective stress are often reduced when rainfall increases (Godt et al., 2009; Lu and Godt, 2013; Oh and Lu, 2015). Therefore, shear strength will also vary with moisture conditions.

Rainfall is a common landslide trigger, increasing the load and porewater pressures, and reducing shear strength. General relationships between varying soil moisture conditions and electrical data, and changes in soil strength are seldom demonstrated, however. Most investigations using field electrical data, such as ERT, for landslide assessment tend to focus on how ERT can be used to elucidate changes in soil moisture (Li et al., 2005; Travelletti et al., 2012; Bittelli et al., 2012; De Vita et al., 2012; Lehmann et al., 2013; Piegari and Di Maio, 2013). Other researchers (Cosenza et al., 2006; Sudha et al., 2009; Siddiqui and Osman, 2013) have attempted to ascertain soil properties pertinent to landslide assessment using field ER data. The aforementioned researchers did not present a comprehensive framework for relating field ER measurements with geotechnical behavior of a partially saturated soil system, subjected to seasonal variation in the moisture conditions, however.

This study establishes a methodology to determine the shear strength of soils from electrical data by comparing in-situ hydrologic parameters and electrical conductivity, along with surface electrical resistivity. The data were evaluated over multiple seasons to assess the effects of transient water fluctuations in shallow colluvial landslides. The results of this study were used to develop a framework of predictive stability models for slope systems. This baseline framework will ultimately inform engineering decisions, planning and development, safety decisions, and infrastructure resilience.

3. Field methodology

Two active landslides in Kentucky were the basis of this study. Each landslide occurs in a different geologic setting and has a different slope history. The landslides occur in relatively horizontally bedded clastic and carbonate sedimentary rocks draped with varying thicknesses of colluvium. The landslides are of different sizes, with different
volumes of material and depths to failure. In addition to the variable geology, site permission, accessibility, and proximity to past landslide activity influenced which sites were chosen.

The Doe Run landslide is located in Erlanger, in northern Kentucky, just south of Cincinnati, Ohio, in the Outer Bluegrass physiographic region (McDowell, 1986). The geology of northern Kentucky and the Cincinnati area consists of interbedded shale (75 to 80%) and limestone (20 to 25%). Clay-rich colluvial soils of varying thickness cover steep slopes and result in high landslide occurrence (Haneberg, 1991).

The monitored slope is a thin translational landslide in which the slide plane occurs along the colluvial-bedrock contact. The colluvium thickness varies from a meter or less upslope to approximately 4 m near the toe. The headscarp and landslide flanks are difficult to observe, except in a small slump at the toe of the slope that exhibits these features well. The length of the downslope axis of the monitored area is approximately 52 m. Fig. 3-1 shows the location of the Doe Run landslide. In the figure, the black dashed line represents the slump downslope at the toe. The dots represent upslope and downslope trenches containing hydrologic sensors. The yellow lines represent the locations of measured surface electrical resistivity surveys.

The Herron Hill landslide is located in Tollesboro, in the Knobs physiographic region (McDowell, 1986), just west of the Cumberland Escarpment. This area is characterized
by steep ridges and conical knobs that are erosional remnants of the Cumberland Escarpment to the east. Landslides are common along lower valley walls in the region. The extent of the slide is difficult to discern, as the entire ridge can be classified as a large landslide complex. Several slumps are visible near the monitored slope. An abandoned road runs across the slide area. The slide occurs in a soft, thick (30 to 40 m) clay shale that exhibits different layers of color and texture. The downslope axis is approximately 153 m. Fig. 3-2 shows the location of the Herron Hill landslide. In the figure, the dots represent upslope and downslope trenches containing hydrologic sensors. The red lines represent the locations of surface electrical resistivity surveys. The black dashed area approximates the landslide area; however, there are several scarps and inset slumps within and outside of the outlined area.

Figure 3-2. Overview of the Herron Hill landslide.

3.1 Geologic conditions

The Doe Run and Herron Hill landslides generally consist of silty clay colluvium that varies in texture and structure. At the Doe Run landslide, the colluvium is a silty clay loam derived from weathered shale. The colluvium can generally be divided into two zones: a light to dark brown silty clay overlying a light brown to greenish-gray soft clayey zone. The transition between the colluvium and underlying bedrock is clayey with shale and limestone rock fragments.
The Herron Hill landslide colluvium and transition to competent bedrock is more complex, and the failure zone is less obvious. The slide occurs in weathered shale and forms soft, severely eroded slopes. A thin, dark brown, silty clay top-soil layer overlies 30 cm of a brownish-gray, crumbly, silty clay loam. This layer transitions to approximately 1.5 m of a soft, brownish-green to light blue clay shale. Below the clay shale is approximately 60 cm of a reddish-brown, hard clay shale with more of a blocky structure.

3.2 Field instrumentation

Two types of sensors were used to determine long-term subsurface hydrologic conditions in the landslides. Campbell Scientific CS655 Water Content Reflectometers monitored soil volumetric water content, bulk electrical conductivity, bulk dielectric permittivity, and temperature. Decagon MPS-6 Dielectric Water Potential Sensors measured soil water potential (soil suction) and temperature. Water potential is the energy state of water in the soil, a determination of a stress state in the soil based on how water propagates through the matrix. The MPS-6 uses a porous material (ceramic disc) with a known static matrix of pores that is buried in the soil and allows for a convergence of hydraulic equilibrium. Because the two mediums (disc and soil) are moving toward equilibrium, measuring the water potential of the disc gives the water potential of the soil.

The hydrologic sensors (CS655 and MPS-6) were installed in trenches that were dug by hand at the Doe Run landslide and excavated by a backhoe at the Herron Hill landslide. Table 3-1 lists the location of each of the trenches and depths for the sensors. The sensors collected data in the upper 1 m of the colluvial soil column. Two CS655 sensors and two MPS-6 sensors were nested vertically in an upslope trench and a downslope trench. These sensors were placed in the undisturbed, up-slope face of the exposed soil in order to measure the natural transient wetting fronts in the soil. As much as possible, the sensors were nested vertically in pairs of each type. Soil stiffness or large rocks prevented a few of the pairs from being placed at the same depth. The trenches were backfilled after the sensors were placed in the ground.
Table 3-1. Locations and types of hydrologic sensors at the Doe Run and Herron Hill landslides.

<table>
<thead>
<tr>
<th>Landslide</th>
<th>Trench Location</th>
<th>Sensor Type</th>
<th>Upper Sensor Depth (cm)</th>
<th>Lower Sensor Depth (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Doe Run</td>
<td>upslope</td>
<td>volumetric water content (CS655)</td>
<td>30</td>
<td>70</td>
</tr>
<tr>
<td></td>
<td></td>
<td>water potential (MPS-6)</td>
<td>30</td>
<td>65</td>
</tr>
<tr>
<td></td>
<td>downslope</td>
<td>volumetric water content</td>
<td>75</td>
<td>130</td>
</tr>
<tr>
<td></td>
<td></td>
<td>water potential</td>
<td>55</td>
<td>130</td>
</tr>
<tr>
<td>Herron Hill</td>
<td>upslope</td>
<td>volumetric water content</td>
<td>90</td>
<td>240</td>
</tr>
<tr>
<td></td>
<td></td>
<td>water potential</td>
<td>100</td>
<td>240</td>
</tr>
<tr>
<td></td>
<td>downslope</td>
<td>volumetric water content</td>
<td>100</td>
<td>240</td>
</tr>
<tr>
<td></td>
<td></td>
<td>water potential</td>
<td>75</td>
<td>168</td>
</tr>
</tbody>
</table>

3.3 Data acquisition

The data were acquired using a Campbell Scientific CR1000 data logger. Data were retrieved in 15-min, hourly, and daily intervals. The hydrologic and electrical data presented in this paper are the average daily values. Generally, the 2015 calendar year and the first half of the 2016 calendar year were wet in these parts of Kentucky. Cumulative rainfall was 1724.6 mm at Doe Run, from May 2015 through September 2016. Cumulative rainfall at Herron Hill was 1473.2 mm from September 17, 2015 to December 7, 2016. In comparison, statewide annual average rainfall was 1285 mm in 2016, 1447 mm in 2015, and 1168 mm 2014 (www.kymesonet.org/summaries.html). For this paper, the monitoring period for the Doe Run landslide was from May 8, 2015 to November 29, 2016 (572 days). The monitoring period for Herron Hill was from September 17, 2015 to December 7, 2016 (449 days).

3.4 Soil properties

Index testing was performed on grab samples taken at trenches at each site. Table 3-2 summarizes the results of the index tests. Natural gravimetric water contents and Atterberg limits were performed according to ASTM standards (D2216 and D4318, respectively). Table 3-2 also includes the soil classification of each sample according to the Unified Soil Classification System. The data in the table show that the soils at the Doe Run landslide are primarily medium-plasticity clays, whereas the soils at the Herron Hill landslide are medium-plasticity silts.
Table 3-2. Soil properties at the Doe Run and Herron Hill landslides.

<table>
<thead>
<tr>
<th>Landslide</th>
<th>Location</th>
<th>Depth (cm)</th>
<th>$\omega_n$ (%)</th>
<th>LL (%)</th>
<th>PI (%)</th>
<th>USCS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Doe Run</td>
<td>upslope</td>
<td>70</td>
<td>41.2</td>
<td>45.2</td>
<td>27</td>
<td>CL</td>
</tr>
<tr>
<td></td>
<td>upslope</td>
<td>120</td>
<td>43.8</td>
<td>43.9</td>
<td>27</td>
<td>CL</td>
</tr>
<tr>
<td>Herron Hill</td>
<td>upslope</td>
<td>120</td>
<td>26</td>
<td>44</td>
<td>18</td>
<td>ML</td>
</tr>
<tr>
<td></td>
<td>downslope</td>
<td>120</td>
<td>31</td>
<td>43</td>
<td>14</td>
<td>ML</td>
</tr>
</tbody>
</table>

$\omega_n$ = Natural gravimetric water content; LL = liquid limit; PI = plasticity index; USCS = Unified Soil Classification System.

4. Seasonal hydrologic observations

As mentioned earlier, volumetric water content and water potential data were measured at the landslides over multiple seasons. The general hydrologic observations at the two landslide sites are summarized as follows:

1. Generally, the more shallow the sensor, the quicker the increase in volumetric water content following a rainfall. Generally, the deeper the sensor, the less the fluctuation. This behavior was particularly evident at the toe of the Doe Run landslide where the slump at the toe was prominent and the sensors were located in the upslope face of the headscarp of the slump.

2. There were two significant dry periods during the monitoring of the Doe Run and Herron Hill landslides. For Doe Run, the main dry period occurred from the end of July to early November 2015. For Herron Hill, the main dry period occurred from the end of August 2016 to the end of October 2016. These periods were typical of seasonal patterns in this part of Kentucky.

4.1 Seasonal variations in volumetric water content

Fig. 3-3 shows the seasonal variation of the volumetric water content at the Doe Run site for the upslope trench at a depth of 70 cm and at the downslope trench at a depth of 75 cm. The data in Fig. 3 show that the colluvial soil was at a saturated or nearly saturated state between May 2015 and July 2015. During this time, the maximum volumetric water content was 0.459 in both the upslope trench at a depth of 70 cm and in the downslope trench at a depth of 75 cm. The average at both locations during this time was approximately 0.45. This initial saturated state was followed by a drying stage from the end of July 2015 to late October 2015. The minimum volumetric water content was 0.321 and 0.276 in the upper and lower trenches, respectively. This drying stage was followed by a wetting stage from October 2015 to December 2015 and then a re-saturation stage from December 2015 to May 2016. A re-drying stage was observed from May 2016 to the end of the monitoring period (July 2016). The downslope location exhibited several smaller wetting and drying cycles, a result of the sensor locations in the slump at the base of the slope.

For the other depths, the minimum volumetric water content in the upper trench was 0.198 at a depth of 30 cm. The minimum volumetric water content in the lower trench
was 0.395 at a depth of 1.3 m. These minimum values also occurred during the October drying period. The magnitude of fluctuations in volumetric water content in the upper part of the colluvium was greater than in the lower, stiffer, clayey zone. The clayey soils tend to hold more water, primarily because the dipole water molecules are attracted to the negatively charged surfaces of the clay particles.

Fig. 3-4 shows the seasonal variation of the volumetric water content at Herron Hill. The figure shows that the maximum volumetric water content was 0.429 in the downslope trench at a depth of 2.4 m. In the upslope trench, a steady wetting to saturation period is evident from early January 2016 to early June 2016. However, there was less fluctuation and distinctive wetting and drying of volumetric water content at the deeper locations. The 2.4 m depth and hard clay shale is most likely contributing to this moisture behavior.

Figure 3-3. Seasonal volumetric-water-content data for Doe Run: (a) upslope at a depth of 70 cm and (b) downslope at a depth of 75 cm.

Figure 3-4. Seasonal volumetric-water-content data for Herron Hill: (a) upslope at a depth of 0.9 m and (b) downslope at a depth of 2.4 m.
4.2 Seasonal variations in water potential

During May 2015 and July 2015 at the Doe Run slide, the MPS-6 measured between $-7.5$ kPa and $-9$ kPa (the lower limit listed in the manufacturer specifications for the sensor is approximately $-9$ kPa). The maximum water potential value was $-450$ kPa on October 22, 2015, upslope at a depth of 65 cm. The maximum water-potential value at the toe (in the headscarp of the slump) was $-318$ kPa on September 29, 2015.

At Herron Hill, the maximum water potential value, reached on October 20, 2016 was $-594$ kPa and occurred upslope at a depth of 1 m. The maximum water-potential value downslope was $-400$ kPa on October 23, 2016. The water-potential sensors at the deeper trench locations stayed at the lower limit of the MPS-6 sensors for the duration of the monitoring period. This observed behavior was most likely due to the depths of the sensors relative to the specific site conditions and geology. Specifically, the upslope location at a depth of 1.68 m is above an old road where water commonly exits the slope in the form of springs and seeps. The downslope location at a depth of 2.4 m is in a soft clay layer on top of a hard, bluish-gray clay shale, which tended to be wetter (i.e. lower water potential values). Fig. 3-5 shows the drying periods and response of the sensors to rainfall. Note the long periods the sensors were near their limit, near or at saturation.
Figure 3-5. Water potential and rainfall for: (a) the Doe Run landslide and (b) the Herron Hill landslide.

5. Field soil-water characteristic curves

Using the in-situ volumetric water content and water-potential sensor data, field Soil Water Characteristic Curves (SWCC) were plotted for each sensor pair, in each trench, at their respective depths. The SWCC data help define the stress state and hydraulic regime in the unsaturated zone of the soil mass. These data are a fundamental part of assessing shear strength for unsaturated soils (Lu and Likos, 2004). A decrease in water potential increases the effective stress within a soil mass, thereby improving slope stability.

Fig. 3-6 shows the in-situ SWCC for the upslope trenches at Doe Run and Herron Hill at depths of 70 cm and 1 m, respectively. The plots show wetting and drying conditions...
within the slope. The water-potential values were converted to absolute values for plotting on a log scale (higher number is drier). The vertical points from 7 to 9 kPa represent the water-potential sensor limit at saturated or near-saturated conditions. The initial wetting or drying corresponds to conditions after the sensors were installed. The periods of wetting, drying, re-wetting, and re-drying coincided with seasonal changes in rainfall and temperature. The largest one-day rainfall total was 64 and 38 mm at Doe Run and Herron Hill, respectively. The sensors' response to rainfall varied depending on the amount of rain and the sensor depth within the landslide.

The Doe Run data indicate that the in-situ SWCC shows a hysteresis effect in that the wetting path is different from the drying path. After the volumetric water content reaches an equilibrium state at saturated conditions, however, the drying returns to the primary drying path; Ng and Xu (2012) referred to this as the penultimate suction path. This behavior is consistent with laboratory data from Sun et al. (2015) and Zhang et al. (2016). For the Herron Hill field SWCC, the paths of the curves vary over the monitoring period, but the stages of wetting and drying are similar to what is observed at Doe Run. Instead of initial saturation conditions, the conditions at Herron Hill were drying when the sensors were installed. The initial drying period began, and after several wetting and drying periods, the water content returned to the primary drying path.

The field SWCCs analyzed in this paper also demonstrate the need to consider the hysteresis effects when developing mechanical behavior models. The field measurements over a range of wetting or drying periods will change the modeled estimates of shear strength. Regardless, the range of data presented is suitable for developing correlations that are a part of the general framework for linking electrical measurements to shear strength.

Figure 3-6. Field soil-water characteristic curves for: (a) the Doe Run landslide upslope at a depth of 70 cm and (b) the Herron Hill landslide upslope at a depth of 90 cm. Each dot represents a daily average.
6. Hydrologic models

6.1 Volumetric water content model

Given that the volumetric water content sensors also measured electrical conductivity (EC), correlative models were developed for hydrologic and EC data. Fig. 3-7 shows the volumetric water content data versus EC measurements for the Doe Run upslope trench at a depth of 70 cm and downslope trench at a depth of 75 cm, and the Herron Hill upslope trench at a depth of 90 cm and downslope trench at a depth of 2.4 m. All data are from a drying path except Fig. 3-7a, which also shows that electrical conductivity and volumetric water content exhibit the hysteresis phenomenon. Distinct wetting and drying paths reflect the moisture conditions within the soil. Rainfall changes the drying to wetting (or vice versa for evapotranspiration), and the timing and magnitude of those changes vary across the slope and at different depths. Typically, the wetting path is used for assessing shear strength and slope stability (Han and Vanapalli, 2015; Guan et al., 2010; Lu and Godt, 2013), but the drying path is typically used in laboratory methodology. However, for field data sets presented here, the wetting path provides much less of a range of values for correlations. Hysteresis will affect the modeled outcomes, and future work to assess the hydrologic behavior of these colluvial soils will occur, but the predictive framework is still the same.
During and after rainfall events, the soils go from dry conditions to near the water potential sensor limit (9 kPa) relatively quickly. These clayey soils do not drain very well; therefore, the drying path is a longer process, perhaps leading to higher shear strength values as opposed to the wetting path. These data were consistent with the other sensor data in the upslope and downslope trenches. In general, the trends of the drying paths and range of volumetric water contents are the same. The Herron Hill downslope location at a depth of 2.4 m shows a minimal range of volumetric water content values. This is likely because of the depth of the sensor and the stiff, reddish-brown clay shale at this depth interval. For the upslope trench at Herron Hill, the deeper sensor location shows less range of volumetric water content and corresponding higher electrical conductivity values. For the downslope trench at Herron Hill, at the deeper sensor location, there is also a minimal range of volumetric water content, but the range and amount of electrical conductivity is less than the more shallow sensors. At Doe Run, in
general, there are higher electrical conductivity values in the deeper sensor locations, even with similar ranges of volumetric water content. This suggests that electrical conductivity change is also controlled by lithology (clays), porosity, and temperature. The in-situ data show that the volumetric water content is a linear function of the electrical conductivity. Specifically, the EC values increased proportionately with water content. The basic form of the linear function is:

$$\theta = \alpha_1(\text{EC}) + \alpha_2$$  
Eq. (3-1)

where $\theta$ = volumetric water content, $\alpha_1$ = the slope of the linear equation, and $\alpha_2$ = the intercept of the linear equation.

This linear function agrees with data presented by several researchers (Cosenza et al., 2006; Perrone et al., 2008; Bryson and Bathe, 2009; Sudha et al., 2009; Kibria and Hossain, 2012) that show a linear relation between EC data and water content within the transition zone of the soil's hydraulic regime. The transition zone is defined as the hydraulic state between saturation and residual saturation (Fredlund et al., 2012).

Table 3-3 gives the slopes and intercepts for the linear functions at each location. All sensor locations showed the same linear trend, but the ranges of volumetric water content and electrical-conductivity values varied significantly at each location. At each site, the different locations represent different hydraulic-stress histories. The two sites also represent different geologic characteristics. Thus, the different slopes and intercepts reflect varying hydraulic-stress histories and geologic characteristics. The quantification of those variations was beyond the scope of this study.

