Innovative Solutions for Slope Stability Reinforcement and Characterization: Vol. II

PARTNERSHIP FOR
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16. Abstract

Soil slope instability concerning highway infrastructure is an ongoing problem in Iowa, as slope failures endanger public safety and continue to result in costly repair work. While in the past extensive research has been conducted on slope stability investigations and analysis, this current research study consists of field investigations addressing both the characterization and reinforcement of such slope failures. While Volume I summarizes the research methods and findings of this study, Volume II provides procedural details for incorporating an infrequently-used testing technique, borehole shear tests, into practice.

Fifteen slopes along Iowa highways were investigated, including thirteen slides (failed slopes), one unfailed slope, and one proposed embankment slope (the Sugar Creek Project). The slopes are mainly comprised of either clay shale or glacial till, and are generally gentle and of small scale, with slope angle ranging from 11º to 23º and height ranging from 6 to 23 m. Extensive field investigations and laboratory tests were performed for each slope. Field investigations included survey of slope geometry, borehole drilling, soil sampling, in-situ Borehole Shear Testing (BST) and ground water table measurement. Laboratory investigations mainly comprised of ring shear tests, soil basic property tests (grain size analysis and Atterberg limits test), mineralogy analyses, soil classifications, and natural water contents and density measurements on the representative soil samples from each slope. Extensive direct shear tests and a few triaxial compression tests and unconfined compression tests were also performed on undisturbed soil samples for the Sugar Creek Project. Based on the results of field and lab investigations, slope stability analysis was performed on each of the slopes to determine the possible factors resulting in the slope failures or to evaluate the potential slope instabilities using limit equilibrium methods. Deterministic slope analyses were performed for all the slopes. Probabilistic slope analysis and sensitivity study were also performed for the slope of the Sugar Creek Project.

Results indicate that while the in-situ test rapidly provides effective shear strength parameters of soils, some training may be required for effective and appropriate use of the BST. Also, it is primarily intended to test cohesive soils and can produce erroneous results in gravelly soils. Additionally, the quality of boreholes affects test results, and disturbance to borehole walls should be minimized before test performance. A final limitation of widespread borehole shear testing may be its limited availability, as only about four to six test devices are currently being used in Iowa. Based on the data gathered in the field testing, reinforcement investigations are continued in Volume III.

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

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The findings, opinions, recommendations, and conclusions expressed in this report are those of the authors and do not necessarily reflect the views of the sponsor and administrations.

EXECUTIVE SUMMARY

Soil slope instability concerning highway infrastructure is an ongoing problem in Iowa, as slope failures endanger public safety and continue to result in costly repair work. This research consists of field investigations addressing both the characterization and reinforcement of such slope failures. The research methods and findings of these investigations are summarized in Volume 1 of this report. Research details of the independent characterization and reinforcement investigations are provided in Volumes 2 and 3, respectively. Combined, the field investigations offer guidance on identifying the factors that affect slope stability at a particular location and also on designing slope reinforcement using pile elements for cases where remedial measures are necessary.

Research Summary

Characterization of slope failures is complicated, because the factors affecting slope stability can be difficult to discern and measure, particularly soil shear strength parameters. Extensive research has been conducted on slope stability investigations and analysis. The current research, however, focused on applying an infrequently-used testing technique comprised of the Borehole Shear Test (BST). This in-situ test rapidly provides effective (i.e., drained) shear strength parameter values of soil. Using the BST device, fifteen Iowa slopes (fourteen failures and one proposed slope) were investigated and documented. Particular attention was paid to highly weathered shale and glacial till soil deposits, which have both been associated with slope failures in the southern Iowa drift region. Conventional laboratory tests, including direct shear tests, triaxial compression tests, and ring shear tests were also performed on undisturbed and reconstituted soil samples to supplement BST results. The shear strength measurements were incorporated into complete evaluations of slope stability using both limit equilibrium and probabilistic analyses.

Remediation of slope failures requires stabilization alternatives that address causes of slope instability. Slope reinforcement using pile elements can be an effective method of remediation in preventing slope movements in weak soils where enhanced drainage does not provide adequate stability. Soil load transfer to pile elements from the downslope soil movement as occurs in slope failures is a complex soil–structure interaction problem. Soil–structure interactions for small-diameter, grouted pile elements subject to lateral soil movement were investigated by conducting full-scale pile load tests, in which piles installed through a shear box into stable soil were loaded by uniform lateral translation of soil. Instrumentation of the shear boxes and pile reinforcement indicated the load distributions that developed along the piles. The load test analyses which followed the pile load tests support the claim that the distributed loads which are mobilized during pile loading depend on the relative displacement between the soil and pile elements. The reliable estimation of these load distributions is important, because the influence of piles on the global stability of the slope depends directly on the pile loading condition.

Research Conclusions

The following conclusions were drawn from slope stability case histories:

- The Borehole Shear Test often measures peak shear strength parameters, which are generally not operative for a slope failure, and sometimes measures the soften shear strength when the measurements are taken near the slip surface. Factors of safety for case histories of slope failures calculated using BSTs were generally greater than unity.
- The ring shear test using reconstituted samples gives residual shear strength parameter values corresponding to relatively large shear displacements. Factors of safety for case histories of slope failures calculated using ring shear test results were generally less than unity.
- Back calculated shear strengths for slope failures that provided factors of safety equal to unity were generally between shear strengths from ring shear tests and Borehole Shear Tests. Slope failures can be attributed to soil softening or progressive failure and may have been caused by high water tables.
- For some slope failures, the use of the BST are useful in better estimating the operative (or the mobilized) shear strength in conjunction with the residual shear strength and back calculated shear strength.
- For the slope failures, the glacial tills generally have lower clay fraction and lower plasticity index than the clay shales. All the tills are classified as low plasticity clay (CL) according to Unified Soil Classification System, while most of the shales are classified as high plasticity clay (CH).
- The peak BST results for the slope failures show that, the glacial tills and the clay shales have similar average values of effective friction angle, which are 22.5^{o} and 22.1^{o}, respectively; but the glacial tills have considerably lower average value of effective cohesion (11.6 kPa) than the clay shales (17.7 kPa). However, the glacial tills have higher residual shear strength (residual friction angle of 8.4^{o} to 26.9^{o}) than the clay shales (residual friction angle of 6.2^{o} to 15.1^{o}).
- Sensitivity analyses showed that soil shear strength is the most sensitive parameter affecting factors of safety. Water table location additionally has a significant influence on slope stability.
- Probabilistic slope stability analyses are useful when a relatively large amount of input parameters are available, such as shear strengths obtained from BSTs. The probability of slope failure is evaluated based on statistical distribution of soil shear strengths.

The following conclusions were drawn from investigating pile reinforcement:

- The installation of slender piles in weak soils offers considerable resistance to lateral soil movement, with improvement factors from the load tests ranging from 1.2 to 6.6. Improvement factors are defined as a ratio of peak loads for reinforced tests and unreinforced tests.
- Pile section moment capacities were mobilized, indicating that a “flexible” pile failure mode was achieved. The depth of maximum moment and pile failure ranged from 1.8 to 5.4 pile diameters below the shear plane.

- The relative soil-pile displacement at the soil surface indicates the behavioral stages of small-diameter piles as (1) mobilization of soil shear stresses and elastic bending of pile, (2) mobilization of pile concrete compressive strength, and (3) incipient pile failure due to pile moment capacity mobilization. The behavioral characteristics of slender piles are controlled by structural pile behavior through moment-curvature relationships as much as they are by soil behavior.
- Displacement-based lateral response analysis methods which use soil p-y curves accurately predict the deflection and bending moment of piles subject to lateral soil movement. From these pile behavior characteristics, pile shear may be calculated and applied to the limit equilibrium equation for evaluating global stability of reinforced slopes.

Recommendations for Implementation

The research findings are expected to benefit civil and geotechnical engineers of government transportation agencies, consultants, and contractors dealing with slope stability, slope remediation, and geotechnical testing in Iowa. In-situ BST measurements provide reliable, site-specific soil parameters for design applications which can lead to substantial cost savings over using empirical estimations for critical soil properties. As the BST is an alternative to expensive and time-consuming laboratory testing, the device is particularly useful in obtaining relatively large amounts of data necessary for probabilistic analyses. Procedures for incorporating Borehole Shear tests into practice are documented in Volume 2 of this report. Nevertheless, some training may be required for effective and appropriate use. The BST is primarily intended to test cohesive soils. The device can produce erroneous results in gravelly soils. Additionally, the quality of boreholes affects test results, and disturbance to borehole walls should be minimized before test performance. A final limitation of widespread Borehole Shear testing may be its limited availability, as only about 4 to 6 test devices are currently being used in Iowa.

The research presented in Volume 3 demonstrates with experimental testing how lateral forces develop along stabilizing piles to resist slope movements. This report then documents a step-by-step procedure that can be used by both state and county transportation agencies to design slope reinforcement using slender piles. A state department of transportation may develop training seminars for all local transportation agencies to provide further guidance in using the proposed design method. This effort may be coordinated with the authors and might be extended so far as to conduct a pilot study to demonstrate the intended process of designing and evaluating the reinforcement solution. While slope reinforcement with slender piles by county transportation agencies is encouraged, such action is recommended to be coordinated with the state department of transportation. This organization can document all such remediation projects to better guide counties using successful and unsuccessful experiences, as the DOT will have working knowledge of other unstable slope characteristics and corresponding reinforcement designs. The proposed slope reinforcement solution has not yet been demonstrated at an Iowa slope failure site. As a result, difficulty in scheduling and bidding a pile reinforcement project and evaluating the effectiveness of the measure may impede successful implementation. Obtaining experience and feedback through data collection or visual inspection, however, will promote incorporation of the research findings into standard slope remediation practice.

Successful implementation of innovative slope stability reinforcement and characterization solutions can be evaluated by documenting the number of slopes reinforced with pile elements and those investigated using BST measurements, respectively. Cost savings of incorporating Borehole Shear testing into site investigation practice will be made evident by comparing costs corresponding to designs for geostructures making use of accurate and reliable soil properties (obtained from BST measurements) to those designs using estimated soil properties and higher factors of safety. Calculating long-term cost savings of slope reinforcement using piles considering maintenance costs associated with alternatives and the cost for rebuilding a failed drainage remediation, for example, can indicate the progress and consequences of implementation.

INTRODUCTION

Problem Statement

Soil slope instability continues to be a problem in Iowa in transportation routes or along highways and roadways. Failures occur in both cut slopes and earth embankment fill slopes. Chu (2001) reported that 48 counties in Iowa have experienced slope stability problems since 1993. A particular case is the Highway 330 slope failure in Jasper County, Iowa, which developed an approximate 35 m long head scarp (Figure 1). Field borings conducted after tension cracks developed showed that the fill soils were 8% to 10% above optimum moisture content, which indicated that the soil was nearly saturated and had developed low shear strength. Slope failures have posed concerns to the public safety, caused construction delays and resulted in costly repair work.

Figure 1. Existing slope failure at Highway 330 in Jasper County, Iowa (White 2003)

Slope failures are complex events and the factors that affect slope stability are difficult to measure, particularly shear strength parameter values of the soil and ground water conditions. Ideally, the stability problems can be discovered and addressed before a slope failure occurs. However, once a failure occurs or a potential failure is identified, information and knowledge of the major factors resulting in the failure are required to develop an effective remediation plan.

It is necessary to evaluate the stability of the concerned slopes, or to investigate the causes of the slope failures, in a rapid and effective way. Although various test methods are available for field investigation, this study focused on the use of the Borehole Shear Test (BST), which has been considered as a simple and quick in-situ testing technique (Handy 1986). The investigations were supplemented by other laboratory tests. Particular emphasis was given to the characterization of the clay shales which have been associated with many slope failures in Iowa.

Objectives

The major objectives in this study are as follows:

- Develop and validate appropriate test procedures for quickly determining in-situ shear strength parameters of soil using the Borehole Shear Test technique
- Illustrate the importance, application and procedure of the proper selection of shear strength parameters for the stability analysis
- Document a number of case histories where failures have been observed or potential failure exists to better understand the failure mechanisms

Organization of the Report

Chapter 2 provides background information relevant to the study, including (1) Regional geology of Iowa; (2) General considerations in slope investigation; (3) Borehole Shear Test and Rock Borehole Shear Test; (4) Residual shear strength and ring shear test; (5) Factor of safety and limit equilibrium slope analysis; and (6) Probabilistic slope analysis.

Chapter 3 presents the field and lab investigation results, slope analysis results and their discussions and conclusions for 15 case histories of slopes. The chapter starts with some general information and overview followed by the details for the 15 case histories.

Chapter 4 summarizes and concludes the results and findings in the study. Finally, Chapter 5 makes some recommendations.

BACKGROUND INFORMATION

In this chapter, some background information that is relevant to the research is provided. This includes the regional geology of Iowa; general considerations in slope investigation; Borehole Shear Test (BST) and Rock BST; residual shear strength and ring shear test; liquid equilibrium slope analysis and probabilistic slope analysis.

Regional Geology of Iowa

Iowa is commonly divided into seven regions based on the various landforms found in each region (Prior 1976 and 1991). Those regions include the Des Moines Lobe, Southern Iowa Drift Plain, Loess Hills, Iowan Surface, Northwest Iowa Plains, Paleozoic Plateau, and Alluvial Plains and are shown in Figure 2. Each region has its own unique landforms and landscape formed by various processes. Most of the landforms of Iowa were formed by water erosion or glacial erosion. Various geologic materials also have influenced the formation of the landforms. The following descriptions of each region are adopted after Prior (1976 and 1991).

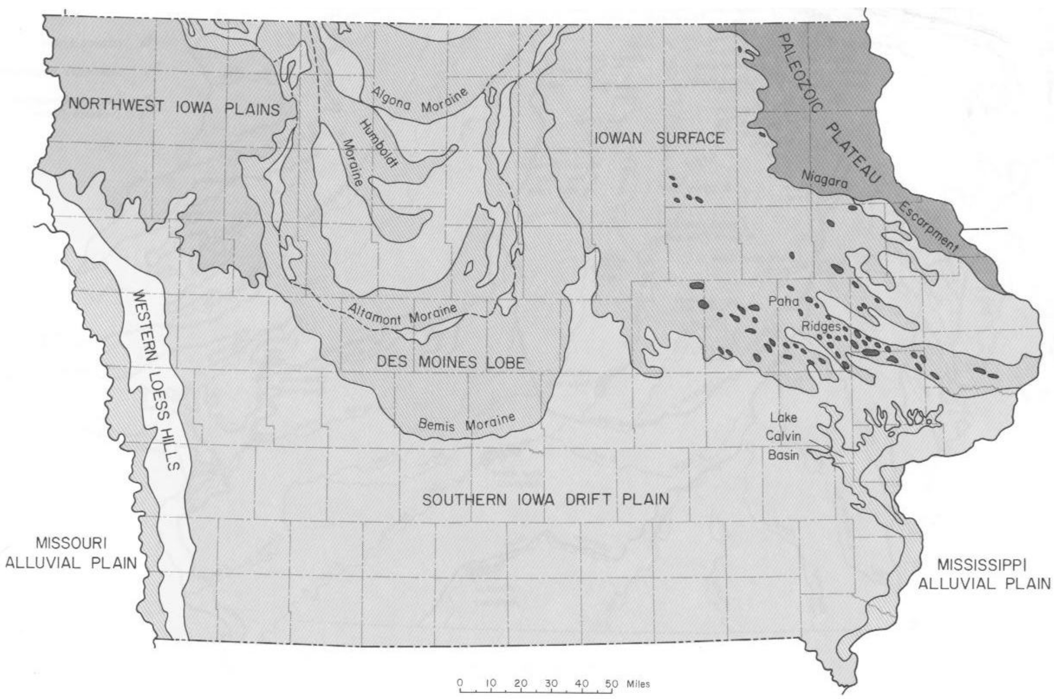


Figure 2. Landform regions of Iowa (Prior 1991)

Des Moines Lobe

Deposits and landforms on the Des Moines Lobe are the best examples of recent glacial erosion and deposition in the state. The Des Moines Lobe landforms formed during the last glacial advance into Iowa about 12,000 to 14,000 years ago. The landforms exhbit rough edges or end moraines, lakes and flat areas, with circular ponds or depressions. Most of the landscape is covered with glacial drift left behind by the glacier. Glacial drift is a deposit of boulders, gravel, sand, silt, and clay left behind by a glacier or by the streams and rivers that drained off the melting ice. In places boulders can be found along fences or in the fields. Present day the rivers that flow across the Lobe have deposited sand and gravel layers (alluvium).

Southern Iowa Drift Plain

The Southern Iowa Drift Plain is the largest in Iowa. The landscapes are characterized by gently rolling hills and valleys. They have been formed by hundreds of thousands of years of erosion and stream development on what was once a landscape similar to that in the Des Moines Lobe region. Often trees or even forests grow in the valleys. Rivers, streams, or creeks at the bottoms of the valleys with their numerous upstream tributaries form a drainage pattern that looks like the branches of a tree. Underlying much of the region is a thin layer of loess, a thick layer of glacial drift, and finally bedrock of limestone, shale, and sandstone. Alluvium is common on the flood plain of the region's drainages. Paleosols also are found in the region.

Loess Hills

The Loess Hills landform region is located along the west edge of Iowa. It formed periodically during the last 150,000 years. Loess is windblown silt that was picked up by winds off the Missouri River valley floor during and between glacial advances and retreats. Loess is thickest along the west edge of Iowa and gradually thins as you go eastward toward central Iowa. Loess is deposited on top of older glacial drift and bedrock. Streams and rivers have eroded valleys in the loess and deposited alluvium on their flood plains. There also are deposits of colluvium in the valleys. The landform region is characterized by steep-sided hills and ridges and tree-covered ravines or side valleys.

Iowan Surface

The Iowan Surface is one of the most difficult regions to interpret geologically. Recent studies indicate that the region formed mainly due to intense erosion in a cold, tundra-like climate. The region is characterized by almost flat land, occasional long hills that early observers called "dolphin-backed hills," and rivers and streams. In the northern part of the region there are numerous sinkholes or depressions caused by the collapse of underground caves and caverns. Glacial drift similar to that found in the Southern Iowa Drift Plain and limestone bedrock underlie the region, and loess remains on the tops of the elongated hills, which geologists call Paha after a Native American word that describes a hill. Colluvium and alluvium are found on some slopes and along flood plains. Erratics (boulders moved by the glaciers from Canada and Minnesota) are common and sometimes very large.

Northwest Iowa Plains

The Northwest Iowa Plains are the highest, driest, and least tree-covered region in the state. The region is characterized by a landscape that is similar to the Iowa Surface: flat to very gently rolling, with long parallel hills and subtle valleys. Trees are typically found only where planted around farmsteads or in some valley bottoms. Glacial drift underlies a thin layer of loess that covers most of the region.

Paleozoic Plateau

The contrast between the Paleozoic Plateau region of Iowa and all of the rest of the state is very obvious. Outcrops of solid bedrock (mostly limestone) are very common. Only a few scattered patches of glacial deposit exist in the region. Valleys are deep, steep, and make great scenic vistas as viewed from the uplands. The bedrock that controls the shape of the land in this region formed in warm tropical sea floors between 300-500 million years ago. The bedrock forms the famous "bluffs" along the edge of the Mississippi River's flood plain. Caves are common and sinkholes or depressions often filled with water are found in portions of this landform region.

Alluvial Plains

This landform is located adjacent to the Mississippi and Missouri Rivers and other large rivers in the state. Characterized by landscapes developed by water erosion and deposition along a river's flood plain it is wide and flat, with features typical of a flowing river. Alluvium deposited by the river and glacial drift or bedrock underlie the region.

General Considerations in Slope Investigation

Many factors are involved in soil slope stability evaluation and analysis. Among those that need to be considered, the main ones include (1) geologic conditions, including soil properties and shear strength; (2) site topography; (3) ground water conditions; (4) construction effects; and (5) seismicity (Abramson et al. 2002; Duncan 1996). Among these factors, shear strength of soil, site topography and ground water conditions are the most critical for embankment slopes and cut slopes. Therefore, these factors will be given particular consideration in slope stability investigation.

Though many apparatus and methods can be used for slope stability investigation, it is neither possible nor necessary to use all of them. Thus, this research will be limited to the use of a few apparatus, which includes in-situ Borehole Shear Test (BST) and some conventional laboratory test devices. The BST is used to rapidly measure in-situ shear strength parameters in a borehole that is drilled either mechanically or by hand-augering. Conventional laboratory tests such as direct shear test, triaxial compression test and ring shear test will also be performed on undisturbed soil samples. As BST is relatively less used, its details and testing procedures are given in the following section.

Borehole Shear Test and Rock Borehole Shear Test

The shear strength of soil is perhaps the most critical factor in slope stability analysis. Many apparatus and methods have been used to obtain the shear strength parameters through both field measurements (e.g., standard penetration test and cone penetration test, etc.) and laboratory measurements (e.g., direct shear test and triaxial test, etc.). Among the various test equipment and apparatus, the Borehole Shear Test (BST) is unique in that it gives a rapid, direct and accurate in situ measurement of both effective cohesion and friction angle (Handy 1986).

The fundamental consideration involved in the BST is to perform a series of direct shear tests on the inside of a borehole (Handy and Fox 1967; Wineland 1975). A BST apparatus is shown in Figure 3. Tests are conducted by expanding diametrically opposed contact shear plates into a borehole under a constant known normal stress, allowing the soil to consolidate, and then by pulling vertically the shear plates and measuring the shear stress. Data points are plotted on Mohr-Coulomb shear envelope (Figure 4) by measuring the maximum shear resistance at successively higher increments of applied normal stresses. Depending on soil type, the total testing time for a typical test with 4 to 5 data points is approximately 30 to 60 minutes (Lutenegger and Hallberg 1981). Because drainage times are cumulative, the BST is normally a consolidated-drained test (Lutenegger and Tierney 1986).

The BST has been successfully used by a number of researchers in different soil conditions, including sandy, silty and clayey soils and shales (e.g., Demartincourt and Bauer 1983; Handy 1986; Lutenegger and Tierney 1986; Millian and Escobar 1987); soft marine clays (Lutenegger and Timian 1987; Demartinecourt and Bauer 1983); hard clays (Handy et al. 1985) and stiff soil (Lutenegger et al. 1978); and unsaturated soils (Miller et al. 1998). Recently, White and Handy (2001) also used the BST to study preconsolidation pressures and soil modulii. In addition, the BST has been used to study a few landslide case histories (e.g., Tice and Sams 1974; Handy 1986). The studies show that the BST is particularly useful for quickly and accurately acquiring the in-situ shear strength parameters of the soil within the slip zone of an active landslide. After the slide activates, soil cohesion appears to become essentially zero (Handy 1986).

A Rock Borehole Shear Test (RBST) is also a portable direct shear device used to evaluate rock shear strength in-situ. The device was developed by Handy and associates at Iowa State University (Handy et al. 1976). The operation mechanism of the RBST is similar to that of the BST, except that the RBST is designed to cater for much higher normal and shear stresses. The maximum rock shear strength that may be measured is 45 MPa, and the range of applied normal stress is 0-86 MPa (Handy et al. 1976). The RBST device consists of three basic parts, i.e. the shear head assembly, the pulling jack, and the console (Figure 5). A number of authors (e.g., Higgins and Rockaway 1979) have reported successful uses of the RBST in measuring the shear strength of rock.

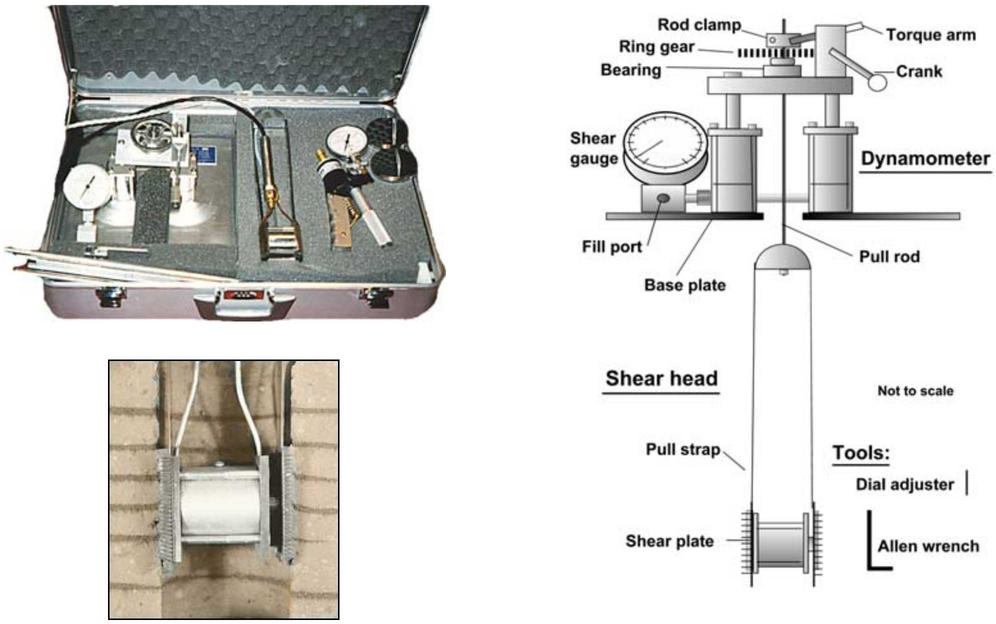


Figure 3. Borehole shear test apparatus (Handy 2001)

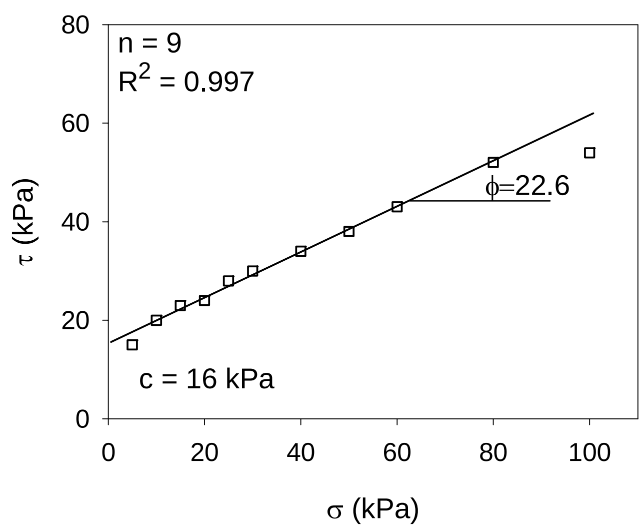


Figure 4. In situ borehole shear test results showing cohesion intercept and friction angle

(a) Pressure console

(b) Shear plate before shearing

(c) Shear plate after shearing

Figure 5. Rock borehole shear testing device

Residual Strength and Ring Shear Test

Skempton (1964, 1985) described the residual strength as the minimum strength of the soil after large displacement. Lambe and Whitman (1979) expressed the residual strength as the ultimate strength of soil in the ultimate conditions during shearing. The shear strength of the soil can drop from its peak value to the residual value after large displacement, and the drop can be significant for materials with large amounts of clay minerals, particularly platy minerals. The formation of the shear surface and achieving the residual strength results in the formation of a new fabric, particularly in material with high clay content. The drop in strength is attributed to the clay particle reorientation parallel to the direction of shearing (Lambe and Whitman 1979; Bromhead 1992). While cohesion provides much of the peak strength, the material has little cohesion once a shear surface is formed (Skempton 1964). Residual strength has been correlated with soil index properties such clay content and Atterberg limit by many researchers (e.g., Voight 1973; Kanji 1974; Lupini et al. 1981; Mesri and Cepeda-Diaz 1986; Collotta et al. 1989; and Stark and Eid 1994). Residual strength is often related to long-term stability problems and for areas with landslide history, bedding planes or folded strata (Skempton 1985). The drop in residual strength from peak strength may cause reactivation of old landslides.

Residual strength parameters are often determined using a rotational ring shear test device. A few types of ring shear apparatus have been reported by Hvorslev (1939), La Gatta (1970), Bishop et al. (1971) and Bromhead (1979). The Bromhead ring shear apparatus (Figures 6 and 7) has become widely used due to its simplicity in operation compared to other previous models. A full description of the apparatus can be found in the technical literature by WF Engineering (1988). In the apparatus, the ring shaped specimen has an internal diameter of 7 cm and an external diameter of 10 cm. Drainage is provided by two porous bronze stones fixed to the upper platen and to the bottom of the container.

Currently, a few testing procedures have been proposed for the use of the Bromhead ring shear apparatus. Stark and Vetell (1992) have shown that the single stage test procedure provides a good estimation of the residual strength at effective normal stress less than 200 kPa. When the effective normal stress is greater than 200 kPa, consolidation of the specimen during the test causes settlement of the upper platen into the lower platen giving higher residual strength values. Anayi et al. (1988) have pointed out that in the preshearing test procedure, the preshearing facilitates the creation of a shear plane and reduces the amount of length of the horizontal displacement required to reach the residual condition. This procedure causes extrusions of a substantial amount of soil during the shear process and therefore, as in the case of the single stage test procedure, gives higher measured residual strength values. Stark and Vetell (1992) also concluded that in the multistage test procedure an additional strength, probably due to wall friction as the top platen settles into the specimen container, develops during consolidation and shear process; hence they proposed the flush test procedure in which, increasing the thickness of the specimen prior to shear reduces the wall friction and gives more trustworthy measured values. This procedure takes substantial time to reach the residual condition when it is conducted at low rate of displacement. In this study, the test procedures (multistage test procedures) described in ASTM (2002g) (D6467-99) were adopted to determine the residual strength of soils. The soil specimen is pre-sheared at a relatively large displacement rate and followed by subsequent shearing under small displacement rate under a few different normal stresses. The plot of shear stress versus normal stress gives the Mohr-Coulomb failure envelope and the residual shear strength parameter values.

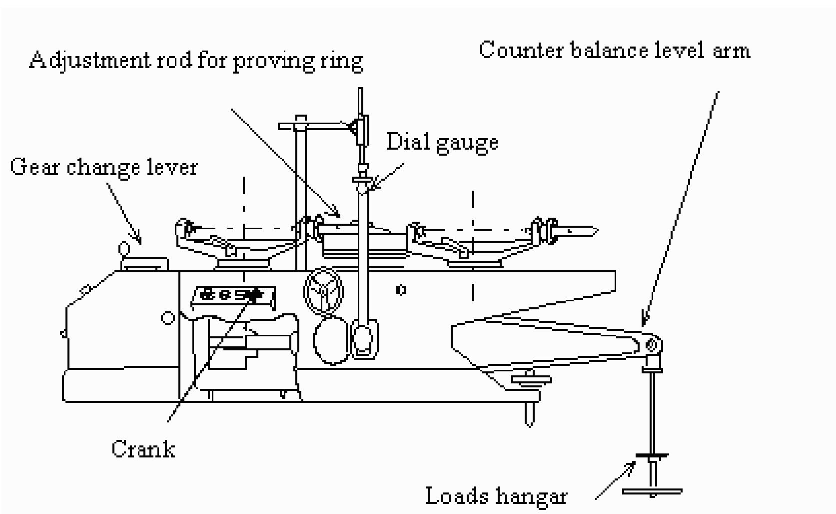


Figure 6. Schematic diagram for the Bromhead ring shear apparatus (Kakou et al. 2001)

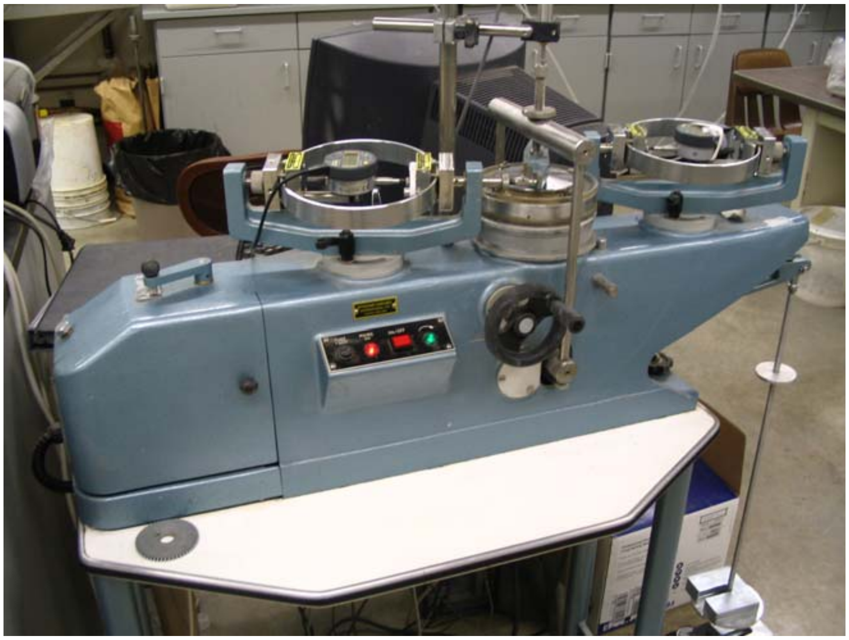


Figure 7. Photograph of the Bromhead ring shear apparatus

Factor of Safety and Limit Equilibrium Slope Analysis

Factor of Safety

Once the slope geometry and subsoil conditions of a slope have been determined, stability of a slope can be evaluated using either published chart solutions or a computer analysis. The primary objectives of a slope stability analysis normally include: (1) to evaluate how safe a slope is, or to calculate the factor of safety for a slope before its failure; and (2) to find out the failure mechanism if a slope has failed in order to provide necessary information for the remedial design.