<table>
<thead>
<tr>
<th>Landslide</th>
<th>Location and Depth</th>
<th>$\alpha_1$</th>
<th>$\alpha_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Doe Run</td>
<td>upslope, 30 cm</td>
<td>1.142</td>
<td>0.030</td>
</tr>
<tr>
<td></td>
<td>upslope, 70 cm</td>
<td>0.469</td>
<td>0.201</td>
</tr>
<tr>
<td></td>
<td>downslope, 75 cm</td>
<td>0.555</td>
<td>0.178</td>
</tr>
<tr>
<td></td>
<td>downslope, 1.3 m</td>
<td>0.465</td>
<td>0.213</td>
</tr>
<tr>
<td>Herron Hill</td>
<td>upslope, 1 m</td>
<td>0.217</td>
<td>0.233</td>
</tr>
<tr>
<td></td>
<td>upslope, 2.4 m</td>
<td>0.059</td>
<td>0.398</td>
</tr>
<tr>
<td></td>
<td>downslope, 1 m</td>
<td>0.143</td>
<td>0.217</td>
</tr>
<tr>
<td></td>
<td>downslope, 2.4 m</td>
<td>0.053</td>
<td>0.364</td>
</tr>
</tbody>
</table>

6.2 Soil water potential-electrical conductivity behavior

The in-situ data show that the EC decreased as a systematic function of increasing water potential (i.e., drying). A few researchers (De Vita et al., 2012; Piegari and Di Maio, 2013; Bryson and Spencer, 2014) have shown that matric suction for a given net normal stress can be correlated with electrical parameters for a particular soil. Currently, there is not an accepted EC-suction model in the literature, however. The in-situ sensors show that the soil water potential–electrical conductivity (SWEC) curve resembles a laboratory-derived SWCC. Therefore, it is proposed that the SWEC can be described
with a logistic power equation similar to the Brutsaert (1967) equation. Furthermore, it is common practice to normalize the water content between the initial saturated water content and the residual water content. According to Fredlund et al. (2012), residual conditions generally reflect a significant change in the behavior of the soil, and normalized water contents isolate physical behavior between saturated and residual water content conditions.

Given the linear relation between in-situ volumetric water content and electrical conductivity, it can be assumed that electrical conductivity can be normalized in a manner similar to that used for water contents in soil-water characteristic curves. The normalized electrical conductivity is:

$$
EC_N = \frac{EC - EC_r}{EC_s - EC_r}
$$

Eq. (3-2)

where $EC_N$ = normalized electrical conductivity, $EC_r$ = electrical conductivity corresponding to residual volumetric water content, and $EC_s$ = electrical conductivity corresponding to saturated volumetric water content. Incorporating normalized electrical conductivity into the logistic power equation yields the following equation:

$$
EC_N = \frac{1}{1 + \left(\frac{\psi}{b}\right)^c}
$$

Eq. (3-3)

When expanded to the base electrical measurements, it is expressed as:

$$
EC = EC_r + \frac{EC_s - EC_r}{1 + \left(\frac{\psi}{b}\right)^c}
$$

Eq. (3-4)

where $\psi = (u_a - u_w)$ = matric suction (water potential), $EC =$electrical conductivity, $EC_s =$ saturated electrical conductivity, $EC_r =$ residual electrical conductivity, $b =$ fitting parameter that is possibly related to the inflection point, and $c =$ fitting parameter that is possibly related to the degree of curvature.

For this study, the fitting parameters were optimized using the Microsoft Excel Equation Solver. The $EC_r$ values could not be determined from direct optimization. Initial efforts showed that $EC_r$ values determined from direct optimization produced supposed $EC_r$ values that corresponded to water-potential values in the transition zone. Thus, valid $EC_r$ values were determined by first fitting the Fredlund and Xing (1994) SWCC equation through the field data to produce a continuous curve. The Fredlund and Xing (1994) equation is given as:

$$
EC = k \left(\frac{1}{1 + \left(\frac{\psi}{b}\right)^c}\right)
$$

Eq. (3-5)

where $k =$ soil constant; $\psi = (u_a - u_w)$ = matric suction (water potential), $EC =$electrical conductivity, $b =$ fitting parameter that is possibly related to the inflection point, and $c =$ fitting parameter that is possibly related to the degree of curvature.

For this study, the fitting parameters were optimized using the Microsoft Excel Equation Solver. The $EC_r$ values could not be determined from direct optimization. Initial efforts showed that $EC_r$ values determined from direct optimization produced supposed $EC_r$ values that corresponded to water-potential values in the transition zone. Thus, valid $EC_r$ values were determined by first fitting the Fredlund and Xing (1994) SWCC equation through the field data to produce a continuous curve. The Fredlund and Xing (1994) equation is given as:
\[
\theta = 1 - \frac{\ln\left( 1 + \frac{\psi}{\psi_r} \right)}{\ln\left( 1 + \frac{\psi_d}{\psi_r} \right)} \left[ \frac{\theta_s}{\ln\left( e + \left( \frac{\psi}{a} \right)^p \right)} \right]^q \quad \text{Eq. (3-5)}
\]

where \( \theta_s \) is saturated volumetric water content, \( e \) is equivalent to 2.71828, \( a, p, \) and \( q \) are fitting parameters, \( \psi_r \) is residual suction, and \( \psi_d \) is the suction at zero saturation (\( \approx 1,000,000 \) kPa). Fig. 3-8 is an example of using the fitting technique for a location at each landslide; saturation is plotted as a function of water potential. The degree of saturation, \( S \), is \( S = \theta/n \), where \( n \) is porosity. The Fredlund and Xing (1994) equation also forces the SWCC to terminate at 1,000,000 kPa. Thus, the residual suction used for fitting is not necessarily the residual suction for the data. For this study, a residual suction value of 3000 kPa was used for fitting the Fredlund and Xing (1994) equation, as recommended by Fredlund et al., (2012).

Figure 3-8. SWCC showing saturation plotted versus water potential; (a) is upslope at Doe Run and (b) is upslope at Herron Hill.

The degree of saturation corresponding to the residual suction, \( S_r \), for the data was obtained by visual inspection of the continuous field SWCC. For this study, the \( S_r \) values were found to occur at a degree of saturation of approximately 20%. With foreknowledge of the porosity, the residual volumetric water content, \( \theta_r \), was obtained from \( \theta_r = S_r n \). The \( EC_r \) values were then determined using the linear relationship presented in Eq. (3-1).

Fig. 3-9 shows the results of using the SWEC equation (Eq. (3-4)) to model water-potential values from near saturation to dry versus electrical conductivity. All models were consistent, showing continuous water potential values through 100,000 kPa. The modeled curves for each landslide are different, which may be a function of geology,
sensor depth, and number of modeled data points. The deeper sensor locations (2.4 m), both upslope and downslope, produced limited SWEC values because of the lack of wetting and drying values during the monitoring period. Table 3-4 gives the input data used in the SWEC for the upslope and downslope locations at both landslides.

The SWEC curves indicate that the b fitting parameter is related to the water potential at the inflection point and the c fitting parameter is related to the slope of the curve.

Table 3-4. Residual and saturated electrical-conductivity values and fitting parameters for the SWEC model.

<table>
<thead>
<tr>
<th>Landslide</th>
<th>Location and Depth</th>
<th>EC_s (dS/m)</th>
<th>EC_r (dS/m)</th>
<th>b</th>
<th>c</th>
</tr>
</thead>
<tbody>
<tr>
<td>Doe Run</td>
<td>upslope, 30 cm</td>
<td>0.23</td>
<td>0.03</td>
<td>207.9</td>
<td>0.88</td>
</tr>
<tr>
<td></td>
<td>upslope, 70 cm</td>
<td>0.53</td>
<td>0.03</td>
<td>305.4</td>
<td>0.45</td>
</tr>
<tr>
<td></td>
<td>downslope, 55 cm</td>
<td>0.45</td>
<td>0.03</td>
<td>122.9</td>
<td>0.49</td>
</tr>
<tr>
<td></td>
<td>downslope, 1.3 m</td>
<td>0.49</td>
<td>0.03</td>
<td>200.0</td>
<td>0.75</td>
</tr>
<tr>
<td>Herron Hill</td>
<td>upslope, 1 m</td>
<td>0.815</td>
<td>0.1</td>
<td>834.0</td>
<td>1.19</td>
</tr>
<tr>
<td></td>
<td>upslope, 2.4 m</td>
<td>1.22</td>
<td>0.1</td>
<td>5000.0</td>
<td>0.64</td>
</tr>
<tr>
<td></td>
<td>downslope, 1 m</td>
<td>1.62</td>
<td>0.1</td>
<td>407.6</td>
<td>2.22</td>
</tr>
<tr>
<td></td>
<td>downslope, 2.4 m</td>
<td>1.30</td>
<td>0.1</td>
<td>5000.0</td>
<td>0.64</td>
</tr>
</tbody>
</table>
Figure 3-9. SWEC model for electrical conductivity and water potential: (a) upslope at Doe Run at 70 cm, (b) downslope at Doe Run at 55 cm, (c) upslope at Herron Hill at 1 m, and (d) downslope at Herron Hill at 1 m. The black dots are in-situ field measurements.

7. Shear strength models

To expand the application of the hydrologic and EC correlations, this research demonstrated that shear strength can be inferred from EC. Shear-strength behavior of soils is controlled by parameters such as net normal stress, matric suction, porosity, interparticle friction, and cementation (in some cases). Vanapalli et al. (1996) proposed a nonlinear shear-strength equation using a normalization of the SWCC between the saturated and residual soil conditions. The Vanapalli et al. (1996) shear-strength equation is an expanded version of the traditional Mohr-Coulomb failure criterion and is expressed as:

\[
\tau_{ff} = c' + (\sigma - u_a)\tan\phi' + \frac{\theta - \theta_r}{\theta_s - \theta_r}(u_a - u_w)\tan\phi' \quad \text{Eq. (3-6)}
\]

where \(\tau_{ff}\) = shear strength, \(c'\) = cohesion at zero matric suction (water potential) and zero net normal stress (effective cohesion), \((\sigma - u_a)\) = net normal stress, \(\sigma\) = total stress, and \(\phi'\) = angle of internal friction associated with net normal stress; the other parameters were defined previously.
The Vanapalli et al. (1996) shear-strength equation links the shear strength to the SWCC. Therefore, if the SWCC has been adjusted to account for variations in initial void ratio (Gallipoli et al., 2003; Zhou et al., 2012) or wetting-drying cycles (Guan et al., 2010; Ng and Xu, 2012; Han and Vanapalli, 2015), and that factor is inherently included in the shear-strength calculations.

7.1 Shear-strength testing

Shear-strength parameters were determined from standard consolidated undrained (CU) triaxial tests in accordance with ASTM method D4767. The triaxial tests were performed using a GeoTac triaxial system manufactured by Trautwein Soil Testing Equipment Co. of Houston, Texas. Data collection and analysis were done using the TruePath software supplied with the triaxial system. The triaxial tests were conducted on three remolded samples collected from each landslide during the trench excavations. At the Doe Run landslide, samples were taken from the soil-bedrock interface in the upslope location at a depth of 70 cm. At the Herron Hill landslide, samples were taken from the downslope trench at a depth of approximately 1.6 m, at a lithologic boundary suspected of being the failure zone.

The remolded test samples were prepared by static compaction using a hydraulic piston. Soil samples were prepared by passing oven-dried soil through a #40 sieve and thoroughly mixing the soil with water to achieve the appropriate in-situ degree of saturation and porosity, which corresponded to the in-situ $\theta_s$. After compaction, the samples were subsequently sealed in plastic bags and stored for 24 h to establish moisture equilibrium. During the isotropic consolidation phase, the samples were consolidated to levels equal to the in-situ $(\sigma - u_a)$. The samples were then sheared at a rate of approximately 0.02% strain per minute. Table 3-5 summarizes the shear-strength and hydrologic parameters used in the Vanapalli et al. (1996) shear strength equation.

<table>
<thead>
<tr>
<th>Landslide</th>
<th>$c'$ (kPa)</th>
<th>$(\sigma - u_a)$ (kPa)</th>
<th>$\phi'$ (deg)</th>
<th>Location</th>
<th>$\theta_s$</th>
<th>$\theta_r$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Doe Run</td>
<td>9.6</td>
<td>13</td>
<td>22</td>
<td>(upslope 70 cm)</td>
<td>0.44</td>
<td>0.09</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>(downslope 1.3 m)</td>
<td>0.43</td>
<td>0.08</td>
</tr>
<tr>
<td>Herron Hill</td>
<td>1.3</td>
<td>29.9</td>
<td>27</td>
<td>(upslope 1 m)</td>
<td>0.41</td>
<td>0.08</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>(downslope 1 m)</td>
<td>0.44</td>
<td>0.09</td>
</tr>
</tbody>
</table>

Fig. 3-10 shows the in-situ shear-strength data, as interpreted by the Vanapalli et al. (1996) shear-strength equation (Eq. (6)), for both landslides. The shear-strength data correlate with the in-situ water potential values along the drying path. The figure shows that the shear strength curves tend to have inflection points (i.e., bends in the curves) approximately corresponding to the air-entry values of the SWCCs.
Figure 3-10. Shear strength versus water potential at the: (a) Doe Run landslide upslope at a depth of 70 cm, (b) Doe Run landslide downslope at a depth of 1.3 m, (c) Herron Hill landslide upslope at a depth of 1 m, and (d) Herron Hill landslide downslope at a depth of 1 m.

7.2 Shear-strength model development

A predictive shear-strength model was developed by combining the linear hydrologic model and the SWEC model with the Vanapalli et al. (1996) shear-strength equation. Specifically, the volumetric water content values were determined from the electrical EC data using Eq. (3-1). The $\theta_s$ values were set equal to the in-situ porosity, and the $\theta_r$ values were assumed to correspond to an $S_r$ of roughly 20%. The SWEC model (Eq. (3-4)) was rearranged to provide water-potential data at given EC values. The aforementioned data, along with the triaxial shear parameters, were used to predict shear strength at given EC values. This complete model is termed the SWEC strength model. Fig. 3-11 compares the field measurements, as interpreted by the Vanapalli et al. (1996) equation with the SWEC strength model. The modeled values have a linear trend for the shallow, upslope locations for both landslides. The deeper locations show curved patterns, perhaps because of the lower shear-strength values and more fluctuation in hydrologic conditions.
8. Surface electrical resistivity

Spatial electrical measurements were obtained from two-dimensional surface electrical resistivity (ER) surveys conducted at the sites. ER surveys consisted of arrays along two lines at the Doe Run landslide and along three lines at Herron Hill. Each measured array was repeated at the same spot on five different dates, using an Advanced Geosciences Inc. SuperSting eight-channel resistivity meter with 84 electrodes. Measurements were conducted using a dipole-dipole electrode configuration, and Earth Imager 2D software processed the inversions from apparent resistivity to 2D resistivity profiles. Electrode spacing for all profiles was 0.9 m. The dipole-dipole electrode configuration and short spacings were chosen to best address limitations with ER investigations for landslides. Measurement errors usually range between 1% and 5% based on data noise, inversion settings, or instrument functionality. All negative apparent resistivities or any relative data misfit greater than 50% were removed. The inversion settings used included a
Smooth Model Inversion and a finite element forward model. Resolution and error can be improved with several specific inversion techniques. However, inversion profiles constructed in this study imaged failure zones, slope geometry, lithologic boundaries, and changes in moisture conditions, which was sufficient for the project framework.

Five ER surveys were run at each site. The surveys occurred between July 1, 2015, and June 14, 2016. Fig. 3-12 shows examples of inverted resistivity profiles measured in the downslope direction. Cooler colors are low resistivity and hotter colors are high resistivity.

![Image](image-url)

**Figure 3-12.** Examples of inverted resistivity profiles measured in the downslope direction at: (a) the Doe Run landslide (measured July 1, 2015) and (b) the Herron Hill landslide, below the old road (measured on May 19, 2016).

Resistivity variations in rock and soil are primarily caused by moisture content, conductivity of pore fluids, grain size, porosity, permeability, pore-water temperature, and lithology (Lucas et al., 2017). For landslides, these variations may be detections of lateral continuity, slide planes, groundwater concentrations, or clays. Slide planes are often associated with increased water content, which would show up as areas of low resistivity. Advantages of using electrical resistivity over other investigative tools include fast gathering of data, repeatability, and being a nonintrusive way to assess the geologic and hydrogeologic conditions. Measurement times were chosen based on weather and field assistance availability, and were conducted during wet conditions (after a storm, for example) and dry conditions. General ER observations are:

- Each resistivity profile exhibits contrasts interpreted as showing the landslide failure zones, landslide type, areas of excess moisture, changes in lithology, and bedrock depth.
- Resistivity profile interpretations are supported by observations of the...
geomorphology on the surface, the colluvial soil mass described in the sensor trenches, and the in-situ hydrologic sensor data.

- Less is known about the depth to failure for the Herron Hill landslide. A failure zone was interpreted, as opposed to a failure plane that was interpreted along the colluvium-bedrock contact at Doe Run.
- Differences in the inverted profiles and resistivity contrasts over time reflect recent rainfall amounts and the soil moisture conditions in the slope.

To support the conditions interpreted from resistivity measurements over time, difference inversions were created using profiles measured at both landslides. Fig. 3-13 shows a difference inversion for the Doe Run landslide as a percent difference in resistivity reflecting two data sets. The profile shows both increases and decreases of subsurface resistivity over time. The difference inversions reflect measurements on July 1, 2015 and September 2, 2015. The slope received 22 mm of rain two days prior to the July measurement and no rain a week prior to the September measurement, therefore several zones showing an increase in resistivity are interpreted to be less conductive because there was less moisture in the soil. These data correspond to volumetric water content data presented previously in Fig. 3-3 that showed the hydraulic regime progressing along a wetting path in July 2015 and progressing along a drying path in September 2015. By extension of the data presented herein, the difference of the surface ER measurements appear to qualitatively correlate to the strength increase (i.e. progressing along the drying path) or strength decrease (i.e. progressing along the wetting path).

The Herron Hill profile (lower part of the slope), shown in Fig. 3-13, presents a percent difference of resistivity between two measurements, on October 7, 2015 (which followed a week with only trace amounts of rain), and on May 19, 2016 (which followed two days and almost 25 mm of rain). Decreases in resistivity can be seen near the surface, supporting the fact that surface ER is sensitive to recent rainfall. A thin zone approximately 4 m below the surface also shows a decrease in resistivity. The increase in resistivity at the deeper zone coincides with the deeper low resistivity zone in Fig. 3-12, which is potentially a deeper failure zone. The difference inversions reflect changes in moisture conditions within the slope, as well as changes in porosity, temperature, and textural changes, perhaps near the interpreted failure zone. Analyzing these changes, and potentially connecting them with changes in shear strength, support using ER as a tool for slope stability.
Figure 3-13. Percent difference of resistivity for ER measurements at the (a) Doe Run and (b) Herron Hill landslides. The positive percent values represent an increase in subsurface resistivity.

8.1 Electrical response with depth

To investigate the correlation between the surface and subsurface electrical data, the EC data from the CS655 sensors were compared with the inverted surface ER data. The surface ER values were extracted from the inverted profiles from the sensor trenches and converted to EC data by taking the inverse of the ER data. Fig. 3-14 shows the inverted surface EC data along with depth. Although the EC profile produces data to depths of 20 m, only data from the top 5 m are shown. These vertical EC profiles show the fluctuations in moisture in the shallow colluvial soil, and where the fluctuations of water content and water potential are occurring. Additional surface electrical resistivity measurements over time will continue in order to define the hydrologic conditions measured within the slope and potentially identify the conditions that lead to failure.
8.2 Adjustment of surface electrical measurements

In general, comparing measurements from the in-situ sensors with those from the inverted surface ER measurements is challenging for several reasons. The moisture conditions and moisture gradient are dictated by soil type, slope morphology, and depth from the surface. The general functionality of the devices measuring electrical data also influences how the data can be compared. The in-situ hydrologic sensors are making discrete measurements at a point, while the electrical resistivity measurements use an inversion process to model a large volume of the slope. Therefore, in order to make a predictive interpretation about electrical conductivity or resistivity measurements and the hydrologic conditions and shear strength of a slope, a multiplying factor was implemented to correlate the in-situ sensor and surface ER measurements. Fig. 3-15 shows electrical measurements from the surface inversions and in-situ sensors at the same trench location and depth for five different measuring events. The figure also shows the surface inversion measurements adjusted to the subsurface sensor data. Multiplying factors of 1.45 and 0.63 were used to adjust the inversion measurements for the Doe Run and Herron Hill landslides, respectively. These are average values for each trench and depth of sensors; therefore, the adjustment is not universal and applies only to these
specific landslides. In general, an advanced physics-based behavioral model with site-specific assumptions will be needed to generalize this approach. Further research is required to develop a more generalized function.

Figure 3-15. Electrical conductivity from the inverted surface measurements adjusted to the in-situ sensor electrical conductivity measurements: (a) upslope at Doe Run at a depth of 70 cm and (b) downslope at Herron Hill at a depth of 0.9 m.

8.3 Limitations of field hydrogeophysical monitoring for shear strength

Field geophysics will almost always include uncertainty when acquiring information related to mechanical behavior of soils, especially with quantifying the relationships between variations of electrical resistivity and fluctuating hydrologic conditions (Jongmans and Garambois, 2007; Perrone et al., 2014). Unlike laboratory tests that conduct small-scale measurements on discrete samples of soil, field measurements of hillslope soils incorporate large volumes of data, and parameters such as soil type, porosity, pore shape, and grain shape, are highly variable over long-term conditions. Bulk electrical conductivity (or surface resistivity) measured in the field is influenced by water content, soil type, temperature, and correlating these parameters with soil behavior is a challenge (Samouelian et al., 2005; Day-Lewis et al., 2005; and Dumont et al., 2016). Measuring techniques, resolution, and sensitivity of distinguishing in-situ, complex soil properties should be properly gaged to the correlations being made. Resultant data and models should reflect the most realistic spatial variations of the soil (Hermans and Irving, 2017). For surface electrical resistivity, inversion, data noise, and measurement error are all limiting factors of resolution when modeling the inversion ER profiles. Near-surface noise, especially for modeling shallow subsurface features, will have an effect on interpretation and any quantitative correlations made with the data. For electrical resistivity tomography, solutions to noisy data or inversion artifacts can be reduced with multiple array and electrode configurations, comparisons with other geophysical methods, and an improvement on time-lapse ERT techniques. Considering the influence of water within colluvial hillslopes and how to support the static images of
normally modeled ERT, improving inversion methodology of time-lapse ER allows for confidence in interpretations and potentially better image quality (Perrone et al., 2014; Lesparre et al., 2017).

However, the methodology used here to correlate field electrical data with shear strength, acknowledges the hysteresis of water behavior within the slope, as well as the non-uniqueness of the surface resistivity, but still establishes a predictive framework that is applicable to hazard assessment and slope stability studies. The resolution of the electrical data, both in situ electrical conductivity and surface resistivity reflect real difference in the soil conditions over time, and it is the range of data that is important for calculating shear strength in this framework.

9. Spatial shear-strength data

Spatial shear-strength data were then obtained from the surface ER survey by first adjusting the surface ER data using the multiplying factors cited above. The adjusted spatial ER data were then input into the hydrologic models (Eqs. (3-1) and (3-3)) and then into the shear strength model (Eq. (3-5)). It must be cautioned at this point that field ER measurements are highly variable. Thus, shear strength derived from ER data, as presented herein, will be highly variable as well. Quantifying that variability was beyond the scope of this study. Nonetheless, the hydrologic and shear strength models do show the potential for giving insight into spatial distribution of shear strength.

The shear strengths and the corresponding spatial coordinates were used to create spatial shear-strength contour maps. Spatial ER values were taken from the Doe Run inverted profile created for the July 1, 2015 data and from the multiplying factor to estimate subsurface EC. The shear strength model was developed from the upslope at 70 cm data. Fig. 3-16 shows the spatial shear strength for the Doe Run data presented herein normalized to the maximum value. This allows for a qualitative assessment of the spatial shear strength. The contour map was created using the Surfer software produced by Golden Software of Golden, CO. It is envisioned that these data would then be input into a slope stability software package to evaluate factors of safety.

Figure 3-16. Spatial shear-strength profile for Doe Run calculated from hydrologic and field electrical measurements.
10. Conclusions

Subsurface soil-water conditions and electrical properties were monitored for two shallow colluvial landslides in Kentucky. Volumetric water content, water potential, and bulk electrical conductivity were collected from in-situ sensors. SWCCs were plotted from the field volumetric water content and water-potential data. Electrical conductivity and hydrologic parameters were then correlated and modeled. The unsaturated soil parameters were then used to calculate shear strength using an extended Mohr-Coulomb failure criterion, so that shear strength could be correlated to water potential and electrical conductivity over time. This technique allowed shear strength to be inferred from the electrical-conductivity data. Surface electrical resistivity was also measured in order to interpret depth to failure and areas of excess moisture within the landslides. The surface ER was also correlated to the hydrologic data and shear strength, using a simple multiplying factor. Repeated ER surveys over time will show differences in resistivity values that can be correlated to variations in hydrologic conditions and shear strength, demonstrating that ER can be a tool for landslide monitoring and slope-stability assessment.
CHAPTER 4

Using 2-D Electrical Resistivity Imaging For Joint Geophysical and Geotechnical Characterization of Shallow Landslides

1. Introduction

Electrical resistivity (ER) is the product of electrical resistance and a cross-sectional area that measures the potential differences at points below the surface, produced by direct electrical currents, to assess the rock or soil's ability to conduct electricity (Burger et al., 2006; Sirles et al., 2012). Results can be 2-D or 3-D inverted tomographic profile images that model a best fit for the field-measured resistivity. The practical application of ER surveys is to analyze the spatial pattern of subsurface resistivity, interpret features in the subsurface, and address geologic, environmental, and engineering questions. For landslides, many of the factors that influence slope stability, such as hydrologic conditions, bedrock type, soil type, and soil thickness, are also factors that control electrical resistance of rock and soils. Interpretation of electrical resistivity tomography (ERT) profiles has proven effective for landslide investigations by identifying the location of the failure zone, slope morphology (scarps, flanks, toe bulges), contrasts in lithology and soil types, moisture regimes, and changes in moisture over time (Bogoslovsky and Ogilvy, 1977; Palmer and Weisgarber, 1988; McCann and Forster, 1990; Godio and Bottino, 2001; Hack, 2001; Lapenna et al., 2005; Mahmut et al., 2006; Jongmans and Garambois, 2007; Colangelo et al., 2008; Perrone et al., 2008, 2014; de Bari et al., 2011; Travelletti et al., 2012; Van Dam, 2012; Gance et al., 2016; Crawford and Bryson, 2018). Other methods, such as drilling exploration, to obtain information about landslide features and slope conditions are often costly and lengthy geotechnical undertakings.

Field investigations of landslides that attempt to correlate geophysics and geotechnical properties are conducted with a wide range of methodologies and rarely try to use electrical data as a predictor of shear strength. The objective of this study is to (1) analyze ERT data from two landslides by identifying specific landslide features, (2) compare differences in multiple ERT surveys over time, and (3) implement a field-based methodology that uses long-term hydrologic monitoring techniques to establish a baseline framework designed to test non-unique electrical measurements and their capability of highlighting changes in shear strength within a slope.

Measurements were conducted along two shallow landslides in different physiographic parts of Kentucky. Dipole-dipole electrode configurations and different electrode spacing were applied, depending on the slope location and length of the profile measurement. Time-lapse inversions calculated resistivity changes in the slope over different measurement times, providing insight into where resistivity changes indicated fluctuations in water content. Knowledge of slope histories, geology, soils, and observable surficial features helps supports the visualization and interpretation of tomograms, but establishing a framework using ERT measurements that can be linked to geotechnical properties of soil can be a benefit to hazard assessment and mitigation efforts.
2. Geologic settings

2.1 Roberts Bend landslide

The Roberts Bend landslide is located along the South Fork of the Cumberland River in Pulaski County, KY., on the edge of the Appalachian Basin. The landslide is in colluvial soils developed on the Paragon Formation (Mississippian), which primarily consists of clay shale, sandstone, limestone, and minor siltstone. The soft, plastic clay shale dominates the bedrock formation and is bluish to greenish gray with red beds that weather to yellowish, red, and green indurated clay layers. The colluvium is primarily silty clay to clay loam with abundant rock fragments derived from the various bedrock lithologies (Table 4-1). Landslides are common in the Paragon, especially during times of heavy precipitation (Taylor et al., 1975). The slope exhibits translational landslide features upslope and rotational slumping downslope.

The slope ranges from approximately 25 degrees upslope near the ridgetop, to approximately 18 degrees midslope, then becomes steep at the toe with near-vertical cliffs that extend down to the South Fork of the Cumberland River (Fig. 4-1). Several flat topographic benches can be traced along contour and are indicators of changes in bedrock lithology. These benches are bedrock controlled, but also influence the movement of the colluvial landslide deposits. The upper part of the slope exhibits older, more muted landslide features, whereas the lower part of the slope exhibits recent scarps and slumps that create hummocky topography and thick toe bulges. The location of the failure zone for the Roberts Bend landslide may be below the colluvium-bedrock contact and within the weathered clay shale, which makes this landslide a good candidate for electrical resistivity surveying.
Figure 4-1. Hillshade image and geologic map of the Roberts Bend landslide area. The red lines are the locations of electrical resistivity surveys. The x's are soil pits that contain hydrologic sensors. The photo is downslope near the landslide toe and before the near-vertical cliff down to the river. (For interpretation of the references to color in this figure legend, the reader is referred to the web version of this article.)

Table 4-1. Soil logs from pits at the Roberts Bend landslide.

<table>
<thead>
<tr>
<th>Midslope Pit (cm)</th>
<th>Soil Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0–5</td>
<td>Dark brown topsoil, organic</td>
</tr>
<tr>
<td>5–30</td>
<td>reddish brown, silty clay, soft</td>
</tr>
<tr>
<td>30–45</td>
<td>dark red, clayey to silty shale, stiff, few rock fragments</td>
</tr>
<tr>
<td>45–75</td>
<td>brownish gray to red, silty clay shale, mottled, few rock fragments</td>
</tr>
<tr>
<td>75–95</td>
<td>grayish green to brown, silty to sandy clay shale, weathered, abundant rock fragments</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Lower Pit (cm)</td>
<td>Soil Description</td>
</tr>
<tr>
<td>0–13</td>
<td>light to dark brown, silty clay</td>
</tr>
<tr>
<td>13–44</td>
<td>light brown to gray, clayey soil, soft, blocky, few rock fragments</td>
</tr>
<tr>
<td>44–75</td>
<td>light gray to greenish gray clay shale, mottled, sandy streaks</td>
</tr>
</tbody>
</table>
2.2 Herron Hill landslide

The Herron Hill landslide is located in Lewis County of northeastern Kentucky. The slide area is characterized by steep ridges and conical knobs that are erosional remnants of steep slopes of the Appalachian Basin to the east. The Herron Hill landslide occurs in the Estill Shale Member of the Crab Orchard Formation (Silurian), which consists of greenish gray clay shale and interbedded limestone. Above the Estill Shale, in ascending order, are the Bisher Limestone (Silurian) and the Ohio Shale (Devonian). The Bisher Limestone is a thin-bedded limestone that ranges from finely crystalline to coarse-textured and sandy. The Ohio Shale is a fissile carbonaceous shale that weathers easily, commonly forming vertical fractures. Translational slides and slumps cause repeated road failures in the area where the slope has been over steepened during construction (Morris, 1965). An old road that cuts across the landslide was abandoned in the mid-1990s because of repeated landslide damage. Persistent seepage occurs from the slope above the old road. The slope ranges from approximately 16 degrees upslope to approximately 6 degrees at the toe, and several recent small slumps are visible along the slope (Fig. 4-2). The colluvium that develops on the Estill Shale is primarily a weak and poorly drained silty clay loam (Table 4-2). The transition from a thin colluvial cover to weathered clay shale occurs just a few centimeters below the surface, making it difficult to distinguish a colluvium-bedrock contact and interpretation of the landslide failure zone more challenging.

Fig. 4-2. Hillshade image and geologic map of the Herron Hill landslide area. The red lines are the locations of electrical resistivity surveys. The x's are soil pits that contain hydrologic sensors. Photo is a small slump at the toe of the landslide.
Table 4-2. Soil logs from pits at the Herron Hill landslide.

<table>
<thead>
<tr>
<th>Upslope Pit (m)</th>
<th>Soil Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0–0.3</td>
<td>Dark brown topsoil, blocky, organic</td>
</tr>
<tr>
<td>0.3–0.6</td>
<td>Brown, silty clay loam</td>
</tr>
<tr>
<td>0.6–1.2</td>
<td>Brownish green clay shale, soft, mottled with reddish brown clay shale, streaks of sand, few rock fragments</td>
</tr>
<tr>
<td>1.2–2.1</td>
<td>Light blue to greenish gray clay shale</td>
</tr>
<tr>
<td>2.1–2.7</td>
<td>Reddish brown clay shale, soft, no structure</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Downslope, Lower Pit (m)</th>
<th>Soil Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0–0.7</td>
<td>Dark brown topsoil, blocky, organic</td>
</tr>
<tr>
<td>0.7–0.3</td>
<td>Brown silty clay loam, soft, few organics</td>
</tr>
<tr>
<td>0.3–1.2</td>
<td>Brown to gray, silty clay shale, soft, weathered fissile, few rock fragments, sand stringers</td>
</tr>
<tr>
<td>1.2–1.8</td>
<td>Greenish gray to brown, silty clay shale, soft to fissile</td>
</tr>
<tr>
<td>1.8–2.1</td>
<td>Light blue to greenish gray clay shale, hard, moderate structure, thin, sandy stringers</td>
</tr>
<tr>
<td>2.1–2.7</td>
<td>Reddish brown clay shale, hard, blocky texture</td>
</tr>
<tr>
<td>2.7–3.5</td>
<td>Gray to brown, weathered shale, fissile, soft, crumbly</td>
</tr>
</tbody>
</table>

3. Materials and methods

3.1 Electrical resistivity tomography

An Advanced Geosciences, Inc. (AGI) SuperSting 8-channel resistivity meter with 84 electrodes was used to make the measurements. Different electrode spacings were deployed on the ground surface depending on the suspected depth of landslide activity and the length of the profile (Table 4-3). Dipole-dipole electrode configurations were used to acquire optimal high-resolution data for the two landslides. Electrode spacing varied from 0.9 m to 1.52 m (A-spacing in Table 4-3) and the n-value designating the separation between current and potential pairs was kept at 6 electrode spacings to obtain the best signal to noise ratio (Advanced Geosciences Inc., 2008). A combination of factory and user settings were used regarding measurement cycle, maximum error, maximum current, measure time, and cable address setup. In a dipole-dipole array, the potential electrodes and current electrodes function independently, allowing for flexible electrode arrangement (current source and potential sink), which results in generally higher resolution for resolving shallow lateral variations and vertical features (Loke, 2000; Hack, 2001; Lapenna et al., 2005; Schrott and Sass, 2008; Perrone et al., 2014). Short spacing also allows for higher image resolution, optimal for landslides anticipated to have shallow (<10 m) failure zones. Other array configurations, such as the common
Wenner and Schlumberger arrays, have high signal-to-noise ratios and moderate lateral-resolution capability (Stummer et al., 2004).

Earth Imager 2D software was used to invert the measured apparent resistivity in the field and create the 2D resistivity profiles. The inversion method incorporates a finite-element model and Smooth Model Inversion, which is recommended for its stability and robustness for all data types (Advanced Geosciences Inc., 2008). For all tomograms, negative apparent resistivities or any relative data misfit >50% were removed. In addition to the Smooth Model Inversion, time-lapse difference inversions were conducted to detect resistivity changes in the subsurface. Configuring the same type of surveys in the same place over time allows a percent difference to be calculated. The time-lapse percent difference is calculated as the ratio of the difference between a base data file value and a monitor data file, relative to the known background data (Advanced Geosciences Inc., 2008).

Table 4-3. Electrical resistivity survey profiles for each landslide.

<table>
<thead>
<tr>
<th>Landslide</th>
<th>Array</th>
<th>Length (m)</th>
<th>A-Spacing (m)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roberts Bend</td>
<td>RB1</td>
<td>57.9</td>
<td>0.91</td>
<td>Parallel to downslope</td>
</tr>
<tr>
<td></td>
<td>RB2</td>
<td>85.3</td>
<td>1.52</td>
<td>Transverse to downslope</td>
</tr>
<tr>
<td>Herron Hill</td>
<td>HH1</td>
<td>47.5</td>
<td>0.91</td>
<td>Parallel to downslope</td>
</tr>
<tr>
<td></td>
<td>HH2</td>
<td>77.7</td>
<td>1.52</td>
<td>Transverse to downslope</td>
</tr>
</tbody>
</table>

3.2 ERT profile results

Electrical resistivity measurements aided in interpreting landslide type, failure zones, lithologic differences, concentrations of moisture, and changes in moisture conditions. The resistivity tomograms were used to map landslide bodies and gain insight into slide type and depth to bedrock. Field observations and soil-profile logs also supported the interpretations of the inverted images.

3.2.1 RB1–4/7/2017

RB1 spans 57.9 m and is oriented in the downslope direction, beginning at a headscarp and continuing downslope across the hummocky landslide body, terminating at the steep cliff above the river (Fig. 4-3). A small high-resistivity anomaly occurs at the headscarp, and is interpreted to be a resistant sandstone or limestone layer. Below the small high-resistivity zone is a series of arcuate low-resistivity areas interpreted to be a series of rotational slumps. These slumps form hummocky topography and suggest that this is an area of thick colluvium and increased moisture content relative to other parts of the slope. A spring with intermittent flow occurs at the scarp face, suggesting that groundwater is saturating this low-resistivity zone. Farther downslope, a discontinuous high-resistivity anomaly intersects the surface, coinciding with a flat topographic bench and the contact between the Paragon and the Bangor Limestone. Lobes of thick colluvium are sliding onto the bench, resulting in the landslide toe bulge. Below the bench is a continuous, 1- to 2-m-thick, low-
resistivity zone that extends toward the cliff and is interpreted to be thin colluvial soil over the limestone. Interpretation of the failure zone becomes more complex along this part of the slope. Surficial geomorphic features and the thin low-resistivity zone indicate shallow displacement, but the presence of deep fractures in the limestone and possibly increased moisture in a deeper low-resistivity zone may also influence slope movement. The large high-resistivity anomaly occurring near the middle of the profile is interpreted to be a discontinuous bedrock layer, perhaps a sandstone or limestone beds.

3.2.2 RB2–4/7/2017

RB2 is oriented transverse to the downslope direction and spans 85 m across the hummocky surface. Clear resistivity contrasts are visible in the inverted profile (Fig. 4-4). A continuous low-resistivity zone, 3 to 4 m thick, occurs near the surface, and is interpreted to be thick colluvium comprising the hummocky toe of the landslide. Underlying the low-resistivity zone is a thin, discontinuous high-resistivity zone, which is interpreted to be lenses of sandstone or limestone of the Paragon Formation. The gaps in the high-resistivity zones may indicate fractures in the bedrock; several fractures can be observed at the surface, along the toe of the landslide near the cliff. A thick, deeper low-resistivity zone occurs approximately 7 m below the surface. This thicker zone is interpreted to be the clay shale of the Paragon and perhaps an area of increased moisture content.

3.2.3 HH1–10/7/2015

HH1 spans 47.5 m and was measured upslope, above the abandoned road that stretches across the landslide. A medium- to high-resistivity zone approximately 1 m thick
occurs just below the surface (Fig. 4-5). This zone corresponds to a drier, crumbly, silty, clay-loam colluvial layer that lies above clayey shale intervals. A distinct, continuous low-resistivity zone occurs approximately 1 m below the surface. This zone is approximately 1.5 m thick and corresponds to a soft, greenish gray to light blue clay shale that transitions to a reddish brown, soft clay shale and is interpreted to be the translational failure zone. A medium-resistivity zone approximately 3 m thick occurs below the continuous low-resistivity zone and ends abruptly downslope at a large low-resistivity area at the end of the profile. The higher resistivity in this zone may be a result of drier, fissile shale. The discontinuous pattern of deeper high-resistivity zones may represent fractures, which would also explain water seeping out of the slope. A large low-resistivity zone toward the toe is interpreted to be an area that accumulates water and is perhaps related to slope modification during road construction.

Figure 4-5. Electrical resistivity tomography profile of HH1.

3.2.4 HH2–10/7/2015

HH2 spans 77.7 m transverse to the downslope direction, below the abandoned road that stretches across the landslide (Fig. 4-6). A near-surface, discontinuous high-resistivity layer approximately 1 m thick is interpreted to be the slope colluvium. The same low-resistivity zone evident in profile HH2 is also evident in this image profile. The 2-m-thick layer, interpreted to be the failure zone, is the soft, greenish gray to light blue clay shale that transitions to a reddish brown, soft clay shale. A medium- to high-resistivity zone approximately 2.5 m thick occurs below the failure zone and corresponds to a drier, weathered, fissile shale horizon.

Figure 4-6. Electrical resistivity tomography profile of HH2.

To support interpretations of the inverted profile images, plots of electrical resistivity points were constructed from a specific location at the surface down to the bottom of the profile. These plots allowed for detailed comparison of measurements from two different times of the year (Fig. 4-7). At Roberts Bend, ERT values measured on two different dates
were extracted along RB1 at 39 m from the beginning of the profile (see Fig. 4-3). The vertical profiles show the location of the failure zone, and the difference in ERT values on the two dates indicates fluctuations in moisture (Fig. 4-7a). The measurement on April 7, 2017, occurred during a wet time of year, and soil moisture was at a steady near-saturated condition. Approximately 72 mm of rain fell in the two weeks prior to the April measurement; 11.5 mm fell three days prior. The October 12, 2017 measurement was during a dry period, in which rainfall was 48 mm two weeks prior to the measurement, and no rain fell four days prior to the October measurement. The Roberts Bend profile shows change in resistivity values in the failure zone, indicative of drier conditions in October.

The vertical plots of resistivity at Herron Hill, measured on two different dates, were extracted 42 m from the beginning of HH1 (see Fig. 4-5). The curves are similar, showing increases and decreases in resistivity at depth (Fig. 4-7b). The October 7, 2015, measurement occurred during a dry period. Approximately 45 mm of rain occurred two weeks prior to the measurement, and only 6 mm fell in the week prior. No rain fell in the three days prior to the measurement. For the May 19, 2016, measurement, approximately 108 mm of rain occurred two weeks prior, and 52 mm fell in the week prior. Approximately 21 mm of rain fell three days prior to the measurements. Higher resistivity values in October (drier) and lower resistivity values in May (wetter) are present in the soil above the failure zone, but this trend does not continue in or below the interpreted failure zone. The April survey at Robert's Bend and the May survey at Herron Hill occurred <24 h after significant rainfall, and differences in resistivity in the upper 1–2 m is presumably related to increased soil moisture. Other factors influencing the differences in inverted tomograms are the antecedent moisture content, smoothing and data misfit, and forward modeling errors.