Stability of a slope is usually analyzed by methods of limit equilibrium, and the factor of safety over the so-called critical slip surface is computed. The factor of safety is defined as the ratio between the shear strength and the shear stress required for the equilibrium of the slope:

$$\text{Factor of Safety} = \frac{\text{Shear strength}}{\text{Shear stress required for equilibrium}} \tag{1}$$

which can be expressed as

$$F = \frac{c + \sigma \tan\phi}{\tau_{eq}} \tag{2}$$

where F = factor of safety, c = soil cohesion, ϕ = soil friction angle, σ = normal stress on the slip surface, and τ_{er} = shear stress required for equilibrium.

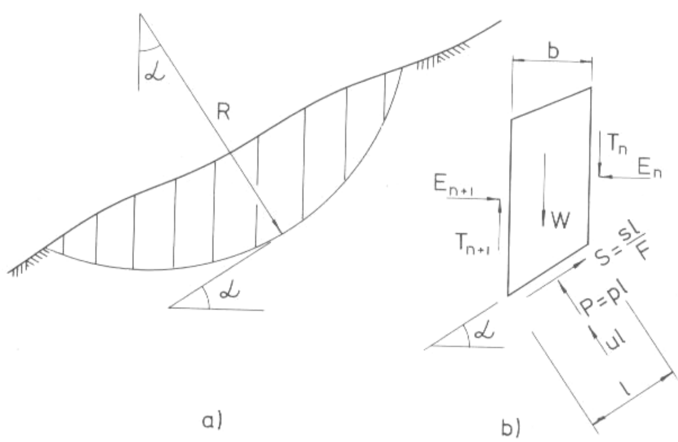
Deterministic slope stability analysis as obtained through equilibrium analysis computes the factor of safety based on a fixed set of conditions and material parameters. In practice, however, there involve many sources of uncertainty in slope stability analysis, e.g., spatial uncertainties (site topography and stratigraphy, etc.) and data input uncertainties (in-situ soil characteristics, soil properties, etc). Probabilistic slope stability analysis allows for the consideration of such uncertainty and variability of the input parameters. Since Borehole Shear Test, which can produce large amount of soil shear strength data in short time, will be the primary in-situ investigation method in the study, it will be an advantage to perform probabilistic analysis to account for the shear strength variability. Handy (1986) illustrated the possible application of probabilistic analysis involving the use of shear strength parameters obtained from BST in a case study.

The details of the equilibrium analysis and probabilistic analysis for slope stability are discussed in the following sections.

Limit Equilibrium Slope Analysis

In equilibrium analysis, the potential sliding mass is subdivided into a series of slices (Figure 8), and a general limit equilibrium formulation (Fredlund et al. 1981; Chugh 1986) can be used in the factor of safety computation. The equations of static that can be generated include

1. Summation of forces in a vertical direction for each slice, where the resulted equations are solved for the normal forces at the bases of the slices;
2. Summation of forces in a horizontal direction for each slice is used to compute the interslice normal forces, where the resulted equations are applied in an integration manner across the sliding mass;
3. Summation of moments about a common point for all slices, where the resulted equations can be rearranged and solved for the moment equilibrium factor of safety, F_m; and
4. Summation of forces in a horizontal direction for all slices, giving rise to a force equilibrium factor of safety, F_f.



(a) Division of sliding mass into slices (b) Forces acting on a typical slice

Figure 8. Method of slices for slope analysis (Chowdhury 1978)

Even with the above static equations, the analysis is still indeterminate, and a further assumption is made regarding the direction of the resultant interslice forces. The direction is assumed to be described by an interslice force function. The factors of safety can then be computed based on moment equilibrium (F_m) and force equilibrium (F_f). These factors of safety may vary depending on the percentage of the interslice force function used in the computation.

Using the same general limit equilibrium formulation, it is also possible to specify a variety of interslice force conditions and satisfy only the moment or force equilibrium conditions. The assumptions made to the interslice forces and the selection of overall force (F_f) or moment (F_m) equilibrium in the factor of safety equation, give rise to the various methods of analysis. A rigorous method satisfies both moment and force equilibrium ($F_f = F_m$).

The available computational methods for slope stability include: (1) Ordinary method of slices (Fellennius 1927); (2) Bishop (1995) simplified method; (3) Janbu (1968) simplified method; (4) Lowe and Karafiath (1960) method; (5) Modified Swedish method (US Army Corps of Engineers 1970); (6) Spencer (1967) method; (7) Bishop (1955) rigorous method; (8) Janbu (1968) generalized method; (9) Sarma (1973) method; and (10) Morgenstern-Price method (Morgenstern and Price 1965). These available methods are categorized by the assumptions made for solving the equations generated in the methods of slices. Fredlund and Krahn (1977), Duncan (1996) and Abramson et al. (2002) made a comprehensive review and summary on these computational methods.

Among the 10 methods that can be used to determine the factor of safety, the simplified Bishop (1955) method, Janbu (1968) method and Morgenstern-Price (1965) method are popular because factor of safety value can be quickly calculated for most slip surfaces (Abramson et al. 2002). However, factor of safety generally varies depending on the selected slip surface. Therefore it is essential to perform a complete, iterative search for the critical slip surface to ensure obtaining the minimum factor of safety, regardless of the computation method of analysis (Duncan 1996).

Probabilistic Slope Analysis

Probabilistic slope stability analysis quantifies the probability of failure of a slope. In general, the input parameters in a probabilistic analysis are considered as the mean values of the parameters, and the variability of the parameters can be specified by entering the standard derivations of the parameters.

Normal Distribution Function

Since soils are naturally formed materials, consequently their physical properties vary from point to point. The variability of soil properties is a major contributor to the uncertainty in the stability of a slope. Laboratory results on natural soils indicate that most soil properties can be considered as random variables conforming to the normal distribution function (Lumb 1966; Tan 1993), which is often referred to as the Gaussian distribution function that is written as:

$$\mathrm{f(x)} = \frac{1}{\sigma\sqrt{2\pi}} \exp\left[-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2\right] \tag{3}$$

where f(x) = relative frequency; σ = standard deviation; and μ = mean value.

A normal curve is bell shaped, symmetric and with the mean value exactly at middle of the curve. A normal curve is fully defined when the mean value, μ and the standard deviation, σ are known. Theoretically, the normal curve will never touch the x axis, since the relative frequency, f(x), will be nonzero over the entire range. However, for practical purposes, the relative frequency can be neglected after ±5 times standard deviation, σ, away from the mean value.

Statistical Analysis

In slope stability analysis, trial factors of safety are assumed to be normally distributed. As a result, statistical analysis can be conducted to determine the mean, standard deviation, the probability density function and the probability distribution function of the slope stability problem. The equations used in the statistical analysis are summarized as follows (Lapin 1983):

Mean factor of safety, μ: $$\mu = \frac{1}{n}\sum_{i=0}^{n} F_i \tag{4}$$

Standard deviation, σ: $$\sigma = \sqrt{\frac{1}{n}\sum_{i=0}^{n}(F_i - \mu)^2} \tag{5}$$

Probability density function: $$\mathrm{f(F)} = \frac{1}{\sigma\sqrt{2\pi}}\exp\left[-\frac{1}{2}\left(\frac{F-\mu}{\sigma}\right)^2\right] \tag{6}$$

Probability distribution function:

$$\mathrm{f(F)} = \mathrm{P}\left[\mathrm{X} \le \mathrm{F}\right] = \int_{-\infty}^{F}\left\{\frac{1}{\sigma\sqrt{2\pi}}\exp\left[-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2\right]\right\}dx \tag{7}$$

where F_i = the trial factors of safety; n = number of trial factors of safety; and F = factor of safety. An example of probability density function and the corresponding probability distribution function are presented in Figures 9 and 10, respectively.

Probability of Failure and Reliability Index

A factor of safety is really an index indicating the relative stability of a slope. It does not represent the actual risk level of the slope due to the variability of input parameters. With probabilistic analysis, two indices, which are known as probability of failure and reliability index, are available to quantify the stability or the risk level of a slope.

The probability of failure is the probability of obtaining a factor of safety less than 1.0, as illustrated in Figure 10. It is computed by integrating the area under the probability density function for factors of safety less than 1.0. The probability of failure can be interpreted in two ways: (1) if a slope were to be constructed many times, what percentage of such slopes would fail; or (2) the level of confidence that can be placed in a design (Mostyn and Li 1993). Nevertheless, the probability of failure is a good index showing the actual level of stability of a slope. In addition, there is also no direct relationship between factor of safety and probability of failure. In other words, a slope with a higher factor of safety may not be more stable than a slope with a lower factor of safety (Harr 1987). For example, a slope with factor of safety of 1.5 and a standard deviation of 0.5 will have a much higher probability of failure than a slope with factor of safety of 1.2 and a standard deviation of 0.1.

The reliability index provides a more meaningful measure of stability than the factor of safety. The reliability index (β) is defined in terms of the mean (μ) and the standard deviation (σ) of the trial factors of safety as (Christian et al. 1994):

$$\beta = \frac{|\mu - 1.0|}{\sigma} \tag{8}$$

The reliability index describes the stability of a slope by the number of standard deviations separating the mean factor of safety from its defined failure value of 1.0. It can also be considered as a way of normalizing the factor of safety with respect to its uncertainty. When the shape of the probability distribution is known, the reliability index can be related directly to the probability of failure.

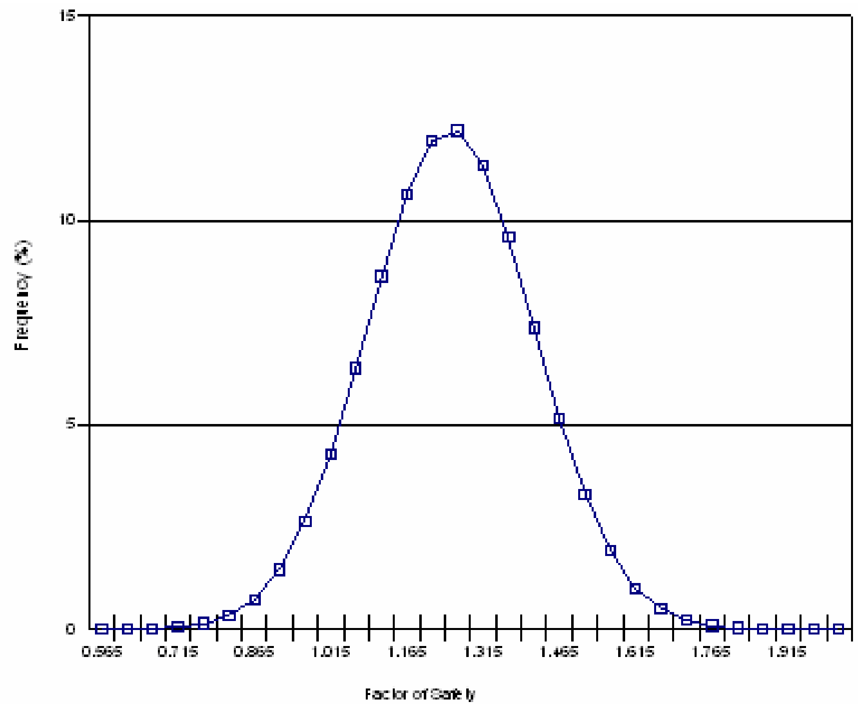


Figure 9. Probability density function (Geo-Slope 2004)

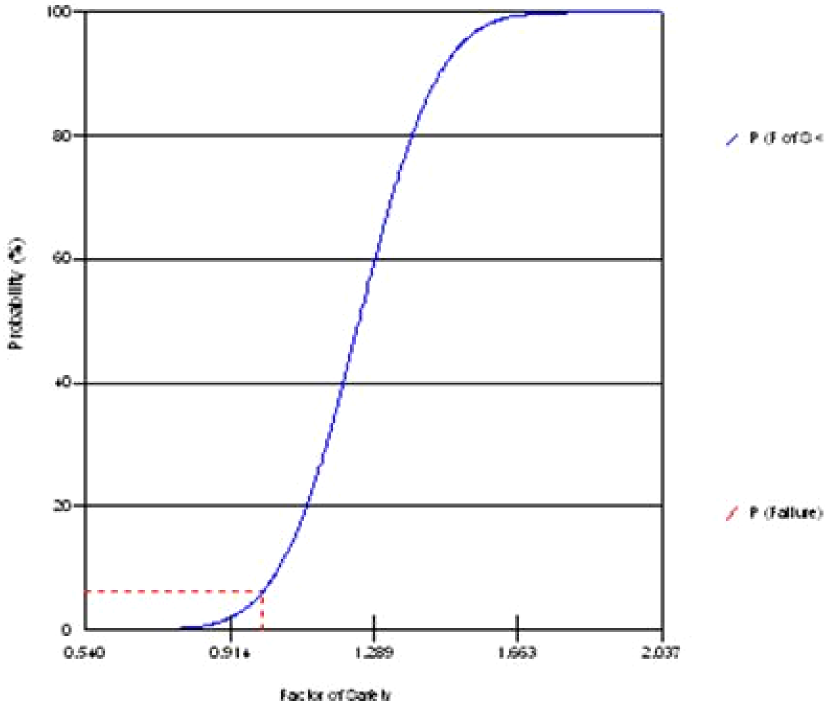


Figure 10. Probability distribution function (Geo-Slope 2004)

Monte Carlo Method

Probabilistic slope stability analyses can be performed using a few methods. One simple but versatile computational procedure is the Monte Carlo simulation (e.g., Tobutt, 1982; Hammond et al. 1992; Chandler 1996) which involves (1) the selection of a deterministic solution procedure; (2) decisions regarding which input parameters are to be modeled probabilistically and the representation of their variability in terms of a normal distribution model using the mean value and standard deviation; (3) the estimation of new input parameters and the determination of new factors of safety many times; (4) the determination of some statistics of the computed factor of safety, the probability density and the probability distribution of the problem.

The critical slip surface is first determined based on the mean value of the input parameters using any of the limit equilibrium methods. Probabilistic analysis is then performed on the critical slip surface, taking into consideration the variability of the input parameters. The variability of the input parameters is assumed to be normally distributed with specified mean values and standard deviations.

During each Monte Carlo trial, the input parameters are updated based on a normalized random number. The factors of safety are then computed based on these updated input parameters. By assuming that the factors of safety are also normally distributed, the mean and the standard deviations of the factors of safety are determined. The probability distribution function is then obtained from the normal curve. The number of Monte Carlo trials in an analysis is dependent on the number of variable input parameters and the expected probability of failure. In general, the number of required trials increases as the number of variable input increases or the expected probability of failure becomes smaller. It is not unusual to do thousands of trials in order to achieve an acceptable level of confidence in a Monte Carlo probabilistic slope stability analysis (Mostyn and Li 1993).

CASE HISTORIES OF SLOPE IN IOWA

In this chapter the investigation of 15 case histories is described. The chapter leads off with an overview of the methods used to investigate the slopes, including the filed and laboratory methodologies used and a description of the analyses undertaken.

Overview of the Study

This study includes 15 case histories of slope in total, which are located besides Highways 34, 169, E57, and 63, involving counties of Monroe, Wapello, Madison, Union, and Boone in Iowa. The locations of the slopes are shown in Figure 11, and the overall information for the slopes is summarized in Table 1. The circles and numbers in the figure indicate the approximate locations and slope numbers in the study, respectively. The major field investigations were carried out between August 2003 and November 2004, and the main laboratory tests were conducted between August 2004 and May 2005.

The slopes are mainly comprised of either clay shale or glacial till, which are commonly encountered in Iowa. Among the 15 slopes, one is a proposed embankment slope that is currently under design (Slope 15, Sugar Creek Project); one is a slope that is not failed (Slope 4); the remainders are all considered failed with apparent failure features. The failed slopes include both embankment slopes (comprising compacted fill) and back-slopes (formed by cutting). The slopes in the study are generally gentle and of small scale with slope angle ranging from 11^o to 23^o and height ranging from 6 to 23m (Table 1).

Extensive field investigations and laboratory tests were performed for the slopes. Field investigations include measurement of slope geometry, boring and soil sampling, in-situ Borehole Shear Test (BST) and groundwater table measurement. Mechanical drilling of boreholes using rotary drilling rig was mainly concentrated on Slope 15 (Sugar Creek Project). A total of 10 boreholes were drilled by CH2M Hill. Slope 7 (Winterset) also has two mechanically drilled boreholes. The remainders of the boreholes for the study were drilled manually using a hand auger due to the site restraints. The mechanically drilled boreholes were as deep as 12m, while the manually drilled boreholes could only reach a maximum depth of 4.2m (14 ft) with 3 to 3.6 m for most cases. These depths appear to be sufficient to provide the necessary subsurface information for the slopes since most of the slides in the study are of relatively small scale (Table 1).

The number of the borehole drilled for each of the slopes ranged from 1 to 4 except for Slope 15, and the number of BSTs performed in each borehole also varied from 1 to 4, both of which depended on the complexity of the site conditions. Ground water levels were monitored and measured, normally within 2 days after boring. The BSTs provided in-situ shear strength parameter values of the soils, which are necessary for the slope analysis together with the ground water conditions.

Laboratory investigations mainly comprised ring shear tests, basic property tests (grain size analysis and Atterberg limits test), mineralogy analyses, soil classifications, natural water contents and density measurements on the representative soil samples from each slope. Extensive direct shear tests and a few triaxial compression tests, unconfined compression tests and consolidation tests were also performed on undisturbed soil samples for Slope 15. All these lab investigations provided further information for the slope study.

Based on the results of field and lab investigation, each slope was analyzed to evaluate the possible factors causing the slope failure or the potential slope instability using limit equilibrium method. Slope stability analyses were performed for all the slopes. Probabilistic analysis was also performed for Slope 15 (Sugar Creek Project) due to the relatively large amount of soil parameters obtained. The computer program Slope/W (Geo-slope 2004) was used to perform all the computations.

For the slopes, three types of soil shear strength parameter values were obtained, which were the in-situ soil strength parameter values from the BST, residual shear strength parameter values from ring shear test, and the possible mobilized shear strength parameter values at failure from back-calculation. The possible failure surface was also estimated based on the failure features of each slope together with back-calculations. This information should be useful when designing a remediation measure using piles. For example, for a slope failure with relatively large displacement, the residual shear strength can be considered along the failure surface, and the piles need to be penetrate through the failure surface. More information can be found in Volume III of the report.

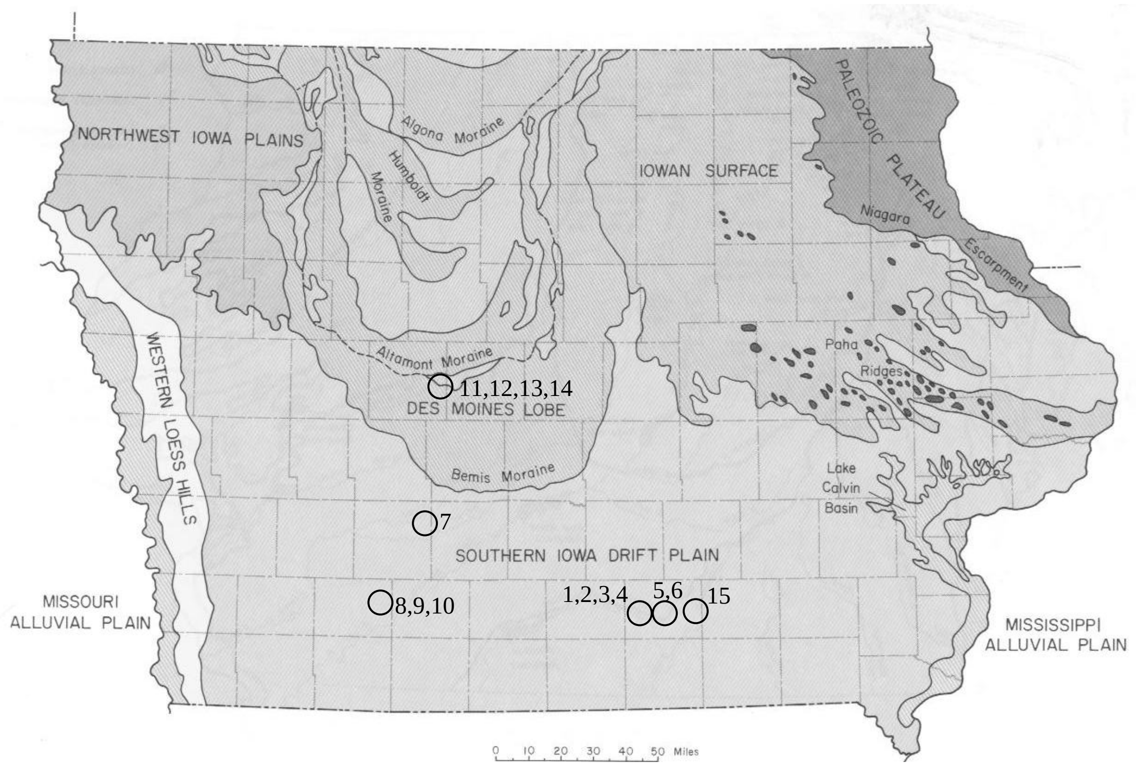


Figure 11. Landform regions of Iowa (Prior 1991) and locations of the slopes investigated

Table 1. Summary of the slopes in the study

Slope	County	Hwy	Location	Type of Slope	Slope Geometry					Site Geology	Nos. of Bore-holes	Nos. of BST
					H:V (V=1)	Slope Angle (deg.)	Max. Height (m)	Max. Length (m)	Width (m)			
1	Monroe	34	MP169.3	Fill	3.3	17	9	30	20	shale and glacial till	1	2
2		34	MP171.7, 3 miles west of Albia	Cut	5.2	11	8	40	70	shale	4	9
3		34	MP175.3	Fill	2.5	22	6	16	20	glacial till	1	4
4		34	MP175.5	Fill	2.5	22	7	18	15	glacial till	1	2
5	Wapello	34	MP178.3	Fill	3.0	18	7	20	25	glacial till	1	4
6		34	MP178.3	Fill	4.0	14	6	22	30	shale	1	1
7	Madison	169	3 miles north of Winterset	Cut	4.4	13	7	33	60	shale	4	9
8	Union	169	2 miles south of Afton	Cut	2.5	22	10	27	60	glacial till	2	4
9		169	2 miles south of Afton	Cut	2.4	23	13	33	40	Mainly shale; glacial till at surface	1	2
10		169	4 miles south of Afton	Cut	2.8	20	7	21	25	shale	1	1
11	Boone	E57	0.5 mile west of Des Moines River, 4.5 miles west of Luther	Cut	3.5	16	23	85	80	glacial till	3	4
12		E57		Cut	3.0	18	20	63	70	glacial till	3	4
13		E57		Cut	3.0	18	16	58	10	glacial till	1	1
14		E57		Fill	4.6	12	10	47	30	glacial till	1	1
15	Wapello	63	Sugar Creek, Ottumwa	Fill (proposed)	3.0	18	19	59	60	silty clay /weathered shale	10	35

General Information

The information on the area geology for each slope was obtained from the Soil Survey Reports (USDA 1975, 1978, 1981a, 1981b, 1984). The details of history of the slopes were generally not well documented. The relevant information was acquired through personal communications with staff of Soils Design of IaDOT and the residents nearby the slopes.

BSTs and ring shear tests were performed according to the procedures as described in Chapter 2. Direct shear tests were performed following ASTM (2002d) (D3080-03) (Standard test Method for Direct Shear Test of Soils under Consolidated Drained Conditions) on undisturbed soil samples under saturated conditions. A shearing rate of 0.02 mm/min was applied, which was sufficiently low for the test to produce effective shear strength parameters. The triaxial tests were performed in accordance with ASTM (2002e) (D4767-95) (Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils), and the unconfined compression tests with ASTM (2002f) (D2166-00) (Standard Test Method for Unconfined Compressive Strength of Cohesive Soil).

The grain size distributions and Atterberg limits of the soil samples were determined following ASTM (2002a) (D422-63) (Standard Test Method for Particle-Size Analysis of Soils) and ASTM (2002b) (D4318-00) (Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils), respectively. The soils were classified using methods in ASTM (2002c) (D2487-00) (Standard Classification of Soils for Engineering Purposes (Unified Soil Classification System)). Soil density was determined using small, relatively "undisturbed" soil samples when obtained from hand auger. Densities were taken for a number of samples and the average value was taken as the representative density value.

X-ray diffraction (XRD) analysis is commonly used to determine the composition of a material. Detailed information about XRD can be found in many literatures such as Cullity (1978) and Moore and Reynolds (1997). In this study, XRD analyses were conducted to investigate the soil clay mineralogy. Random oriented bulk sample from air dry soil was used in the test, and the corresponding x-ray diffractogram was generated. The minerals identified are summarized at the bottom of a diffractogram.

For the slope analyses, Morgenstern and Price (1965) method and Bishop (1955) simplified method were adopted due to their popularity and familiary. The factors of safety (FS) obtained from these two methods were found to be essentially same as can be seen in the results. The computations of the slope analysis were performed using the computer program SLOPE/W (Geo-slope 2004). FS was calculated on different slip surfaces and the minimum FS was determined. Three types of slip surface could be searched or defined in SLOPE/W, which included circular, block specified and fully specified slip surface. Circular search of slip surface was performed by the program with the range of the center and radius for the slip circle being defined. A block-specified slip surface consisted of several line segments defined by two grids of intersection points. Slip surfaces were created by connecting each point in the left block with each point in the right block, and then projecting each point to the surface at specified angles. This type of slip surface was suitable to "guide" the slip surface passing through a specified soil range. The fully specified slip surface was most suitable for a known or observed slip surface.

Slope 1 (HWY34 MP169.3)

Site Conditions

Location

The slope is a fill slope and is located at the north side of Highway 34 MP169.3, Monroe County (Figure 12).

History

The exact time of failure was unknown. Failure or deformation may mostly have occurred during 2000-2003. No evidence of fresh movement was found when it was investigated in July 2004.

Area Geology

According to the USDA (1984) Soil Survey Report, most of the soils in Monroe County formed in loess, glacial till, or alluvium. A few of the soils formed in colluvium, eolian sand or shale residuum. The major Pleistocene deposits are glacial tills ranging from 0 to more than 90m in thickness. Shale residuum is the oldest parent material in the county. The shale consists of a series of beds deposited during the Des Moines sedimentary cycle in the Pennsylvanian period. These beds include shale of different colors and textures, conglomerates, and a few organic layers such as layers of coal.

Field Investigations

Slope Geometry

The slope (Figure 13) had an overall sloping angle of about 17 degree (H:V = 3.3:1), a maximum length of 30 m and a maximum height of 9 m (Figure 14). The width of the slope was about 20 m (along the highway). The top of the slope had filled with a strip of stones as remediation measures when the slope was investigated. The strip was about 2 m wide and extended along the slope beside the highway. The depth of the stone fill was unknown. There also existed a gentle hump at the mid surface of the slope indication the failure of the slope. A slope profile that is perpendicular to the highway indicating the slip direction was developed (Figure 14)

Site Geology

A 3.0 m deep borehole was drilled manually in the slope. The borehole revealed that the slope was formed with mixture of backfilled light grey clay shale and brown glacial till. The soils were generally soft to medium stiff. The boring log is shown in Appendix (Figure A1).

Ground Water Level

Ground water level in the borehole was measured after boring and was found to be located near the bottom of the borehole. The depth of the ground water level near the toe of the slope was estimated to be in the range of 1.5 to 2.5 m as indicated by the moist ground surface (Figure 14).

Borehole Shear Test Results

BSTs were conducted at a depth of 2.1 m in the borehole, and the soil tested was most likely shale based on the field observation. The results are presented in Figure 15. The results showed that $\phi' = 10^o$ and c' = 8 kPa for the shale, which was relatively low. The residual shear strength parameters from BST was $\phi' = 6^o$ and c' = 12kPa.

Lab Investigations

Basic Properties

Basic properties for representative soil samples were investigated and the results are summarized in Table 2. The results show that both the shale and glacial till samples have very low sand content of less than 5%, clay content of less than about 50% and liquid limit around 40%. All the soils are classified as low plasticity clay (CL) by USCS.

Ring Shear Test Results

A ring shear test was conducted for the glacial till sample from a depth of 2.4 m in the borehole and the result is presented in Figure 16. The result indicated that the soil had residual friction angle ϕ_r' of 22.1^o with small c_r' of 3.2 kPa.

Mineralogy and Morphology

The x-ray diffractogram (XRD) for the random oriented bulk shale sample at depth of 0.3m in the borehole is given in Appendix Figure A2. The minerals identified are summarized at the bottom of the diffractogram, and include quartz, montmorillonite, kaolinite, illite, calcite and cristobalite.

Slope Analysis

Soil Properties

The field and lab test results show that the slope mainly consists of low plasticity clay shale and glacial till. Based on the field investigation, it appeared that the soils were mixed and could not be sorted into clear layers. Therefore, a uniform slope was assumed for slope stability analysis. A unit weight of 18.0 kN/m^3 as determined in lab was used.

Method of Slope Analysis

In the slope analysis, the slip surface was assumed to be circular (Figure 17) because circular slip surface gave the minimum factor of safety (FS). Back-calculations were also performed to determine the average shear strength of the soil giving unity FS.

Results and Discussions

Three analyses were performed using different shear strength parameter values for the soil (Table 3), and slip surface corresponding to minimum FS passed through the top of slope where the failure zone existed (filled with stones). The results show that FS was close to 1.0 using shear strength parameter values obtained from BST. FS was about 1.5 using shear strength parameter values obtained from ring shear test. The back-calculated shear strength values for FS = 1.0 was essentially the same as the shear strength parameter values from BST. These results suggested that the slope was most likely unstable when it was investigated as FS was close to 1.0 based on the information from BST. The instability was further suggested by the repair work (rip-rap) at the top of the slope. The instability was due to the relatively low shear strength of the soil as exhibited by the clay shale; and also possibly due to the relatively high ground water level.

Conclusions

BST was used to obtain the shear strength parameter values of the soil in the slope. The shale in the slope had a relatively low shear strength with $\phi' = 10^{o}$ and $c' = 8$ kPa. These values together with the measured ground water condition were used to evaluate the stability of the slope. The factor of safety was found to be close to 1.0, and the potential slip surface was circular passing through near the top of the slope where failure zone already existed. The analyses indicated the slope was close to an unstable state when it was investigated, and this was probably mainly due to the low shear strength of the shale measured and the relatively high ground water table in the slope.

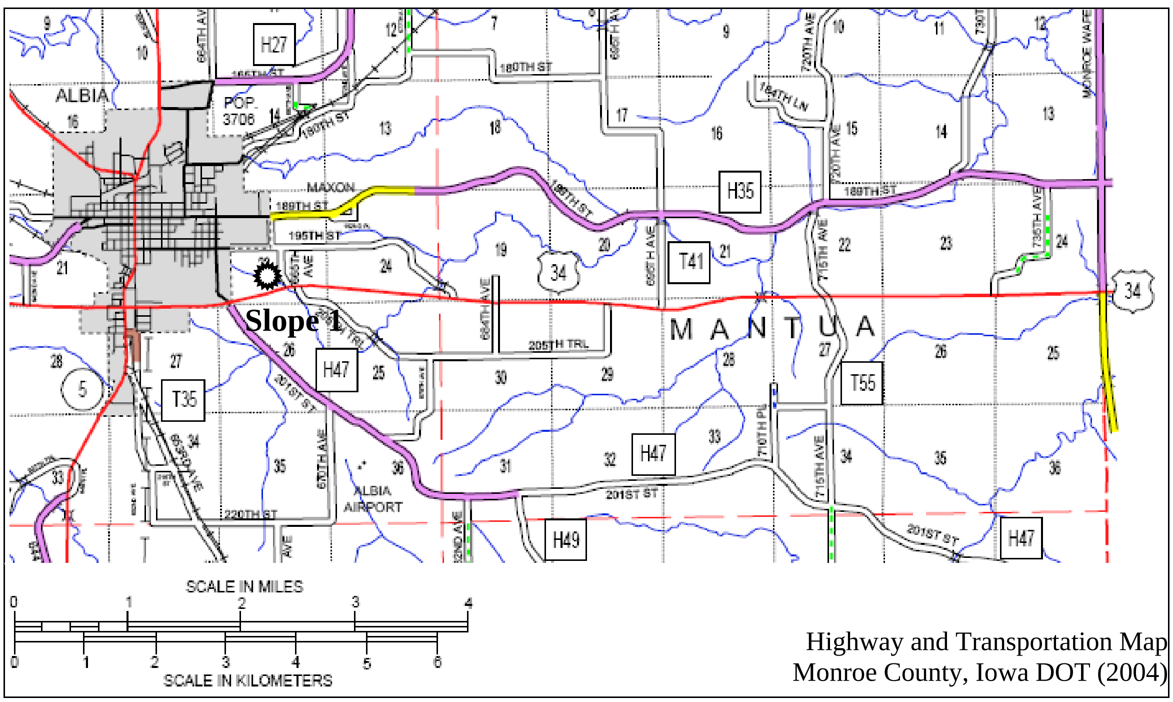


Figure 12. Location of slope 1 (Hwy 34, MP 169.3, Monroe Co.)

(a) Looking east, showing the rip-rap (photo taken by Thompson, 10/27/03)

(b) Looking south (photo taken by Yang, 08/12/05)

(c) Looking east, upward, close view of the slope (photo taken by Yang, 04/01/05)

Figure 13. Photographs for slope 1

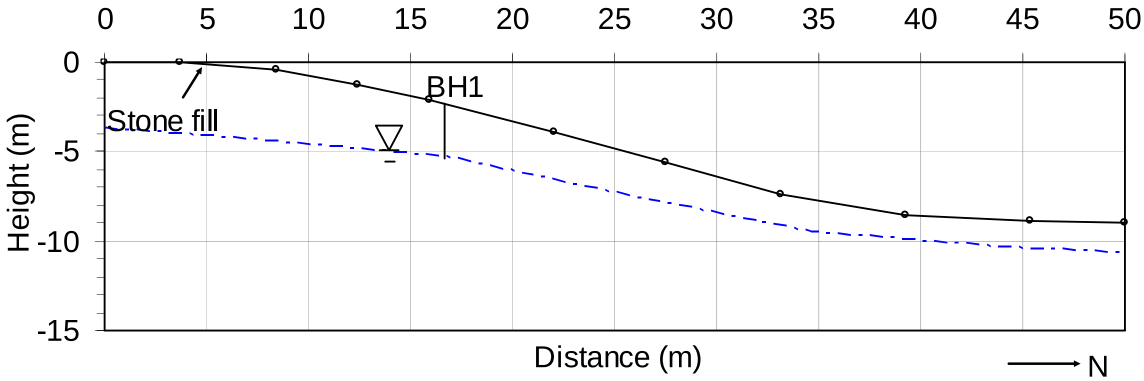


Figure 14. Cross-section for slope 1

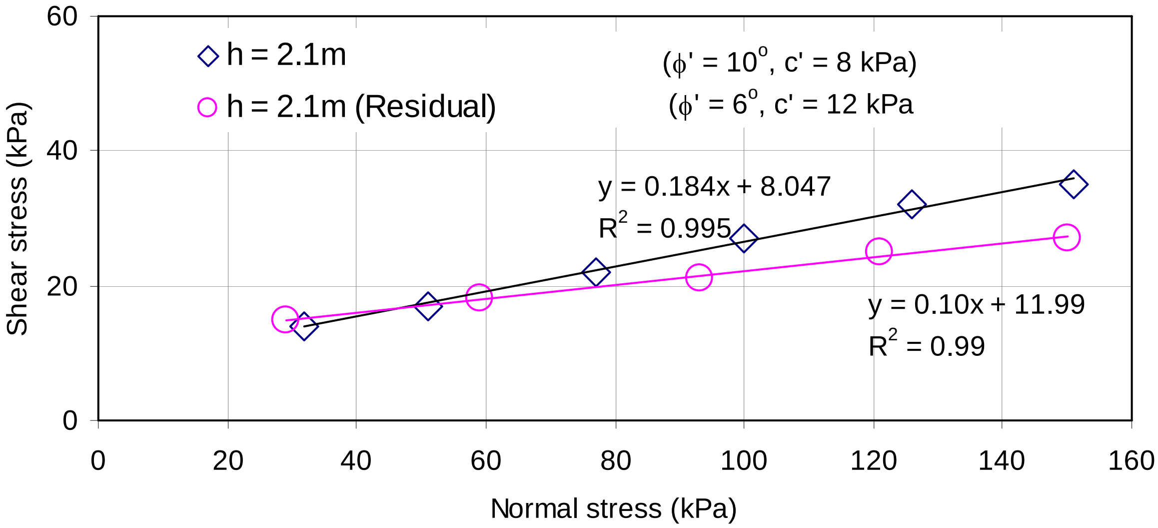


Figure 15. BST results for the shale in slope 1

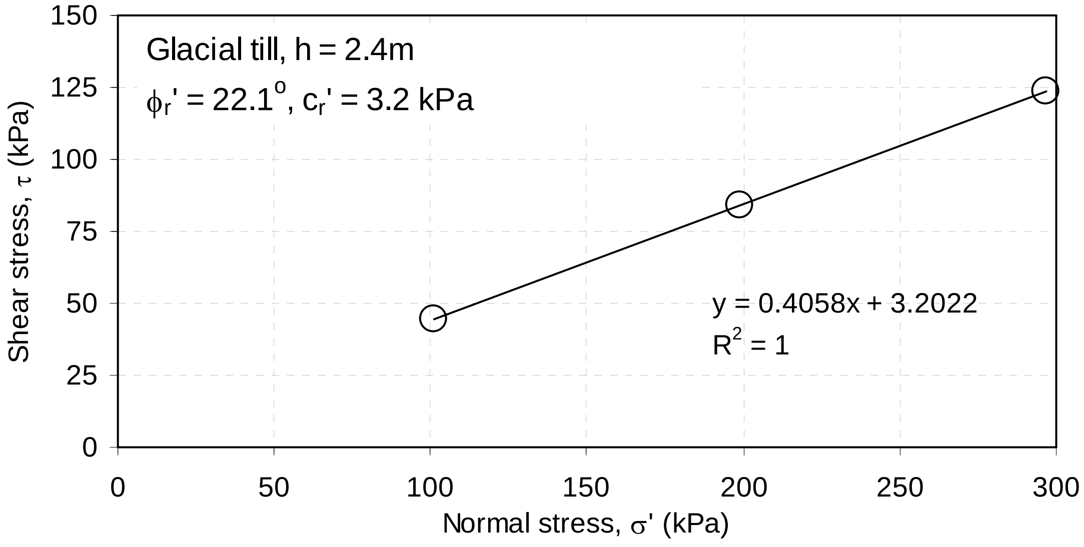


Figure 16. Ring shear test results for the glacial till in slope 1

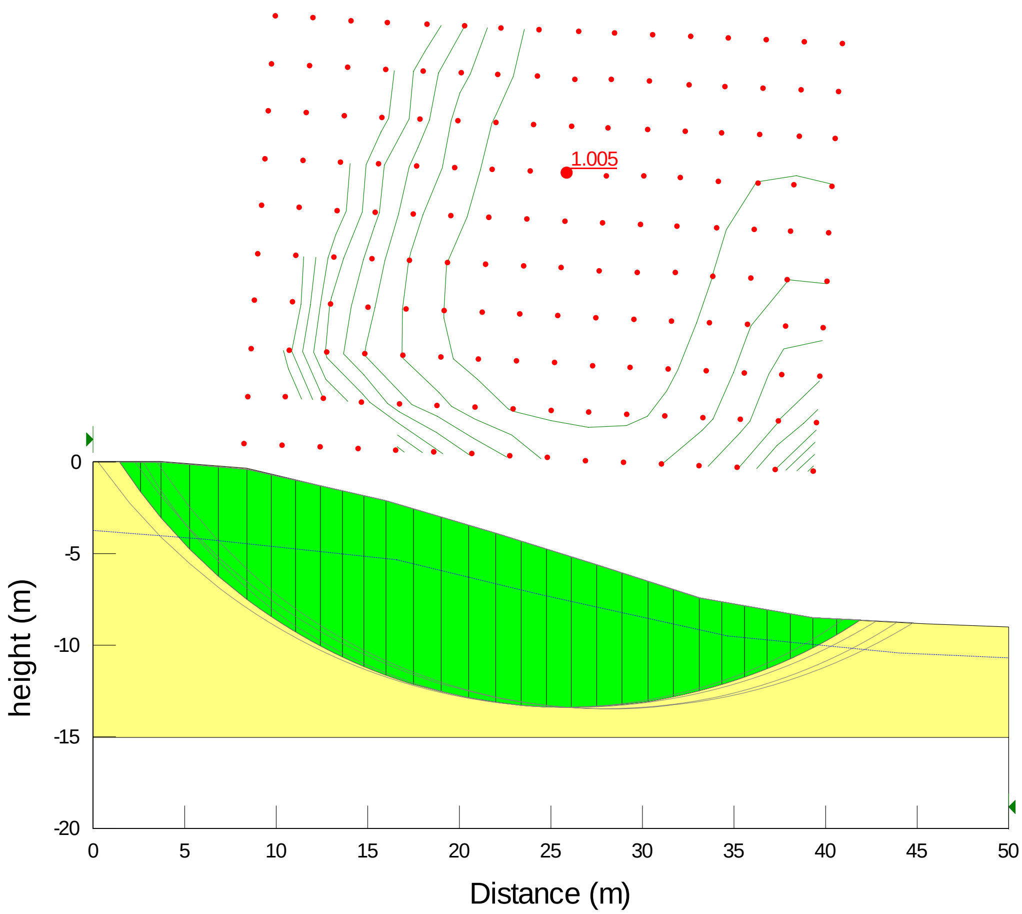


Figure 17. Stability analysis for slope 1

Table 2. Summary of basic properties for soils in slope 1

Soil	Depth (m)	Grain Size			Atterberg Limit			Classification		Water Content (%)	Total density (kN/m3)
		Sand (%)	Silt (%)	Clay (%)	LL (%)	PL (%)	PI (%)	USCS	AASHTO		
Shale	0.3	3	62	35	45	24	21	CL	A-7-6	27.8	17.1
Glacial till	2.4	5	66	28	39	20	19	CL	A-6	24.7	18.0

Table 3. Summary of slope analysis results for slope 1

Analysis No.	Shear Strength			Factor of Safety	
	Source	ϕ' (deg.)	c' (kPa)	M-P	Bishop
1	BST	10	8	1.005	1.005
2	Ring Shear	22.1	3.2	1.525	1.522
3	Back-calculated	10	7.9	1.000	1.000

M-P: Morgenstern-Price method

Slope 2 (Hwy34 Mp171.7)

Site Conditions

Location

The slope is a cut slope and is located at the north side of Highway 34 MP171.7, three miles west of Albia, Monroe County (Figure 18).

History

The exact time of failure was unknown. Failure or deformation may have occurred prior to 2001 based on discussion with a nearby resident. The failure features of scarp and hump of the slide appeared quite old when it was investigated in July 2004.

Area Geology

According to the USDA (1984) Soil Survey Report, most of the soils in Monroe County formed in loess, glacial till, or alluvium. A few of the soils formed in colluvium, eolian sand or shale residuum. The major Pleistocene deposits are glacial tills ranging from 0 to more than 90m in thickness. Shale residuum is the oldest parent material in the county. The shale consists of a series of beds deposited during the Des Moines sedimentary cycle in the Pennsylvanian period. These beds include shale of different colors and textures, conglomerates, and a few organic layers such as layers of coal.

Field Investigations

Slope Geometry

The slope (Figure 19) had an overall sloping angle of about 11 degrees (H:V = 5.2:1), a maximum length of 40 m and a maximum height of 8m (Figure 20). It had a curved scarp near the top with a maximum height of 1.5 m. The scarp extended along the two wings of the slope and ended at the toe of the slope. The width of the slope (at the toe) was about 70 m (along the highway). There were a few small humps at the surface of the slope. There was also a small ditch located at the toe of slope. The ditch was parallel to the highway.

Site Geology

A total of four boreholes were drilled manually along the maximum length of the slope, and the direction of the profile is perpendicular to the highway (Figure 20). The boreholes showed that the slope was covered with about 0.15 m thick topsoil underlain with brown to grey, highly weathered shales. The shales were generally medium stiff to stiff, with the lower portion being soft to medium stiff. A thin layer of coal was found near surface in BH4 (near the toe of the slope). The boring logs are shown in Appendix (Figures A3 to A6).

Ground Water Level

Ground water level for each borehole was measured after boring and was shown on the slope profile (Figure 20). The ditch at the toe of the slope had a little flowing water, which might indicate a shallow ground water level near the toe.

Borehole Shear Test Results

BSTs were performed at various depths of the boreholes. The results are presented in Figure 21 and Table 4. The results show that ϕ' for the shales ranged from 11^o to 40^o, and c' varied from 7 to 22 kPa.

Lab Investigations

Basic Properties

Basic properties for representative soil samples were investigated and the results are summarized in Table 5. The results show that all the shale samples have very low sand content of less than 5%, high clay content of about 50% and liquid limit larger than 50%. All the shales are classified as high plasticity clay (CH) by USCS.

Ring Shear Test Results

Ring shear tests were conducted for two soil samples and the results are presented in Figure 22. The results indicated that all the tests gave consistent values of ϕ_r' ranging from 6^o to 7^o with small c_r' values.

Mineralogy and Morphology

The x-ray diffractogram (XRD) for the random oriented bulk shale sample at depth of 0.6m in BH4 is given in Appendix Figure A7. The minerals identified are summarized at the bottom of the diffractogram, which include quartz, montmorillonite, kaolinite, illite, gypsum, and cristobalite.

Slope Analysis

<u>Soil Properties</u>

The field and lab test results show that the slope mainly consists of high plasticity weathered clay shales. Based on the field visual inspections and BST results, the shales are interpreted as three layers as shown in the Figure 20, with the Layer 2 being relatively weak. Slope stability analysis is performed based on this interpretation, and the soil properties used in the analysis are shown in Table 6.

<u>Method of Slope Analysis</u>

In the slope analysis, the slip surface was specified as passing through the observed scarp and the weak shale layer of Layer 2 (Figure 23). Current slope geometry and geometry before failure were considered for the analysis; and both observed ground water conditions (low GWT) and assumed ground water conditions (high GWT) were used. The high GWT was located at the surface along the whole slope profile and represented the worst possible ground water condition. Back-calculations were also performed to determine the shear strength parameter values of the weak Layer 2 giving a unity factor of safety (FS).

<u>Results and Discussions</u>

The results of the slope analysis are given in Table 7, and the different shear strength failure envelopes including those obtained from back-calculations for the weak Layer 2 are presented in Figure 24. The results show that FSs are larger than unity under different conditions of GWT and slope geometry using shear strength parameter values obtained from BSTs (Analyses 1, 4, 7 and 10 in Table 7). FSs are close to unity or slightly less than unity under different conditions of GWT and original geometry using shear strength parameter values obtained from the ring shear test (Analyses 2, 5 8 and 11 in Table 7). The back-calculated shear strength parameter values (Analyses 3, and 9 in Table 7) are rather close to the residual shear strength parameter values from ring shear test; or have the same friction angles with those obtained from BST (for Analyses 6 and 12).

All these results suggest that the slope may have most likely failed under the conditions as in Analysis 12 in Table 7, i.e. the slope failure took place under a high GWT that was located near the surface. In this situation, the shear strength of the weak Layer 2 developed or mobilized has similar ϕ' value as measure from BST but with zero *c'* value. The BST results may represent the peak shear strength of the shale; and the back-calculated results, which has same ϕ' with that of BST and near zero c', indicated the softened shear strength (or mobilized shear strength) of the shale during the slope failure (Figure 24). The residual shear strength as obtained from the ring shear test was the ultimate shear strength corresponding to a slide with relatively large displacement.

Conclusions

BSTs were used to characterize the slope. An underlying relatively weak shale layer was detected. The shear strength parameter values obtained from BST for each soil layer were used for the slope analyses to investigate the possible causes of the failure.

The slope most likely failed under near surface GWT conditions with the slip surface passing through the relatively weak shale layer. The weak shale layer has minimum peak shear strength parameter values of $\phi' = 11^o$ and c' = 13 kPa as measured by BST. It has softened shear strength parameter values of $\phi' = 10.85^o$ and c' = 0 kPa during the slide mobilization as obtained-from back-calculation; and residual shear strength parameter values of $\phi' = 6.8^o$ and c' = 1.6 kPa as indicated by ring shear test. The different shear strength parameter values together with the location of the slip surface can be considered when slope remediation design is considered.

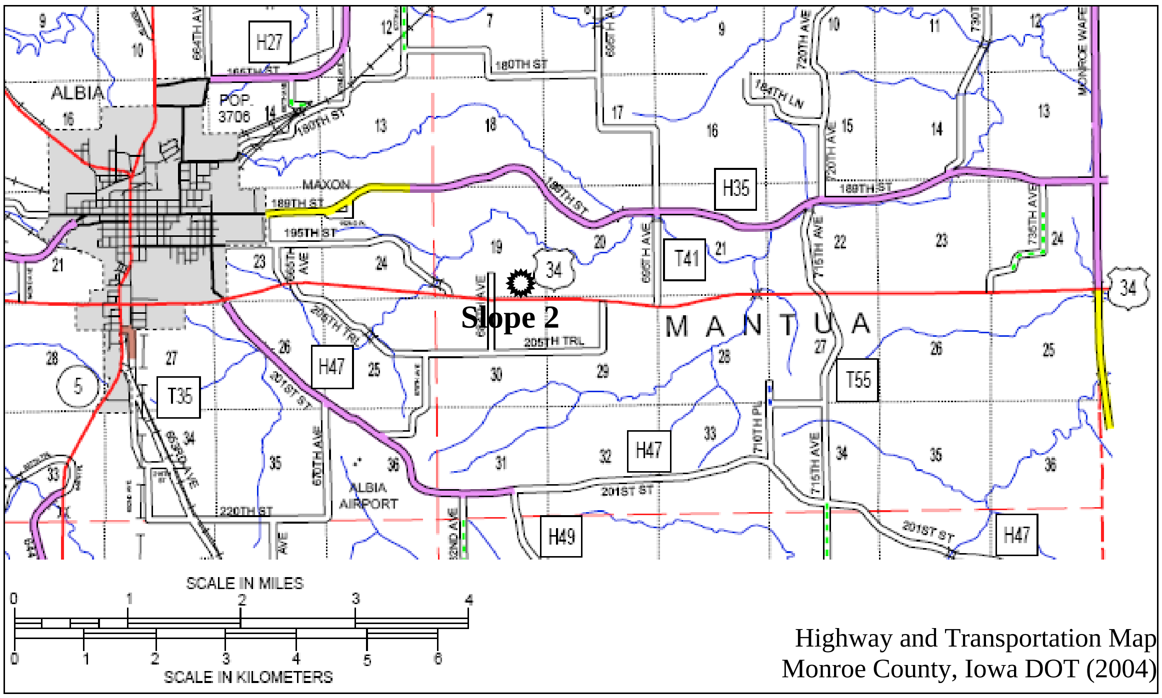


Figure 18. Location of slope 2 (Hwy 34, MP 171.7, Monroe Co.)

(a) Looking north, overview of the slope (photo taken by Yang, 04/01/05)

(b) Looking north, showing the scarp (photo taken by Thompson, 10/27/03)

(c) Looking east, showing the ditch at the toe of the slope (photo taken by Thompson, 10/27/03)

Figure 19. Photographs for slope 2

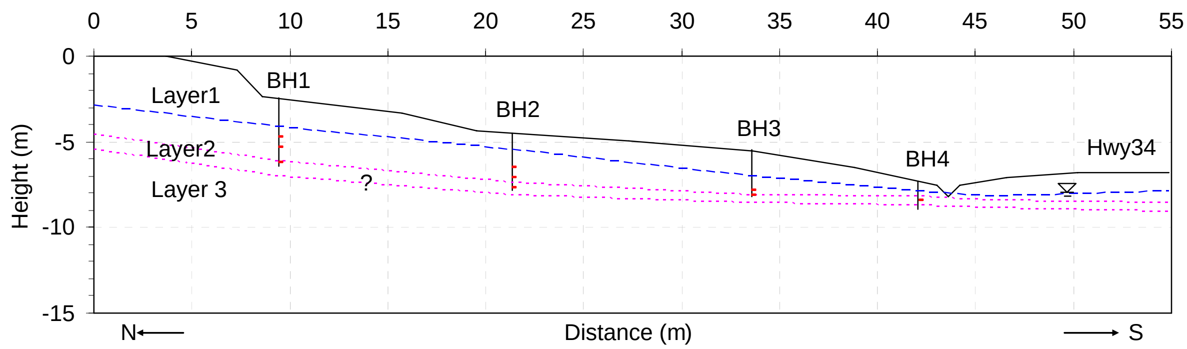


Figure 20. Cross-section for slope 2

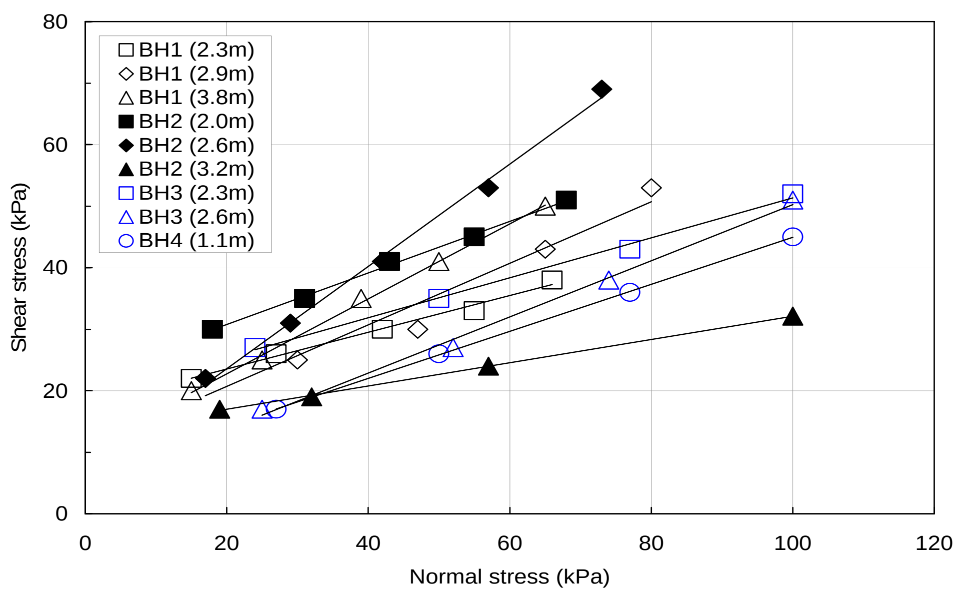


Figure 21. BST results for slope 2

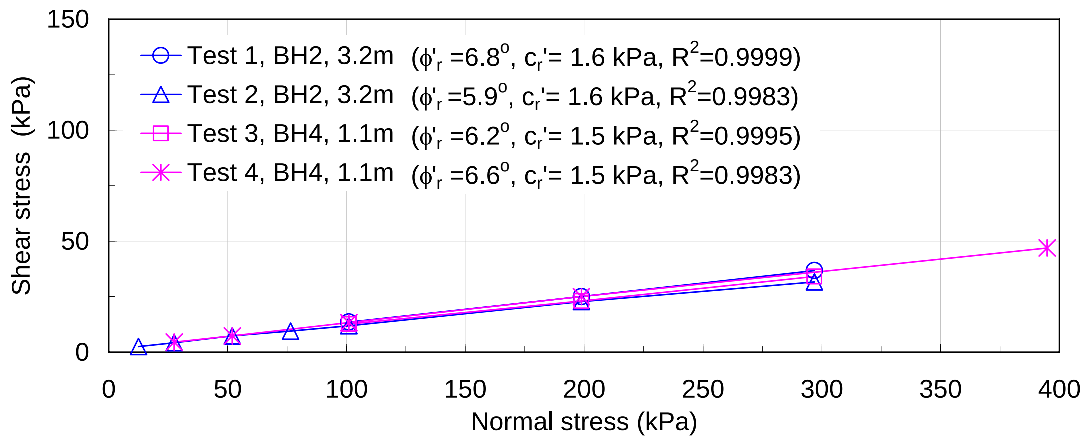


Figure 22. Ring shear test results for slope 2

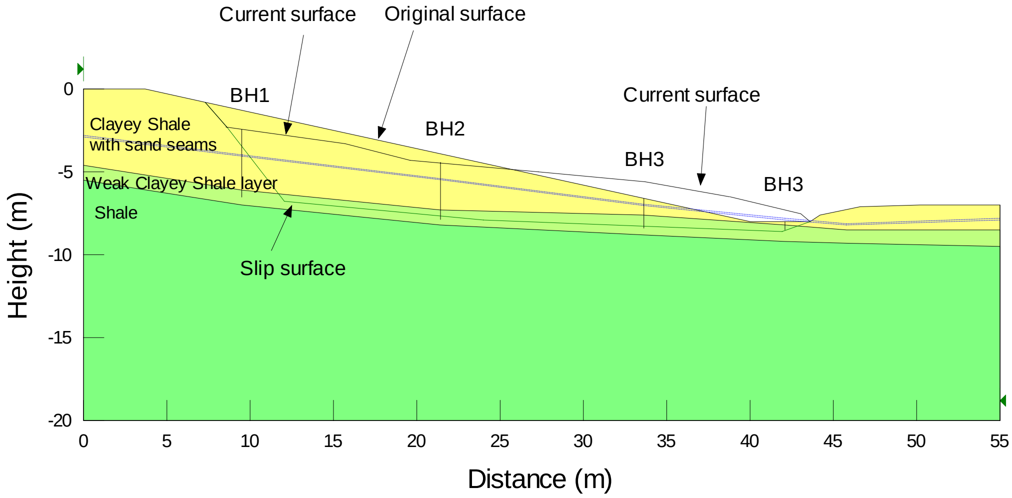


Figure 23. Slope profile for slope 2

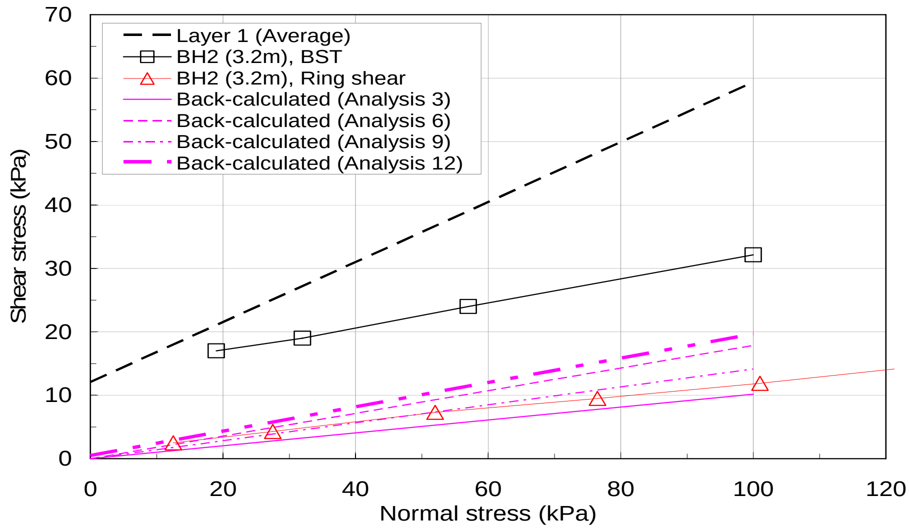


Figure 24. Shear strength parameter values for the weak shale layer (layer 2) in slope 2

Table 4. Summary of BST results for slope 2

BH	Depth (m)	ϕ' (deg.)	c' (kPa)	R^2	Data points
1	2.3	17	18	0.989	5
1	2.9	27	11	0.954	5
1	3.8	31	11	0.998	5
2	2.0	23	22	0.997	4
2	2.6	40	7	0.996	5
2	3.2	11	13	0.999	4
3	2.3	18	19	0.996	4
3	2.6	25	5	0.995	4
4	1.1	21	7	1.000	4

Table 5. Summary of basic properties of soils in slope 2

BH	Depth (m)	Soil	Grain Size: Sand (%)	Grain Size: Silt (%)	Grain Size: Clay (%)	Atterberg Limit: LL (%)	Atterberg Limit: PL (%)	Atterberg Limit: PI (%)	Classification: USCS	Classification: AASHTO	Water content (%)	Total density (kN/m3)
1	2.3	shale									13.2	
1	2.9	shale									19.3	
1	3.8	shale									26.1	19.1
2	2.0	shale									20.1	
2	2.6	shale									25.0	
2	3.2	shale	2	46	52	64	24	40	CH	A-7-6	25.2	19.0
3	2.6	shale									23.5	18.9
4	0.6	shale	2	51	47	59	28	31	CH	A-7-6	30.9	
4	1.1	shale	5	47	48	60	25	35	CH	A-7-6	33.6	18.0

Table 6. Soil Properties used for the slope analysis for slope 2

Layer	Soil	Unit weight (kN/m^3)	ϕ' (deg.)	c' (kPa)	Remark
1	Shale	19.0	25	12	Average
2	Shale	19.0	Varied	Varied	
3	Shale	22.0	30	100	Assumed

Table 7. Summary of slope analysis results for slope 2

Analysis No.	Ground Surface	Water Table: Source	Water Table: Position	Shear Strength for the Weak Layer 2: Source	ϕ' (deg.)	c' (kPa)	Factor of Safety: M-P	Factor of Safety: Bishop
1	Current (after failure)	Measured	Low	BST	11	13	3.192	3.240
2				Ring Shear	6.8	1.6	1.325	1.338
3				Back-calculated	5.8	0	1.000	1.008
4		Assumed	High (near surface)	BST	11	13	2.742	2.787
5				Ring Shear	6.8	1.6	1.003	1.014
6				Back-calculated	10.13	0	1.000	1.009
7	Original (before failure)	Measured	Low	BST	11	13	2.450	2.473
8				Ring Shear	6.8	1.6	1.088	1.093
9				Back-calculated	8.05	0	1.000	1.002
10		Assumed	High (near surface)	BST	11	13	2.215	2.241
11				Ring Shear	6.8	1.6	0.902	0.909
12				Back-calculated	10.85	0	1.000	1.004

M-P: Morgenstern-Price method

Slope 3 (Hwy34 Mp175.3)

Site Conditions

Location

The slope is a fill slope and is located at the south side of Highway 34 MP175.3, Monroe County (Figure 25).

History

The exact time of failure was unknown. The failure features of scarp and hump of the slide appeared quite old when it was investigated in July 2004.

Area Geology

According to the USDA (1984) Soil Survey Report, most of the soils in Monroe County formed in loess, glacial till, or alluvium. A few of the soils formed in colluvium, eolian sand or shale residuum. The major Pleistocene deposits are glacial tills ranging from 0 to more than 90m in thickness. Shale residuum is the oldest parent material in the county. The shale consists of a series of beds deposited during the Des Moines sedimentary cycle in the Pennsylvanian period. These beds include shale of different colors and textures, conglomerates, and a few organic layers such as layers of coal.

Field Investigations

Slope Geometry

The slope (Figure 26) and had an overall sloping angle of about 22 degree (H:V = 2.5:1), a maximum length of 16 m and a maximum height of 6m (Figure 27). The width of the slope (at the toe) was about 20 m (along the highway). It had a scarp near the top with a maximum height of 0.8 m. There was a hump at the surface of the slope. There was also a small ditch located at the toe of slope.