3.3 Time-lapse difference inversion

Electrical resistivity surveys conducted on different dates at the same location allowed
for difference inversion modeling that produced time-lapse changes of resistivity within the slope. The number of electrodes used in each array was the same, and the arrays were generally placed with the same orientation and length. Field flags ensured the starting and ending points for each array were in the same location on the slope, but each subsequent electrode position depended on vegetation, ease of stake hammering, and user decisions regarding stake placement. The position of the electrodes as a result of landslide movement was not considered, as the variation of stake positions for each survey based on the field conditions is likely greater than actual slope displacement. A difference inversion was calculated using a base data set measured at Roberts Bend on April 7, 2017, and a single monitored data set measured on October 12, 2017. The result is a comparison of before and after measurements (Fig. 4-8). Large areas of 50 to 100% increase in resistivity are prominent near the surface, primarily extending from mid-slope down to the toe. The increase in subsurface resistivity indicates less moisture in the slope. Upslope near the surface, however, calculated resistivity decreased. This area is interpreted to be the thick section of hummocky colluvium, which stores water for a long period.

A time-lapse difference inversion was also calculated for the Herron Hill profiles (Fig. 4-9). This inverted profile is a percent difference of resistivity between measurements on October 7, 2015 (which followed a week with only trace amounts of rain) and May 19, 2016 (which followed two days with almost 25 mm of rain). Just below the surface is a thin, discontinuous zone calculated as a percent increase in resistivity, which is interpreted to be drier soil above the failure zone. A continuous zone approximately 4 m below the surface had a percent decrease of subsurface resistivity, which is interpreted to be the failure zone and impermeable clay-shale layer shown in Figs. 5 and 6. The difference is interpreted to be an increase in saturated soil following rainfall in May 2016.

The difference inversions partially reflect changes in moisture conditions, but may also reflect changes in porosity, temperature, and textural changes during slope movement. In order to establish consistency with the difference inversions, adjustments were made to which SuperSting data files were used, as well as the percentage of removed data for each difference rerun. Additional time-lapse measurements are needed to image the slope conditions and interpret the differences.

Figure. 4-8. Time-lapse difference inversion for RB1 between surveys on April 7, 2017 and October 12, 2017.
4. Using electrical resistivity to support geotechnical correlations

4.1 Framework overview

Building on the practical applications ERT, a field-based methodology was developed to correlate electrical data and hydrologic soil parameters that assess shear strength of colluvial soils. Sensors that measured volumetric water content and water potential were buried and nested vertically in the upslope face of soil pits on the landslides. Campbell Scientific CS655 water-content reflectometers monitored soil volumetric water content, bulk electrical conductivity, and temperature. Decagon MPS-6 Dielectric water-potential sensors measured soil-water potential (soil suction).

In order to consider the factors that influence a non-unique solution of modeling electrical data, normalized in-situ measurements allowed for the calculation of an effective electrical conductivity parameter (Eq. (4-1)). Some of these factors include sensor specifications, measurement frequency, moisture content, and temperature. The effective electrical conductivity considers these factors together and allows modeling of a smooth function from saturated to residual conditions. Measuring electrical conductivity and water potential allowed for a modified field soil-water characteristic curves (SWCC), termed the soil-water electrical curve (SWEC) to be constructed (Crawford and Bryson, 2018). Traditional SWCC data, which compares water content and water potential, help define the stress state and describes how water moves through pores in the unsaturated zone of the soil mass, and are a fundamental part of assessing shear strength (Fredlund et al., 1995; Vanapalli et al., 1996; Lu and Likos, 2004). The SWEC is described with a logistic power equation similar to the Brutsaert (1967) equation for modeling SWCCs from saturated to dry soil conditions (Eq. (2)). Effective electrical conductivity is similarly calculated using the residual and saturated EC values.

\[
EC_e = \frac{EC - EC_r}{EC_s - EC_r}
\]

Eq. (4-1)
\[
EC_e = \frac{1}{1 + \left( \frac{(u_a - u_w)}{b} \right)^c}
\]
Eq. (4-2)

where \( EC_e \) = effective electrical conductivity, \( EC_r \) = residual electrical conductivity, \( EC_s \) = saturated electrical conductivity, \( (u_a - u_w) \) = water potential, \( b \) and \( c \) = fitting parameters. The fitting parameters were optimized using the Microsoft Excel Equation Solver, and are likely related to inflection points and curvature of the SWEC as well as the soil horizons from which data is collected (Table 4-4). Multiple coefficients are used for Herron Hill because of the depth from which data was acquired. Calculating effective conductivity also allows consideration of the hysteresis effect of wetting and drying of the soil over time. Typically, wetting paths are considered for assessing shear strength and landslide initiation, but drying paths are commonly used in laboratory studies (Han and Vanapalli, 2015; Guan et al., 2010). The in-situ data presented are values from a drying path at both the Herron Hill and Roberts Bend landslides (Fig. 4-10).

Table 4-4. Effective electrical conductivity and SWEC fitting parameters.

<table>
<thead>
<tr>
<th>Landslide</th>
<th>Location and depth</th>
<th>( EC_s ) (dS/m)</th>
<th>( EC_r ) (dS/m)</th>
<th>( b )</th>
<th>( c )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roberts Bend</td>
<td>Downslope 44 cm</td>
<td>0.517</td>
<td>0.03</td>
<td>1433.7</td>
<td>0.33</td>
</tr>
<tr>
<td>Herron Hill</td>
<td>Upslope 1 m</td>
<td>0.815</td>
<td>0.006</td>
<td>834.1</td>
<td>1.29</td>
</tr>
<tr>
<td>Herron Hill</td>
<td>Upslope 2.3 m</td>
<td>0.81</td>
<td>0.01</td>
<td>3000</td>
<td>0.32</td>
</tr>
</tbody>
</table>

In addition, volumetric water content is typically presented as a linear function of electrical conductivity (Cosenza et al., 2006; Perrone et al., 2008; Bryson and Bathe, 2009; Sudha et al., 2009; Kibria and Hossain, 2012). A linear relationship was established between volumetric water content and electrical conductivity, using a linear equation that includes the range of the in-situ field data (Fig. 4-11). Volumetric water content was represented as effective degree of saturation (\( S_e \))
Figure 4-10. Soil-water electrical curve (SWEC) and in-situ data converted to effective electrical conductivity ($EC_e$): (a) Herron Hill upslope at 1 m and (b) Roberts Bend downslope at 44 cm. The points are field measurements.

Figure 4-11. Linear regression models for in-situ effective degree of saturation and effective electrical conductivity: (a) is Herron Hill upslope at 1 m and (b) is Roberts Bend downslope at 44 cm.

and effective electrical conductivity (Eq. (4-3)), which allows corrections to the influencing factors of a non-unique solution, as well as any hysteresis effect (Godt et al., 2009; Lu and Godt, 2013).

\[
S_e = \frac{\theta - \theta_r}{\theta_s - \theta_r} = \alpha_1(\text{EC}_e) + \alpha_2 \quad \text{Eq. (4-3)}
\]

where $S_e =$ effective degree of saturation, $\theta =$ volumetric water content, $\theta_r =$ residual volumetric water content, $\theta_s =$ saturated volumetric water content, $\text{EC}_e =$ effective electrical conductivity, $\alpha_1 =$ the slope of the linear equation, and $\alpha_2 =$ the intercept of the linear equation. Based on the differences in geologic settings and soil types of each landslide, (also reflected in the shape of the SWEC curves), the linear model slope
coefficients are likely related to sensor measurement frequency and soil horizon properties (Table 4-5). These relationships provide insight into how water moves through the soil support the field-based SWEC model (Bordoni et al., 2017; Crawford and Bryson, 2018).

Table 4-5. Saturated and residual volumetric water content and slope intercept values for the linear function correlating $S_e$ and $EC_e$.

<table>
<thead>
<tr>
<th>Landslide</th>
<th>Location and depth</th>
<th>$\theta_s$</th>
<th>$\theta_r$</th>
<th>$\alpha_1$</th>
<th>$\alpha_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roberts Bend</td>
<td>Downslope 44 cm</td>
<td>0.50</td>
<td>0.1</td>
<td>0.663</td>
<td>0.365</td>
</tr>
<tr>
<td>Herron Hill</td>
<td>Upslope 1 m</td>
<td>0.41</td>
<td>0.08</td>
<td>0.451</td>
<td>0.540</td>
</tr>
<tr>
<td>Herron Hill</td>
<td>Upslope 2.3 m</td>
<td>0.46</td>
<td>0.09</td>
<td>0.133</td>
<td>0.804</td>
</tr>
</tbody>
</table>

Once the SWECs and linear volumetric water content relationships are established, several shear strength models are suitable for analyzing hydrologic parameters needed to calculated shear strength of unsaturated soils. Vanapalli et al. (1996) proposed a nonlinear shear-strength equation linking the shear strength to unsaturated soil parameters. Their equation uses volumetric water content and water potential between the saturated and residual soil conditions, the same parameters needed for field-based soil-water characteristic curves. The Vanapalli et al. (1996) shear-strength equation is an expanded version of the traditional Mohr-Coulomb failure criterion (Eq. (4-4)).

$$
\tau_{ff} = c' + (\sigma - u_a) \tan \phi' + \frac{\theta - \theta_r}{\theta_s - \theta_r} (u_a - u_n) \tan \phi' \tag{4-4}
$$

where $\tau_{ff} = \text{shear strength}$, $c' = \text{cohesion at zero matric suction (water potential) and zero net normal stress (effective cohesion)}$, $(\sigma - u_a) = \text{net normal stress}$, $\sigma = \text{total stress}$, $\theta = \text{volumetric water content}$, $\theta_s = \text{saturated volumetric water content}$, $\theta_r = \text{residual volumetric water content}$, and $\phi' = \text{angle of internal friction associated with net normal stress}$. Shear strength parameters used are shown in Table 4-6. A predictive shear-strength model combines the in-situ hydrologic correlations with the modified soil-water characteristic curve for each landslide. Using Eq. (4-4), shear strength was calculated and correlated with electrical conductivity for the range of wetting and drying conditions (Fig. 4-12). In both landslides, the ranges of shear strength are different, but presented in terms of the SWEC model, strength and effective electrical conductivity are adjusted to account for non-uniqueness associated with electrical measurements. The plots show, as electrical conductivity increases (with presumably higher moisture contents) shear strength decreases.
Table 4-6. Shear strength and volumetric water content parameters used in Eq. (4). Shear strength parameters were determined from standard consolidated undrained (CU) triaxial tests in accordance with ASTM method D4767.

<table>
<thead>
<tr>
<th>Landslide</th>
<th>$c'$ (kPa)</th>
<th>$(\sigma - u_a)$ (kPa)</th>
<th>$\phi'$ (deg)</th>
<th>Location</th>
<th>$\theta_s$</th>
<th>$\theta_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roberts Bend</td>
<td>10</td>
<td>8.3</td>
<td>24</td>
<td>(downslope 44 cm)</td>
<td>0.50</td>
<td>0.1</td>
</tr>
<tr>
<td>Herron Hill</td>
<td>1.3</td>
<td>29.9</td>
<td>27</td>
<td>(upslope 1 m)</td>
<td>0.41</td>
<td>0.08</td>
</tr>
</tbody>
</table>

Figure 4-12. Comparison of the SWEC shear strength model and effective electrical conductivity: (a) Herron Hill landslide upslope at 1 m and (b) Roberts Bend landslide downslope at 44 cm.

4.2 Electrical resistivity tomography approach and results

The framework for acquiring shear strength was established using in-situ (field) hydrologic measurements, but can be expanded using ERT. There are challenging limitations of implementing ERT in this framework, primarily related to different measurement devices and techniques. The in-situ sensors (CS655 water-content reflectometers) make bulk measurements at a point using a high-speed oscillator output of a certain frequency, which is based on electromagnetic wave propagation between sensor rods. The wave propagation is dependent upon the dielectric permittivity of the surrounding soil. The electrical resistivity measurements use a direct current to model resistivity in a large volume of the slope. The SuperSting measurements mainly operate in the time-domain, making multiple measurements of injected current. Despite the different techniques, ERT can be used in this framework to correlate electrical measurements and geotechnical properties. The calculated effective electrical conductivity ($EC_e$) fundamentally normalizes the data in order for the SWEC fitting parameters and the linear model intercepts to be considered. Using ERT (with resistivity was converted to conductivity), effective electrical conductivity was calculated. Saturated EC ($EC_s$) was determined as the greatest value, at a particular depth, selected from all survey measurements (Table 4-7). Multiple coefficients were used at Herron Hill to reflect the different soil horizons because in-situ data was collected at a greater depth, compared to Roberts Bend, thus it was easier to delineate the soil horizons and associated properties. The residual EC ($EC_r$) was then calculated using Eq. (4-1), solving for
*ECr*. Water potential and effective saturation (*Se*) were calculated using Eq. (4-2), and shear strength was determined using the expanded Mohr-Coulomb Eq. (4-4). Surface ERT measurements implemented into this methodology, particularly using effective electrical conductivity, show a predicted shear strength and how it varies with ERT values (Fig. 4-13). The correlation of *ECe* and shear strength for both landslides show similar curves of decreasing *ECe* with increasing shear strength. The plot demonstrates the framework of using a range of surface ERT measurements to predict shear strength is valid.

Table 4-7. Saturated and residual electrical values taken from the surface ERT measurements.

<table>
<thead>
<tr>
<th>Landslide</th>
<th>Location and depth</th>
<th><em>ECs</em> (dS/m)</th>
<th><em>ECr</em> (dS/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roberts Bend</td>
<td>Downslope 0–44 cm</td>
<td>0.16</td>
<td>0.01</td>
</tr>
<tr>
<td>Herron Hill</td>
<td>Upslope 0–44 cm</td>
<td>0.54</td>
<td>0.12</td>
</tr>
<tr>
<td>Herron Hill</td>
<td>Upslope 44 cm–2.3 m</td>
<td>0.81</td>
<td>0.02</td>
</tr>
</tbody>
</table>

Figure 4-13. Effective electrical conductivity calculated from a surface ERT survey (ER values were converted to EC) and shear strength at the Herron Hill and Roberts Bend landslides.

To spatially evaluate shear strength using this framework, the ERT from four surveys at Herron Hill and three surveys at Roberts Bend (vertical profiles from surface down to approximate interpreted failure zone) were used to calculate a normalized shear strength, \( \tau_0 / \tau_0 (\text{max}) \) (Fig. 4-14). The plots show correlations between surface ER and shear strength at depth, and identify where there are changes in shear strength within the slope. Generally, the shear strength profile curves are similar for each ERT survey and correlate with resistivity contrasts delineated in the image profiles. At Herron Hill, the pattern of shear strength coincided with the different soil interfaces and location of the failure zone. An increase in shear strength occurs between 0.4 m and 1 m below the surface, which coincides with a medium to high resistivity zone (Figs 4-5 and 4-7) and the brownish-green clay shale with sand streaks and rock fragments. Shear strength then decreases with depth from the top of the failure zone down, which coincides with the
large low-resistivity zone at the end of profile HH1. At Roberts Bend, shear strength is low until an increase at approximately 20 cm below the surface. An increase in shear strength follows until about 40 cm, where a decrease in shear strength begins. The decrease in shear strength coincides with the beginning of the low-resistivity zone shown in the Roberts Bend image profiles (Figs 4-3 and 4-7). Below the interpreted failure zone, approximately 60 cm below the surface, the magnitude of the shear strength values is speculated because of a lack of soil data at that depth.

Figure 4-14. Normalized shear strength at depth calculated from ERT surveys: (a) Herron Hill and (b) Roberts Bend. The deeper dashed line represents the approximate top of the failure zone interpreted from the image profiles, see Figs. 3 and 5.

These results primarily reflect data collected from one location on the slope of each landslide. The framework assumption is that the model coefficients are specific to particular soil horizons and associated geotechnical parameters. Therefore, shear strength is presented as normalized shear strength allowing for an evaluation at depth that shows general similarity in soil behavior. Evaluating specific soil strength associated with model coefficients, additional slope locations, and greater constrain on soil horizons will be conducted in future research.

5. ER and shear strength discussion

Using ER to investigate landslides is limited by resolution of the inversion process, near surface data noise, variable long-term conditions of heterogeneous soils, as well as the functionality of the in-situ hydrologic sensors (Godio and Bottino, 2001; Jongmans and Garambois, 2007; Perrone et al., 2014). Particularly for shallow colluvial landslides, detecting and interpreting resistivity anomalies surrounding a slide plane are often difficult because of the noise signals associated with highly weathered rocks and thin soil on a slope. The time of year (wet versus dry season, for example) that resistivity is measured can also produce varying results, having an effect on inversion modeling, interpretation, and any quantitative correlations made with the data. Assessing ERT results that distinguish between what may be a lithologic change and a concentrated groundwater zone, especially considering any soil behavioral parameters such as plasticity or porosity, is
challenging. For example, landslide failure zones commonly occur within clayey soils or clayey shales, and using ERT to delineate the difference between concentrations of water and clays is difficult. This distinction highlights the importance of using time-lapse resistivity, however, which has become widely used to characterize hydrologic processes in the unsaturated zone (Lesparre et al., 2017). The percent differences calculated in time-lapse inversions have the potential to be associated with differences in shear strength at different moisture conditions or different seasons.

Developing reliable methodologies that can identify hydrologic and electrical relationships in a landslide mass, and then connect them with material property variations in the slope, can provide insight into moisture change over time occurring along the slip plane, or in other parts of the landslide body. Geophysics and geotechnical correlations can be supported by long-term hydrologic monitoring and multiple resistivity surveys that capture the different hillslope conditions. Having a framework to assess shear strength from ERT will add necessary support geophysical and geotechnical investigations.

6. Conclusions

2-D electrical resistivity measurements were conducted on two landslides in Kentucky. Dipole-dipole electrode configurations were used along profiles of varying length, electrode spacings, and slope positions. The modeled inversion profiles were used to interpret landslide failure zones, characteristics of landslide type, lithologic boundaries, soil thickness, and changes in moisture conditions. Time-lapse inversions showed changes in hillslope hydrologic conditions, as surveys were conducted during wet and dry seasons allowing increases and decreases in resistivity to be calculated.

A field-based framework that correlates shear strength and hydrologic parameters was expanded in order to allow shear strength to be calculated from surface ERT, thus creating plots of shear strength throughout the slope. A field-based SWEC model, similar to a traditional SWCC, calculated effective electrical conductivity and an associated water potential. The effective electrical conductivity parameter was calculated in order to consider the non-unique solution of modeling electrical data and general soil differences over time such as the hysteresis effect of wetting and drying. A linear model was established between an effective degree of saturation and \( EC_e \). Both models provided the framework coefficients to calculate shear strength. An unsaturated soils shear strength equation was used to calculate shear strength based on in-situ electrical measurements and ERT, demonstrating a field-based framework to forecast the correlation between electrical data and shear strength. Changes in shear strength were observed at depth in both landslides, indicating landslide failure zones, specific soil horizons, and areas of low resistivity, thus providing a spatial view of shear strength throughout the slope. Implementing this methodology to forecast shear strength of hillslope materials under different hydrologic conditions can serve as a basis for various hazard assessments and landslide mitigation techniques.
CHAPTER 5

Long-term Landslide Monitoring Using Soil-Water Relationships and Electrical Data to Estimate Suction Stress

1. Introduction

Complex spatial and temporal variables control the movement of water through hillslope colluvial soils. Some of the factors that influence soil-moisture fluctuation in shallow colluvial soils are soil type, thickness, porosity, permeability, slope morphology, antecedent moisture conditions, and bedrock geology (Baum et al., 2010; Smith et al., 2014; Sorbino and Nicotera, 2013; Giuseppe et al., 2016). The stability of shallow colluvial landslides is highly influenced by fluctuating water content and stresses in the unsaturated zone (Haneberg, 1991; Godt et al., 2009, 2012; Bittelli et al., 2012; Lu and Godt, 2013; Suradi et al., 2014). The mechanisms for these stresses in the unsaturated zone are often described as stress-state variables that explain overall stress influence on the soil (Lu, 2008; Fredlund et al., 2012). Gravity, pore-water pressure, and temperature are the mechanisms which primarily influence water potential (matric suction). Water potential (pore air pressure minus pore water pressure; i.e., \( u_a - u_w \)) and effective stress are often reduced when rain infiltrates the slope (Godt et al., 2012; Baum et al., 2010; Lu and Godt, 2013; Oh and Lu, 2015). Small perturbations of wetting and drying tip the balance of an equilibrium stress state in landslides and greatly influence the initiation and extent of slope movement (Iverson and George, 2017).