Site Geology

One borehole was drilled manually on the slope (Figure 27). The borehole showed that the slope was composed of yellowish brown glacial till which was generally soft to medium stiff. The boring logs are shown in Appendix (Figure A8).

Ground Water Table

Ground water level was measured after boring and was shown on the slope profile (Figure 27). The ditch at the toe of the slope had a little flowing water, which might be indicative of the ground water level near the toe.

Borehole Shear Test Results

BSTs were conducted at various depths of the borehole. The results are presented in Figure 28 and Table 8. The results show that ϕ' ranged from 19^o to 39^o, and *c*' varied from 14 to 22 kPa.

Lab Investigations

<u>Basic Properties</u>

Basic properties for representative soil sample were investigated and the results are summarized in Table 9. The results showed that the glacial till sample comprised 32% sand, 29% silt and 39% of clay. Its liquid limit was smaller than 50%. The soil was classified as low plasticity clay (CL) by USCS.

<u>Ring Shear Test Results</u>

Ring shear test was conducted for one soil sample and the result is presented in Figure 29. The result indicated that the glacial till has residual shear strength parameter values of $\phi_r' = 10.5^o$ with small $c_r' = 1.0$ kPa.

<u>Mineralogy and Morphology</u>

The x-ray diffractogram (XRD) for the random oriented bulk till sample at depth of 0.6 m is given in the Appendix Figure A9. The minerals identified are summarized at the bottom of the diffractogram, which include quartz, montmorillonite, kaolinite and illite.

Slope Analysis

<u>Soil Properties</u>

The field and lab test results show that the slope mainly consists of low plasticity glacial tills. Based on the field visual inspections and BST results, the soil was interpreted as two layers of different shear strengths as shown in the Figure 27. Slope stability analysis was performed based on this interpretation, and the soil properties used in the analysis are shown in Table 10.

<u>Method of Slope Analysis</u>

In the slope analysis, the slip surface was assumed to be circular and passing through the observed scarp (Figure 30). The observed ground water table (GWT) condition was used. Back-calculation was also performed to determine the average shear strength parameter values of the top layer giving unity factor of safety (FS).

<u>Results and Discussions</u>

Three slope analyses were performed and the results are summarized in Table 11. The results show that FS was larger that unity using shear strength parameter values obtained from BST. FS was smaller than unity using shear strength parameter values obtained from the ring shear test. The back-calculated shear strength parameter values were relatively close to the residual shear strength values from ring shear test.

The results suggest that the slope may have most likely failed under the conditions as in the Analysis 3 (back-calculation). The back-calculated shear strength parameter values indicated the possible average mobilized (or softened) shear strength during the slope failure. Also, the BST results in Table 11 may represent the peak shear strength of the till. The residual shear strength as obtained from the ring shear test was the ultimate shear strength corresponding to a slide with relatively large displacement. Once the slope movement was initiated, the shear strength of the soil started to drop from the peak value to the softened value, and may eventually drop to the residual value if the displacement was sufficiently large.

Conclusions

BST was used to investigate the slope and the shear strength parameter values were used for the slope analysis. The slope most likely failed with a circular slip surface passing the observed scarp. The relatively weak top layer has an average peak shear strength values of $\phi' = 20^0$ and c' = 21 kPa as measured by BST. It has a softened (mobilized) shear strength values of $\phi' = 12^0$ and c' = 2 kPa during the slide mobilization; and a residual shear strength of $\phi_r' = 10.5^0$ and c_r' = 1.0 kPa.

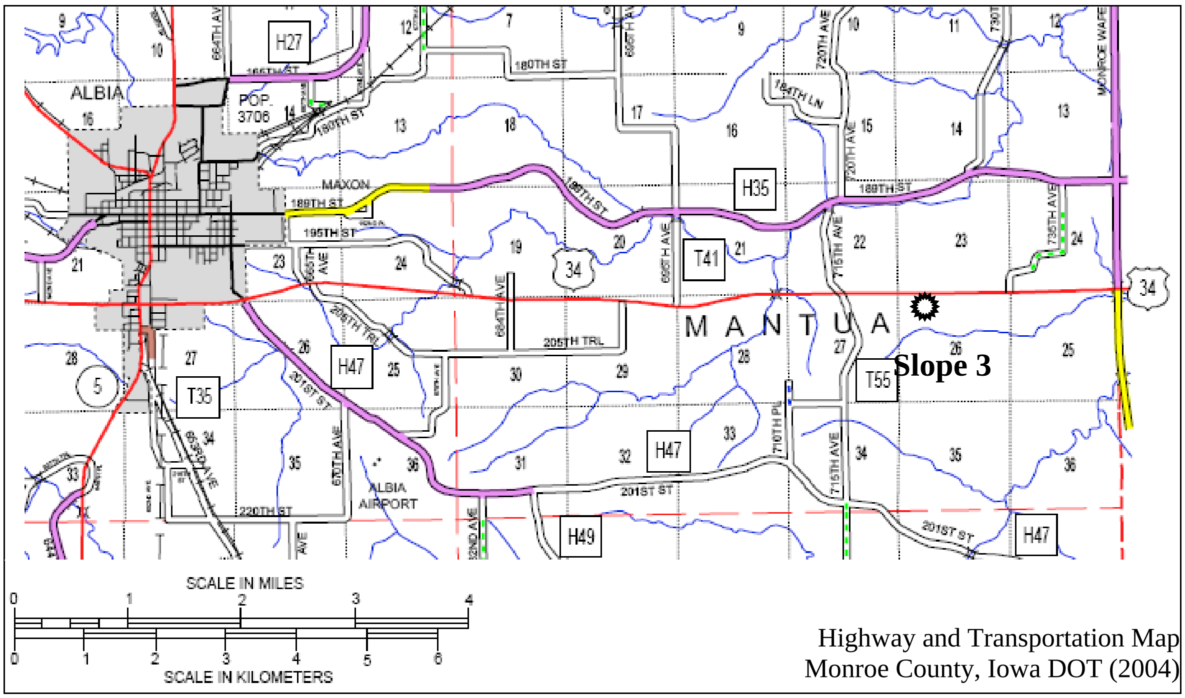


Figure 25. Location of slope 3 (Hwy 34, MP 175.3, Monroe Co.)

(a) Looking northwest, overview of the slope (photo taken by Yang, 07/18/04)

(b) Looking northwest, close view of the bulge and the scrap (photo taken by Yang, 04/01/05)

Figure 26. Photographs for slope 3

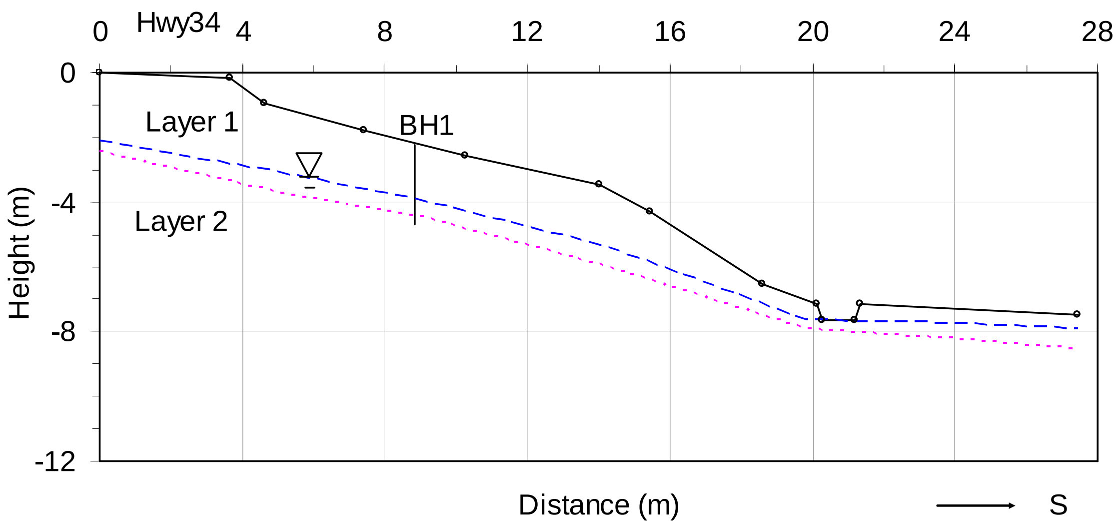


Figure 27. Cross-section for slope 3

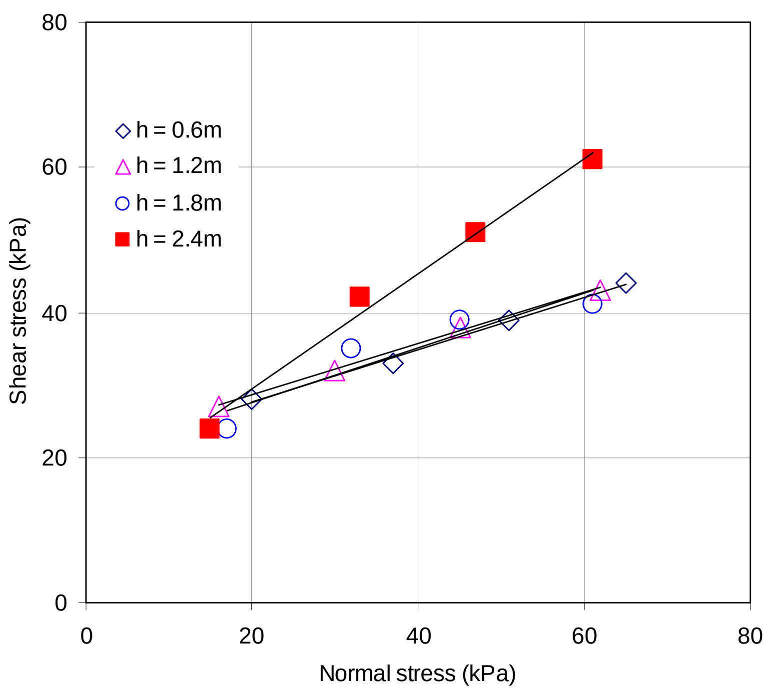


Figure 28. BST results for slope 3

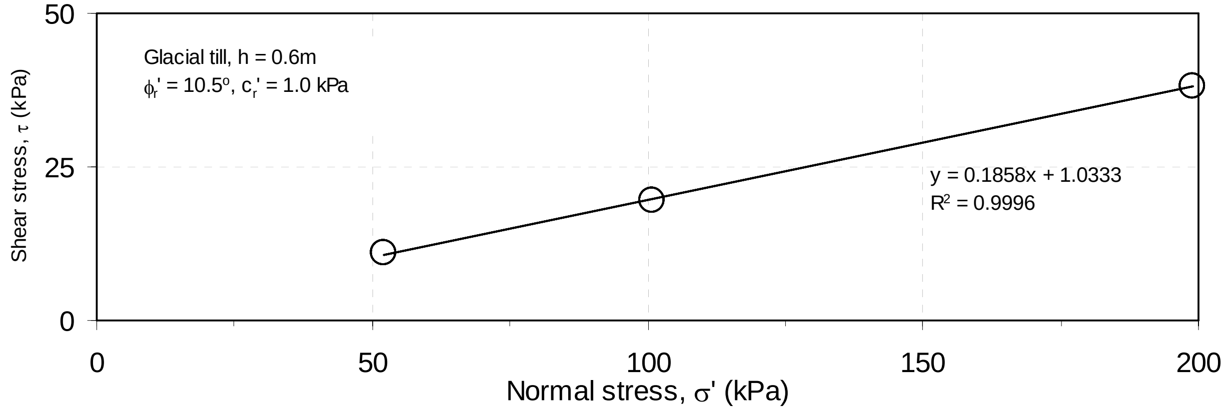


Figure 29. Ring shear test results for slope 3

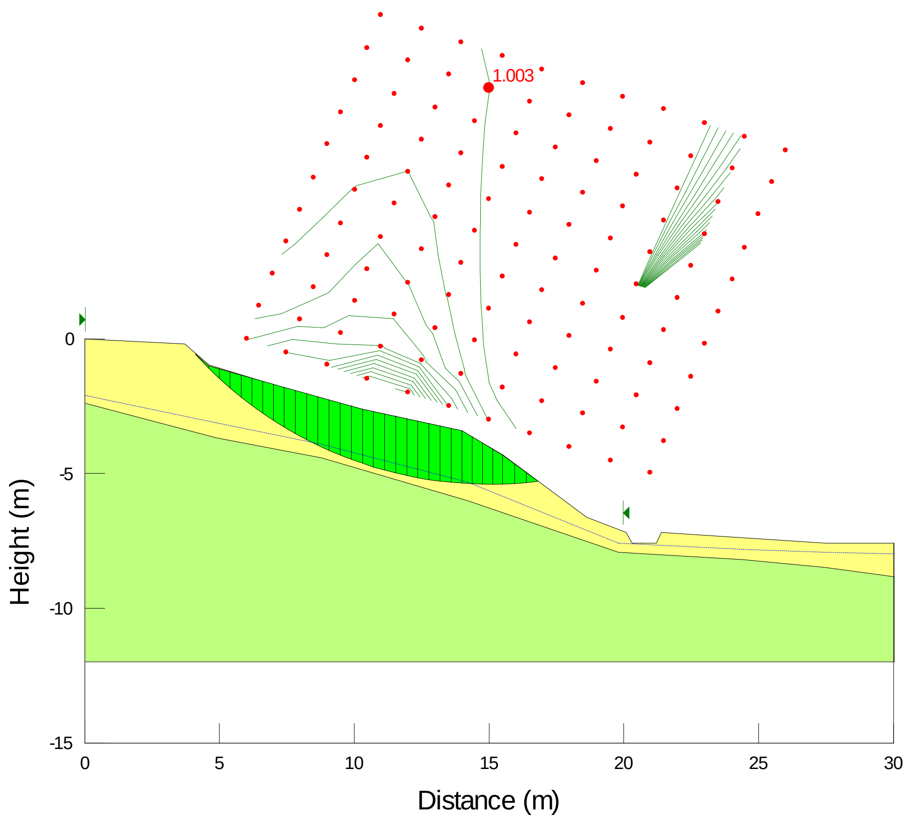


Figure 30. Slope stability analysis for slope 3

Table 8. Summary of BST results for slope 3

Depth (m)	Soil	ϕ' (deg.)	c' (kPa)	R^2	Data points
0.6	Glacial till	20	20	0.995	4
1.2	Glacial till	19	22	0.996	4
1.8	Glacial till	21	20	0.870	4
2.4	Glacial till	39	14	0.989	4

Table 9. Summary of basic properties for soils in slope 3

Depth (m)	Soil	Grain Size			Atterberg Limit			Classification		Water	Total
		Sand (%)	Silt (%)	Clay (%)	LL (%)	PL (%)	PI (%)	USCS	AASHTO	content (%)	density (kN/m3)
0.6	glacial. till	32	29	39	42	22	20	CL	A-7-6	23.1	19.0
1.2	glacial. till									27.5	
2.4	glacial. till									26.3	19.1

Table 10. Soil properties used for the slope analysis for slope 3

Layer	Unit weight	ϕ' (deg.)	c' (kPa)	Remarks
1	19.0	20	21	Depends on analysis
2	21.0	39	14	

Table 11. Summary of slope analysis results for slope 3

Analysis No.	Shear Strength for Layer 1			Factor of Safety	
	Source	ϕ' (deg.)	c' (kPa)	M-P	Bishop
1	BST (average)	20	21	2.710	2.706
2	Ring Shear	10.5	1.0	0.788	0.769
3	Back-calculated	12	2	1.003	1.003

M-P: Morgenstern-Price method

Slope 4 (Hwy34 Mp175.5)

Site Conditions

Location

The slope is a fill slope and is located at the north side of Highway 34, MP175.5, Monroe County (Figure 31).

History

There was no apparent evidence of failure for the slope when it was investigated in July 2004. The purpose of the investigation was to find out what were the conditions for a typical stable slope along Hwy 34.

Area Geology

According to the USDA (1984) Soil Survey Report, most of the soils in Monroe County formed in loess, glacial till, or alluvium. A few of the soils formed in colluvium, eolian sand or shale residuum. The major Pleistocene deposits are glacial tills ranging from 0 to more than 90m in thickness. Shale residuum is the oldest parent material in the county. The shale consists of a series of beds deposited during the Des Moines sedimentary cycle in the Pennsylvanian period. These beds include shale of different colors and textures, conglomerates, and a few organic layers such as layers of coal.

Field Investigations

Slope Geometry

The slope (Figure 32) had an overall sloping angle of about 22 degrees (H:V = 2.5:1), a maximum length of 18 m and a maximum height of 7 m (Figure 33). There was a small hump near the toe of the slope. There was also a small ditch located at the toe of slope which was parallel to the highway.

Site Geology

One borehole was drilled manually on the slope (Figure 33). The borehole showed that the slope composed of fill of brown glacial till which was soft to medium stiff. The boring log is shown in Appendix (Figures A10).

Ground Water Level

Ground water level for in the borehole was measured after boring and was shown on the slope profile (Figure 33). The ditch at the toe of the slope had water, which was used to establish the GWT near the toe.

Borehole Shear Test Results

BSTs were conducted at a depth of 1.5m in the borehole. The results are presented in Figure 34. The results show that $\phi' = 18^o$ and $c' = 9$ kPa.

Lab Investigations

Basic Properties

Basic properties for representative soil sample were investigated and the results are summarized in Table 12. The results show that all the soil samples have a relatively low clay content of less than 40%, and liquid limit is less than 50%. All the glacial tills were classified as low plasticity clay (CL) by USCS.

Ring Shear Test Results

Ring shear test was conducted for the soil sample of 1.5 m deep and the results are presented in Figure 35. The results indicated residual shear strength parameter values of ϕ_r' = 10.1^o with small c_r' = 2.3 kPa.

Mineralogy and Morphology

The x-ray diffractogram (XRD) for the random oriented bulk till sample at depth of 0.3m in the borehole is given in the Appendix (Figure A11). The minerals identified are summarized at the bottom of the diffractogram, and include quartz, montmorillonite, kaolinite and illite.

Slope Analysis

Soil Properties

The field and lab test results show that the slope mainly consists of low plasticity clay of glacial till. Based on the field visual inspections and BST results, the soil in the slop is assumed to be uniform for slope analysis.

Method of Slope Analysis

In the slope analysis, the potential slip surface was assumed to be circular (Figure 36). Back-calculation was also performed to determine the shear strength parameter values of the soil giving unity factor of safety (FS).

Results and Discussions

Three slope analyses were performed and the results are given in Table 13. The results show that FS was larger than unity using shear strength parameter values obtained from BST. FS was smaller than one using shear strength parameter values obtained from the ring shear test. The back-calculated shear strength parameter values were relatively close to the shear strength parameter values from BST with a smaller *c*' value. The difference in c' values provided the margin of safety.

The results indicate that the slope was stable under the conditions when it was investigated. The shear strength parameter values from back-calculation indicate the average shear strength parameter values that need to be mobilized if the slope movement is initiated. Also, the BST results in Table 13 may represent the peak shear strength of the shale. The residual shear strength as obtained from the ring shear test was the ultimate shear strength corresponding to a slide with relatively large displacement.

Conclusions

BST was used to characterize the slope which did not fail when it was investigated. The shear strength parameter values obtained from BST were used for the slope analysis to investigate the factor of safety of the slope. The FS was found to be 1.36 for the slope, which had a circular potential slip surface passing the top and the toe of the slope. The soil had peak shear strength values of $\phi' = 18^0$ and c’ = 9 kPa as measured by BST. The average shear strength values of the soil will be $\phi' = 18^0$ and c’ = 3.5 kPa in order to mobilize the slope. The residual shear strengths of soil were $\phi_r' = 10.1^0$ and c_r' = 2.3 kPa.

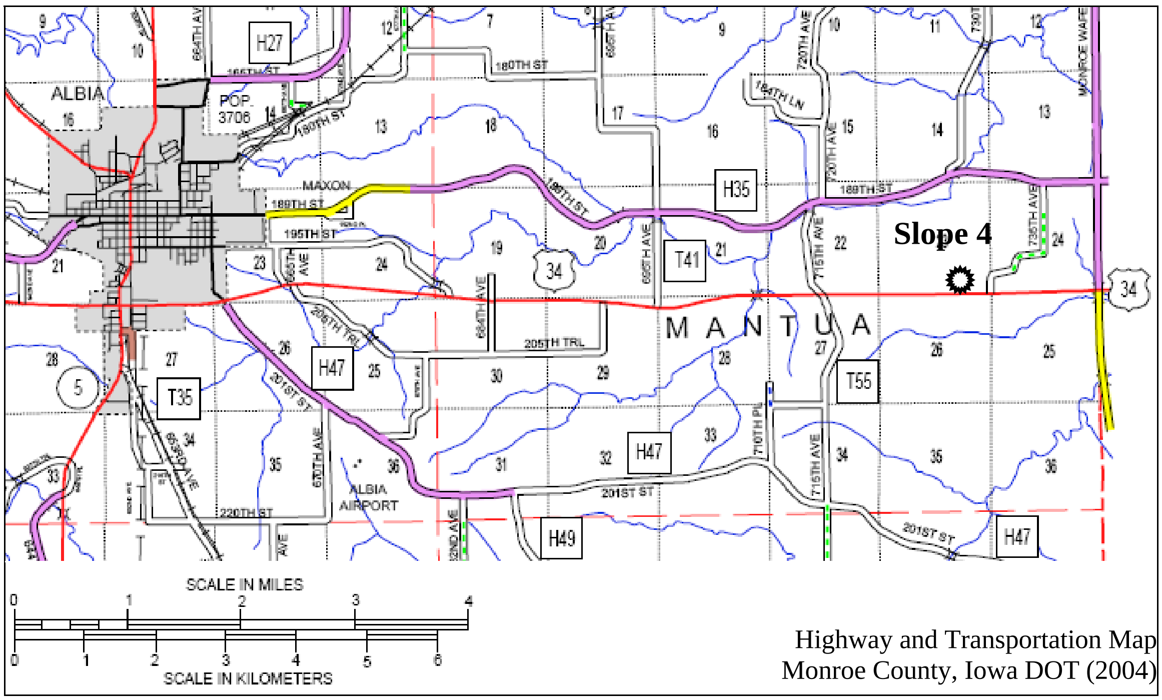


Figure 31. Location of slope 4 (Hwy 34, Mp 175.5, Monroe Co.)

Figure 32. Slope 4, looking east (photo taken by Yang, 04/01/05)

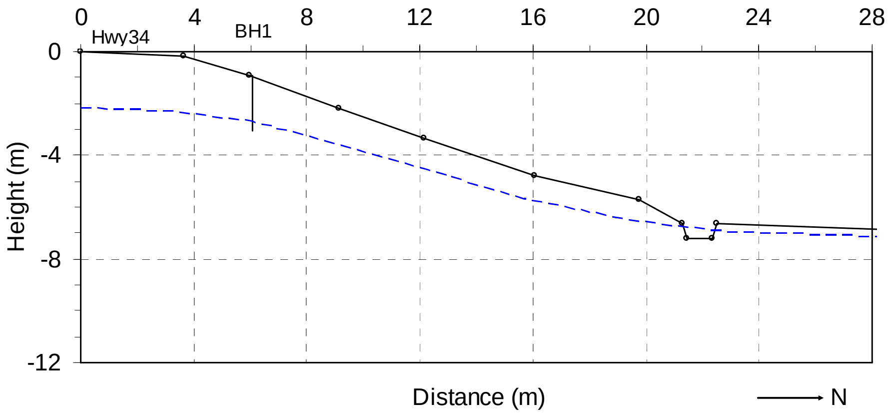


Figure 33. Cross-section for slope 4

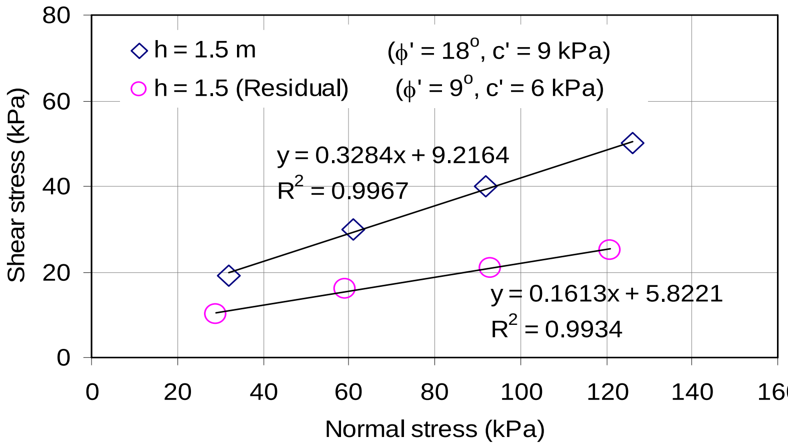


Figure 34. BST results for slope 4

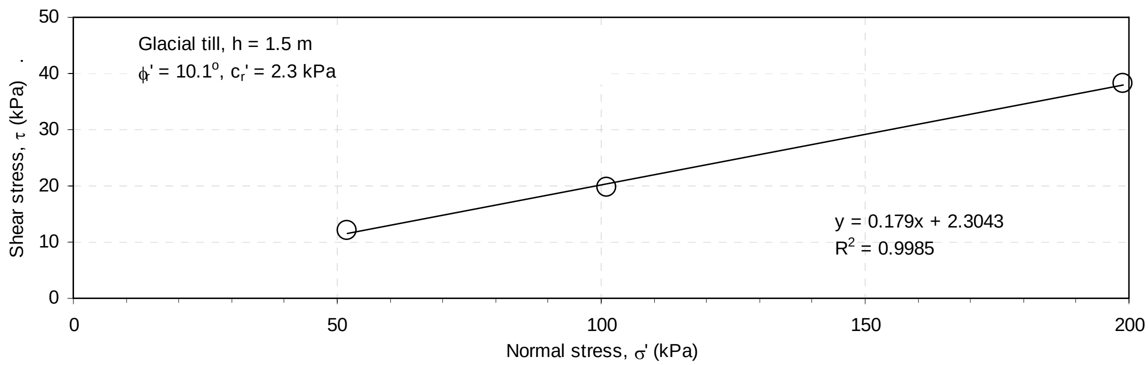


Figure 35. Ring shear test result for slope 4

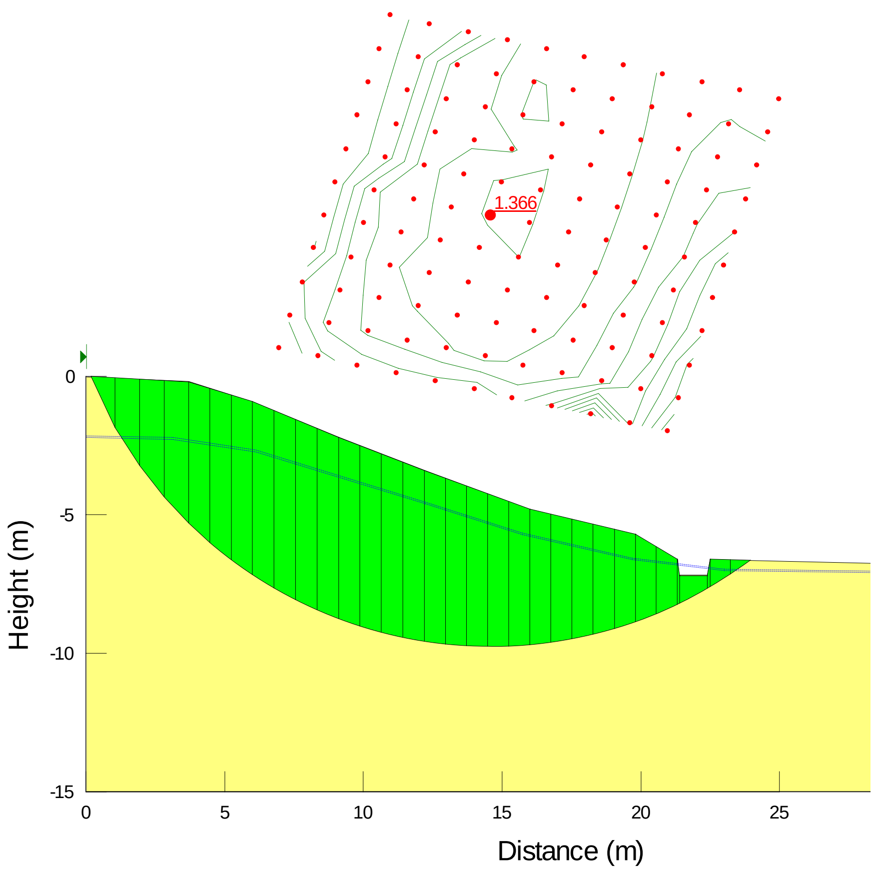


Figure 36. Slope stability analysis for slope 4

Table 12. Summary of basic property results for slope 4

Soil	Depth (m)	Grain Size			Atterberg Limit			Classification		Water Content (%)	Total density (kN/m3)
		Sand (%)	Silt (%)	Clay (%)	LL (%)	PL (%)	PI (%)	USCS	AASHTO		
G. till	0.3	17	45	38	42	21	21	CL	A-7-6	24.7	
G. till	1.5	25	35	40	43	20	23	CL	A-7-6	27.9	18.2

Table 13. Summary of slope analysis results for slope 4

Analysis No.	Shear Strength			Factor of Safety	
	Source	ϕ' (deg.)	c' (kPa)	M-P	Bishop
1	BST	18	9	1.366	1.364
2	Ring Shear	10.1	2.3	0.575	0.573
3	Back-calculated	18.0	3.5	1.001	0.998

M-P: Morgenstern-Price method

Slope 5 (Hwy34 Mp178.3n)

Site Conditions

Location

The slope is a fill slope and is located at the north side of Highway 34 MP178.3, Wapello County (Figure 37).

History

The exact time of failure was unknown. The failure features of scarp and hump of the slide appeared quite old when it was investigated in July 2004.

Area Geology

According to the USDA (1981b) Soil Survey Report, most of soils formed in glacial till, loess and alluvium. Clayey shale is the oldest parent material forming the bedrock of the project site. The bedrock surface closely parallels to the existing ground surface.

Field Investigations

Slope Geometry

The slope (Figure 38) had an overall sloping angle of about 18 degree (H:V = 3.0:1), a maximum length of 20 m and a maximum height of 7 m (Figure 39). The width of the slope (at the toe) was about 25 m (along the highway). The slope had a scarp near the top with a maximum height of 0.8 m. It had a 2 m wide subsidence filled with stones (rip-rap) at top in some area of the slope. There also small hump near the toe. A small ditch was located at the toe of slope.

Site Geology

One borehole was drilled manually on the slope (Figure 39). The borehole showed that the slope was filled with yellowish brown glacial till which was generally soft to medium stiff. The boring log is shown in the Appendix (Figure A12).

Ground Water Level

Ground water level was measured after boring and was shown on the slope profile (Figure 39). The ditch at the toe of the slope had a little water, which was used to establish the ground water level near the toe.

Borehole Shear Test Results

BSTs were conducted at various depths of the borehole. The results are presented in Figure 40 and Table 14. The results show that ϕ' ranged from 13^o to 31^o, and *c*' varied from 3 to 7 kPa.

Lab Investigations

Basic Properties

Basic properties for representative soil sample were investigated and the results are summarized in Table 15. The results showed that the glacial till sample comprised 16% sand, 47% silt and 37% of clay. Its liquid limit was smaller than 50%. The soil was classified as low plasticity clay (CL) by USCS.

Ring Shear Test Results

Ring shear test was conducted for one soil sample and the result is presented in Figure 41. The results indicate that the glacial till had residual shear strength parameter values of $\phi_r' = 8.4^o$ with a small $c_r' = 2.7$ kPa.

Mineralogy and Morphology

The x-ray diffractogram (XRD) for the random oriented bulk till sample at depth of 0.6 m is given in the Appendix (Figure A13). The minerals identified are summarized at the bottom of the diffractogram, and include quartz, calcite, kaolinite and illite.

Slope Analysis

Soil Properties

The field and lab test results show that the slope mainly consists of fill of low plasticity glacial till. Based on the field visual inspections and BST results, the soil was assumed to be uniform for the slope stability analysis.

Method of Slope Analysis

In the slope analysis, the slip surface was assumed to be circular passing through the observed scarp (Figure 42). The observed ground water level condition was used. Back-calculation was also performed to determine the average shear strength parameter values of the soil giving unity factor of safety (FS).