The objective of this study was to monitor long-term hydrologic conditions in an active landslide, establish hydrologic relationships across the slope, and analyze specific parameters that influence how water behaves throughout the soil. Volumetric water content, water potential, electrical conductivity, effective saturation, and suction stress were either directly measured, or derived from measured values across the landslide to establish field soil-water characteristic curves (SWCC) and suction-stress characteristic curves (SSCC). The parameters used in defining these relationships were analyzed along with rainfall and landslide movement data that were collected between October 2015 and March 2018. Building on a comparison of field data and constitutive equations that model soil-water relationships, a new equation that incorporates electrical conductivity as a predictor of suction stress was used. Monitoring active landslides can provide the data sets to assess landslide movement and slope stability, and set up a foundation for future monitoring of field conditions (Reid et al., 2008).

2. Physiographic and Geologic Setting

We investigated the Roberts Bend landslide, near Burnside, Kentucky, along the western edge of the Appalachian Plateau. The study area is on a variably steep forested slope adjacent to a sharp meander in the South Fork of the Cumberland River (Fig. 5-1). The
lowermost part of the slope becomes very steep with near-vertical cliffs that reach down to the river. Elsewhere, the slope ranges from approximately 18° to 25° between midslope and the ridgetop. The local relief between river and the ridgetop is about 145 m. Several flat topographic benches can be traced along contour and are indicators of changes in bedrock lithology. These benches are bedrock controlled, but also influence the movement of the surficial deposits. A U.S. Forest Service gravel road crosses the slope.

The landslide is a complex of shallow and possibly deep-seated landslides of various relative ages. The overall morphology of the landslide complex varies above and below the Forest Service road. Upslope of the road, landslide features are somewhat subdued, except for a prominent headscarp that defines the upper extent of the landslide area. Several, recent, shallow nested landslides exist below the road. The downslope nested landslide has well-defined scarps, distinct flank features, hummocks, and a toe bulge. Rotation in the head of this area is evident from back-tilting of trees and the ground surface. The underlying bedrock consists of light greenish gray to reddish brown clay-shale with interbeds of sandstone, limestone, and minor dolomite and siltstone. Throughout the region, the distinct hummocky topography which forms on the shale is susceptible to landslides, especially when wet (Taylor et al., 1975).
2.1. Soil Descriptions and Classification

Three pits were dug by hand, two above the road and one below, in order to describe the colluvium (Table 5-1). Soil-classification parameters are shown in Table 5-2. Natural gravimetric water contents and Atterberg limits were determined according to ASTM standards (D2216 and D4318, respectively). The Unified Soil Classification System designations were determined according to the Natural Resources Conservation Service soils data for Kentucky.
Table 5-1. Colluvial soil descriptions from the sensor pits.

<table>
<thead>
<tr>
<th>Above road pit, upslope Depth (cm)</th>
<th>Soil description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0–10</td>
<td>Dark brown topsoil, organic</td>
</tr>
<tr>
<td>10–20</td>
<td>Silty clay, brown</td>
</tr>
<tr>
<td>20–45</td>
<td>Silty clay, light brown, few rock fragments</td>
</tr>
<tr>
<td>45–75</td>
<td>Gray to bluish clay-shale, soft, slightly fissile, few rock fragments</td>
</tr>
<tr>
<td>75–95</td>
<td>Red clay-shale, stiff, weathered</td>
</tr>
<tr>
<td>95–100</td>
<td>Weathered shale</td>
</tr>
<tr>
<td>Above road pit, midslope Depth (cm)</td>
<td>Soil description</td>
</tr>
<tr>
<td>0–5</td>
<td>Dark brown topsoil, organic</td>
</tr>
<tr>
<td>5–30</td>
<td>Reddish brown, silty clay, soft</td>
</tr>
<tr>
<td>30–45</td>
<td>Dark red, clayey to silty shale, stiff, few rock fragments</td>
</tr>
<tr>
<td>45–75</td>
<td>Brownish gray to red, silty clay-shale, mottled, few rock fragments</td>
</tr>
<tr>
<td>75–95</td>
<td>Grayish green to brown, silty to sandy clay-shale, weathered, abundant rock fragments</td>
</tr>
<tr>
<td>Below road pit, downslope Depth (cm)</td>
<td>Soil description</td>
</tr>
<tr>
<td>0–13</td>
<td>Light to dark brown, silty clay</td>
</tr>
<tr>
<td>13–44</td>
<td>Light brown to gray, clayey soil, soft, blocky, few rock fragments</td>
</tr>
<tr>
<td>44–75</td>
<td>Light gray to greenish gray clay-shale, mottled, sandy streaks</td>
</tr>
</tbody>
</table>

Table 5-2. Soil properties at selected locations.

<table>
<thead>
<tr>
<th>Location</th>
<th>Depth (cm)</th>
<th>$\omega_n$ (%)</th>
<th>LL (%)</th>
<th>PI (%)</th>
<th>USCS</th>
</tr>
</thead>
<tbody>
<tr>
<td>upslope</td>
<td>70</td>
<td>16</td>
<td>32.5</td>
<td>10</td>
<td>SC</td>
</tr>
<tr>
<td>downslope</td>
<td>44</td>
<td>24</td>
<td>34.3</td>
<td>12</td>
<td>CL-ML</td>
</tr>
</tbody>
</table>

$\omega_n$ = Natural gravimetric water content, LL = liquid limit, PI = plasticity index, USCS = Unified Soil Classification System.

3. Instrumentation and Methods

3.1. Hydrologic Instrumentation

Two types of sensors captured continuous, shallow hydrologic conditions in the landslide. A Campbell Scientific CS655 water-content reflectometer measured soil volumetric water content, bulk electrical conductivity, and temperature. The other type of
sensor was the Decagon MPS-6 dielectric water-potential sensor for measuring soil-water potential and temperature. The hydrologic sensors were installed in each of the soil pits upslope, midslope, and downslope near the toe. The sensors were placed in the undisturbed, upslope face of the exposed soil at various depths depending on identified soil horizons and textural differences. Pits and sensor depths are shown in Table 5-3. Each sensor type was nested vertically, creating a pair of each type at a particular soil horizon. Soil stiffness or large rocks prevented a few pairs from being at the exact same depth. The deeper sensors in the upslope and midslope pits, above the Forest Service road, were placed at what was interpreted to be the colluvium–weathered bedrock contact. The soil pits downslope at the toe did not reach weathered bedrock. Figure 5-2 is a schematic diagram of the landslide pits and instrumentation locations. Rainfall was measured using a Rain Wise Inc. tipping bucket rain gauge and a stand-alone RainLog 2.0 data logger. The logger has a 1-minute resolution, and the rain gauge is calibrated at 0.25 mm/tip.

Table 5-3. Hydrologic sensor locations and depths. VWC = volumetric water content, WP = water potential.

<table>
<thead>
<tr>
<th>Pit location and sensor type</th>
<th>Upper and lower sensor depths (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Above road, upslope VWC</td>
<td>45, 70</td>
</tr>
<tr>
<td>Above road, upslope WP</td>
<td>45, 70</td>
</tr>
<tr>
<td>Above road, midslope VWC</td>
<td>30, 65</td>
</tr>
<tr>
<td>Above road, midslope WP</td>
<td>30, 65</td>
</tr>
<tr>
<td>Below road, VWC</td>
<td>25, 44</td>
</tr>
<tr>
<td>Below road, WP</td>
<td>25, 44</td>
</tr>
</tbody>
</table>

Figure 5-2. Schematic diagram of the Roberts Bend landslide showing locations of soil pits (dark brown triangles), base stations with dataloggers, cable-extension transducer (CET), and inferred failure zones. Photograph is of the upslope soil pit, showing hydrologic sensors.
3.2. Cable-Extension Transducer (CET)

Landslide movement downslope at the toe of the landslide was measured with a cable-extension transducer (CET). Commonly referred to as a wire extensometer (Coe et al., 2003), the CET is a stainless-steel cable that measures absolute linear positions. The cable was attached to a potentiometer that was enclosed in a protective case. The CET output signal was voltage, which was then converted to linear displacement. One end of the CET system was located on what was assumed to be a stable part of the slope, and the cable was stretched from there across the landslide toe bulge, where it was anchored to a pole in the ground (Fig. 5-3). The CET recorded extension and retraction movements.

![CET installation in the landslide toe](image)

Figure 5-3. The cable-extension transducer inserted in the downslope toe bulge of the landslide. A horizontal cable extends from a pole grouted into the toe bulge and is attached to a larger pole grouted into flat, stable ground directly downslope. Photo is looking upslope.

3.3. Data Acquisition

Two data- landslide toe. Data from the hydrologic sensors and CET were acquired using a Campbell Scientific CR1000 collection base stations were installed: one above the Forest Service road and the other near the datalogger and a power supply system (battery, solar panel, and charging regulator). Campbell Scientific PC200W software was used for data collection and compilation. Sensors readings were taken using a 15-second scan
interval, and retrieved data in 15-minute, hourly, and daily average value data tables. Table 5-4 shows selected data parameters collected at the landslide.

Table 5-4. Hydrologic-sensor data-collection parameters.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Sensor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Volumetric water content</td>
<td>m³/m³</td>
<td>CS655</td>
</tr>
<tr>
<td>Electrical conductivity</td>
<td>dS/m</td>
<td>CS655</td>
</tr>
<tr>
<td>Temperature</td>
<td>°C</td>
<td>CS655 and MPS-6</td>
</tr>
<tr>
<td>Water potential</td>
<td>kPa</td>
<td>MPS-6</td>
</tr>
<tr>
<td>Landslide extension</td>
<td>cm</td>
<td>CET</td>
</tr>
</tbody>
</table>

4. Hydrologic Observations and Landslide Movement

4.1 Rainfall

Total annual rainfall for 2016 and 2017 at Roberts Bend was 1,281 and 1,353 mm, respectively. Annual accumulative rainfall the last five years in nearby Whitley City, Kentucky recorded by the Kentucky Mesonet (www.kymesonet.org/summaries.html) are shown in Table 5-5. Generally, these rainfall totals suggest that 2016 was a drier year than normal and 2017 was closer to normal rainfall amounts. This can perhaps explain the overall higher volumetric water contents and the short-lived drying period in the downslope soil location in 2017. The largest one-day rainfall during the monitoring period was 65 mm on July 7, 2016.

Table 5-5. Yearly accumulative precipitation amounts measured at the Kentucky Mesonet climate monitoring station near Roberts Bend.

<table>
<thead>
<tr>
<th>Year</th>
<th>Accumulative Precipitation (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2017</td>
<td>1526</td>
</tr>
<tr>
<td>2016</td>
<td>1176</td>
</tr>
<tr>
<td>2015</td>
<td>1567</td>
</tr>
<tr>
<td>2014</td>
<td>1402</td>
</tr>
<tr>
<td>2013</td>
<td>1584</td>
</tr>
</tbody>
</table>

4.2 Volumetric-Water-Content Response

Volumetric water content and water potential were measured along different parts of the slope and at various depths so that clear seasonal wetting and drying periods could be observed. Seasonal fluctuations in the volumetric water content of the soils, which ranged from 0.14 to 0.52, indicated distinct periods of wetting and drying during the calendar year. In general, drying occurred during the meteorological summer and fall likely in response to increased evapotranspiration (Czikowsky and Fitzjarrald, 2004). The duration and magnitude of drying and wetting paths within the soil were different for each slope.
location and soil depth, suggesting that differences in soil texture, porosity, and permeability contributed to the soil moisture profile. Generally, there was one drying period at the landslide in a calendar year, lasting approximately 3 to 4 months. Other times of the year, the volumetric water content fluctuated only slightly with each rainfall, but generally maintained a level of near-saturated or saturated conditions.

The annual range in volumetric water content of the soil and the response to storm rainfall varied with depth and location on the landslide. Above the Forest Service road (upslope and midslope), volumetric-water-content values ranged from 0.15 upslope during a dry period to 0.45 midslope during near-saturated times (Fig. 5-4). The shallower the sensor, the quicker the response to rainfall and greater magnitude of increase in volumetric water content. The midslope sensors, at both depths, show a gradual wetting period following the drying period, which then reaches the near saturated condition. This wetting period is not evident in the upslope sensors. The magnitude of the drying period in 2017 was less (more rain causing higher volumetric water contents) for all sensor locations compared to 2016.
Figure 5-4. Average daily and cumulative rainfall (a) and volumetric water content (b) for upslope and midslope (above the Forest Service road). Clear patterns of wetting, near-saturated conditions, and drying are evident.

Compared to upslope, higher volumetric water contents were measured downslope, below the Forest Service road, in the hummocky part of the landslide (Fig. 5-5). The magnitudes of wetting and drying were less there than in the upslope locations. The gradual wetting phase is evident in the deeper location (44 cm), but not the shallow location (25 cm). The drying period in the shallow sensors occurs as two intervening periods while the deeper soils remained exceptionally wet. The shallow sensor measured volumetric water content that ranged from 0.16 to 0.39 whereas the volumetric water content at the deeper sensor ranged from only 0.41 to 0.51. For the largest one-day rainfall on July 7, 2016, there was minimal response from the deeper sensor at 44 cm. The soft, clayey soils downslope hold moisture longer and do not allow large increases in moisture compared to the coarser soils above the road. Also, during the near-saturated times, the measurements from the water potential sensors remained around 9 kPa, which is the manufacturer’s stated limit of sensor measurement. The long-term, steady levels of
volumetric water content and water potential support the inference that the sensors were at near-saturated or saturated conditions.

Figure 5-5. Average daily and cumulative rainfall (a) and volumetric water content (b) downslope (below the Forest Service road). Clear patterns of wetting, near-saturated conditions, and drying are evident over the monitoring period.

Figure 5-6 shows the differences in storm response between the pits upslope and downslope during a storm that occurred as soil moisture transitioned from relatively dry to elevated in late 2016. On Nov. 30, 2016, at the end of the dry season, almost 40 mm of rain fell. There was negligible response from the upslope volumetric-water-content sensors and an immediate response from the midslope sensors, and the shallow sensor (30 cm) had a greater magnitude of increase in volumetric water content (Fig. 5-6a). The downslope sensor location response to rainfall during the same period, end of drying to the near-saturated state, was generally more subtle (Fig. 5-6b). The overall increase in volumetric water content in the shallow sensor was greater (25 cm). Subsequent rainfall on Dec. 6, 2016, was 37 mm, which caused an increase in volumetric water content from 0.23 to 0.31 at 25 cm depth, but almost no change in the soil moisture at 44 cm. A sharp
increase in volumetric water content did not occur in the deeper sensor (44 cm) until 25 mm of rainfall on Dec. 12, 2016.

4.3. Landslide Movement

Field observations indicate recent landslide activity downslope (where there are well-defined scarps, flanks, and a hummocky surface), but not upslope where landslide features are more subtle. Generally, increased water potential values only existed for a range of approximately 103 to 123 days during both 2016 and 2017, and thus contributed to stability only seasonally from approximately early August to early December. The total movement of the landslide over the monitored period was approximately 3 cm. A comparison of the cumulative horizontal displacement of the toe of the landslide to rainfall shows seasonal periods of movement with varying average velocity (Fig. 5-7); separated by a period where movement suspended. The CET is limited to a linear position along a horizontal line. The deviations (positive movements and peaks in Fig 5-7) from shortening may result from various causes; ground rotation causing the anchor pole on bulge to rotate backwards, ground rotation that caused the CET pole to rotate forward, ice on the cable, expansive soils related to moisture changes, and thermal changes in cable.

The landslide was active at the start of monitoring in October 2015, and the average velocity gradually decreased through January 2016. Movement suspended in early February, but renewed during a 4-day storm that began on February 15. Movement continued, but with a gradual decrease in average velocity through late April. Subsequently, very little movement occurred, but a short burst of millimeter-scale displacement corresponded, in part, with the second wettest storm during the period of monitoring that began on December 19. In 2017, two periods of movement corresponded
with near saturated conditions between late January and the end of April, when movement suspended. Movement resumed around the middle of June and persisted for slightly over 3 months. The third wettest storm of the monitoring period occurred in early August about midway during this period of movement. No additional movement was detected in the remainder of the year.

Figure 5-7. CET cumulative horizontal displacement (red line) and rainfall. Periods of increased velocity (arrows) mostly correspond with the wettest multi-day storms.

For a closer look, CET movement and volumetric water from Nov. 24, 2016, to Dec. 29, 2016, a range from the drying period to a wetting phase, were plotted (Fig. 5-8). The CET movement showed minor change at the end of the dry period, followed by a sharp increase (~0.4 cm) in cumulative displacement, and then a leveling out that coincided with several rainfalls. The sharp decrease in cumulative displacement (slide advancement) occurred as the soils became wet and trend toward near-saturated conditions.
4.4. Effective Saturation and Suction Stress

The effective degree of saturation and suction stress can be derived from the measurements of volumetric water content and water potential in the landslide. The relationship between effective degree of saturation and water potential is a form of a soil-water characteristic curve. This relationship indicates differences in soil type and how water moves through the soil, and is often used as a predictor of shear strength (Vanapalli et al., 1996; Guan et al., 2010). Field-based SWCCs have been shown to be a good method to analyze hydrologic behavior in heterogeneous soils by supporting observations of wetting and drying; the curves can then be fitted to several models (Fredlund and Xing, 1994; Fredlund et al., 2011; Lu et al, 2014; Bordoni et al, 2017; Crawford and Bryson, 2018). The effective degree of saturation ($S_e$) is a normalized volumetric water content, is unitless, and calculated as:

\[ S_e = \frac{\theta}{\theta_s} \]

Figure 5-8. Volumetric water content and horizontal displacement during the transitions from dry to through a wetting phase toward near-saturated conditions in late 2016. Cumulative movement renews by late December.
\[ S_e = \frac{\theta - \theta_r}{\theta_s - \theta_r} \]  

(5-1)

where \( S_e \) = effective degree of saturation, \( \theta \) = measured volumetric water content, \( \theta_s \) = saturated volumetric water content, and \( \theta_r \) = residual volumetric water content.

Suction stress is the product of effective saturation and water potential, and can vary within the unsaturated zone depending on soil type, moisture conditions, and depth below the surface. As in the colluvial soil, the moisture conditions in the unsaturated zone are anisotropic relative to changes in grain fabric and degree of saturation, thus making moisture condition an important factor to analyze in regard to slope movement (Lu and Likos, 2006). Suction stress (Lu and Godt, 2013; Chen et al., 2017) can be expressed as:

\[ \sigma^s = \frac{\theta - \theta_r}{\theta_s - \theta_r} (u_a - u_w) \]  

(5-2)

where \( \sigma^s \) = suction stress and \( (u_a - u_w) \) = water potential (\( u_a \) = pore air pressure and \( u_w \) = pore water pressure).

As the soil becomes more saturated, suction stress is reduced and can contribute to triggering of landslides (Bittelli et al., 2012). In clayey soils, in which water potential has a large range, suction stress during infiltration could be reduced by as much as 500 kPa (Lu and Godt, 2013). Analyzing suction stress over time and correlating it with rainfall can be a proxy for changes in effective stress in a hillslope soil, during wetting and drying (Lu and Likos, 2004; Lu, 2008; Lu et al., 2010; Lu and Godt, 2013; Dong and Lu, 2017). Increases in rainfall will increase the pore-water pressure in a soil system, thus causing a decrease in effective stress and shear strength. The highest suction stress values occur during dry periods. For the 2016 dry period, maximum suction stress values were similar across the slope: approximately 200 kPa (Fig. 5-9).

For the 2017 dry period, the upslope and midslope suction-stress values were similar to those for 2016, but the downslope values only reached about 30 and 12 kPa at 25 and 44 cm depth, respectively. Recognizing magnitudes of increases and decreases of suction stress allows the entire slope to be used as a comparison of quantitative relationships that are established.
Figure 5-9. Suction stress and rainfall, midslope and downslope, during the 2016 drying period. Rainfall measurements are daily values.

4.5. Suction Stress and Landslide Movement

Once general suction-stress behavior across the slope was determined, suction stress could be correlated with landslide movement. In a manner similar to our examination of suction stress and rainfall, we examined suction stress of the soils, beginning with near-saturated conditions over a drying period (Fig. 5-10). Only downslope soils are shown in Figure 5-10, because that is where the CET is measuring movement. Across the 2016 drying period, the landslide toe advanced during near-saturated times (low suction stress) and movement leveled out as drying occurred (Fig. 5-10). The 2017 data show significant difference in the suction stress correlation with movement as the suction stress in the deeper soil horizon (44 cm) increased minimally during drying.
5. Field Characteristic Curves

5.1. Soil-Water Characteristic Curves

The field SWCCs establish soil-water relationships, and the models used to extend the curves from wet to dry periods are similar to equations that calculate a suction-stress characteristic curve. The field SWCCs also demonstrate that the hysteresis effect must be considered. We acknowledge that an analysis of wetting curves for soils may provide insight into slope conditions that trigger landslides (i.e., positive pore pressures that indicate the increased likelihood of a landslide). However, for this study we used drying path data to analyze hydrologic relationships, compare long-term field conditions to empirical relations, and establish new models for assessing stress-state variables. The wetting curves contain sharp fluctuations and represent a short amount of time compared to drying conditions, which have a wide range of values, exhibit a clear indication of saturation stages, and a clear linear correlation between volumetric water content and water potential representing the primary transition zone (Fig. 5-11a).

In order to use relevant fitting parameters, the field SWCCs were modeled using the van Genuchten (1980) equation for volumetric water content as a function of water potential (Eq. 5-3). The fitting parameters were optimized using Microsoft Excel Equation Solver (Table 5-6). The drying curve data were modeled with Equation 3 (Fig. 5-11b).

\[
\theta = \theta_r + \frac{(\theta_s - \theta_r)}{[1 + (\alpha(u_a - u_w))]^n} \theta_s
\]  

(5-3)

where \(\theta\) = volumetric water content, \(\theta_s\) = saturated volumetric water content, \(\theta_r\) = residual volumetric water content. \(n\) = fitting parameter, \(m\) = fitting parameter, and \(\alpha\) = fitting
parameter. The saturated volumetric water content was determined by the observed consistent values at the end of the near-saturated stage. The residual volumetric water content used was 0.1, determined by the Equation 5-3 fitting curve. Residual volumetric content corresponds to a residual water potential of approximately 3,000 kPa at all slope locations except downslope at 44 cm depth. For the deeper soil location downslope (Fig. 11b), the break in the curve is not as evident as the other SWCCs, and it does not reach residual because of the clayey soil horizon.

Figure 5-11. Soil-water characteristic curve for soil downslope at 44 cm depth (a), demonstrating the hysteresis effect and wetting and drying paths, and (b) the modeled curve using the drying path and Equation 3. Each point is a daily average value.

Table 5-6. SWCC fitting parameters calculated from van Genuchten (1980) for the landslide soils and the derived residual and saturated volumetric water content.

<table>
<thead>
<tr>
<th>Location</th>
<th>α</th>
<th>n</th>
<th>m</th>
<th>θ&lt;sub&gt;r&lt;/sub&gt;</th>
<th>θ&lt;sub&gt;s&lt;/sub&gt;</th>
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</thead>
<tbody>
<tr>
<td>Upslope 70 cm</td>
<td>0.44</td>
<td>5.07</td>
<td>0.03</td>
<td>0.1</td>
<td>0.39</td>
</tr>
<tr>
<td>Midslope 70 cm</td>
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<td>0.1</td>
<td>0.41</td>
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<tr>
<td>Downslope 25 cm</td>
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<td>5.36</td>
<td>0.03</td>
<td>0.1</td>
<td>0.38</td>
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<tr>
<td>Downslope 44 cm</td>
<td>0.40</td>
<td>4.93</td>
<td>0.01</td>
<td>0.1</td>
<td>0.51</td>
</tr>
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</table>

5.2. Suction-Stress Characteristic Curves

Long-term field hydrologic relationships allowed constitutive equations that model suction-stress characteristic curves (SSCC) to be compared. We established field SSCCs along the drying path and then compared the field hydrologic monitoring data with an empirical closed-form equation developed by Lu et al. (2010) that is similar to a van Genuchten (1980) equation that has been used to model SSCCs (Eq. 5-4). Combining Eq. 5-2 and 5-3 yields:
where $\sigma^s$ = suction stress, $(u_a - u_w)$ = water potential, $n$ = fitting parameter, $m$ = fitting parameter, and $\alpha$ = fitting parameter.

SSCCs were plotted using the drying-path data across the landslide. The Lu et al (2010) predictive curves perform well, establishing a model related to the fitting parameters that were proven by the field SWCCs (Fig. 5-12). Generally, as the suction stress decreases, effective saturation increases. The shape of the curves differ for each soil location across the landslide and with soil depth. The upslope and midslope subsurface moisture conditions have a larger range of moisture content than the downslope conditions do. The curves of the upslope and midslope soils suggest that water flow is more dynamic, compared to downslope, with greater ranges of saturation and resulting suction stress.

Range of suction stress is minimal in the downslope locations, in the most active part of the landslide. The suction stress increases gradually as $S_e$ decreases, with a more narrow range of $S_e$ in both downslope sensor locations (25 cm and 44 cm). The deeper location has significantly higher suction stress, ranging from 0.75 to 0.90. The deeper downslope location fluctuates less between wetting and drying. We hypothesize that the light gray, fine-grained, greenish gray clay-shale horizon, with lower permeability and slope morphology that appears to concentrate water toward this part of the slope contributes to the long-term high levels of water content and narrow range of suction stress downslope. The SSCC for the downslope locations shows a gentle slope, suggesting less dynamic changes of saturation, particularly at the deeper location (44 cm) where water is being retained in the soil.
6. Soil-Water Electrical Curve Model

Traditional SWCC and SSCC data help define the stress state and hydraulic regime in the unsaturated zone of the soil mass, and are a fundamental part of assessing shear strength (Lu and Likos, 2004). Crawford and Bryson (2018) established a framework that uses electrical conductivity in similar soil-water relationship constitutive equations as a predictor of shear strength. Normalized electrical conductivity ($EC_e$) is calculated similarly to effective saturation, using the residual and saturated values (Eq. 5-5). This allows construction of a soil-water electrical curve that plots $EC_e$ and water potential, termed an SWEC.

$$EC_e = \frac{EC - EC_r}{EC_s - EC_r}$$  \hspace{1cm} (Eq. 5-5)

where $EC_e$ = effective electrical conductivity, $EC_r$ = residual electrical conductivity, and $EC_s$ = saturated electrical conductivity.

Figure 5-12. Selected suction-stress characteristic curves upslope, midslope, and downslope. Each point is a daily average along the drying curve.
The SWEC is described with a logistic power equation similar to the Brutsaert (1967) equation for modeling SWCCs from saturated to dry soil conditions (Eq. 5-6):

\[
EC_e = \frac{1}{1 + \left( \frac{u_a - u_w}{b} \right)^c}
\]  

(Eq. 5-6)

where \((u_a - u_w) = \text{water potential and} \ b \text{ and} \ c = \text{fitting parameters. The fitting parameters were optimized using the Microsoft Excel Equation Solver, and, similar to the SWCC, are likely related to inflection points in the curve.}

Solving Eq. 5-6 for water potential yields Equation 5-7, thus establishing the use of electrical conductivity as a predictor of suction stress:

\[
(u_a - u_w) = b \left( \frac{1}{EC_e} - 1 \right)^c
\]  

(Eq. 5-7)

This relationship primarily hinges on the linear relationship between \(S_e\) and \(EC_e\) (Eq. 5-8):

\[
S_e = \frac{\theta - \theta_r}{\theta_s - \theta_r} = \alpha_1(CE_e) + \alpha_2
\]  

(Eq. 5-8)

where \(S_e = \text{effective degree of saturation,} \ \theta = \text{volumetric water content,} \ \theta_r = \text{residual volumetric water content,} \ \theta_s = \text{saturated volumetric water content,} \ CE_e = \text{effective electrical conductivity,} \ \alpha_1 = \text{the slope of the linear equation, and} \ \alpha_2 = \text{the intercept of the linear equation. Variables and fitting parameters for the SWEC model are shown in Table 5-7.}

Table 5-7. Electrical variables and fitting parameters calculated from Equations 5-6 and 5-8.

<table>
<thead>
<tr>
<th>Location</th>
<th>(\alpha_1)</th>
<th>(\alpha_2)</th>
<th>(b)</th>
<th>(c)</th>
<th>(EC_r)</th>
<th>(EC_s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upslope 70 cm</td>
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<td>0.066</td>
<td>301.9</td>
<td>0.55</td>
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<td>0.075</td>
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<tr>
<td>Midslope 70 cm</td>
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<td>143.2</td>
<td>0.85</td>
<td>0.01</td>
<td>0.147</td>
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<tr>
<td>Downslope 25 cm</td>
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<td>0.36</td>
<td>0.01</td>
<td>0.075</td>
</tr>
<tr>
<td>Downslope 44 cm</td>
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<td>0.352</td>
<td>1762.2</td>
<td>0.32</td>
<td>0.01</td>
<td>0.52</td>
</tr>
</tbody>
</table>

The components of the SWEC model are shown in plots of daily averages of the drying curves across the slope (Fig. 5-13).
Figure 5-13. The SWEC curves (top row) and the linear relationship of effective degree of saturation as a function of electrical conductivity (bottom row) midslope and downslope. Points are daily averages along a drying curve.

6.1. Using Effective Electrical Conductivity to Predict Suction Stress

The SWEC framework was used to calculate suction stress in terms of electrical conductivity. Substituting Equations 5-7 and 5-8 into Equation 5-2 yields:

$$\sigma^* = \left[ \alpha_1 (EC_e) + \alpha_2 \right] b \left( \frac{1}{EC_e} - 1 \right)^2$$  

(Eq. 5-9)

Suction stress as a function of electrical conductivity shows states of saturation as a traditional SSCC exhibit (Fig. 5-14). In general, steeper curves were developed for downslope measurements compared to the more gently sloped curves for measurements in coarser soils upslope. Lu and Likos (2006) demonstrated that the SSCC can be divided into moisture regimes of transient water behavior, from saturated to dry. The same regimes can be identified by analyzing suction stress using the SWEC model. The
regimes are best expressed and labeled on the midslope location in Figure 5-14. A clear zone of saturation, the interpreted air-entry value (AEV), the transition zone, and a residual drying regime are all identified. The saturated zone exhibits consistent volumetric water content and electrical conductivity values, as interpreted in the near-saturated zone in Figures 5-4 and 5-5. As the soil dries, and suction stress and water potential reach the AEV, capillary interparticle stresses develop. The transition zone is defined by the range of in-situ measurements, with suction stresses ranging from 250 kPa to approximately 7 kPa. The shape of the curve changes in this zone depending on the soil location and soil type. The residual regime includes high values of suction stress and water potential and minor changes in volumetric water content and electrical conductivity. The nonlinear curves can be used for estimating permeability, water storage, and shear-strength functions.

Figure 5-14. Suction stress modeled from electrical-conductivity data upslope, midslope, and downslope. AEV = interpreted air-entry value.

Using the SWEC model to calculate suction stress demonstrates that electrical conductivity measured in the field can be effectively used to correlate with suction stress over time. The greatest changes (decreases) in cumulative movement occur during times of wetting and near-saturated conditions, and increases in suction stress occur as
movement levels out and the soil dries. The predicted curves match the in-situ data (Fig. 5-15a). Spanning the 2016 drying period and after an approximately 0.2 cm decrease in cumulative movement, the suction stress increased to 150 kPa in the shallow soil location (25 cm) and to approximately 300 kPa at the deeper location (44 cm) (Fig. 5-15b). Uncertainty remains with the variables potentially related to the positive movements that are deviations from consistent landslide acceleration (positive displacement from approximately September 26, 2016 to October 21, 2016). However, the predicted suction stress does correspond to times of landslide acceleration and what is interpreted is no or minimal movement. The fluctuations in suction stress are important for deciphering the conditions that may lead to slope movement, and using electrical data is an alternative to traditional hydrologic means of hazard assessment. Although the large range of field-measured water potential and suction stress is in-part due to hysteresis, the SSCC using the SWEC framework proves that constitutive equations are valid for long-term soil monitoring, and are applicable for geotechnical engineering practices and a practical support of laboratory procedures. The electrical conductivity data show the potential of using near-surface geophysics, electrical-resistivity measurements in particular, to support hydrologic correlations and predict suction stress and shear strength (Crawford and Bryson, 2018; Crawford et al., 2018).

Figure 5-15. Comparison of suction stress measured in situ and derived from the SWEC model and Equation 9 (a) and suction stress derived from the SWEC model compared with cumulative displacement across the drying period (b), for the downslope location.

7. Summary

A long-term monitoring project at a landslide in Kentucky measured volumetric water content, water potential, and electrical conductivity at various locations across the slope. Slope location, soil type, and soil depths control the variable magnitudes of these parameters and their response to rainfall. Field-derived soil-water characteristic curves were developed in order to support the calculation of suction stress. We modeled suction stress characteristic curves using a constitutive equation from Lu et al. (2010) that effectively predict long-term, field hydrologic behavior. Suction stress was also correlated with water content, rainfall, and landslide movement.
A new framework was developed to establish the field characteristic curves that use electrical conductivity. Suction stress was compared with effective electrical conductivity by using the van Genuchten (1980) and Brutsaert (1967) SWCC equations, written in terms of effective electrical conductivity. Combining this correlation with the linear relationship between effective saturation and effective electrical conductivity (SWEC model) established a new equation to calculate suction stress in terms of electrical conductivity. Steeper curves were calculated in the downslope soils, and more gently sloped curves were developed for the coarser soils upslope. Moisture regimes commonly identified with traditional SWCCs or SSCCs revealed the transient water behavior from saturated to dry. The same regimes are identified with the analysis of suction stress using the SWEC model.

Establishing the SSCC using the SWEC framework proves that the constitutive equations are valid for long-term soil monitoring and that development of new models using electrical data to predict hydrologic parameters is viable. The practical applications of such correlations include new frameworks from which to assess the soil conditions and geotechnical parameters needed to investigate landslide occurrence.
CHAPTER 6

Conclusions

A long-term monitoring project at three landslides in Kentucky measured volumetric water content, water potential, and electrical conductivity at various locations across the slope. Slope location, soil type, and soil depths control the variable magnitudes of these parameters and their response to rainfall. Field-derived soil-water characteristic curves were developed in order to calculate and model suction stress, and use effective saturation and effective electrical conductivity to construct soil-water electrical curves (SWEC). The SWEC model derives a new equation to predict suction stress and shear strength. Moisture regimes commonly identified with traditional soil-water characteristic curves, or soil suction characteristic curves, indicate the transient water behavior from saturated to dry. The same regimes are identified using the SWEC model. The SWEC model is the foundation for using an extended Mohr-Coulomb failure criterion that incorporates electrical conductivity and surface electrical resistivity to predict shear strength and suction stress.

This study also showed that 2-D surface ERT and borehole electrical resistivity measurements resulted in inverted resistivity sections with distinct contrasts that correlate to landslide soil horizons, failure zone depth, and groundwater conditions. Low-resistivity zones were indicators of high moisture content (along with high clay content) and correlated to the failure surface of the landslide. Time-lapse inversions showed changes in hillslope hydrologic conditions, as surveys were conducted during wet and dry seasons allowing increases and decreases in resistivity to be calculated.

The unsaturated soils shear strength equation was used to calculate shear strength based on in-situ electrical measurements and ERT, demonstrating a field-based framework to forecast the correlation between electrical data and shear strength. Changes in shear strength were observed at depth in all landslides, indicating landslide failure zones, specific soil horizons, and areas of low resistivity, thus providing a spatial view of shear strength throughout the slope. Implementing this methodology to forecast shear strength of hillslope materials under different hydrologic conditions can serve as a basis for various hazard assessments, slope stability studies, and landslide mitigation techniques.
APPENDIX A.

Campbell Scientific Sensor Programs
Campbell Scientific Sensor Programs

Doe Run and Herron Hill landslides

'CR1000

'Revision History:

'Rev2: 4/30/2015, search on "Rev2" to find changes
' Description: added SW12V(1)

'Declare Variables and Units
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Public PTemp_C
Public CS65X(6)
Public CS65X_2(6)
Public CS65X_3(6)
Public CS65X_4(6)
Public SDI12(2)
Public SDI12_2(2)
Public SDI12_3(2)
Public SDI12_4(2)

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Alias CS65X(2)=EC_1
Alias CS65X(3)=T_1
Alias CS65X(4)=P_1
Alias CS65X(5)=PA_1
Alias CS65X(6)=VR_1
Alias CS65X_2(1)=VWC_2
Alias CS65X_2(2)=EC_2
Alias CS65X_2(3)=T_2
Alias CS65X_2(4)=P_2
Alias CS65X_2(5)=PA_2
Alias CS65X_2(6)=VR_2
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Alias CS65X_3(2)=EC_3
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Alias CS65X_4(2)=EC_4
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Alias SDI12_2(1)=WP_2
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Alias SDI12_3(1)=WP_3
Alias SDI12_3(2)=Temp_3
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Alias SDI12_4(2)=Temp_4

Units BattV=Volts
Units PTemp_C=Deg C
Units VWC_1=m^3/m^3
Units EC_1=dS/m
Units T_1=Deg C
Units P_1=unitless
Units PA_1=nSec
Units VR_1=unitless
Units VWC_2=m^3/m^3
Units EC_2=dS/m
Units T_2=Deg C
Units P_2=unitless
Units PA_2=nSec
Units VR_2=unitless
Units VWC_3=m^3/m^3
Units EC_3=dS/m
Units T_3=Deg C
Units P_3=unitless
Units PA_3=nSec
Units VR_3=unitless
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<td>Average(1,P_3,FP2,False)</td>
</tr>
<tr>
<td>Average(1,PA_3,FP2,False)</td>
</tr>
<tr>
<td>Average(1,VR_3,FP2,False)</td>
</tr>
<tr>
<td>Average(1,VWC_4,FP2,False)</td>
</tr>
<tr>
<td>Average(1,EC_4,FP2,False)</td>
</tr>
<tr>
<td>Average(1,T_4,FP2,False)</td>
</tr>
<tr>
<td>Average(1,P_4,FP2,False)</td>
</tr>
<tr>
<td>Average(1,PA_4,FP2,False)</td>
</tr>
<tr>
<td>Average(1,VR_4,FP2,False)</td>
</tr>
<tr>
<td>Average(1,WP_1,FP2,False)</td>
</tr>
<tr>
<td>Average(1,Temp_1,FP2,False)</td>
</tr>
<tr>
<td>Average(1,WP_2,FP2,False)</td>
</tr>
<tr>
<td>Average(1,Temp_2,FP2,False)</td>
</tr>
<tr>
<td>Average(1,WP_3,FP2,False)</td>
</tr>
<tr>
<td>Average(1,Temp_3,FP2,False)</td>
</tr>
<tr>
<td>Average(1,WP_4,FP2,False)</td>
</tr>
<tr>
<td>Average(1,Temp_4,FP2,False)</td>
</tr>
</tbody>
</table>

EndTable

DataTable(Table3,True,-1)
  DataInterval(0,1440,Min,10)
  Average(1,VWC_1,FP2,False)
  Average(1,EC_1,FP2,False)
  Average(1,T_1,FP2,False)
  Average(1,P_1,FP2,False)
  Average(1,PA_1,FP2,False)
  Average(1,VR_1,FP2,False)
  Average(1,VWC_2,FP2,False)
  Average(1,EC_2,FP2,False)
  Average(1,T_2,FP2,False)
  Average(1,P_2,FP2,False)
  Average(1,PA_2,FP2,False)
  Average(1,VR_2,FP2,False)
  Average(1,VWC_3,FP2,False)
  Average(1,EC_3,FP2,False)
  Average(1,T_3,FP2,False)
  Average(1,P_3,FP2,False)
Average(1, PA_3, FP2, False)
Average(1, VR_3, FP2, False)
Average(1, VWC_4, FP2, False)
Average(1, EC_4, FP2, False)
Average(1, T_4, FP2, False)
Average(1, P_4, FP2, False)
Average(1, PA_4, FP2, False)
Average(1, VR_4, FP2, False)
Average(1, WP_1, FP2, False)
Average(1, Temp_1, FP2, False)
Average(1, WP_2, FP2, False)
Average(1, Temp_2, FP2, False)
Average(1, WP_3, FP2, False)
Average(1, Temp_3, FP2, False)
Average(1, WP_4, FP2, False)
Average(1, Temp_4, FP2, False)
Maximum(1, VWC_1, FP2, False, False)
Maximum(1, EC_1, FP2, False, False)
Maximum(1, T_1, FP2, False, False)
Maximum(1, P_1, FP2, False, False)
Maximum(1, PA_1, FP2, False, False)
Maximum(1, VR_1, FP2, False, False)
Maximum(1, VWC_2, FP2, False, False)
Maximum(1, EC_2, FP2, False, False)
Maximum(1, T_2, FP2, False, False)
Maximum(1, P_2, FP2, False, False)
Maximum(1, PA_2, FP2, False, False)
Maximum(1, VR_2, FP2, False, False)
Maximum(1, VWC_3, FP2, False, False)
Maximum(1, EC_3, FP2, False, False)
Maximum(1, T_3, FP2, False, False)
Maximum(1, P_3, FP2, False, False)
Maximum(1, PA_3, FP2, False, False)
Maximum(1, VR_3, FP2, False, False)
Maximum(1, VWC_4, FP2, False, False)
Maximum(1, EC_4, FP2, False, False)
Maximum(1, T_4, FP2, False, False)
Maximum(1, P_4, FP2, False, False)
Maximum(1, PA_4, FP2, False, False)
Maximum(1, VR_4, FP2, False, False)
Maximum(1, WP_1, FP2, False, False)
Maximum(1, Temp_1, FP2, False, False)
Maximum(1, WP_2, FP2, False, False)
Maximum(1, Temp_2, FP2, False, False)
Maximum(1, WP_3, FP2, False, False)
Maximum(1, Temp_3, FP2, False, False)
| Minimum(1,VWC_1,FP2,False,False) |
| Minimum(1,EC_1,FP2,False,False) |
| Minimum(1,T_1,FP2,False,False) |
| Minimum(1,P_1,FP2,False,False) |
| Minimum(1,PA_1,FP2,False,False) |
| Minimum(1,VR_1,FP2,False,False) |
| Minimum(1,VWC_2,FP2,False,False) |
| Minimum(1,EC_2,FP2,False,False) |
| Minimum(1,T_2,FP2,False,False) |
| Minimum(1,P_2,FP2,False,False) |
| Minimum(1,PA_2,FP2,False,False) |
| Minimum(1,VR_2,FP2,False,False) |
| Minimum(1,VWC_3,FP2,False,False) |
| Minimum(1,EC_3,FP2,False,False) |
| Minimum(1,T_3,FP2,False,False) |
| Minimum(1,P_3,FP2,False,False) |
| Minimum(1,PA_3,FP2,False,False) |
| Minimum(1,VR_3,FP2,False,False) |
| Minimum(1,VWC_4,FP2,False,False) |
| Minimum(1,EC_4,FP2,False,False) |
| Minimum(1,T_4,FP2,False,False) |
| Minimum(1,P_4,FP2,False,False) |
| Minimum(1,PA_4,FP2,False,False) |
| Minimum(1,VR_4,FP2,False,False) |
| Minimum(1,WP_1,FP2,False,False) |
| Minimum(1,Temp_1,FP2,False,False) |
| Minimum(1,WP_2,FP2,False,False) |
| Minimum(1,Temp_2,FP2,False,False) |
| Minimum(1,WP_3,FP2,False,False) |
| Minimum(1,Temp_3,FP2,False,False) |
| Minimum(1,WP_4,FP2,False,False) |