Results and Discussions

Three slope analyses were performed and the results are given in Table 16. The results show that FS was larger that unity using shear strength parameter values obtained from BST. FS was smaller than 1.0 using shear strength parameter values obtained from ring shear test. The back-calculated shear strength parameter values were between those from BST and ring shear test.

All these results suggest that the slope may have most likely failed under the conditions as in Analysis 3 (the back-calculation). The shear strength parameter values from back-calculation indicate the average mobilized shear strength during the slope failure. Also, the BST results in Table 16 may represent the average peak shear strength of the shale. The residual shear strength as obtained from the ring shear test was the ultimate shear strength corresponding to a slide with relatively large displacement. Thus the actual failure condition would appear to be a softened condition.

Conclusions

BST was used to characterize the slope. The shear strength parameter values obtained from BST were used for the slope analysis to investigate the possible cause of the failure. The slope most likely failed with a circular slip surface passing the scarp. The soil has a peak shear strength values of $\phi' = 16^0$ and c' = 25 kPa as measured by BST. It has a softened (mobilized) shear strength values of $\phi' = 13.2^0$ and c' = 2.7 kPa during the slide mobilization; and a residual shear strength of $\phi_r' = 8.4^0$ and c_r' = 2.7 kPa.

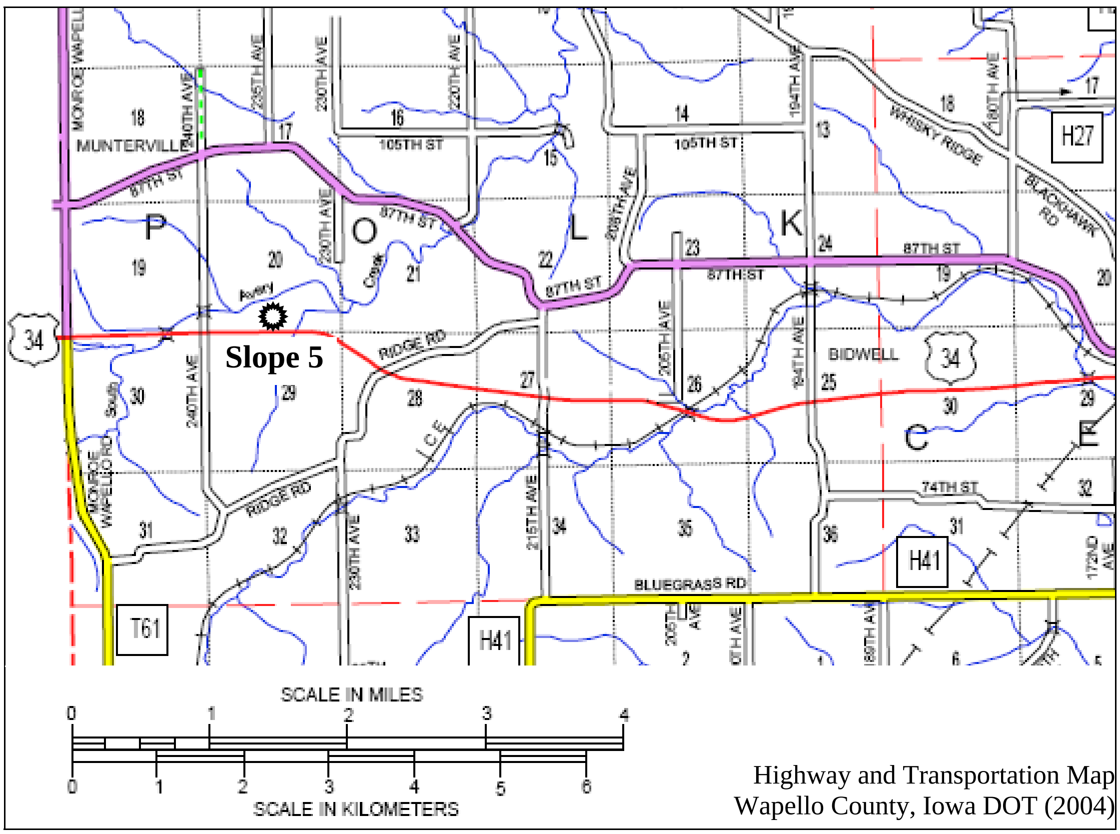


Figure 37. Location of slope 5 (Hwy 34, MP178.3N, Wapello Co.)

(a) Looking south, showing the scarp of the slope (photo taken by Thompson, 06/04/04)

(b) Looking southwest, showing field investigation (photo taken by Thompson, 06/04/04)

Figure 38. Photographs for slope 5

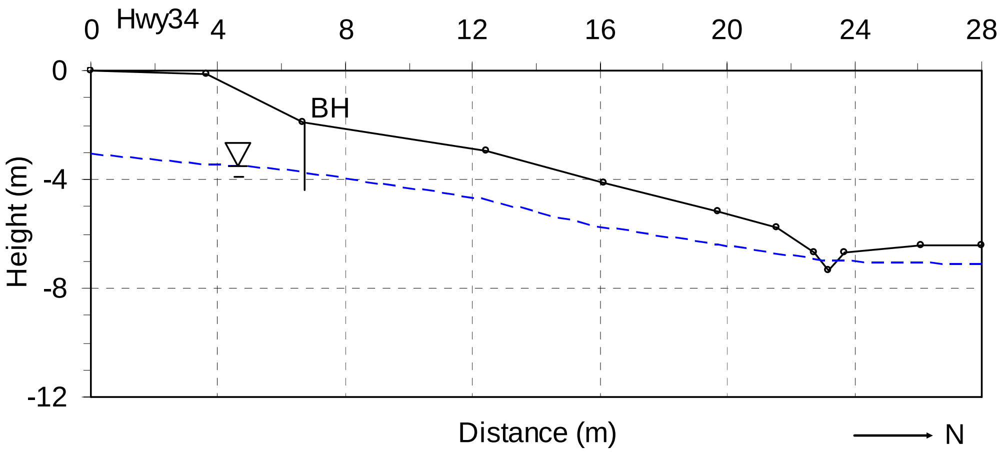


Figure 39. Cross-section for slope 5

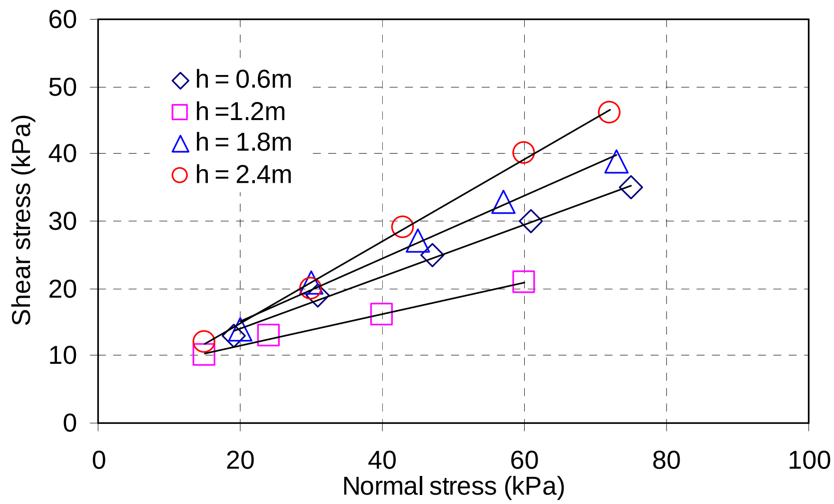


Figure 40. BST results for slope 5

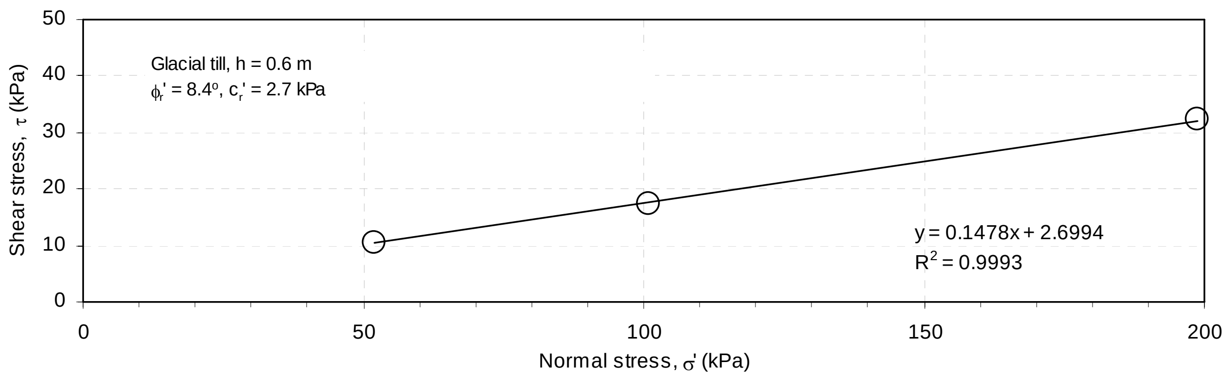


Figure 41. Ring shear test results for slope 5

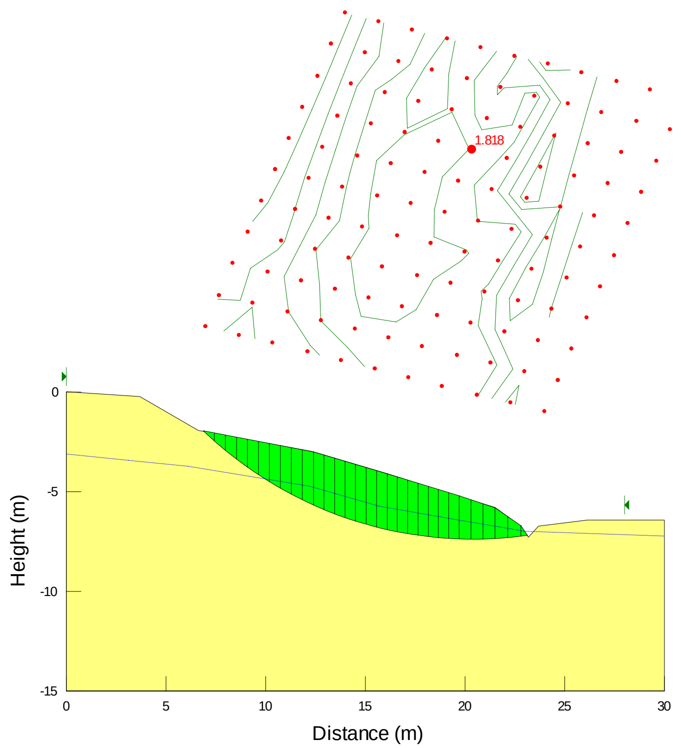


Figure 42. Slope stability analysis for slope 5

Table 14. Summary of BST results for slope 5

Depth (m)	Soil	ϕ' (deg.)	c' (kPa)	R^2	Data points
0.6	Glacial till	21	6	0.996	5
1.2	Glacial till	13	7	0.993	4
1.8	Glacial till	25	6	0.990	5
2.4	Glacial till	31	3	0.998	5

Table 15. Summary of basic property results for slope 5

Depth (m)	Soil	Grain Size			Atterberg Limit			Classification		Water	Total
		Sand (%)	Silt (%)	Clay (%)	LL (%)	PL (%)	PI (%)	USCS	AASHTO	content (%)	density (kN/m^3)
0.6	glacial. till	16	47	37	44	23	21	CL	A-7-6	24.1	18.4
2.4	glacial. till										17.9

Table 16. Summary of slope analysis results for slope 5

Analysis No.	Shear Strength			Factor of Safety	
	Source	ϕ' (deg.)	c' (kPa)	M-P	Bishop
1	BST (average)	23	5	1.817	1.818
2	Ring Shear	8.4	2.7	0.737	0.737
3	Back-calculated	13.2	2.7	0.997	0.998

M-P: Morgenstern-Price method

Slope 6 (Hwy34 Mp175.3s)

Site Conditions

Location

The slope is a full slope and is located beside Highway 34 MP178.3 (south side), Wapello County (Figure 43).

History

The exact time of failure was unknown. The failure features of scarp and hump of the slide appeared quite old when it was investigated in July 2004.

Area Geology

According to the USDA (1981b) Soil Survey Report, most of soils formed in glacial till, loess and alluvium. Clayey shale is the oldest parent material forming the bedrock of the project site. The bedrock surface closely parallels to the existing ground surface.

Field Investigations

Slope Geometry

The slope (Figure 44) had an overall sloping angle of about 14 degrees (H:V = 4.0:1), a maximum length of 22 m and a maximum height of 6 m (Figure 45). The width of the slope (at the toe) is about 30 m (along the highway). It had a scarp near the top with a maximum height of 1.0 m. There was a hump at the surface of the mid of the slope.

Site Geology

One borehole was drilled manually on the slope (Figure 45). The borehole showed that the slope was backfilled with grey and brown shale which was generally soft to medium stiff. The boring log is shown in Appendix (Figure A14).

Ground Water Level

Ground water level was measured after boring and was found to be located near the bottom of the borehole as shown on the slope profile (Figure 45). This ground water level condition was use for slope analysis.

Borehole Shear Test Results

BST was conducted at a depth of 1.8 m in the borehole. The results are presented in Figure 46. The results show that ϕ' was 16^{o} and *c*' was 25 kPa.

Lab Investigations

Basic Properties

Basic properties for a representative soil sample were investigated and the results are summarized in Table 17. The results showed that the shale sample comprised 18% sand, 43% silt and 39% of clay. Its liquid limit was smaller than 50%. The soil was classified as low plasticity clay (CL) by USCS.

Ring Shear Test Results

Ring shear test was conducted for one soil sample and the result is presented in Figure 47. The result indicated that the glacial till has residual shear strength values of $\phi_r' = 9.3^o$ with small $c_r' = 1.1$ kPa.

Mineralogy and Morphology

The x-ray diffractogram (XRD) for the random oriented bulk shale sample at depth of 0.3 m is given in the Appendix (Figure A15). The minerals identified are summarized at the bottom of the diffractogram, and include quartz, montmorillonite, kaolinite and illite.

Slope Analysis

Soil Properties

The field and lab test results show that the slope mainly consists of low plasticity clay shales. Based on the field visual inspections and BST results, the soil was assumed to be uniform for the slope stability analysis.

Method of Slope Analysis

In the slope analysis, the slip surface was assumed to be circular passing through the observed failure zone near the top of the slope (Figure 48). The observed ground water table (GWT) condition was used. Back-calculation was also performed to determine the average shear strength parameter values of the soil giving unity factor of safety (FS).

Results and Discussions

Three slope analyses were performed and the results are given in Table 18. The results show that FS was larger that unity using shear strength parameter values obtained from BST. FS was smaller than one using shear strength parameter values obtained from ring shear test. The back-calculated shear strength parameter values to achieve FS = 1.0 have same ϕ' with that of BST but with much smaller c' than BST.

The results suggest that the slope may have most likely failed under the conditions as in back-calculation (Analysis 3). The shear strength parameter values from back-calculation indicated the average mobilized shear strength during slope failure. Also, the BST results in Table 18 may represent the peak shear strength of the shale. The residual shear strength as obtained from the ring shear test was the ultimate shear strength corresponding to a slide with relatively large displacement. Thus the actual failure conditions would appear to be a softened condition.

Conclusions

BST was used to characterize the slope. The shear strength parameter values obtained from BST were used for the slope analysis to investigate the possible cause of the failure. The slope most likely failed with a circular slip surface passing the failure zone near the top of the slope. The soil in the slope had a peak shear strength values of $\phi' = 16^0$ and c' = 25 kPa as measured by BST. It had a softened (mobilized) shear strength values of $\phi' = 16^0$ and c' = 1.1 kPa during the slide mobilization; and a residual shear strength of $\phi_r' = 9.3^0$ and c_r' = 1.1 kPa.

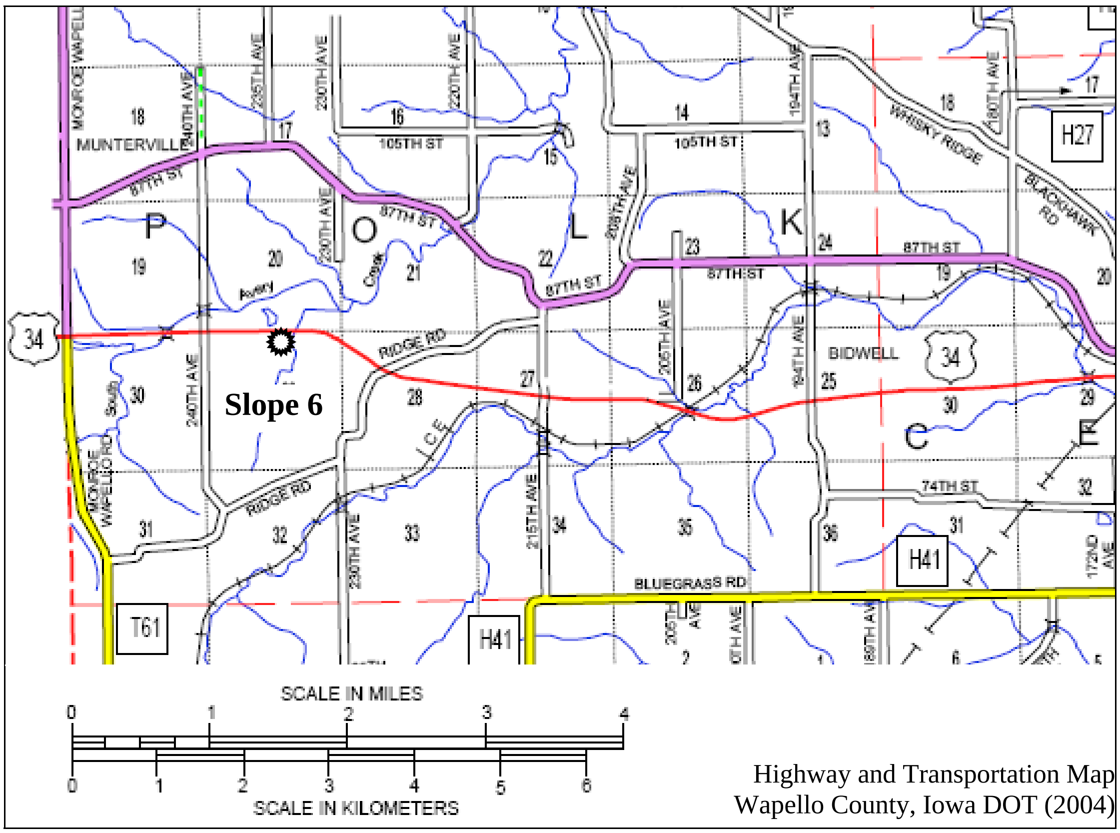


Figure 43. Location of slope 6 (Hwy 34, MP175.3S, Wapello Co.)

(a) Looking northeast, showing the slope failure at top (photo taken by Thompson, 06/04/04)

(b) Looking west, field investigation (photo taken by Thompson, 06/04/04)

Figure 44. Photographs for slope 6

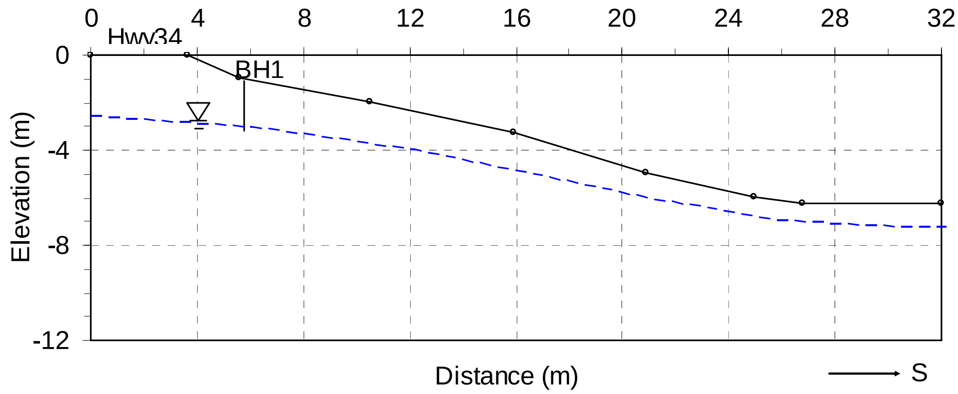


Figure 45. Cross-section for slope 6

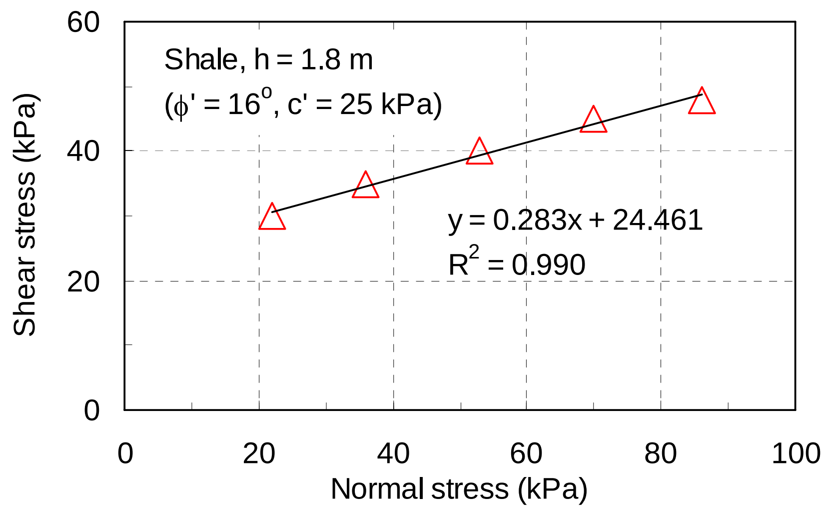


Figure 46. BST results for slope 6

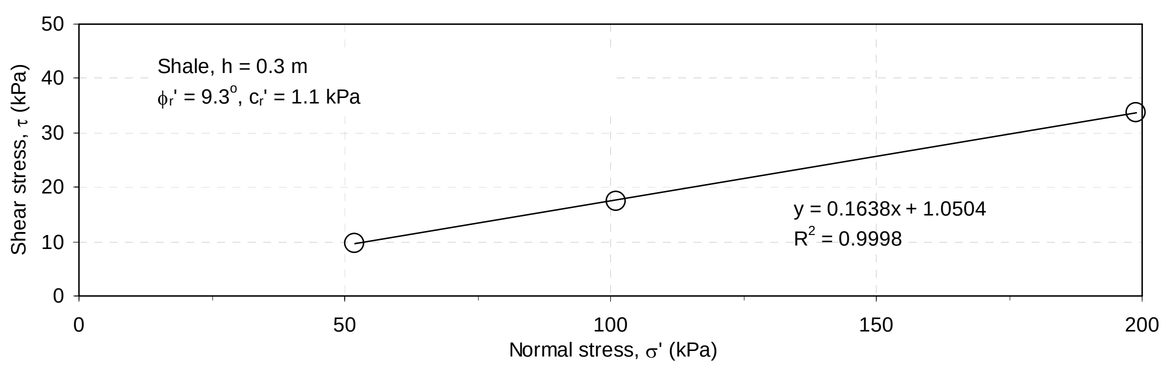


Figure 47. Ring shear test results for slope 6

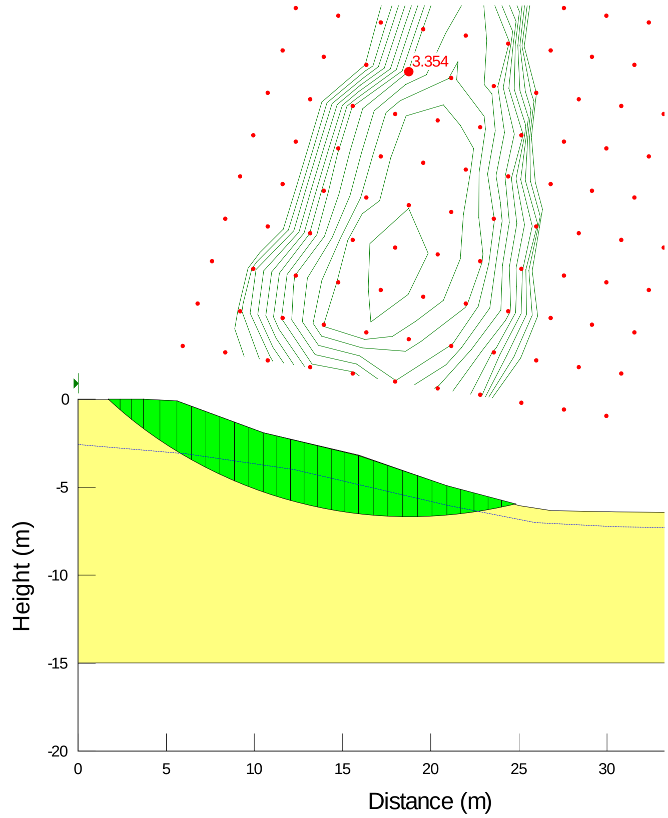


Figure 48. Slope stability analysis for slope 6

Table 17. Summary of basic property results for slope 6

Depth (m)	Soil	Grain Size			Atterberg Limit			Classification		Water content (%)	Total density (kN/m3)
		Sand (%)	Silt (%)	Clay (%)	LL (%)	PL (%)	PI (%)	USCS	AASHTO		
0.3	shale	18	43	39	44	25	19	CL	A-7-6	25.3	
1.8	shale									27.0	18.0

Table 18. Summary of slope analysis results for slope 6

Analysis No.	Shear Strength			Factor of Safety	
	Source	ϕ' (deg.)	c' (kPa)	M-P	Bishop
1	BST	16	25	3.354	3.362
2	Ring Shear	9.3	1.1	0.626	0.626
3	Back-calculated	16.0	1.1	1.014	1.014

M-P: Morgenstern-Price method

Slope 7 (Hwy169 Winterset)

Site Conditions

Location

The slope is a cut slope and is located at the west side of Highwy 169, about 3 miles north of Winterset, Madison County (Figure 49).

History

The slope started to move during 2003, and failed in 2004 thus brought IaDOT's attention. The scarp of the slide appeared to be newly formed when the slide was first investigated in June 2004. The slope was repaired around November 2004 by cutting.

Area Geology

According to the USDA (1975) Soil Survey Report, the soils of Madison County formed from loess, glacial till, alluvium, shale, limestone and sandstone. Near Winterset, the soils mainly formed in moderately well drained loess and glacial till, and slowly permeable shales.

Field Investigations

Slope Geometry

The slope (Figure 50) had an overall sloping angle of about 13 degree (H:V = 4.4:1), a maximum length of 33 m and a maximum height of 7 m (Figure 51). It has a nearly straight, steep scarp near the top with a maximum height of 1.7 m. The scarp extends along the side of highway for about 70 m, which is the maximum the width of the slide. There are a few transverse cracks at the mid of the slope and a small hump near the toe of the slope.

Site Geology

A total of four boreholes were drilled following the sliding direction (perpendicular to the highway) of the slope to establish a representative profile for the slope (Figure 51). Two of the boreholes (BH1 and BH2) were drilled with a rotary drill rig, and two (BH3 and BH4) were drilled manually using a hand auger. The boreholes show that the slope is covered with about 2 m thick brown silty clay overlying clay shales. The shales are divided into three layers based on the field visual classifications and the in-situ shear strength of the soil as measured by BST. The boring logs are shown in the Appendix (Figures A16 to A19).

Ground Water Table

Ground water level for each borehole was measured after the boring and was shown on the slope profile (Figure 51). The ground surface near the toe of the slope was very wet, and minor seepage of water out of the slope was observed. These observations were used to establish the ground water table conditions.

Borehole Shear Test results

BSTs were conducted at various depths of the boreholes. The results were presented in Figure 52 and Table 19. The results show that ϕ' ranged from 18^o to 35^o, and *c*' varied from 11 to 45 kPa.

Lab Investigations

Basic Properties

Basic properties for representative soil samples were investigated and the results are summarized in Table 20. The results show that all the shale samples have very low sand content of less than 3%. The soils have relatively high clay content of about 35% and liquid limit larger than 50%. All the shales are classified as high plasticity clay (CH) by USCS.

Ring Shear Test Results

Ring shear tests were conducted on two soil samples with two tests for each sample to compare the results. The results are presented in Figure 53 and Table 21. The results indicated that tests on each sample gave similar values of ϕ_r' with small c_r' values. ϕ_r' was 15.1^o and 16.3^o for sample in BH2, and 12.0^o and 13.0^o for sample in BH3.

Slope Analysis

Soil Properties

The field and lab test results show that the slope mainly consists of high plastic weathered clayey shales. Based on the field visual classifications and BST results, the soils were interpreted as four layers with different shear strength as shown in the Figure 51, with the Layer 2 exhibiting relatively low shear strength. Slope stability analysis was performed based on this interpretation of slope profile, and the soil properties used in the analysis were listed in Table 22.

Method of Slope Analysis

In the slope analysis, the slip surface was specified passing through the observed scarp and the weak shale layer of Layer 2 (Figure 54) for analyses No. 1 to 12. Both current slope geometry and geometry before failure were considered for the analysis; and both observed ground water conditions (low GWT) and assumed ground water conditions (high GWT) were used. The high GWT was located at the surface along the whole slope profile and represented the worst ground water conditions. Back-calculations were also performed to determine the shear strength parameter values of the weak Layer 2 giving unity factor of safety (FS). An additional analysis 13 of back-calculation was performed assuming uniform soil with circular slip surface passing the scarp (Figure 55).

Results and Discussions

The results of the slope analyses are summarized in Table 23. The results show that FS is much larger than unity (between 4 and 5) under different conditions of GWT and slope geometry using shear strength parameter values obtained from BST (Analyses 1, 4, 7 and 10). FS is about 1.5 to 2.0 under different conditions using shear strength parameter values obtained from ring shear test (Analyses 2, 5, 8 and 11). The back-calculated shear strength parameter values for FS =1.00 are lower than the residual shear strength values (Analyses 3, 6, 9 and 12) indicating the average shear strength along the slip surface mobilized during the failure of the slope is lower than the measured residual shear strength. This due to two possible reasons: the actual residual shear strength may be lower than what has been measured using ring shear test due to the spatial soil variation; the slip surface may not be exactly the one assumed in the analysis.

The Analysis 13, which assumed uniform soils (applying uniform shear strength parameter values for the slope) and circular slip surface passing near the scarp, gave a back-calculated shear strength parameter values that is exactly the same as the lower values of residual shear strength parameter as obtained from the ring shear tests. The ground water level condition was low for this analysis.

Based on slope analysis results, the slope most likely failed under a high GWT condition with a relatively low mobilized shear strength (Analysis 12) and flat slip surface; or failed under a low GWT with a mobilized shear strength close to the residual shear strength measured from ring shear test and with slip surface that is more close to circular shape (Analysis 13). The uncertainty of the back-calculated shear strength was due to the conditions involved exactly during the slope failure which were not observed. The BST results may represent the peak shear strength of the shale; and the back-calculated results indicate the probable softened shear strength (or mobilized shear) of the shale during slope failure (Figure 7). The residual shear strength as obtained from ring shear test is the ultimate shear strength corresponding to a slide with relatively large displacement.

Conclusions

BST was used to characterize the slope and gave shear strength parameter values for each soil layer. These values were used for the slope analysis considering various possible geometry and ground water table conditions. The slope most likely failed under the near surface GWT conditions with a flat slip surface passing through the relatively weak shale layer. The weak shale layer has peak shear strength parameter values of $\phi' = 18^o$ and c’ = 20 kPa as measured by BST, and residual strength parameter values of $\phi' = 12.0^o$ and c’ = 3.5 kPa as measured by ring shear test. The average shear strength mobilized during the slope failure was estimated to be close to the residual shear strength assuming a circular slip surface.