EndTable

'Main Program
BeginProg
'Main Scan
Scan(15,Sec,1,0)

SW12 (1 ) ’ Rev2

'Default Datalogger Battery Voltage measurement 'BattV'
Battery(BattV)
'Default Wiring Panel Temperature measurement 'PTemp_C'
PanelTemp(PTemp_C,_60Hz)
'CS650/655 Water Content Reflectometer measurements 'VWC_1',
'EC_1', and 'T_1'
   SDI12Recorder(CS65X(),1,"0","M3!",1,0)
'CS650/655 Water Content Reflectometer measurements 'VWC_2',
'EC_2', and 'T_2'
   SDI12Recorder(CS65X_2(),3,"0","M3!",1,0)
'CS650/655 Water Content Reflectometer measurements 'VWC_3',
'EC_3', and 'T_3'
   SDI12Recorder(CS65X_3(),5,"0","M3!",1,0)
'CS650/655 Water Content Reflectometer measurements 'VWC_4',
'EC_4', and 'T_4'
   SDI12Recorder(CS65X_4(),7,"0","M3!",1,0)
'Reset Generic SDI-12 Sensor measurements
Move(SDI12(),2,NaN,1)
'Generic SDI-12 Sensor measurements 'WP_1', and 'Temp_1'
   SDI12Recorder(SDI12(),1,"1","M!",1,0)
'Reset Generic SDI-12 Sensor measurements
Move(SDI12_2(),2,NaN,1)
'Generic SDI-12 Sensor measurements 'WP_2', and 'Temp_2'
   SDI12Recorder(SDI12_2(),3,"2","M!",1,0)
'Reset Generic SDI-12 Sensor measurements
Move(SDI12_3(),2,NaN,1)
'Generic SDI-12 Sensor measurements 'WP_3', and 'Temp_3'
   SDI12Recorder(SDI12_3(),5,"3","M!",1,0)
'Reset Generic SDI-12 Sensor measurements
Move(SDI12_4(),2,NaN,1)
'Generic SDI-12 Sensor measurements 'WP_4', and 'Temp_4'
   SDI12Recorder(SDI12_4(),7,"4","M!",1,0)
'Call Data Tables and Store Data
CallTable Table1
CallTable Table2
CallTable Table3
   NextScan
EndProg

Roberts Bend landslide

'CR1000
'Created by Short Cut (3.1)
'CET programming added for SN H2507762A
'Program to read 2 Decagon MPS-6s based on Chris Chambers code

'Declare Variables and Units
Public BattV
Public PTemp_C
Public CS65X(6) 'rev3, changed array from 3 to 6
Public CS65X_2(6) 'rev3, changed array from 3 to 6
Public extens
Public SensorOut(2)
Public SensorOut_2(2)

Alias CS65X(1)=VWC
Alias CS65X(2)=EC
Alias CS65X(3)=T
Alias CS65X_2(1)=VWC_2
Alias CS65X_2(2)=EC_2
Alias CS65X_2(3)=T_2
Alias SensorOut(1) = Tension
Alias SensorOut(2) = Temp
Alias SensorOut_2(1) = Tension_2
Alias SensorOut_2(2) = Temp_2
Units BattV=Volts
Units PTemp_C=Deg C
Units VWC=m^3/m^3
Units EC=dS/m
Units T=Deg C
Units VWC_2=m^3/m^3
Units EC_2=dS/m
Units T_2=Deg C
Units extens=cm
Units Temp = Deg_C
Units Tension = kPa
Units Temp_2 = Deg_C
Units Tension_2 = kPa

'Define Data Tables

DataTable(Table15,True,-1)' Rev1, changed to M4
DataInterval(0,15,Min,10)

Average(1,VWC,FP2,False)
Average(1,EC,FP2,False)
Average(1,T,FP2,False)

Average(1,VWC_2,FP2,False)
Average(1,EC_2,FP2,False)
Average(1,T_2,FP2,False)

Average (1,Tension,FP2,False)
Average (1,Temp,FP2,False)
Average (1,Tension_2,FP2,False)
Average (1,Temp_2,FP2,False)
Average(1,extens,ieee4,False)
Average(1,PTemp_C,FP2,False)
Average (1,BattV,FP2,False)

EndTable

DataTable(Table60,True,-1) ' Rev1, changed to M4
DataInterval(0,60,Min,10)
Average(1,VWC,FP2,False)
Average(1,EC,FP2,False)
Average(1,T,FP2,False)
Average(1,VWC_2,FP2,False)
Average(1,EC_2,FP2,False)
Average(1,T_2,FP2,False)
Average (1,Tension,FP2,False)
Average (1,Temp,FP2,False)
Average (1,Tension_2,FP2,False)
Average (1,Temp_2,FP2,False)
Average(1,extens,ieee4,False)
Average(1,PTemp_C,FP2,False)
Average (1,BattV,FP2,False)

EndTable

DataTable(Table24,True,-1) ' Rev1, changed to M4
DataInterval(0,1440,Min,10)
Average(1,VWC,FP2,False)
Average(1,EC,FP2,False)
Average(1,T,FP2,False)
Average(1,VWC_2,FP2,False)
Average(1,EC_2,FP2,False)
Average(1,T_2,FP2,False)
Average (1,Tension,FP2,False)
Average (1,Temp,FP2,False)
Average (1, Tension_2, FP2, False)
Average (1, Temp_2, FP2, False)

Average(1, extens,ieee4, False)

Average(1, PTemp_C, FP2, False)
Average (1, BattV, FP2, False)

EndTable

'Main Program
SequentialMode

BeginProg

'Main Scan
Scan(15, Sec, 1, 0)

'Default Datalogger Battery Voltage measurement 'BattV'
Battery(BattV)

'Default Wiring Panel Temperature measurement 'PTemp_C'
PanelTemp(PTemp_C, _60Hz)

'CS650/655 Water Content Reflectometer measurements 'VWC', 'EC', and 'T'
SDI12Recorder(CS65X(), 7, "1", "M3!", 1, 0) ' Rev1, changed to M4 'Rev3 changed to M3

'CS650/655 Water Content Reflectometer measurements 'VWC_2', 'EC_2', and 'T_2'
SDI12Recorder(CS65X_2(), 7, "2", "M3!", 1, 0) ' Rev1, changed to M4 'Rev3 changed to M3

'CET SN H2507762A
ExciteV (1,2500,0)
VoltSe(extens,1,mV2500,8,True,0, _60Hz,,06496,0)

'MPS-6 code for sensor 5
PortSet (9,1)

'Delay for at least 250 mSec for sensor to enter SDI-12 mode.
Delay (0,1,Sec)

'Query sensor for 2 SDI-12 outputs. Default address for all Decagon Digital sensors is 0.
SDI12Recorder (SensorOut(),1,0,"M!",1.0,0)

'Turn SW12V off
PortSet (9,0)

'MPS-6 code for sensor 6
PortSet (9,1)

'Delay for at least 250 mSec for sensor to enter SDI-12 mode.
Delay (0,1,Sec)
'Query sensor for 2 SDI-12 outputs. Default address for all Decagon Digital sensors is 0.
   SDI12Recorder (SensorOut_2(),3,0,"M!",1.0,0)
   'Turn SW12V off
   PortSet (9,0)

'Call Data Tables and Store Data

   CallTable Table15 ' Rev1, changed to M4
   CallTable Table60 ' Rev1, changed to M4
   CallTable Table24 ' Rev1, changed to M4

   NextScan
   EndProg
APPENDIX B.

Sensor Installation Guidelines
Sensor Installation Guidelines

Manuals and the PC200W software are available from the ‘Resource CD’ that ships with the order, or from the CSI website:
https://www.campbellsco.com/documents

Please refer to the product manuals for details.

Installation procedures:

1. Change SDI-12 addresses in the (4) MPS6 probes to 1, 2, 3, and 4.

2. Mount the ENC 12/14 enclosure to a user-supplier vertical pipe.

3. PS150 Power Supply:

   Attach the cable from the battery inside the PS150 power supply to the mating connector located above the ‘Charge’ terminals.
   Toggle the power switch to ‘ON’, and make sure the red LED turns on.

4. Wire the sensors to the CR1000 as shown below:

PC200W Software:

1. Install the PC200W software on the laptop computer (available from the Resource CD or CSI website).

   Connect to the CR1000 with the PC200W software using PN 17394 ‘USB to serial adaptor’ connected to the RS232 port on the CR1000.

   A test program was loaded in the CR1000 at the CSI factory that measures the (4) CS655 and (4) MPS6 probes every 15 seconds and stores data in (3) data tables.

2. When PC200W is first started, the EZSetup Wizard is launched. Click the Next button and follow the prompts to select the CR1000, the COM port on the computer that will be used for communications, 115200 baud, and Pakbus Address 1. When prompted with the option to “Test Communications” click the Finish button.

3. Program the CR1000 (an example program was loaded into the CR1000 at the factory)

   Click the CONNECT button to establish communications.
   Once connected, click SET CLOCK button.
   Click the SEND PROGRAM button and send the program file Crawford_Rev1.cr1.

4. Monitor sensors real-time
Click the CONNECT button to establish communications. 
Click the MONITOR VALUES tab. Variables from the Public Table are displayed by default, verify that the sensor measurements are reasonable.

5. Collect data

Click on the COLLECT DATA tab. Select “Table1”, “new data, append to file”, and click the COLLECT button. Do the same for Table2 and Table3 data tables.

6. View / Graph (1 or 2 columns) data

Click on the VIEW button.
Select FILE|OPEN and select the filename, e.g. C:\PC200W\CR1000Series_Table1.dat.

Options are accessed by using the menus or by selecting the toolbar icons. If you move and hold the mouse over a toolbar icon for a few seconds, a brief description of that icon's function will appear.

Click the Expand Tabs icon to display the data in columns with column headings. Change the Tab Width to 14. To graph a column of data, click on a column to select it, then click the Show Graph (1 Y axis) icon on the toolbar. Use the Show Graph (2 Y axis) to graph two columns of data.

Sensor Wiring:

CS650/655 Water Content Reflectometer (VWC, EC, T, P, PA, and VR) (1)
SDI-12 address 0 (default)

G: Black
G: Clear
G: Orange
12V: Red
C1: Green

CS650/655 Water Content Reflectometer (VWC, EC, T, P, PA, and VR) (2)
SDI-12 address 0 (default)

G: Black
G: Clear
G: Orange
12V: Red
C3: Green

CS650/655 Water Content Reflectometer (VWC, EC, T, P, PA, and VR) (3)

SDI-12 address 0 (default)

G: Black
G: Clear
G: Orange
12V: Red
C5: Green

CS650/655 Water Content Reflectometer (VWC, EC, T, P, PA, and VR) (4)

SDI-12 address 0 (default)

G: Black
G: Clear
G: Orange
12V: Red
C7: Green

SDI-12 Sensor (1) - Note: Set SDI-12 address to ‘1’
G: Ground
12V: Power
C1: Data Line

SDI-12 Sensor (2) - Note: Set SDI-12 address to ‘2’
G: Ground
12V: Power
C3: Data Line

SDI-12 Sensor (3) - Note: Set SDI-12 address to ‘3’
G: Ground
12V: Power
C5: Data Line

SDI-12 Sensor (4) - Note: Set SDI-12 address to ‘4’
G: Ground
12V: Power
C7: Data Line

Measurement Labels (as displayed for the Public Variables in PC200 ‘Monitor Data’ tab):

Default Measurements
BattV
PTemp_C

CS650/655 Water Content Reflectometer (VWC, EC, T, P, PA, and VR) (1)
VWC_1
EC_1
T_1
P_1
PA_1
VR_1

CS650/655 Water Content Reflectometer (VWC, EC, T, P, PA, and VR) (2)
VWC_2
EC_2
T_2
P_2
PA_2
VR_2

CS650/655 Water Content Reflectometer (VWC, EC, T, P, PA, and VR) (3)
VWC_3
EC_3
T_3
P_3
PA_3
VR_3

CS650/655 Water Content Reflectometer (VWC, EC, T, P, PA, and VR) (4)
VWC_4
EC_4
T_4
P_4
PA_4
VR_4

SDI-12 Sensor (1)
WP_1
Temp_1

SDI-12 Sensor (2)
WP_2
Temp_2

SDI-12 Sensor (3)
WP_3
Temp_3
SDI-12 Sensor (4)
  WP_4
  Temp_4

Data Table Descriptions:

Table1:

Interval: 15 MIN
Fields:
  BattV  Units: Volts
  PTemp_C  Units: Deg C
  VWC_1  Units: m^3/m^3
  EC_1  Units: dS/m
  T_1  Units: Deg C
  P_1  Units: unitless
  PA_1  Units: nSec
  VR_1  Units: unitless
  VWC_2  Units: m^3/m^3
  EC_2  Units: dS/m
  T_2  Units: Deg C
  P_2  Units: unitless
  PA_2  Units: nSec
  VR_2  Units: unitless
  VWC_3  Units: m^3/m^3
  EC_3  Units: dS/m
  T_3  Units: Deg C
  P_3  Units: unitless
  PA_3  Units: nSec
  VR_3  Units: unitless
  VWC_4  Units: m^3/m^3
  EC_4  Units: dS/m
  T_4  Units: Deg C
  P_4  Units: unitless
  PA_4  Units: nSec
  VR_4  Units: unitless
  WP_1  Units: KPa
  Temp_1  Units: C
  WP_2  Units: KPa
  Temp_2  Units: C
  WP_3  Units: KPa
  Temp_3  Units: C
  WP_4  Units: KPa
  Temp_4  Units: C
**Data Table 2:**

Interval: 60 MIN  
Fields:
- BattV_Min Units: Volts  
- VWC_1_Avg Units: m³/m³  
- EC_1_Avg Units: dS/m  
- T_1_Avg Units: Deg C  
- P_1_Avg Units: unitless  
- PA_1_Avg Units: nSec  
- VR_1_Avg Units: unitless  
- VWC_2_Avg Units: m³/m³  
- EC_2_Avg Units: dS/m  
- T_2_Avg Units: Deg C  
- P_2_Avg Units: unitless  
- PA_2_Avg Units: nSec  
- VR_2_Avg Units: unitless  
- VWC_3_Avg Units: m³/m³  
- EC_3_Avg Units: dS/m  
- T_3_Avg Units: Deg C  
- P_3_Avg Units: unitless  
- PA_3_Avg Units: nSec  
- VR_3_Avg Units: unitless  
- VWC_4_Avg Units: m³/m³  
- EC_4_Avg Units: dS/m  
- T_4_Avg Units: Deg C  
- P_4_Avg Units: unitless  
- PA_4_Avg Units: nSec  
- VR_4_Avg Units: unitless  
- WP_1_Avg Units: KPa  
- Temp_1_Avg Units: C  
- WP_2_Avg Units: KPa  
- Temp_2_Avg Units: C  
- WP_3_Avg Units: KPa  
- Temp_3_Avg Units: C  
- WP_4_Avg Units: KPa  
- Temp_4_Avg Units: C

**Data Table 3:**

Interval: 1440 MIN  
Fields:
- VWC_1_Avg Units: m³/m³  
- EC_1_Avg Units: dS/m  
- T_1_Avg Units: Deg C  
- P_1_Avg Units: unitless  
- PA_1_Avg Units: nSec
VR_1_Avg Units: unitless
VWC_2_Avg Units: m^3/m^3
EC_2_Avg Units: dS/m
T_2_Avg Units: Deg C
P_2_Avg Units: unitless
PA_2_Avg Units: nSec
VR_2_Avg Units: unitless
VWC_3_Avg Units: m^3/m^3
EC_3_Avg Units: dS/m
T_3_Avg Units: Deg C
P_3_Avg Units: unitless
PA_3_Avg Units: nSec
VR_3_Avg Units: unitless
VWC_4_Avg Units: m^3/m^3
EC_4_Avg Units: dS/m
T_4_Avg Units: Deg C
P_4_Avg Units: unitless
PA_4_Avg Units: nSec
VR_4_Avg Units: unitless
WP_1_Avg Units: KPa
Temp_1_Avg Units: C
WP_2_Avg Units: KPa
Temp_2_Avg Units: C
WP_3_Avg Units: KPa
Temp_3_Avg Units: C
WP_4_Avg Units: KPa
Temp_4_Avg Units: C
VWC_1_Max Units: m^3/m^3
EC_1_Max Units: dS/m
T_1_Max Units: Deg C
P_1_Max Units: unitless
PA_1_Max Units: nSec
VR_1_Max Units: unitless
VWC_2_Max Units: m^3/m^3
EC_2_Max Units: dS/m
T_2_Max Units: Deg C
P_2_Max Units: unitless
PA_2_Max Units: nSec
VR_2_Max Units: unitless
VWC_3_Max Units: m^3/m^3
EC_3_Max Units: dS/m
T_3_Max Units: Deg C
P_3_Max Units: unitless
PA_3_Max Units: nSec
VR_3_Max Units: unitless
VWC_4_Max Units: m^3/m^3
EC_4 Max Units: dS/m
T_4 Max Units: Deg C
P_4 Max Units: unitless
PA_4 Max Units: nSec
VR_4 Max Units: unitless
WP_1 Max Units: KPa
Temp_1 Max Units: C
WP_2 Max Units: KPa
Temp_2 Max Units: C
WP_3 Max Units: KPa
Temp_3 Max Units: C
VWC_1 Min Units: m^3/m^3
EC_1 Min Units: dS/m
T_1 Min Units: Deg C
P_1 Min Units: unitless
PA_1 Min Units: nSec
VR_1 Min Units: unitless
VWC_2 Min Units: m^3/m^3
EC_2 Min Units: dS/m
T_2 Min Units: Deg C
P_2 Min Units: unitless
PA_2 Min Units: nSec
VR_2 Min Units: unitless
VWC_3 Min Units: m^3/m^3
EC_3 Min Units: dS/m
T_3 Min Units: Deg C
P_3 Min Units: unitless
PA_3 Min Units: nSec
VR_3 Min Units: unitless
VWC_4 Min Units: m^3/m^3
EC_4 Min Units: dS/m
T_4 Min Units: Deg C
P_4 Min Units: unitless
PA_4 Min Units: nSec
VR_4 Min Units: unitless
WP_1 Min Units: KPa
Temp_1 Min Units: C
WP_2 Min Units: KPa
Temp_2 Min Units: C
WP_3 Min Units: KPa
Temp_3 Min Units: C
WP_4 Min Units: KPa
Temp_4 Min Units: C

Public Table:
Fields:
BattV Units: Volts
APPENDIX C.

Triaxial Test Notes for the Doe Run Landslide
Triaxial Test Notes for the Doe Run Landslide

Sample: Doe Run 2

Location: Downslope, in slump scarp          Depth: 3.6 ft (1.1m)

CU TEST        Target Consolidation Effective Stress – 20 psi

Started 7/19/2016
γ\text{wet} = 120.3 \text{pcf}  
γ\text{dry} = 95.4 \text{pcf}  
W_s = 1016.1 \text{g}  
W_w = 267.6 \text{g}  
W_t = 1283.7 \text{g}  

Weight = 1239.2 \text{g}  
Height = 151 \text{mm}  
Diameter = 72 \text{mm}  

Backpressure:       Target Backpressure: 75 psi          Effective stress: 5 psi

Table C-1. Backpressure data

<table>
<thead>
<tr>
<th>Time</th>
<th>Cell pressure (psi)</th>
<th>Pore-pressure (psi)</th>
<th>Pore pressure transducer (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2:38pm</td>
<td>7.1</td>
<td>2.1</td>
<td>0.15</td>
</tr>
<tr>
<td>5:30pm</td>
<td>31.6</td>
<td>26.6</td>
<td>24.5</td>
</tr>
<tr>
<td>7/20/16</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8:12am</td>
<td>80.0</td>
<td>75.0</td>
<td>72.3</td>
</tr>
<tr>
<td>B-value check</td>
<td>0.97 in 2 min 50 sec (after 17hr 35 min)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Stopped Backpressure, pressures returned to Backpressure levels
Total time: 17hrs. 35min. 30 sec.

7/20/2016 (cont.)

Consolidation: – started @ 8:25am

Target effective stress = 20 psi   Effective stress rate = 2psi/hr

Table C-2. Consolidation data.

<table>
<thead>
<tr>
<th>Time</th>
<th>Current Effective Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9:30am</td>
<td>7.1</td>
</tr>
<tr>
<td>1:45pm</td>
<td>15.6</td>
</tr>
<tr>
<td>5:30pm</td>
<td>20.0</td>
</tr>
</tbody>
</table>

Consolidation reached in ~9 hrs., creep mode initiated (green light), let stand overnight
cell pressure = 95.0 psi, pore-pressure = 74.9 psi, pore-pressure transducer = 72.3 psi
7/21/2016

Shear: – started @ 8:15am

Strain rate = 5% per hour, Max strain = 20%, Max vert effective stress = 150 psi

Table C-3. Shear stage data.

<table>
<thead>
<tr>
<th>Time</th>
<th>Current Strain (%)</th>
<th>Current Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9:35am</td>
<td>6.6</td>
<td>20.9</td>
</tr>
<tr>
<td>12:15pm</td>
<td>20.0</td>
<td>32.7</td>
</tr>
</tbody>
</table>

Load at end was 152.0 lbs., pore-pressure transducer read 82.1
After ~4hrs current strain was 20.0%, test stopped

ENDED TEST

Sample: Doe Run 3

Location: Downslope, in slump scarp         Depth: 3.6 ft (1.1m)

CU TEST          Target Consolidation Effective Stress – 30 psi

Started 7/26/2016

\( \gamma_{wet} = 120.3 \text{ pcf} \)
\( \gamma_{dry} = 95.4 \text{ pcf} \)
\( W_s = 1016.1 \text{ g} \)
\( W_w = 267.6 \text{ g} \)
\( W_t = 1283.7 \text{ g} \)

Weight = 1230.5 g
Height = 151 mm
Diameter = 69 mm

Backpressure:  Target Backpressure: 75 psi     Effective stress: 5 psi

Table C-4. Backpressure data

<table>
<thead>
<tr>
<th>Time</th>
<th>Cell pressure (psi)</th>
<th>Pore-pressure (psi)</th>
<th>Pore pressure transducer (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10:53am</td>
<td>6.6</td>
<td>1.8</td>
<td>-0.6</td>
</tr>
<tr>
<td>1:30pm</td>
<td>21.8</td>
<td>16.7</td>
<td>13.7</td>
</tr>
<tr>
<td>5:20pm</td>
<td>47.8</td>
<td>42.8</td>
<td>38.3</td>
</tr>
<tr>
<td>7/27/16</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8:30am</td>
<td>80.0</td>
<td>74.9</td>
<td>72.0</td>
</tr>
<tr>
<td>B-value check</td>
<td>0.98 in 4 minutes</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Stopped Backpressure, pressures returned to Backpressure levels
7/27/2016 (cont.)

Consolidation – started @ 8:49am
Target effective stress = 30 psi Effective stress rate = 2psi/hr

Table C-5. Consolidation data.

<table>
<thead>
<tr>
<th>Time</th>
<th>Current Effective Stress (psi)</th>
<th>Cell Pressure (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12:45pm</td>
<td>13.4</td>
<td>87.8</td>
</tr>
<tr>
<td>6:43pm</td>
<td>25.5</td>
<td>99.7</td>
</tr>
<tr>
<td>7/28/16</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9:00am</td>
<td>29.6</td>
<td>104.2</td>
</tr>
</tbody>
</table>

Total time: 1 day, 0 hr, 11 min

Consolidation reached in ~15 hrs., creep mode initiated (green light on)

7/28/2016 (cont.)

Shear – started @ 9:05am

Strain rate = 5% per hour, Max strain = 20%, Max vert effective stress = 150 psi

Table C-6. Shear stage data.

<table>
<thead>
<tr>
<th>Time</th>
<th>Current Strain (%)</th>
<th>Current Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10:05am</td>
<td>4.9</td>
<td>29.5</td>
</tr>
<tr>
<td>2:25pm</td>
<td>20.0</td>
<td>45.5</td>
</tr>
</tbody>
</table>

Load at end was 170.0 lbs., pore-pressure transducer read 86.9
After ~4hrs current strain was 20.0%, test stopped

ENDED TEST

Sample: Doe Run 1

Location: Upslope Depth: 2.5 ft (75cm)

CU TEST Target Consolidation Effective Stress – 40 psi

Started 8/15/2016

\[ \gamma_{wet} = 120.3 \text{ pcf} \]
\[ \gamma_{dry} = 95.5 \text{ pcf} \]
\[ W_s = 1016.1 \text{ g} \]
\[ W_w = 267.6 \text{ g} \]
\[ W_t = 1283.7 \text{ g} \]

Weight = 1236.1 g (losing soil during compaction)
Height = 152 mm
Diameter = 72 mm

Backpressure – started @10:35am    Target Backpressure: 75 psi    Effective stress: 5 psi

Table C-7. Backpressure data.

<table>
<thead>
<tr>
<th>Time</th>
<th>Cell pressure (psi)</th>
<th>Pore-pressure (psi)</th>
<th>Pore pressure transducer (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1:20pm</td>
<td>20.8</td>
<td>15.9</td>
<td>10.8</td>
</tr>
<tr>
<td>5:30pm</td>
<td>44.0</td>
<td>39.0</td>
<td>33.3</td>
</tr>
<tr>
<td>8/16/16</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8:05am</td>
<td>80.0</td>
<td>74.9</td>
<td>71.9</td>
</tr>
<tr>
<td>B-value check</td>
<td>0.96 in 10 min</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Stopped Backpressure, pressures returned to Backpressure levels
Total time: 22 hrs. 2min.

8/16/2016 (cont.)

Consolidation – started @ 8:37am

Target effective stress = 40 psi    Effective stress rate = 2psi/hr

Table C-8. Consolidation data

<table>
<thead>
<tr>
<th>Time</th>
<th>Current Effective Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1:45pm</td>
<td>15.7</td>
</tr>
<tr>
<td>8/17/2016</td>
<td></td>
</tr>
<tr>
<td>8:45am</td>
<td>39.9</td>
</tr>
</tbody>
</table>

Consolidation reached in ~20 hrs., creep mode initiated (green light), let stand extra 4 hrs
cell pressure = 114.4.0 psi, pore-pressure = 74.4 psi, pore-pressure transducer = 71.8 psi

8/17/2016

Shear – started @ 8:58am

Strain rate = 5% per hour, Max strain = 20%, Max vert effective stress = 150 psi

Table C-9. Shear stage data.

<table>
<thead>
<tr>
<th>Time</th>
<th>Current Strain (%)</th>
<th>Current Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2:30pm</td>
<td>20.0</td>
<td>48.7</td>
</tr>
</tbody>
</table>

Load at end was 181.0 lbs., pore-pressure transducer read 94.2 psi

ENDED TEST
APPENDIX D.

CU Triaxial Test Results for the Doe Run Landslide
Table D-1. Shear strength parameters for 3 CU tests

<table>
<thead>
<tr>
<th>Doe Run</th>
<th>20 psi</th>
<th>30 psi</th>
<th>40 psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_1$</td>
<td>29.4</td>
<td>41.1</td>
<td>48.1</td>
</tr>
<tr>
<td>$\sigma_3$</td>
<td>12.5</td>
<td>17.1</td>
<td>19.5</td>
</tr>
<tr>
<td>$\mu$</td>
<td>8.8</td>
<td>11.9</td>
<td>18.9</td>
</tr>
<tr>
<td>$\sigma_{1\text{effective}}$</td>
<td>20.5</td>
<td>29.3</td>
<td>29.2</td>
</tr>
<tr>
<td>$\sigma_{3\text{effective}}$</td>
<td>3.6</td>
<td>5.2</td>
<td>0.58</td>
</tr>
<tr>
<td>$p'$</td>
<td>9.2</td>
<td>13.2</td>
<td>10.2</td>
</tr>
<tr>
<td>$q'$</td>
<td>16.9</td>
<td>24.1</td>
<td>28.6</td>
</tr>
<tr>
<td>$s'$</td>
<td>12.1</td>
<td>17.2</td>
<td>14.9</td>
</tr>
<tr>
<td>$t=t'$</td>
<td>8.5</td>
<td>12.1</td>
<td>14.3</td>
</tr>
<tr>
<td>$t'/s'$</td>
<td>0.4328</td>
<td>4.9</td>
<td>25.6</td>
</tr>
</tbody>
</table>

Figure D-1. Stress-strain plot (a) and excess pore-water pressure (b) for 3 CU tests

Figure D-2. Stress paths for CU tests.
APPENDIX E.

Triaxial Test Notes for the Herron Hill Landslide
Triaxial Test Notes for the Herron Hill Landslide

Sample: Herron Hill 2
Location: Midslope  Depth: 5-6 ft (1.5-1.8m)

CU TEST  Started 6/7/2016  Target Consolidation Effective Stress – 20 psi

\[ \gamma_{\text{wet}} = 125 \text{ pcf} \]
\[ \gamma_{\text{dry}} = 106 \text{ pcf} \]
\[ W_s = 1133 \text{ g} \]
\[ W_w = 182 \text{ g} \]
\[ W_t = 1315 \]

Weight = 1337 g  
Height = 148 mm  
Diameter = 71.5 mm

Backpressure:  Target Backpressure: 75 psi  Effective stress: 5 psi

<table>
<thead>
<tr>
<th>Time</th>
<th>Cell pressure (psi)</th>
<th>Pore-pressure (psi)</th>
<th>Pore pressure transducer (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12:30pm</td>
<td>5.0</td>
<td>0.0</td>
<td>na</td>
</tr>
<tr>
<td>2:00pm</td>
<td>13.9</td>
<td>9.0</td>
<td>0.73</td>
</tr>
<tr>
<td>5:00pm</td>
<td>26.9</td>
<td>21.9</td>
<td>15.7</td>
</tr>
<tr>
<td>6/8/16</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7:30am</td>
<td>80.0</td>
<td>75.0</td>
<td>72.1</td>
</tr>
<tr>
<td>B-value check</td>
<td>0.96 in 11 minutes</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stopped Backpressure</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

6/8/2016

Consolidation – started @ 8:00am  
Target effective stress = 20 psi  
Effective stress rate = 2%/hr  
Consolidation reached in 10 hrs., creep mode initiated (green light)  
cell pressure = 94.8 psi, pore-pressure = 74.9 psi, pore-pressure transducer = 72.3 psi

Shear – started @ 8:20 pm  
Strain rate = 5% per hour, Max strain = 20%, Max vert effective stress = 150 psi  
6/9/2016 @9:40am  Stopped test
Sample: Herron Hill 1

Location: Midslope  Depth: 4.5 ft (1.37m)

CU TEST  Started 6/28/2016

$\gamma_{wet} = 125$ pcf
$\gamma_{dry} = 106$ pcf
$W_s = 1133$ g
$W_w = 182$ g
$W_t = 1315$

Target Consolidation Effective Stress – 30 psi

Weight = 1319 g
Height = 147 mm
Diameter = 71 mm

Backpressure:  Target Backpressure: 75 psi  Effective stress: 5 psi

Table E-2. Backpressure data

<table>
<thead>
<tr>
<th>Time</th>
<th>Cell pressure (psi)</th>
<th>Pore-pressure (psi)</th>
<th>Pore pressure transducer (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12:20pm</td>
<td>15.5</td>
<td>10.5</td>
<td>0.6</td>
</tr>
<tr>
<td>3:30pm</td>
<td>28.8</td>
<td>23.8</td>
<td>3.7</td>
</tr>
<tr>
<td>5:15pm</td>
<td>36.6</td>
<td>31.4</td>
<td>6.1</td>
</tr>
<tr>
<td>8:30pm</td>
<td>46.8</td>
<td>41.9</td>
<td>15.3</td>
</tr>
<tr>
<td>6/29/16</td>
<td>80.0</td>
<td>75.0</td>
<td>71.1</td>
</tr>
<tr>
<td>8:45am</td>
<td>80.0</td>
<td>75.0</td>
<td>71.1</td>
</tr>
<tr>
<td>B-value check</td>
<td>0.75 in 30 min, stopped and restarted Backpressure</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9:30am</td>
<td>79.9</td>
<td>74.8</td>
<td>72.5</td>
</tr>
<tr>
<td>11:30</td>
<td>80.0</td>
<td>75.0</td>
<td>71.6</td>
</tr>
<tr>
<td>1:30pm</td>
<td>80.0</td>
<td>75.0</td>
<td>71.7</td>
</tr>
<tr>
<td>5:00pm</td>
<td>80.0</td>
<td>75.0</td>
<td>71.9</td>
</tr>
<tr>
<td>6/30/16</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8:00am</td>
<td>80.0</td>
<td>75.0</td>
<td>72.2</td>
</tr>
<tr>
<td>B-value check</td>
<td>0.92 in 2 hrs</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Ended Backpressure

Consolidation  6/30/2016

Ramped pressure down to empty cell pressure pump, ramped pressures back up to Backpressure levels
Target effective stress = 30 psi
Started Consolidation @ 10:30am

Table E-3. Consolidation data.

<table>
<thead>
<tr>
<th>Time</th>
<th>Current Effective Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10:30am</td>
<td>5.0</td>
</tr>
<tr>
<td>12:50pm</td>
<td>9.5</td>
</tr>
<tr>
<td>9:00pm</td>
<td>25.8</td>
</tr>
<tr>
<td>7/1/2016</td>
<td></td>
</tr>
<tr>
<td>8:00am</td>
<td>30.0</td>
</tr>
</tbody>
</table>

Creep mode initiated (green light), stopped Consolidation

Shear

5%/hr
Max Vert Effective Stress = 150 psi
Max Strain = 20%

@10:50 am Current Strain = 12.88%, load = 263 lbs
cell pressure = 105.2 psi, pore-pressure = 73.2 psi, pore-pressure transducer = 80.75

Shear stopped at 18.09% strain after 3 hr 37 minutes

Stopped test

Sample: Herron Hill 3

Location: Upslope, above old road  Depth: 5 ft (1.5m)

CU TEST    Started 7/6/2016    Target Consolidation Effective Stress – 40 psi

\[ \gamma_{wet} = 125 \text{ pcf} \]
\[ \gamma_{dry} = 106 \text{ pcf} \]
\[ W_s = 1133 \text{ g} \]
\[ W_w = 182 \text{ g} \]
\[ W_t = 1315 \text{ g} \]

Weight = 1313 g
Height = 148 mm
Diameter = 71 mm

Backpressure
Target Backpressure: 75 psi
Effective stress: 5 psi
Table E-4. Backpressure data.

<table>
<thead>
<tr>
<th>Time</th>
<th>Cell pressure (psi)</th>
<th>Pore-pressure (psi)</th>
<th>Pore pressure transducer (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>11:13am</td>
<td>5.0</td>
<td>0.0</td>
<td>na</td>
</tr>
<tr>
<td>11:18am</td>
<td>6.23</td>
<td>1.27</td>
<td>-1.29</td>
</tr>
<tr>
<td>2:00pm</td>
<td>18.2</td>
<td>13.2</td>
<td>2.75</td>
</tr>
<tr>
<td>4:20pm</td>
<td>28.4</td>
<td>23.5</td>
<td>15.5</td>
</tr>
<tr>
<td>9:00pm</td>
<td>46.3</td>
<td>41.4</td>
<td>15.5</td>
</tr>
</tbody>
</table>

Paused and ramped pp down to refill pp pump, manually reduced load

7/7/16

8:15am

80.0  75.0  71.8

B-value check  0.97 in 6 min 30 sec

Stopped Backpressure, pressures returned to Backpressure levels

7/7/2016

Consolidation – started @ 8:35am

Target effective stress = 40 psi  Effective stress rate = 2psi/hr

Table E-5. Consolidation data.

<table>
<thead>
<tr>
<th>Time</th>
<th>Current Effective Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>11:00am</td>
<td>9.64</td>
</tr>
<tr>
<td>1:40pm</td>
<td>15.1</td>
</tr>
<tr>
<td>6:30pm</td>
<td>24.8</td>
</tr>
<tr>
<td>7/8/2016</td>
<td></td>
</tr>
<tr>
<td>8:00am</td>
<td>40.0</td>
</tr>
</tbody>
</table>

7/8/2016 (cont.)

Consolidation in 20 hrs., creep mode initiated (green light), let stand another 3 hrs.

cell pressure = 115.0 psi, pore-pressure = 75.0 psi, pore-pressure transducer = 72.2 psi

Shear – started @ 11:05am

Strain rate = 5% per hour, Max strain = 20%, Max vert effective stress = 150 psi

After 3hrs 53 minutes current strain was 19.44%, test stopped load=307.8 lbs

ENDED TEST
APPENDIX F.

CU Triaxial Test Results for the Herron Hill Landslide
CU Triaxial Test Results for the Herron Hill Landslide

Table F-1. Shear strength parameters for 3 CU tests.

<table>
<thead>
<tr>
<th></th>
<th>20 psi</th>
<th>30 psi</th>
<th>40 psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Herron Hill</td>
<td>51.9</td>
<td>61.7</td>
<td>78.9</td>
</tr>
<tr>
<td>σ₁</td>
<td>22.7</td>
<td>28.0</td>
<td>36.8</td>
</tr>
<tr>
<td>μ</td>
<td>6.7</td>
<td>10.9</td>
<td>13.41</td>
</tr>
<tr>
<td>σ₁ effective</td>
<td>41.2</td>
<td>50.8</td>
<td>65.5</td>
</tr>
<tr>
<td>σ₃ effective</td>
<td>15.9</td>
<td>17.1</td>
<td>23.4</td>
</tr>
<tr>
<td>p’</td>
<td>25.7</td>
<td>28.3</td>
<td>37.4</td>
</tr>
<tr>
<td>q’</td>
<td>29.2</td>
<td>33.7</td>
<td>42.2</td>
</tr>
<tr>
<td>s’</td>
<td>30.5</td>
<td>33.9</td>
<td>44.4</td>
</tr>
<tr>
<td>t=t’</td>
<td>14.6</td>
<td>16.8</td>
<td>21.1</td>
</tr>
<tr>
<td>t’/s’</td>
<td>0.4499</td>
<td>1.178</td>
<td>26.7</td>
</tr>
</tbody>
</table>

Figure F-1. Stress-strain plot (a) and excess pore-water pressure (b) for 3 CU tests

Figure F-2. Stress paths for CU tests.
APPENDIX G.

Un-interpreted Electrical Resistivity Profiles
Meadowview Landslide

MVS1, 7/26/2013

MVS1-2, 11/13/2013

MVS2, 7/26/2013

MVS2-2, 11/13/2013

MVS3, 7/26/2013
Doe Run Landslide

DR1, 7/1/2015

DR1A, 9/2/2015

DR1B, 11/16/2015
DR1C, 2/5/2016

Herron Hill Landslide

HH1A, 3/12/2015

HH1B, 10/7/2015
HH2C, 12/7/2015

HH2D, 2/19/2016

HH2E, 5/19/2016

HH3A, 3/12/2015

HH3B, 10/7/2015
HH3E, 5/19/2016

Roberts Bend Landslide

RB1, 2/9/2017

RB1A, 4/7/2017

RB2, 2/9/2017

RB2A, 4/7/2017
REFERENCES


Sass, O.; Bell, R.; and Glade, T., 2008. Comparison of GPR, 2D-resistivity and traditional techniques for the subsurface exploration of the Öschingen landslide, Swabian Alb (Germany). Geomorphology 93, (1) 89–103.


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EDUCATION

Ph.D. Candidate: Fall 2010 – present, Earth and Environmental Sciences, University of Kentucky

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PROFESSIONAL EXPERIENCE


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AWARDS

2017 Certificate of Meritorious Service, from the Geological Society of America Environmental and Engineering Geology Division for efforts made on behalf of the Division.

Peer-reviewed journals and conference papers


**Crawford, M.M.**, and Bryson L.S., 2016, Field observations of an active landslide in Kentucky, 1st International Conference on Natural Hazards and Infrastructure, Chania, Greece, June 28-30, 10 p.


Kentucky Geological Survey Publications


Maps


6 Surficial Geologic Map Kentucky Geological Survey Contract Reports
Example citation: Crawford, M.M., and Murphy, M.L., Quaternary geologic map of the Quicksand 7.5-minute quadrangle, Kentucky, Kentucky Geological Survey Contract Report 33, Series 12, scale 1:24,000, 1 sheet.