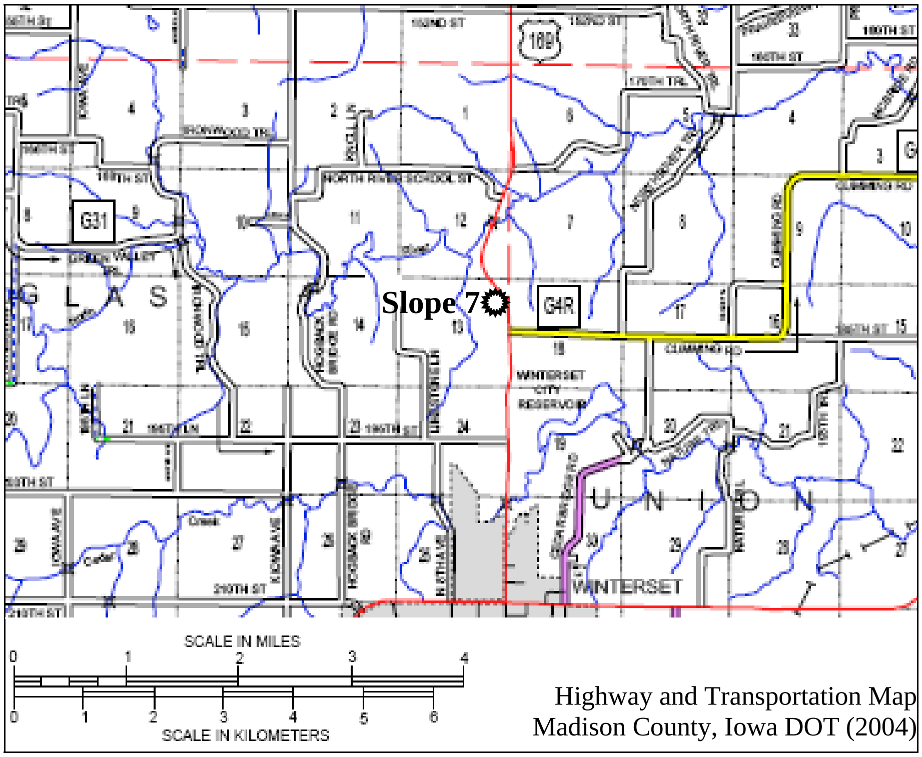


Figure 49. Location for slope 7 (Hwy 169, Winterset, Madison Co.)

(a) Looking south, showing the scarp and graben of the slope (photo taken by Thompson, 05/26/04)

(b) Looking north, showing the overview of the slope (photo taken by Thompson, 05/26/04)

(c) Looking northeast, showing the scarp of the slope (photo taken by Thompson, 05/26/04)

Figure 50. Photographs for slope 7

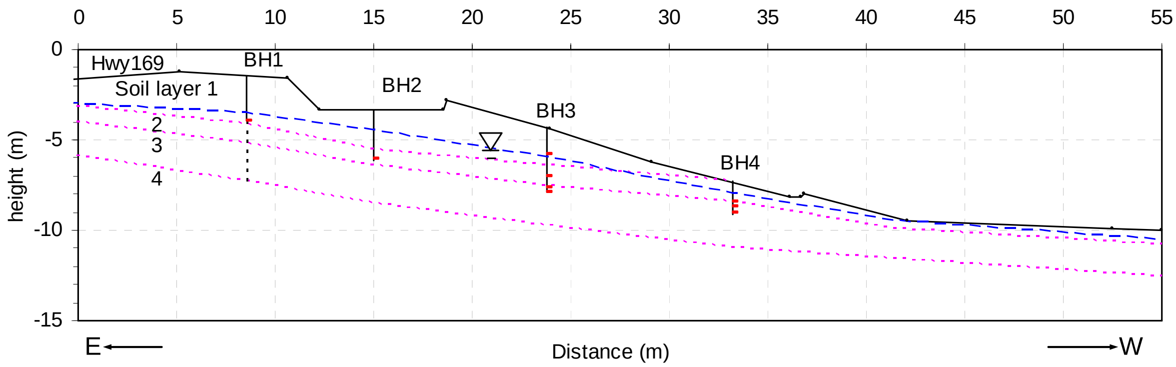


Figure 51. Cross-section for slope 7

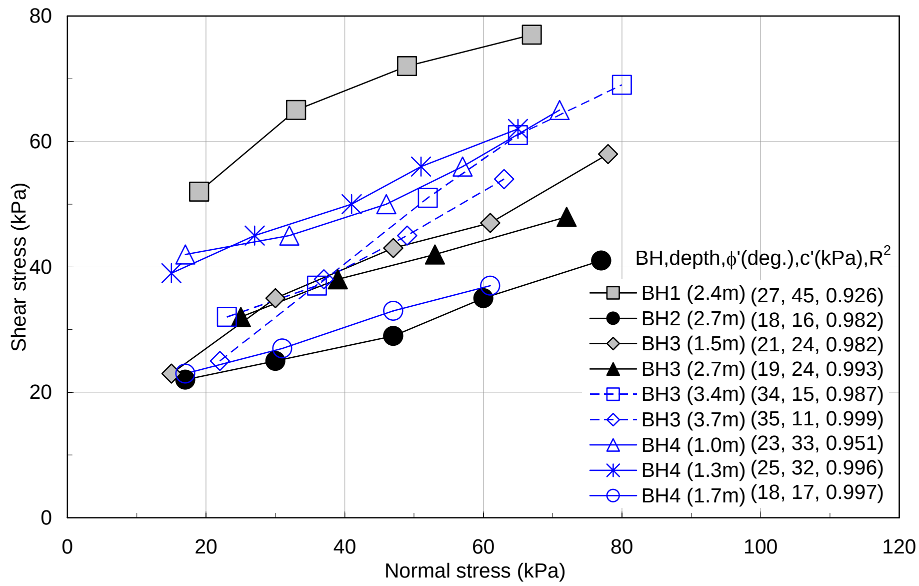


Figure 52. BST results for slope 7

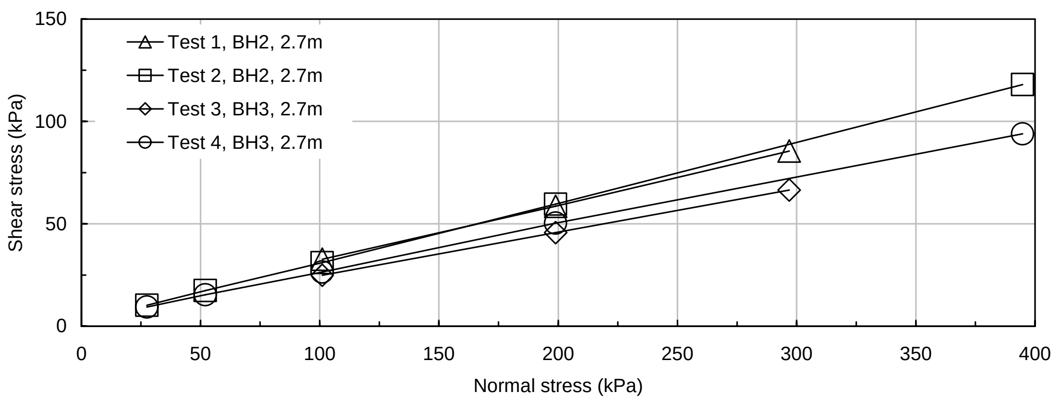


Figure 53. Ring shear test results for shales in slope 7

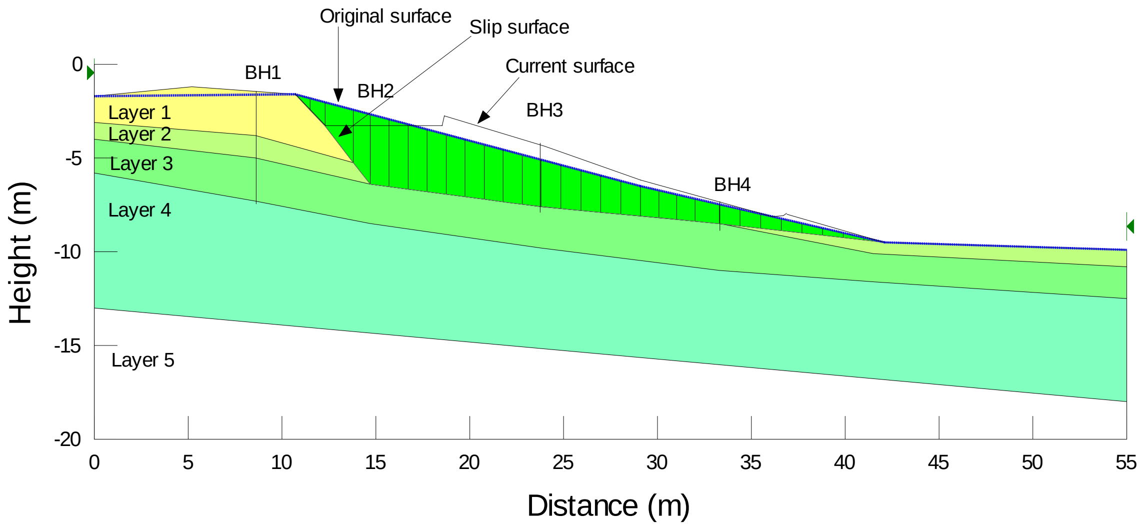


Figure 54. Slope profile for stability analysis for slope 7

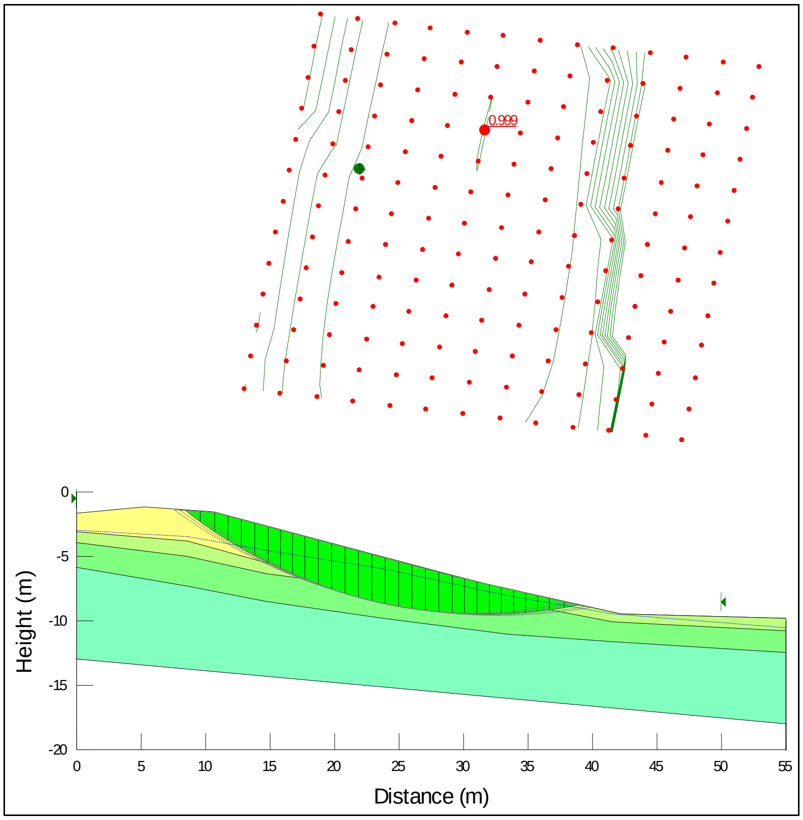


Figure 55. Analysis 13 assuming homogeneous soils and circular slip surface for slope 7

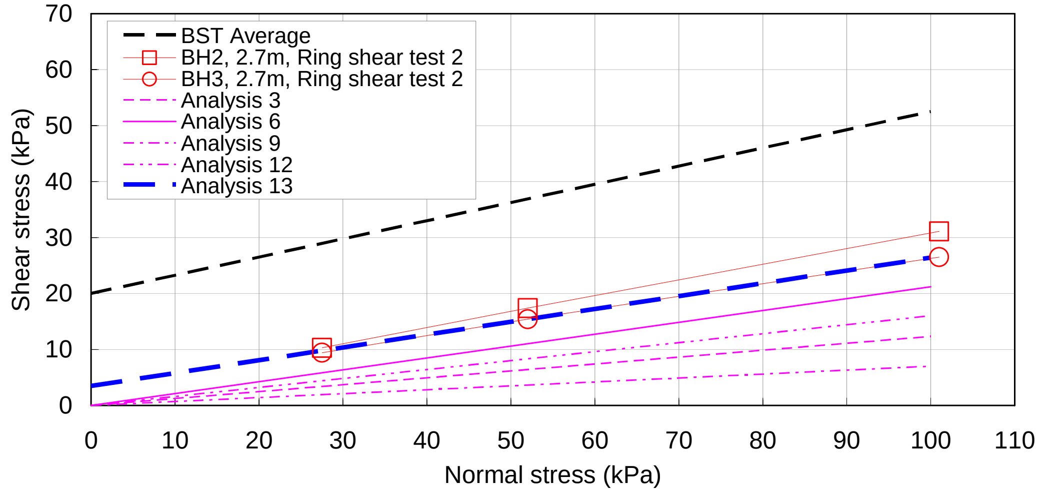


Figure 56. Different shear strength for the weak shale layer 2 in the slope 7

Table 19 Summary of BST results for slope 7

BH	Depth (m)	ϕ' (deg.)	c' (kPa)	R^2	Data points
1	2.4	27	45	0.926	4
2	2.7	18	16	0.982	5
3	1.5	21	24	0.982	5
3	2.7	19	24	0.993	4
3	3.4	34	15	0.987	5
3	3.7	35	11	0.999	4
4	1.0	23	33	0.951	5
4	1.3	25	32	0.996	5
4	1.7	18	17	0.997	4

Table 20. Summary of basic property test results for slope 7

BH	Depth (m)	Grain Size			Atterberg Limit			Classification		Water Content (%)	Total density (kN/m3)
		Sand (%)	Silt (%)	Clay (%)	LL (%)	PL (%)	PI (%)	USCS	AASHTO		
2	2.7	1	64	35	50	23	27	CH	A-7-6	16.2	18.9
3	2.7	3	60	37	59	24	35	CH	A-7-6	29.0	19.2

Table 21. Summary of ring shear test results for slope 7

BH	Depth (m)	ϕ_r' (deg.)	c' (kPa)	R^2	Data points	ϕ_r' (c'=0)
2	2.7	15.1	3.3	0.9999	3	16.3
2	2.7	16.3	1.9	0.9999	5	16.7
3	2.7	12.0	3.5	1.0000	3	12.8
3	2.7	13.0	3.5	0.9996	5	13.7

Table 22. Soil properties used for the slope analysis for slope 7

Layer	Unit weight (kN/m^3)	ϕ' (deg.)	c' (kPa)	Remarks
1	18.0	24	35	Average
2	19.0	(18)	(20)	Various
3	19.5	29	23	Average
4	21.0	35	100	Assumed

Table 23. Summary of slope analysis results for slope 7

Analysis No.	Ground Surface	Water Table		Shear Strength for the Weak Layer 2			Factor of Safety	
		Method	Position	Source	Friction angle (deg.)	Cohesion (kPa)	M-P	Bishop
1	Current (after slide)	Measured	Low	BST (Average)	18	20	4.895	4.966
2				Ring Shear	12.0	3.5	1.968	1.964
3				Back-calculated	7.03	0.0	1.000	0.943
4		Assumed	High (near surface)	BST (Average)	18	20	4.359	4.427
5				Ring Shear	12.0	3.5	1.586	1.582
6				Back-calculated	11.98	0.0	0.995	0.967
7	Original (before slide)	Measured	Low	BST (Average)	18	20	4.364	4.378
8				Ring Shear	12.0	3.5	1.962	1.962
9				Back-calculated	8.0	0	1.020	0.973
10		Assumed	High (near surface)	BST (Average)	18	20	3.881	3.923
11				Ring Shear	12.0	3.5	1.587	1.581
12				Back-calculated	9.1	0.0	0.998	0.968
13	Original (before slide)	Measured	Low	Back-calculated	12.9	3.5	1.000	0.997

Note: Analysis 13 assumes Layers 1, 2, and 3 have same c' and ϕ'; circular slip surface passing the scarp.
M-P denotes Morgenstern-Price method.

Slope 8 (Hwy169 Afton Slope A)

Site Conditions

Location

The slope is a cut slope and is located at the east side of Highway 169, 2 miles south of Afton, Union County (Figure 57).

History

The time of the slope failure was unknown. The scarp and the hump generally appeared old in August 2004 when the slope was investigated.

Area Geology

According to the USDA (1978) Soil Survey Report, most of the soils in Union County formed from loess, glacial till, alluvium and shale. Glacial tills dominate the area near Afton, ranging to a depth of 30 m or more.

Field Investigations

Slope Geometry

The slope (Figure 58) had an overall sloping angle of about 22 degree (H:V = 2.5:1), a maximum length of 27 m and a maximum height of 10 m (Figure 59). The width of the slope (at the toe) is about 60 m (along the highway). It had a scarp near the middle of the slope surface with a maximum height of 0.5 m. There was a hump near the toe of the slope. There was also a 2 m wide shallow ditch located at the toe of slope.

Site Geology

Two boreholes were drilled manually on the slope (Figure 59). The boreholes showed that the slope was made of yellowish brown glacial till which was generally soft to medium stiff. The boring logs are shown in Appendix (Figures A.20 and A.21).

Ground Water Table

Ground water level was measured after boring and was shown on the slope profile (Figure 59). The ditch at the toe of the slope was wet and could be indicative of shallow water level near the toe.

Borehole Shear Test Results

BSTs were conducted at various depths of the boreholes. The results were presented in Figure 60 and Table 24. The results show that ϕ' ranged from 16^o to 31^o, and c' varied from 12 to 35 kPa.

Lab Investigations

Basic Properties

Basic properties for representative soil sample were investigated and the results were summarized in Table 25. The results showed that the glacial till sample comprised 34% of sand, 37% of silt and 29% of clay. The liquid limit was lower than 50%. The soil was classified as low plasticity clay (CL) by USCS.

Ring Shear Test Results

Ring shear test was conducted for one soil sample and the result is presented in Figure 61. The result indicated that the glacial till has residual shear strength parameter values of $\phi_r' = 14.2^o$ with $c_r' = 0$ kPa.

Slope Analysis

Soil Properties

The field and lab test results show that the slope mainly consists of low plasticity glacial till. Based on the field visual inspections and BST results, the soil was assumed to be uniform for the slope stability analysis.

Method of Slope Analysis

In the slope analysis, the slip surface was assumed to be circular (Figure 62). The observed ground water table (GWT) condition was used. Back-calculation was also performed to determine the average shear strength parameter values of the soil giving unity factor of safety (FS).

Results and Discussions

Three slope analyses were performed and the results are given in Table 26. The results show that FS was larger that unity using shear strength parameter values obtained from BST. FS was smaller than one using shear strength parameter values obtained from ring shear test. The back-calculated shear strength parameter values are between those of from BST and ring shear test.

All these results suggest that the slope may have most likely failed under the conditions as in Analysis 3. The shear strength parameter values from back-calculation indicated the average mobilized shear strength during slope failure. Also, the BST results in Table 5 may represent the peak shear strength of the shale. The residual shear strength as obtained from the ring shear test was the ultimate shear strength corresponding to a slide with large displacement.

Conclusions

BST was used to characterize the slope. The shear strength parameter values obtained from BST were used for the slope analysis to investigate the possible cause of the failure. The slope most likely failed with a circular slip surface passing the top of the slope. The soil in the slope has an average peak shear strength values of $\phi' = 26^0$ and $c' = 20$ kPa as measured by BST. It has a softened (mobilized) shear strength values of $\phi' = 14.2^0$ and $c' = 11.5$ kPa during the slide mobilization; and a residual shear strength of $\phi_r' = 14.2^0$ and $c' = 0$ kPa.

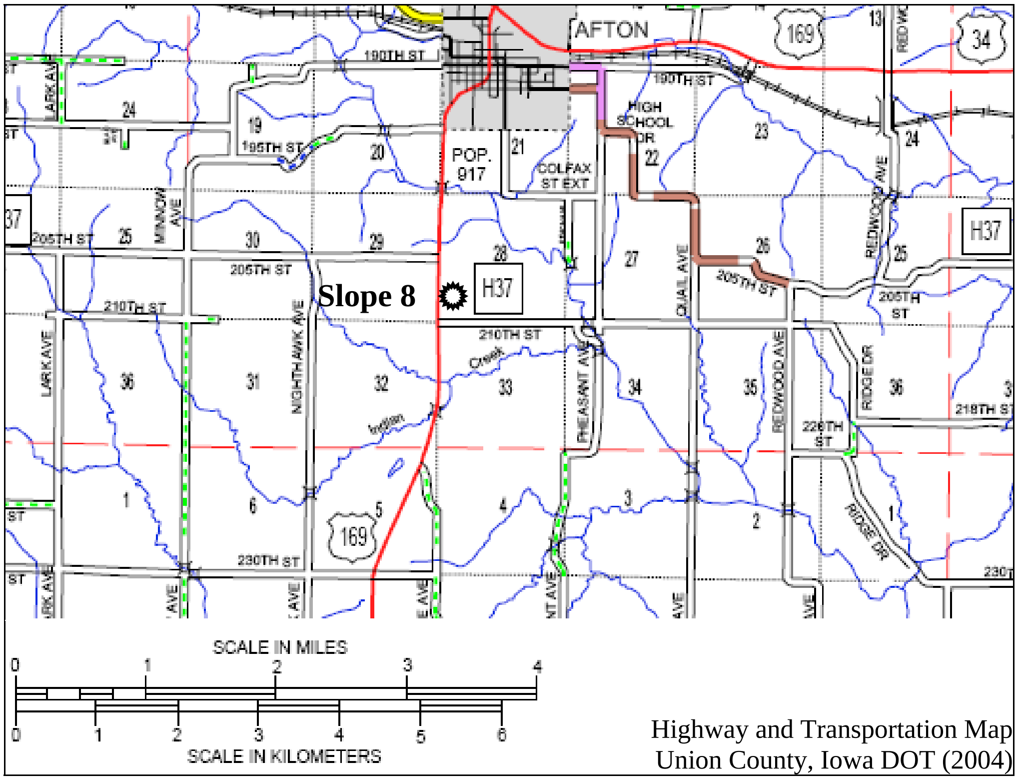


Figure 57. Location of slope 8 (Hwy 169, 2 miles south of Afton, Union Co.)


(a) Looking east, showing the overview of the slope (photo taken by Thompson, 06/04/04)

(b) Looking east, showing the bulge and scarp of the slope (photo taken by Thompson, 06/04/04)

(c) Looking southwest, showing the overview of the slope (photo taken by Thompson, 06/04/04)

Figure 58. Photographs for slope 8

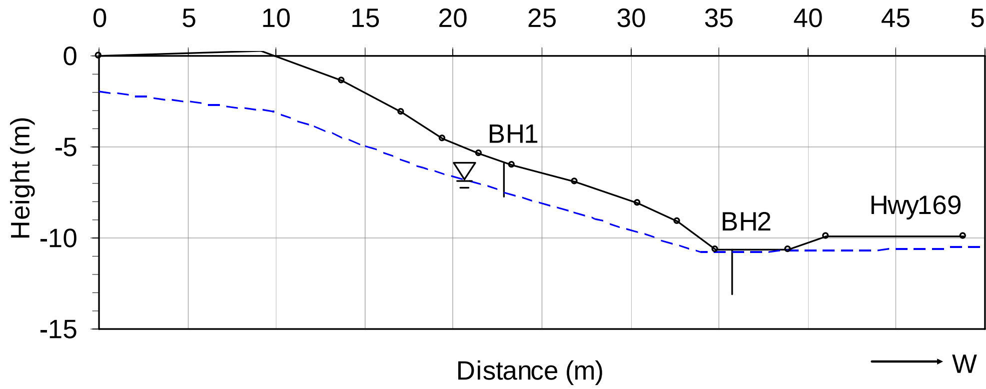


Figure 59. Cross-section for slope 8

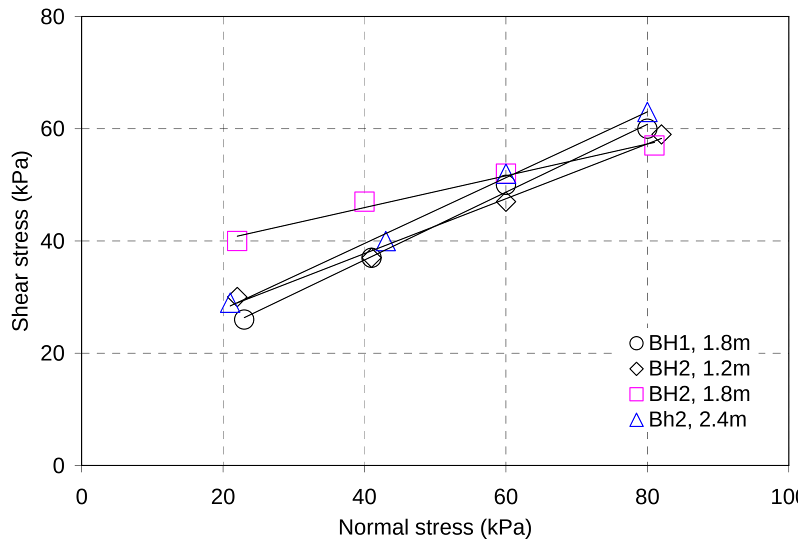


Figure 60. BST results for slope 8

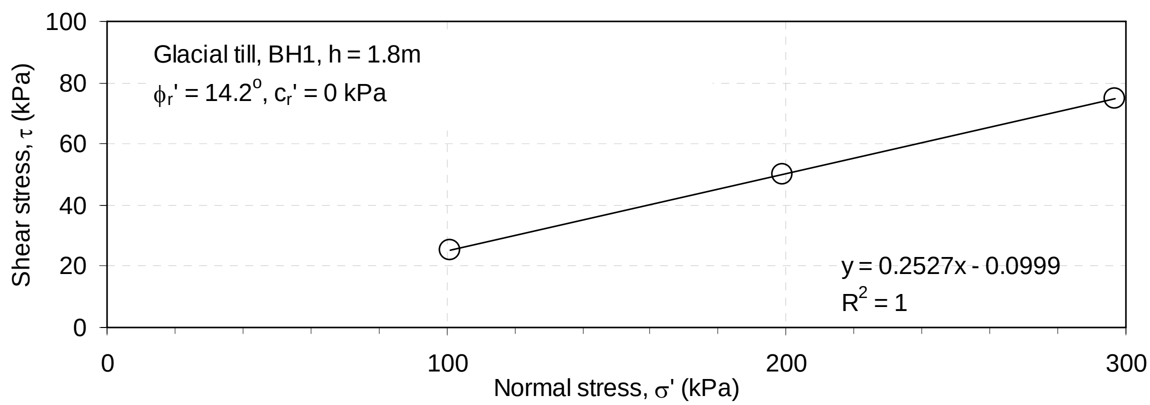


Figure 61. Ring shear test results for slope 8

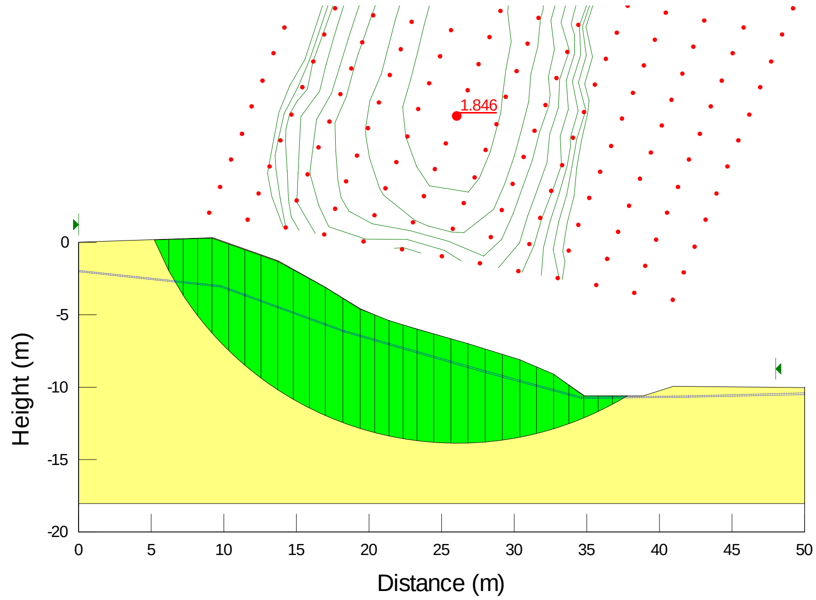


Figure 62. Slope stability analysis for slope 8

Table 24. Summary of BST results for slope 8

BH	Depth (m)	Soil	ϕ' (deg.)	c' (kPa)	R^2	Data points
1	1.8	g. till	31	12	0.996	4
2	1.2	g. till	26	18	0.993	4
2	1.8	g. till	16	35	0.986	4
2	2.4	g. till	30	16	0.996	4

Table 25. Summary of basic properties for the soils in slope 8

BH	Depth (m)	Soil	Grain Size			Atterberg Limit			Classification		Water content (%)	Total density (kN/m3)
			Sand (%)	Silt (%)	Clay (%)	LL (%)	PL (%)	PI (%)	USCS	AASHTO		
1	1.8	g. till	34	37	29	40	19	21	CL	A-7-6	19.0	18.1
2	1.2	g. till									16.7	
2	1.8	g. till									16.7	
2	2.4	g. till									16.9	19.2

Table 26. Summary of slope analysis results for slope 8

Analysis No.	Shear Strength			Factor of Safety	
	Source	ϕ' (deg.)	c' (kPa)	M-P	Bishop
1	BST (average)	26	20	1.846	1.844
2	Ring Shear	14.2	0.0	0.561	0.556
3	Back-calculated	14.2	11.5	0.999	0.999

M-P: Morgenstern-Price method

Slope 9 (Hwy169 Afton Slope B)

Site Conditions

Location

The slope is a cut slope and is located at the west side of Highway 169, 2 miles south of Afton, Union County (Figure 63).

History

The exact time of failure of the slope was unknown. The scarp and the hump generally appeared old in August 2004 when the slope was investigated.

Area Geology

According to the USDA (1978) Soil Survey Report, most of the soils in Union County formed from loess, glacial till, alluvium and shale. Glacial tills dominate the area near Afton, ranging to a depth of 30 m or more.

Field Investigations

Slope Geometry

The slope (Figure 64) had an overall sloping angle of about 23 degree (H:V = 2.4:1), a maximum length of 33 m and a maximum height of 13 m (Figure 65). The width of the slope (at the toe) is about 40 m (along the highway). There was a hump near the toe of the slope. There was also a shallow ditch located at the toe of slope.

Site Geology

One borehole was drilled manually on the slope (Figure 65). The borehole showed that the slope was composed of 0.6 m thick brown glacial till overlying grey or brown shale. The soils were generally soft to medium stiff. The boring log is shown in the Appendix (Figures A22).

Ground Water Table

Ground water level was measured after boring and was shown on the slope profile (Figure 65). The ditch at the toe of the slope was wet and could be indicative of shallow water level near the toe.

Borehole Shear Test Results

BSTs were conducted at various depths of the boreholes. The results are presented in Figure 66. The results show that ϕ' ranged from 18^o to 23^o, and c' was 20 kPa.

Lab Investigations

Basic Properties

Basic properties for representative soil sample were investigated and the results are summarized in Table 27. The results showed that the shale sample at the depth of 1.8m comprised 17% of sand, 41% of silt and 42% of clay. The liquid limit was higher than 50%. The shale was classified as high plasticity clay (CH) by USCS.

Ring Shear Test Results

Ring shear test was conducted for one shale sample and the result is presented in Figure 67. The result indicated that the shale has residual shear strength parameter values of $\phi_r' = 7.1^o$ with $c_r' =$ 1.6 kPa.

Mineralogy and Morphology

The x-ray diffractogram (XRD) for the random oriented bulk shale sample at depth of 0.6 m is given in Appendix Figure A23. The minerals identified are summarized at the bottom of the diffractogram, and include quartz, montmorillonite and cristobalite.

Slope Analysis

Soil Properties

The field and lab test results show that the slope mainly consists of a thin layer of low plasticity glacial till underlain with high plasticity clay shale. Based on the field visual inspections and BST results, the clay shale was assumed to be uniform for the slope. The glacial till is also assumed to have same shear strength with the shale for simplicity. Slope stability analysis was performed accordingly.

Method of Slope Analysis

In the slope analysis, the slip surface was assumed to be circular (Figure 68). The observed ground water table (GWT) condition was used. Back-calculation was also performed to determine the average shear strength parameter values of the shale giving unity factor of safety (FS).

Results and Discussions

Three slope analyses were performed and the results are given in Table 28. The results show that FS was larger that unity using shear strength parameter values obtained from BST. FS was smaller than one using shear strength parameter values obtained from ring shear test. The back-calculated shear strength parameter values have same ϕ' with that of the BST values, but with much lower c'.

The results suggest that the slope may have most likely failed under the conditions as in Analysis 3 (back-calculation). The shear strength parameter values from back-calculation indicated the average mobilized shear strength during slope failure. Also, the BST results in Table 28 may represent the peak shear strength of the shale. The residual shear strength as obtained from the ring shear test was the ultimate shear strength of the soil under large displacement.

Conclusions

BST was used to characterize the slope. The shear strength parameter values obtained from BST for the soil were used the slope analysis to investigate the possible cause of the failure. The slope most likely moved with a circular slip surface passing the top of the slope. The soil has an average peak shear strength values of $\phi' = 21^0$ and c' = 20 kPa as measured by BST. It has a softened (mobilized) shear strength values of $\phi' = 21^0$ and c' = 5.1 kPa during the slide mobilization; and a residual shear strength of $\phi_r' = 7.1^0$ and c' = 1.6 kPa.

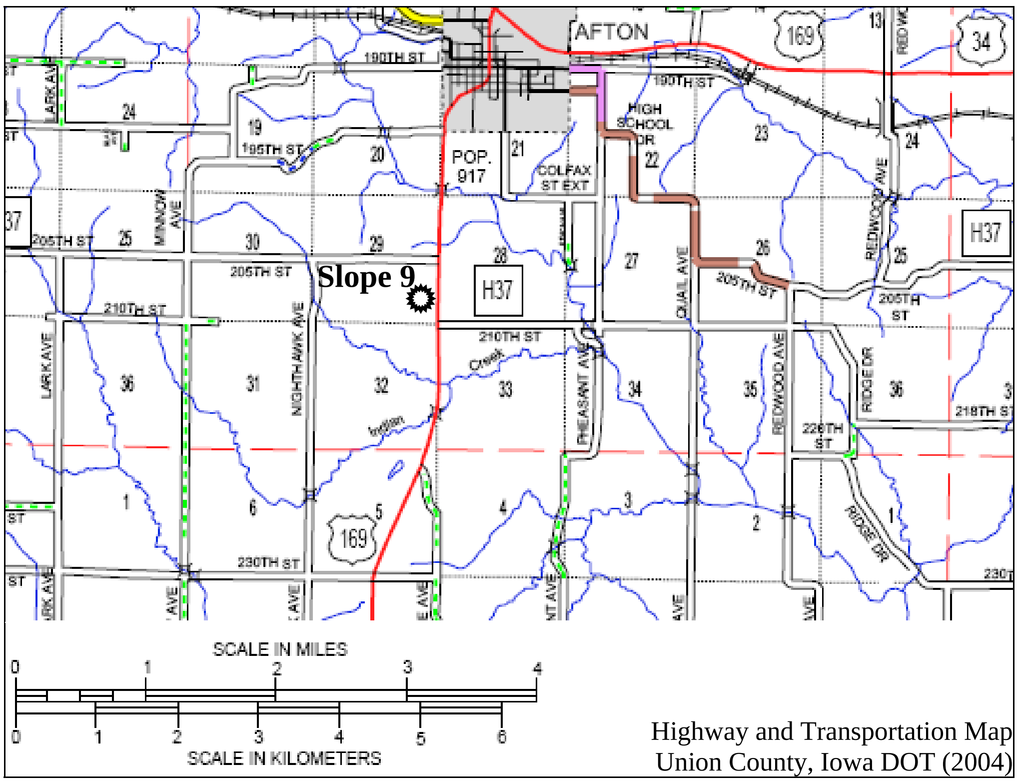


Figure 63. Location of slope 9 (Hwy 169, 2 miles south of Afton, Union Co.)

(a) Looking south, showing the overview of the slope (photo taken by Thompson, 06/04/04)

(b) Looking north, showing the overview of the slope (photo taken by Yang, 04/16/05)

Figure 64. Photographs for slope 9

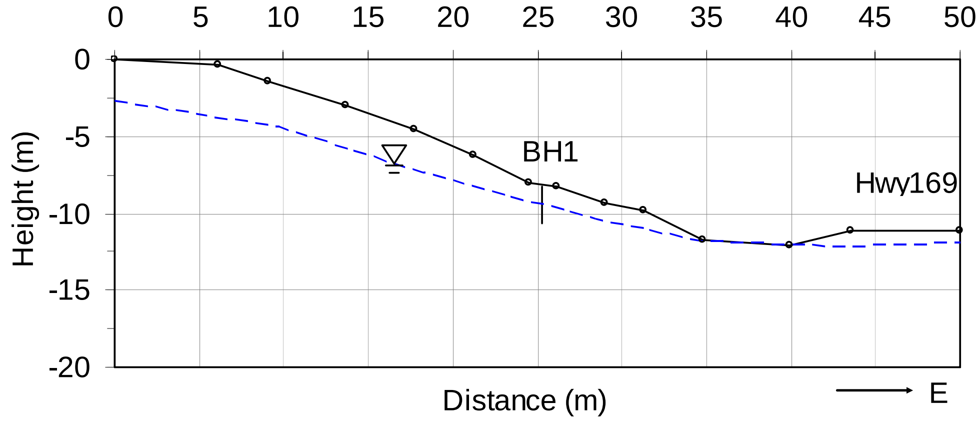


Figure 65. Cross-section for slope 9

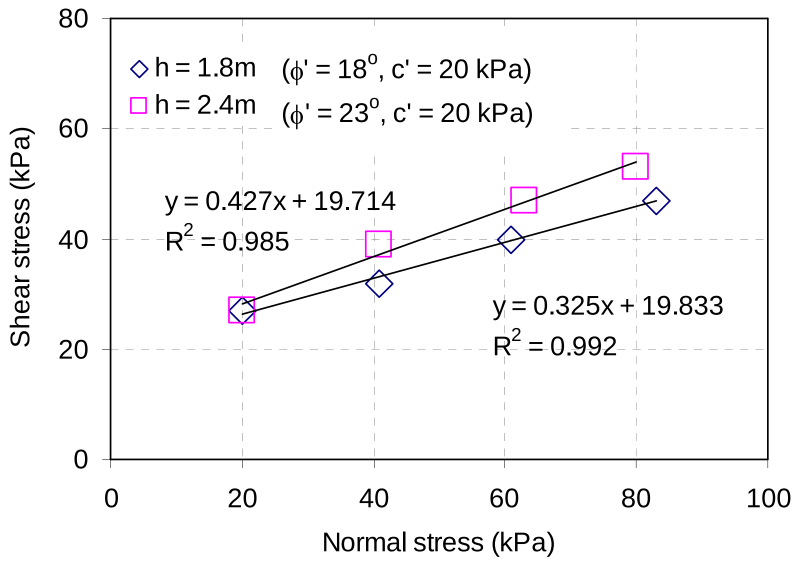


Figure 66. BST results for slope 9

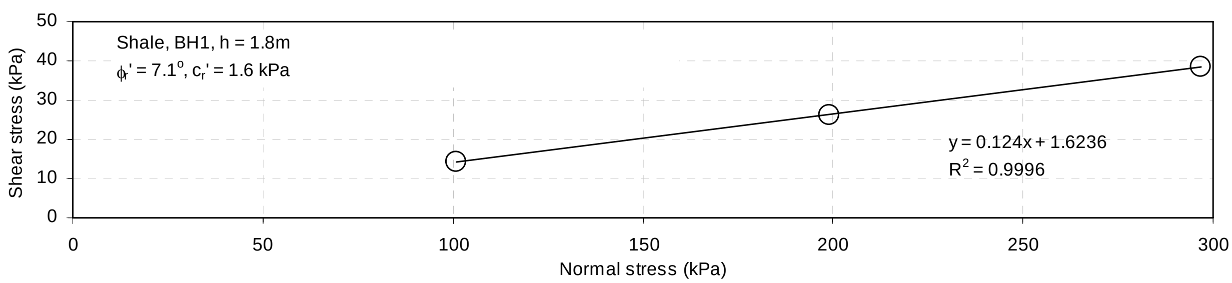


Figure 67. Ring shear test results for slope 9

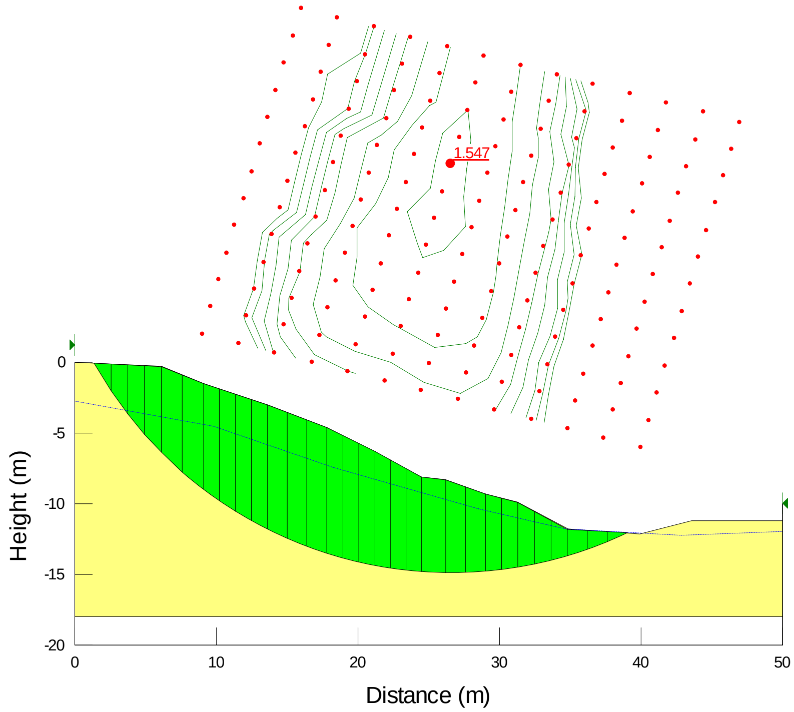


Figure 68. Slope stability analysis for slope 9

Table 27. Summary of basic property results for slope 9

BH	Depth (m)	Soil	Grain Size: Sand (%)	Grain Size: Silt (%)	Grain Size: Clay (%)	Atterberg Limit: LL (%)	Atterberg Limit: PL (%)	Atterberg Limit: PI (%)	Classification: USCS	Classification: AASHTO	Water content (%)	Total density (kN/m3)
1	0.3	g. till	35	34	31	36	17	19	CL	A-6	18.3	
1	0.6	shale	9	44	48	61	27	34	CH	A-7-6	29.2	
1	1.8	shale	17	41	42	61	25	36	CH	A-7-6	26.0	18.5
1	2.4	shale									21.7	

Table 28. Summary of slope analysis results for slope 9

Analysis No.	Shear Strength for Shale: Source	Shear Strength for Shale: ϕ' (deg.)	Shear Strength for Shale: c' (kPa)	Factor of Safety: M-P	Factor of Safety: Bishop
1	BST (average)	21	20	1.547	1.546
2	Ring Shear	7.1	1.6	0.323	0.322
3	Back-calculated	21.0	5.1	1.001	0.996

M-P: Morgenstern-Price method

Slope 10 (Hwy169 Afton Slope C)

Site Conditions

Location

The slope is a cut slope and is located at the east side of Highway 169, 4 miles south of Afton, Union County (Figure 69).

History

The time of the slope failure was unknown. The scarp and the hump generally appeared old in August 2004 when the slope was investigated.

Area Geology

According to the USDA (1978) Soil Survey Report, most of the soils in Union County formed from loess, glacial till, alluvium and shale. Glacial tills dominate the area near Afton, ranging to a depth of 30 m or more.

Field Investigations

Slope Geometry

The slope (Figure 70) had an overall sloping angle of about 20 degree (H:V = 2.8:1), a maximum length of 21 m and a maximum height of 7 m (Figure 71). The width of the slope (at the toe) is about 25 m (along the highway). There was a scarp at middle of the slope and a hump near the toe of the slope. There was also a shallow ditch located at the toe of slope (beside the highway).

Site Geology

One borehole was drilled manually on the slope (Figure 71). The borehole showed that the slope was composed of grey or brown clay shale. The soils were generally soft to medium stiff. The boring log is shown in Appendix (Figure A.24).

Ground Water Level

Ground water level was measured after boring and was shown on the slope profile (Figure 71). The ditch at the toe of the slope was wet and could be indicative of sallow water level near the toe.

Borehole Shear Test Results

BST was conducted at the depth of 2.4 m in the borehole. The results are presented in Figure 72. The results show that ϕ' was 11^o, and *c*' was 8 kPa.

Lab Investigations

Basic Properties

Basic properties for representative soil sample were investigated and the results are summarized in Table 29. The results showed that the shale sample at the depth of 2.4 m comprised 10% of sand, 53% of silt and 37% of clay. Its liquid limit was larger than 50%. The shale was classified as high plasticity clay (CH) by USCS.

Ring Shear Test Results

Ring shear test was conducted for one shale sample and the result is presented in Figure 73. The result indicated that the shale has residual shear strength parameter values of $\phi_r' = 10.6^o$ with $c_r' = 3.4$ kPa.

Slope Analysis

Soil Properties

The field and lab test results show that the slope mainly consists of high plasticity clay shale. Based on the field visual inspections and BST results, the clay shale was assumed to be uniform for the slope stability analysis.

Method of Slope Analysis

In the slope analysis, the slip surface was assumed to be circular (Figure 74) passing the observed scarp. The observed ground water table (GWT) condition was used. Back-calculation was also performed to determine the average shear strength parameter values of the shale giving unity factor of safety (FS).

Results and Discussions

Three slope analyses were performed and the results are given in Table 30. The results show that FS was larger that unity using shear strength parameter values obtained from BST. FS was smaller than one using shear strength parameter values obtained from ring shear test. The back-calculated shear strength parameter values have the same ϕ' with that of the BST values, but with lower c' value.

The results suggest that the slope may have most likely failed under the conditions as in Analysis 3. The shear strength parameter values from back-calculation indicated the average mobilized shear strength during slope failure. Also, the BST results in Table 30 may represent the peak shear strength of the shale. The residual shear strength as obtained from the ring shear test was the ultimate shear strength corresponding to a slide with large displacement.

Conclusions

BST was used to characterize the slope. The shear strength parameter values obtained from BST the soil were used for the slope analysis to investigate the possible cause of the failure. The slope most likely failed with a circular slip surface passing the scarp of the slope. The slope has an average peak shear strength values of $\phi' = 11^0$ and c' = 8 kPa as measured by BST. It has a softened (mobilized) shear strength values of $\phi' = 11^0$ and c' = 4.4 kPa during the slide mobilization; and a residual shear strength of $\phi_r' = 10.6^0$ and c' = 3.4 kPa. The results indicated a softening response of the slope movement.

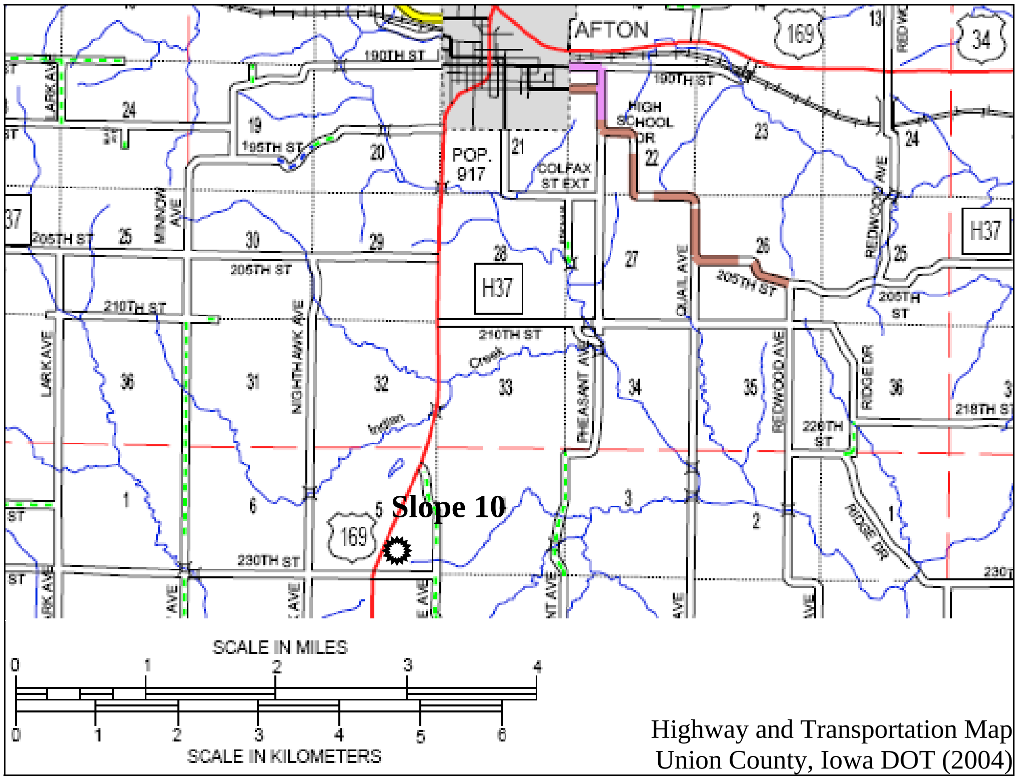


Figure 69. Location of slope 10 (Hwy 169, 4 miles south of Afton, Union Co.)

(a) Looking east, showing the overview of the slope (photo taken by Yang, 08/11/04)

(b) Looking southeast, showing the overview of the slope (photo taken by Yang, 04/16/05)

Figure 70. Photographs for slope 10

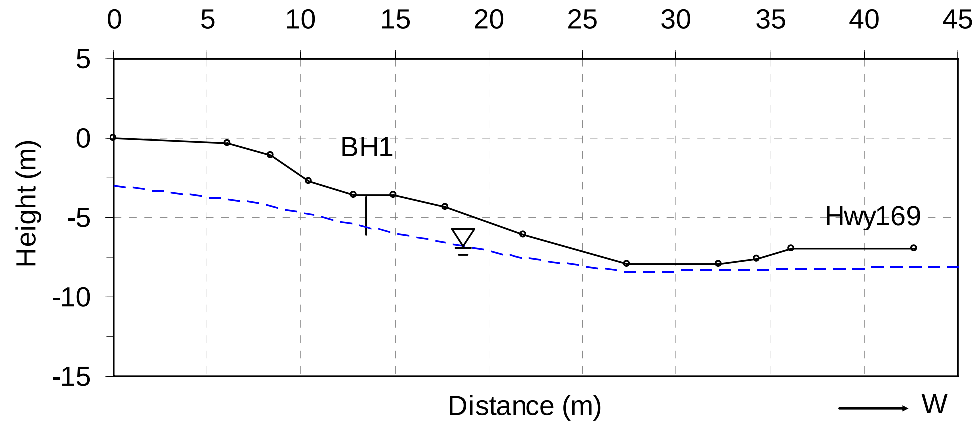


Figure 71. Cross-section for slope 10

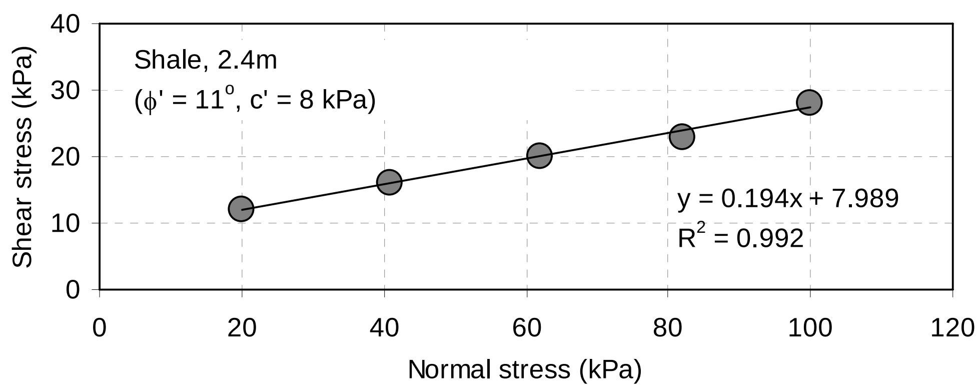


Figure 72. BST results for slope 10

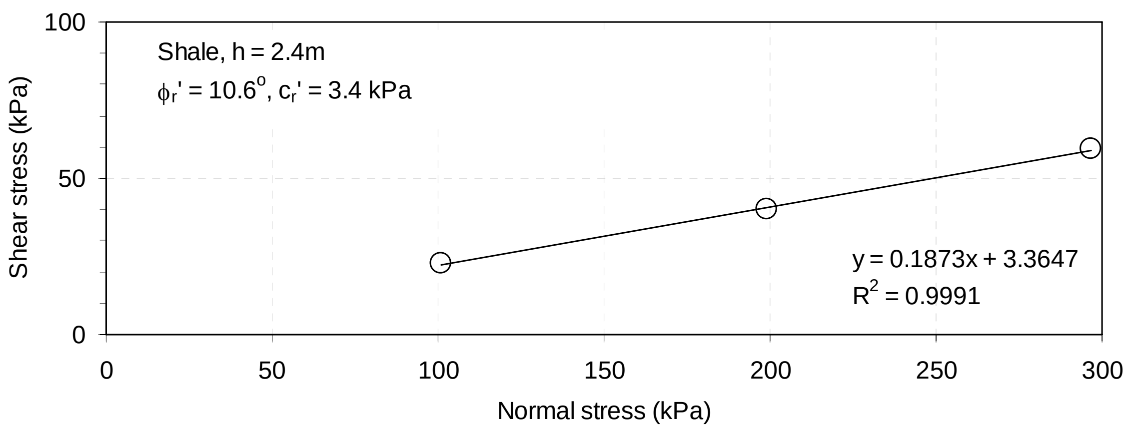


Figure 73. Ring shear test results for slope 10

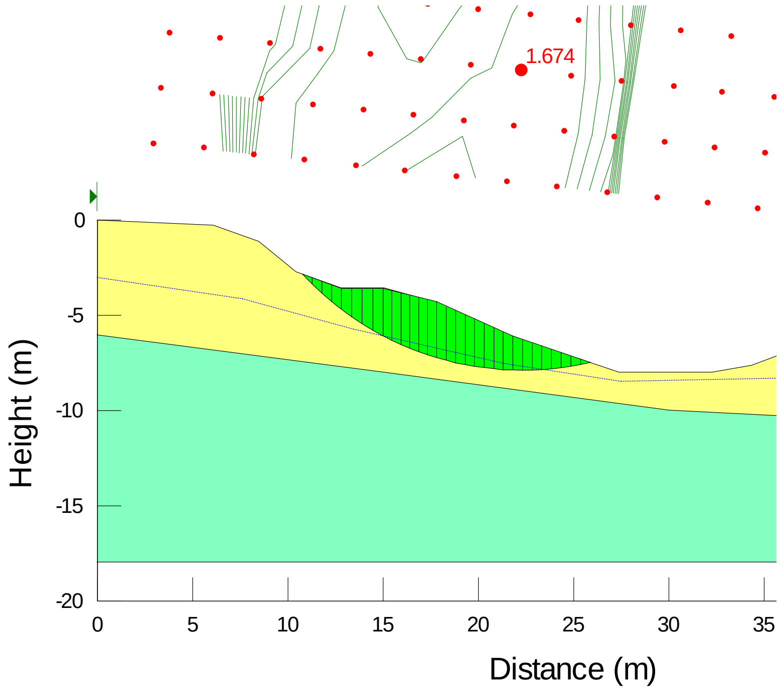


Figure 74. Slope stability analysis for slope 10

Table 29. Summary of basic properties of soil for slope 10

Depth (m)	Soil	Grain Size			Atterberg Limit			Classification		Water content (%)	Total density (kN/m³)
		Sand (%)	Silt (%)	Clay (%)	LL (%)	PL (%)	PI (%)	USCS	AASHTO		
2.4	Shale	10	53	37	51	20	31	CH	A-7-6	23.8	18.0

Table 30. Summary of slope analysis results for slope 10

Analysis No.	Shear Strength for Shale			Factor of Safety	
	Source	ϕ' (deg.)	c' (kPa)	M-P	Bishop
1	BST (average)	11	8	1.674	1.675
2	Ring Shear	10.6	3.4	0.885	0.886
3	Back-calculated	11.0	4.42	1.000	1.000

M-P: Morgenstern-Price method

Slope 11 (Hwy E57 Luther Slope A)

Site Conditions

Location

The slope is a cut slope that is located at the south side of Highway E57, 0.5 mile west of Des Moines River, 4.5 miles west of Luther, Boone County (Figure 75). This slope is connected with Slope 12, and is very close to Slopes 13 and 14. Since Slopes 11 and 12 have quite different geometries and cross-sections, they are treated separately in this and the following sections.

History

The exact history of the development for the slope was not recorded. The air-photo taken on 1994 did not appear to show apparent evidence of landslide, but the air-photos taken in 2002 and 2004 show the scarp and the image of landslide (Figure 76). The scarp and the humps generally appeared old in August 2003 when the slope was first investigated. The bushes and vegetations were well grown on the slope surface (Figure 77). There was also newly repaired pavement near the toe of the slope.

Area Geology

According to the USDA (1981a) Soil Survey Report, the soils of Boone County formed in glacial till and sediment from glacial till, glacial outwash and alluvium, etc. Glacial till is the parent material of most of the soils. Most of the soils formed in glacial till deposited by the most recent, the Wisconsin Glaciations. Sandstone and shale are the oldest parent materials in the county, which were deposited during the Pennsylvanian and Permian Periods.

Field Investigations

Slope Geometry

The slope had an overall sloping angle of about 16 degrees (H:V = 3.5:1), a maximum length of 85 m and a maximum height of 23 m (Figure 78). The width of the slope is about 80 m along the highway. It had a scarp near the top with a maximum height of 5 m. It also had a few cracks near the middle and the toe of the slope. The maximum widths of the cracks were about 0.3 m. There was also a hump near the toe of the slope. A 2 m wide shallow ditch was located at the toe of slope (beside the highway).

Site Geology

Three boreholes were drilled manually on the slope following the direction of the slope movement (Figure 78). The maximum depth of the boreholes was 4.1 m. The boreholes showed that the slope was made of yellowish brown glacial till which was generally soft to medium stiff. Shale was found in some outcrops near the bank of the Des Moines River but was not seen in the slope. The boring logs are shown in the Appendix (Figures A25 to A27).

Ground Water Level

Ground water level was observed to be located near the bottom of the boreholes after boring and was shown on the slope profile (Figure 78). The ditch at the toe of the slope was wet, indicating the ground water level was shallow near the toe.

Borehole Shear Test Results

BSTs were conducted near the bottom of the boreholes to obtain the shear strength of the soils. The results are presented in Figure 79 and Table 31. The results show that ϕ' ranged from 15^o to 22^o, and c' varied from 7 to 12 kPa.

Lab Investigations

Basic Properties

Basic properties for representative soil samples were investigated and the results are summarized in Table 32. The results showed that the glacial till sample comprised 48% of sand, 33% of silt and 19% of clay. Its liquid limit was only 28%. The soil was classified as low plasticity clay (CL) by USCS.

Ring Shear Test Results

Ring shear test was conducted for one soil sample and the result is presented in Figure 80. The result indicated that the glacial till has residual shear strength parameter values of $\phi_r' = 25.7^o$ with $c_r' = 3.8$ kPa, which relatively close the shear strength parameter values as obtained from BST.

Slope Analysis

Soil Properties

The field and lab test results show that the slope mainly consists of low plasticity glacial till. Based on the field visual inspections and BST results, the soil was assumed to be uniform for the slope stability analysis.

Method of Slope Analysis

In the slope analysis, the slip surface was assumed to be circular passing the observed scarp (Figure 81). The observed ground water table (GWT) condition was used. Back-calculation was also performed to determine the average shear strength parameter values of the soil giving unity factor of safety (FS).

Results and Discussions

Three slope analyses were performed and the results are given in Table 33. The results show that FS was larger that 1.0 using the average shear strength parameter values obtained from BST and shear strength parameter values obtained from ring shear test. The back-calculated shear strength parameter values have slightly lower ϕ' than that BST with zero c' value.

All these results suggest that the slope may have most likely failed under the conditions as in Analysis 3 (back-calculation). The shear strength parameter values from back-calculation indicated the average mobilized shear strength during slope failure. Also, the BST results in Table 33 may represent the peak shear strength of the shale. The residual shear strength as obtained from the ring shear test was the ultimate shear strength corresponding to a slide with large displacement. This value was higher than the average value obtained from BST and the values obtained from back-calculation, which may be due to the soil variability. The soil sample for the ring shear test may not be exactly the same with what BST has been performed. Another possible reason was that the soil may have exhibited a hardening response during shearing.

Conclusions

BST was used to characterize the slope. The shear strength parameter values obtained from BST for the soil were used for the slope analysis to investigate the possible cause of the failure. The slope most likely failed with a circular slip surface passing through the observed scarp of the slope. The soil in the slope has an average peak shear strength parameter values of $\phi' = 20^0$ and c' = 10 kPa as measured by BST. It has a softened (mobilized) shear strength parameter values of $\phi' = 17.7^0$ and c' = 0 kPa during the slide mobilization; and residual shear strength parameter values of $\phi_r' = 25.7^0$ and c' = 3.8 kPa as measured by ring shear test. The measured residual strength may be the upper bound strength value for the glacial till in the slope, and the soil may exhibit hardening response during shearing.

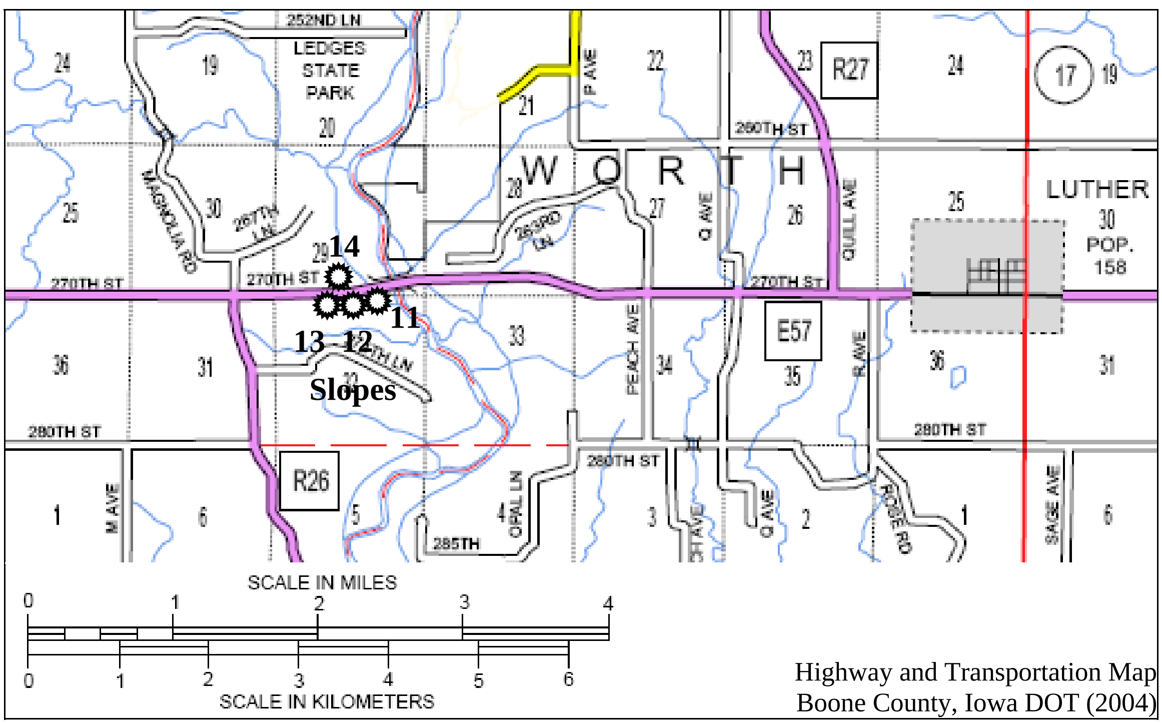
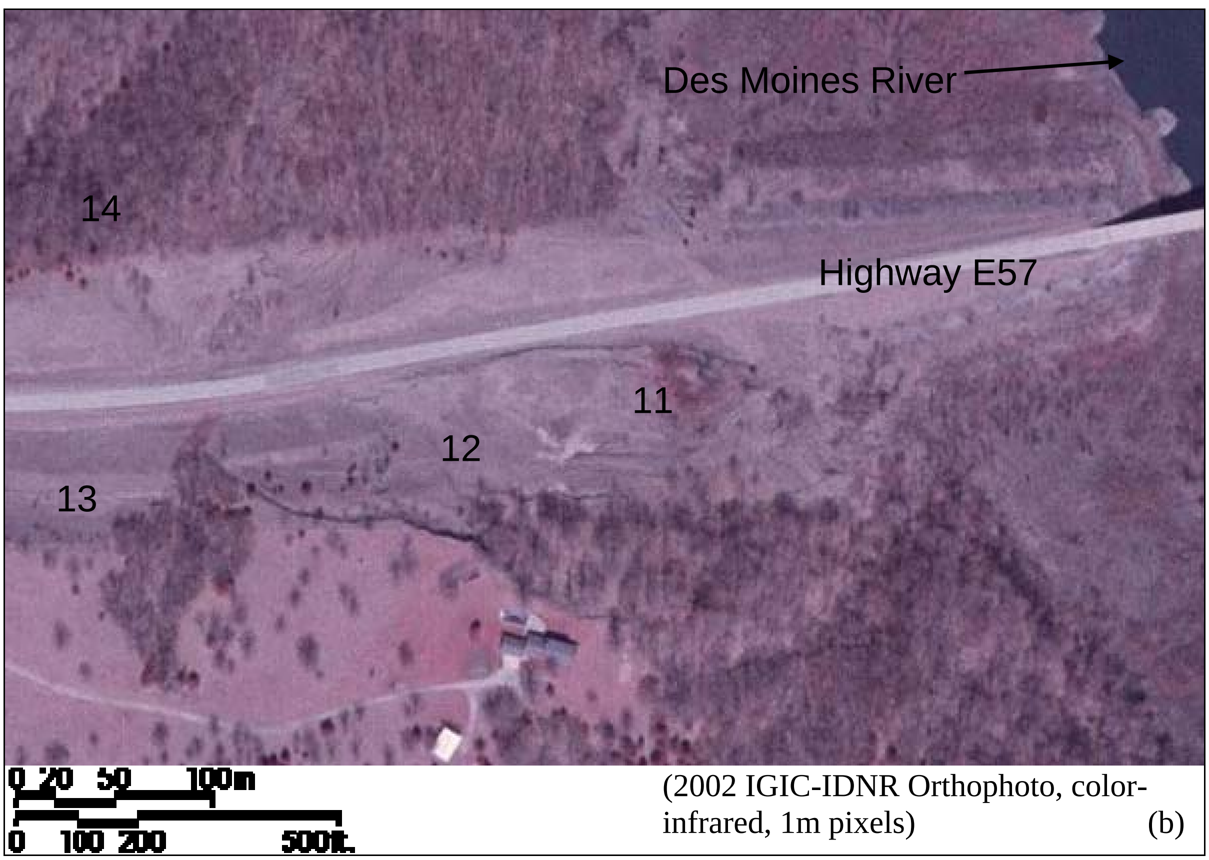


Figure 75. Location of slopes 11-14 (Hwy E57, 4.5 miles west of Luther, Boone Co.)

(1994 USGS Orthophoto, gray-scale, 1m pixels) (a)



(2002 IGIC-IDNR Orthophoto, color-infrared, 1m pixels) (b)

(2004 USDA Orthophoto, natural color, 2m pixels) (c)

Figure 76. Air-photos for slopes 11 to 14 (Source: http://cairo.gis.iastate.edu/)


(a) Looking southwest, overview of the slopes 11 and 12 (photo taken by Thompson, 08/27/03)


(b) Looking southwest, overview of the slopes 11 and 12 (photo taken by Thompson, 03/30/04)


(c) Looking southeast, overview of the slope 11 (photo taken by Thompson, 03/30/04)


(d) Looking southwest, scarps of the slope 11 (photo taken by Thompson, 03/30/04)


(e) Looking southwest, overview of the slope 12 (photo taken by Thompson, 03/30/04)

Figure 77. Photographs for slopes 11 and 12

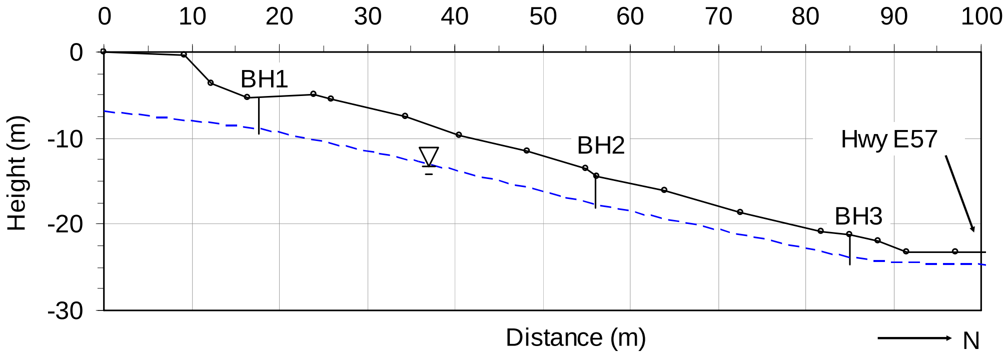


Figure 78. Cross-section for slope 11

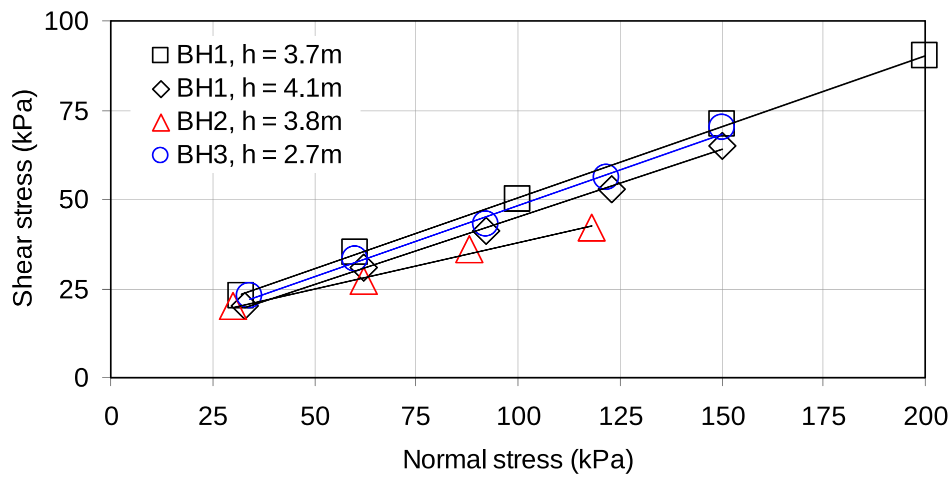


Figure 79. BST results for slope 11

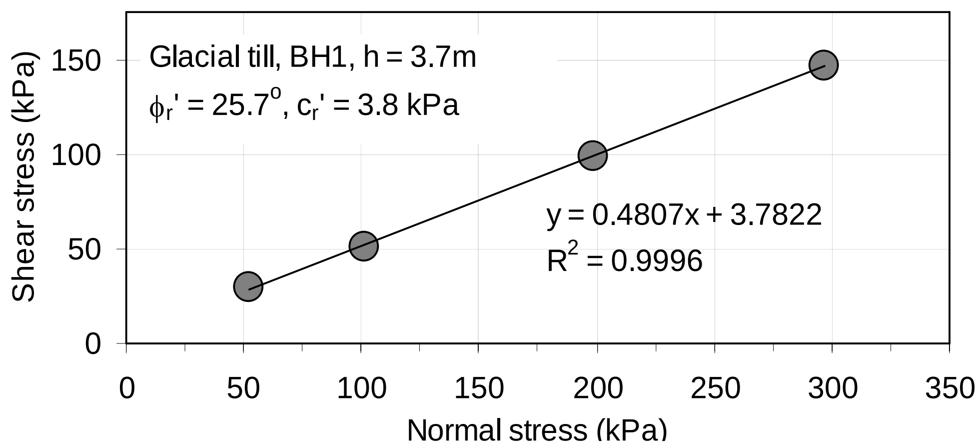


Figure 80. Ring shear test results for slope 11

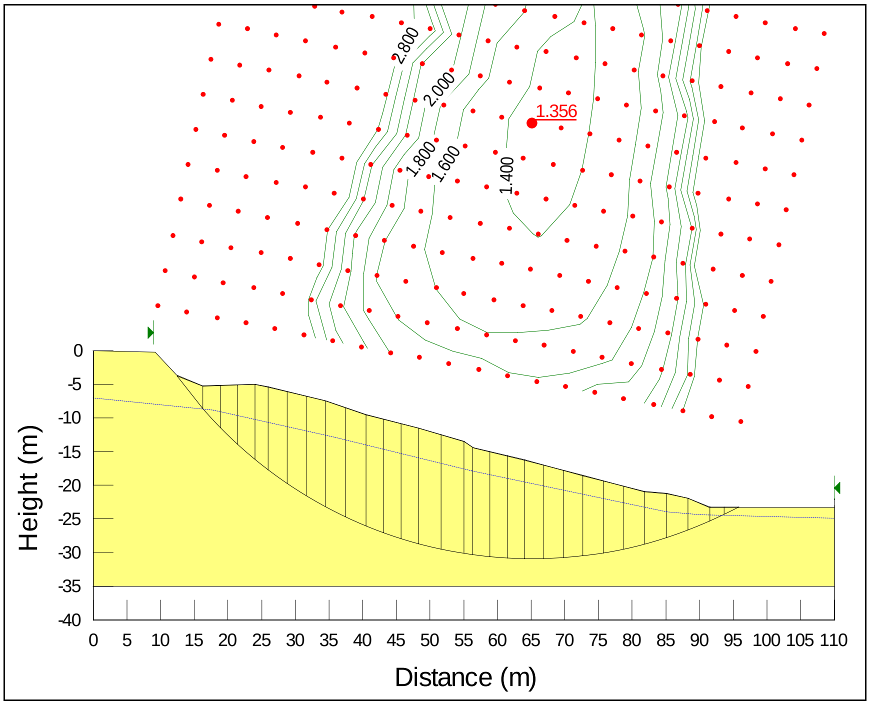


Figure 81. Slope stability analysis for slope 11

Table 31. Summary of BST results for slope 11

BH	Depth (m)	Soil	ϕ' (deg.)	c' (kPa)	R^2	Data points
1	3.7	Glacial till	22	11	1.000	5
1	4.1	Glacial till	21	7	0.998	5
2	3.8	Glacial till	15	12	0.990	4
3	2.7	Glacial till	22	9	0.993	5

Table 32. Summary of basic property results for slope 11

BH	Depth (m)	Grain Size			Atterberg Limit			Classification		Water content (%)	Total density (kN/m3)
		Sand (%)	Silt (%)	Clay (%)	LL (%)	PL (%)	PI (%)	USCS	AASHTO		
1	3.7	48	33	19	28	13	14	CL	A-6	17.8	19.2
2	3.8									13.1	
3	2.7									15.0	19.4

Table 33. Summary of slope analysis results for slope 11

Analysis No.	Shear Strength			Factor of Safety	
	Source	ϕ' (deg.)	c' (kPa)	M-P	Bishop
1	BST (average)	20	10	1.356	1.353
2	Ring Shear	25.7	3.8	1.539	1.534
3	Back-calculated	17.7	0.0	1.001	0.997

M-P: Morgenstern-Price method

Slope 12 (Hwy E57 Luther Slope B)

Site Conditions

Location

The slope is a cut slope that is located at the south side of Highway E57, 0.5 mile west of Des Moines River, 4.5 miles west of Luther, Boone County (Figure 75, see the section for Slope 11). This slope is connected with Slope 11, and is very close to Slopes 13 and 14. Since Slopes 11 and 12 have quite different geometry in their cross-section, they were treated separately in this and the following section.

History

The exact history of the development for the slope was not recorded. The air-photo taken on 1994 did not appear to show apparent evidence of landslide, but the air-photos taken in 2002 and 2004 show the scarp and the image of landslide (Figure 76, see the section for Slope 11). The scarp and the humps generally appeared old in August 2003 when the slope was first investigated. The bushes and vegetations were well grown on the slope surface (Figure 77, see the section for Slope 11). There was also newly repaired pavement near the toe of the slope.

Area Geology

According to the USDA (1981a) Soil Survey Report, the soils of Boone County formed in glacial till and sediment from glacial till, glacial outwash and alluvium, etc. Glacial till is the parent material of most of the soils. Most of the soils formed in glacial till deposited by the most recent, the Wisconsin Glaciations. Sandstone and shale are the oldest parent materials in the county, which were deposited during the Pennsylvanian and Permian Periods.

Field Investigations

Slope Geometry

The slope had an overall sloping angle of about 18 degree (H:V = 3.0:1), a maximum length of 63 m and a maximum height of 20 m (Figure 82). The width of the slope is about 70 m along the highway. It had a scarp near the top with a maximum height of 2 m. It also had a few cracks and a hump near the middle of the slope. The maximum widths of the cracks were about 0.3 m. A small ditch was located at the toe of slope (beside the highway).

Site Geology

Three boreholes were drilled manually on the slope following the direction of the slope movement (Figure 82). The maximum depth of the boreholes was 4.1 m. The boreholes showed that the slope was made of yellowish brown glacial till which was generally soft to medium stiff. Shale was found in some outcrops near the bank of the Des Moines River but was not seen in the slope. The boring logs are shown in Appendix (Figures A.28 to A.30).

Ground Water Level

Ground water level was observed only in the boreholes of BH1 and B3 after the boring. GWL was not observed in BH2 near the center of the slope (Figure 82). The ditch at the toe of the slope was wet indicating the GWT was shallow near the toe.

Borehole Shear Test Results

BSTs were conducted near the bottom of the boreholes to obtain the shear strength of the soils. The results are presented in Figure 83 and Table 34. The results show that ϕ' ranged from 12^o to 36^o, and *c*' varied from 1 to 10 kPa.

Lab Investigations

Basic Properties

Basic properties for representative soil sample were investigated and the results are summarized in Table 35. The results showed that the glacial till sample comprised 32% of sand, 45% of silt and 23% of clay. Its liquid limit was only 31%. The soil was classified as low plasticity clay (CL) by USCS.

Ring Shear Test Results

Ring shear test was conducted for one soil sample and the result is presented in Figure 84. The result indicated that the glacial till has residual shear strength parameter values of $\phi_r' = 24.3^o$ with $c_r' = 0$ kPa, which is relatively close the average shear strength parameter values as obtained from BST.

Slope Analysis

Soil Properties

The field and lab test results show that the slope mainly consists of low plasticity glacial till. Based on the field visual inspections and BST results, the soil was assumed to be uniform for the slope stability analysis.

Method of Slope Analysis

In the slope analysis, the slip surface was assumed to be circular passing the observed scarp (Figure 85). The observed ground water table (GWT) condition was used. Back-calculation was also performed to determine the average shear strength parameter values of the soil giving unity factor of safety (FS).

Results and Discussions

Three slope analyses were performed and the results are given in Table 36. The results show that FS was larger that 1.0 using the average shear strength parameter values obtained from BST. FS was slightly larger than one using shear strength parameter values obtained from the ring shear test. The back-calculated shear strength parameter values have slightly lower ϕ' than those from BST.

All these results suggest that the slope may have most likely failed under the conditions as in Analysis 3 (FS = 1.0). The shear strength parameter values from back-calculation indicated the average mobilized shear strength during slope failure under the assumed ground water conditions. The back-calculated shear strength was higher than the residual strength, which could be attributed to two possible reasons: (1) the residual strength was not representative of the average soil conditions along the slip surface; (2) the ground water level could be higher than the observed (adopted) water level, thus the shear strength at failure should be higher than the back-calculated strength. The BST results in Table 36 may represent the average peak shear strength of the shale. The residual shear strength as obtained from the ring shear test was the ultimate shear strength corresponding to a slide with large displacement.

Conclusions

BST was used to characterize the slope. The shear strength parameter values obtained from BST were used for the slope analysis to investigate the possible cause of the failure. The slope most likely failed with a circular slip surface passing the observed scarp near the top of the slope. The soil in the slope has an average peak shear strength values of $\phi' = 24^0$ and c’ = 6 kPa as measured by BST; and a residual shear strength of $\phi_r' = 24.3^0$ and c’ = 0 kPa.

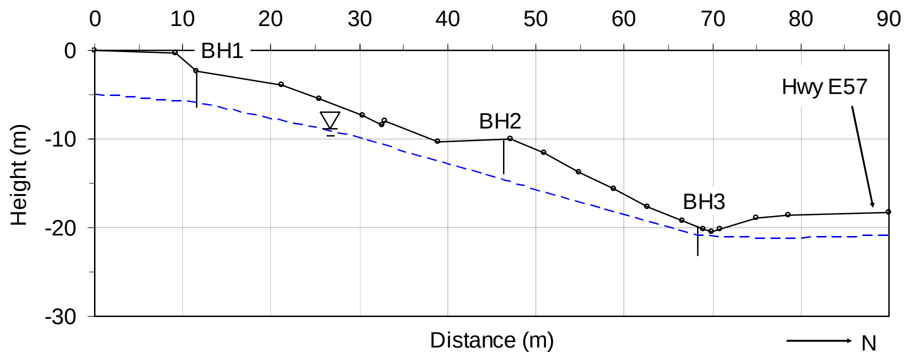


Figure 82. Cross-section for slope 12

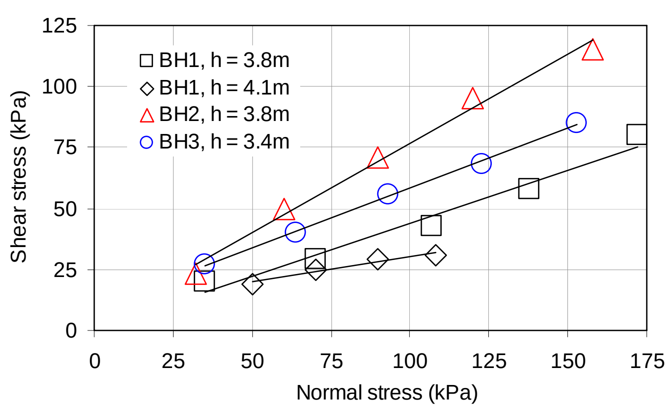


Figure 83. BST results for slope 12

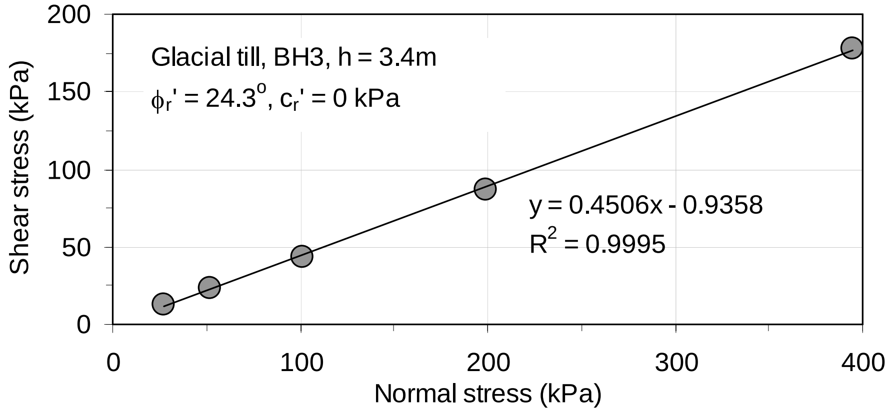


Figure 84. Ring shear test results for slope 12

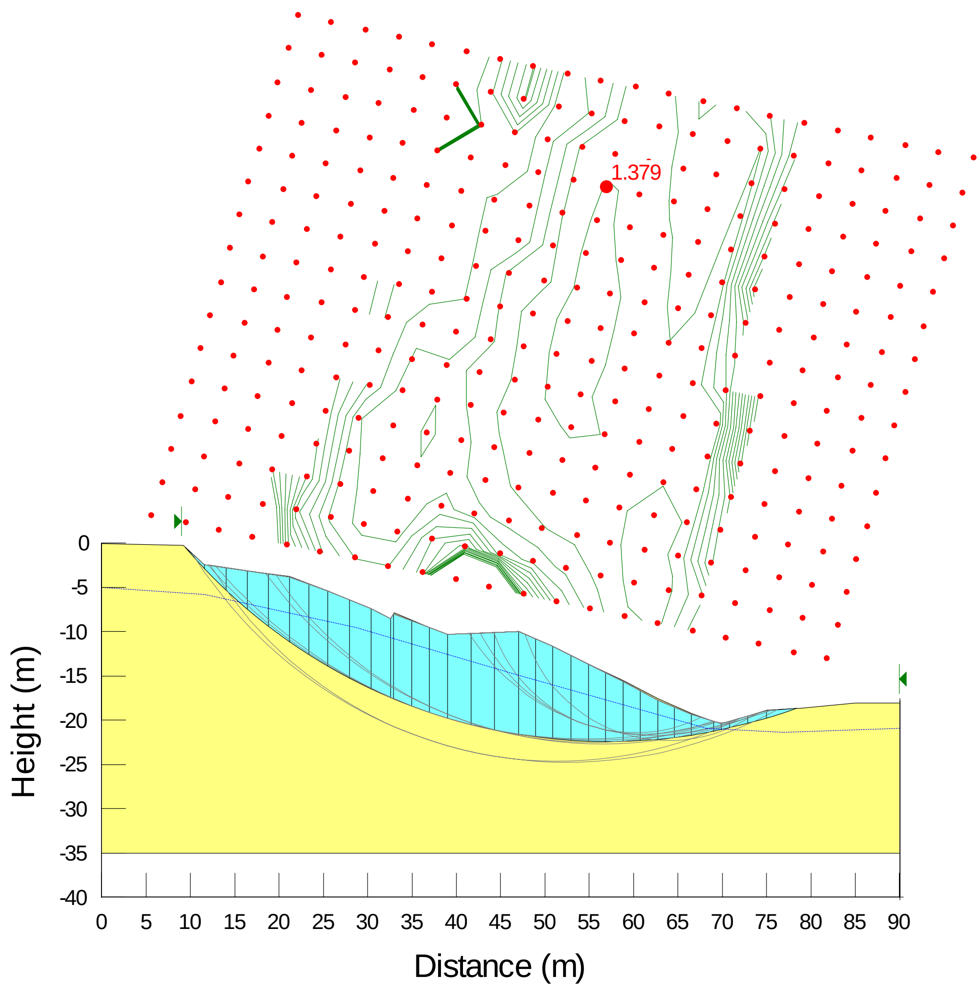


Figure 85. Slope stability analysis for slope 12

Table 34. Summary of BST results for slope 12

BH	Depth (m)	Soil	ϕ' (deg.)	c' (kPa)	R^2	Data points
1	3.8	Glacial till	23	1	0.971	5
1	4.1	Glacial till	12	10	0.962	4
2	3.8	Glacial till	36	4	0.990	5
3	3.4	Glacial till	26	10	0.998	5

Table 35. Summary of basic property results for slope 12

BH	Depth (m)	Grain Size			Atterberg Limit			Classification		Water content (%)	Total density (kN/m3)
		Sand (%)	Silt (%)	Clay (%)	LL (%)	PL (%)	PI (%)	USCS	AASHTO		
1	3.7									21.0	19.6
2	3.8									15.0	
3	2.7	32	45	23	31	14	17	CL	A-6	15.0	19.2

Table 36. Summary of slope analysis results for slope 12

Analysis No.	Shear Strength			Factor of Safety	
	Source	ϕ' (deg.)	c' (kPa)	M-P	Bishop
1	BST (average)	24	6	1.379	1.376
2	Ring Shear	24.3	0.0	1.194	1.191
3	Back-calculated	20.7	0.0	1.000	0.997

M-P: Morgenstern-Price method

Slope 13 (Hwy E57 Luther Slope C)

Site Conditions

<u>Location</u>

The slope is a cut slope that is located at the south side of Highway E57, 0.5 mile west Des Moines River, 4.5 miles west of Luther, Boone County (Figure 75, see the section for Slope 11). This slope is very close to Slopes 11 and 12.

<u>History</u>

The exact history of the development for the slide was not recorded. The air-photo did not appear to show apparent evidence of landslide (Figure 76, see the section for Slope 11). The scarp and the hump generally appeared old in August 2003 when the slope was first investigated. The bushes and vegetations grew well on the slope surface (Figure 86).

<u>Area Geology</u>

According to the USDA (1981a) Soil Survey Report, the soils of Boone County formed in glacial till and sediment from glacial till, glacial outwash and alluvium, etc. Glacial till is the parent material of most of the soils. Most of the soils formed in glacial till deposited by the most recent, the Wisconsin Glaciations. Sandstone and shale are the oldest parent materials in the county, which were deposited during the Pennsylvanian and Permian Periods.

Field Investigations

<u>Slope Geometry</u>

The slope had an overall sloping angle of about 18 degree (H:V = 3.0:1), a maximum length of 58 m and a maximum height of 16 m (Figure 87). The width of the slope with slide is about 10 m parallel to the highway. It had a scarp near the center of the slope with a maximum height of 0.5 m. It also had a hump downside of the scarp near the middle of the slope.

<u>Site Geology</u>

One borehole was drilled manually on the slope near the scarp (Figure 87). The maximum depth of the borehole was 3.1 m. The borehole showed that the slope was made of yellowish brown glacial till which was generally soft to medium stiff. Shale was found in some outcrops near the bank of the Des Moines River but was not seen in the slope. The boring log is shown in the Appendix (Figure A31).

<u>Ground Water Level</u>

Ground water level was observed at the bottom of the borehole after boring (Figure 87). The ground water level was estimated for the whole slope based on the observation in the borehole.

Borehole Shear Test Results

BST was conducted near the bottom of the borehole to obtain the shear strength of the soil. The results are presented in Figure 88. The results show that $\phi' = 16^o$, and $c' = 8$ kPa.

Lab Investigations

Basic Properties

Basic properties for representative soil sample were investigated and the results are summarized in Table 37. The results showed that the glacial till sample comprised 47% of sand, 34% of silt and 19% of clay. Its liquid limit was only 27%. The soil was classified as low plasticity clay (CL) by USCS.

Ring Shear Test Results

Ring shear test was conducted for one soil sample and the result is presented in Figure 89. The result indicated that the glacial till has residual shear strength parameter values of $\phi_r' = 26.0^o$ with $c_r' = 0.1$ kPa.

Slope Analysis

Soil Properties

The field and lab test results show that the slope mainly consists of low plasticity glacial till. Based on the field visual inspections and BST results, the soil was assumed to be uniform for the slope. Slope stability analysis was performed accordingly.

Method of Slope Analysis

In the slope analysis, the slip surface was assumed to be circular passing the observed scarp (Figure 90). The slope was assumed to be uniform for the slope analysis. The observed ground water level condition was used. Back-calculation was also performed to determine the average shear strength parameter values of the soil giving unity factor of safety (FS).

Results and Discussions

Three slope analyses were performed and the results are given in Table 38. The results show that FS was larger that 1.0 using the average shear strength parameter values obtained from BST. FS was also larger than 1.0 using shear strength parameter values obtained from ring shear test. The back-calculated shear strength parameter values have slightly lower ϕ' than that from BST and with a zero c' value.

All these results suggest that the slope may have most likely failed under the conditions as in Analysis 3 (the back-calculation). The shear strength parameter values from back-calculation indicated the average mobilized shear strength during slope failure. Also, the BST results in Table 38 may represent the peak shear strength of the shale. The residual shear strength as obtained from the ring shear test was the ultimate shear strength corresponding to a slide with large displacement. This value was higher than the average value obtained from BST and the values obtained from back-calculation, which may be due to the soil variability. The soil sample for the ring shear test may not be exactly the same with what BST has been performed on. Another possible reason is that the soil may have exhibited ductile or hardening response with respect to shearing.

Conclusions

BST was used to characterize the slope. The shear strength parameter values obtained from BST were used for the slope analysis to investigate the possible cause of the failure. The slope most likely failed with a circular slip surface passing the observed scarp near mid of the slope. The soil in the slope had an average peak shear strength values of $\phi' = 16^0$ and c' = 8 kPa as measured by BST. It had a mobilized shear strength values of $\phi' = 14.3^0$ and c' = 0 kPa during the slide mobilization. The residual shear strength of the soil was $\phi_r' = 26.0^0$ and c' = 0.1 kPa. The measured residual strength could be the upper bound strength value for the glacial till in the slope. The soil could have exhibited strain-hardening response with respect to shearing.

Figure 86. Slope 13, looking southwest (photo taken by Yang, 10/10/04)

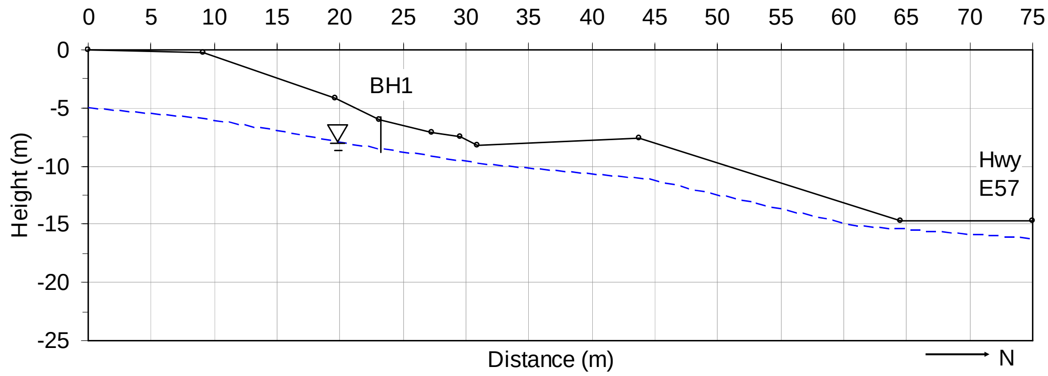


Figure 87. Cross-section for slope 13

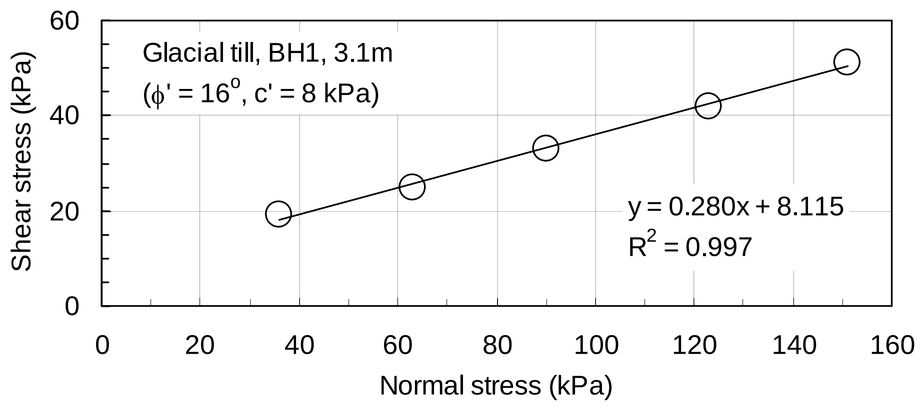


Figure 88. BST results for slope 13

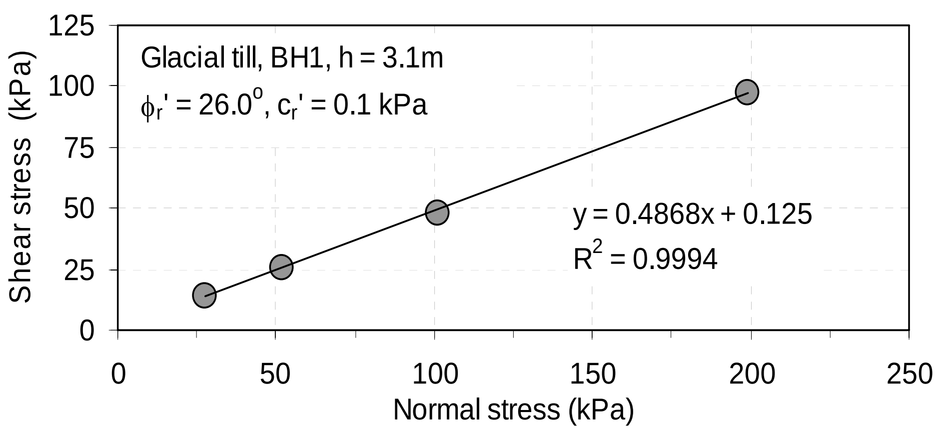


Figure 89. Ring shear test results for slope 13

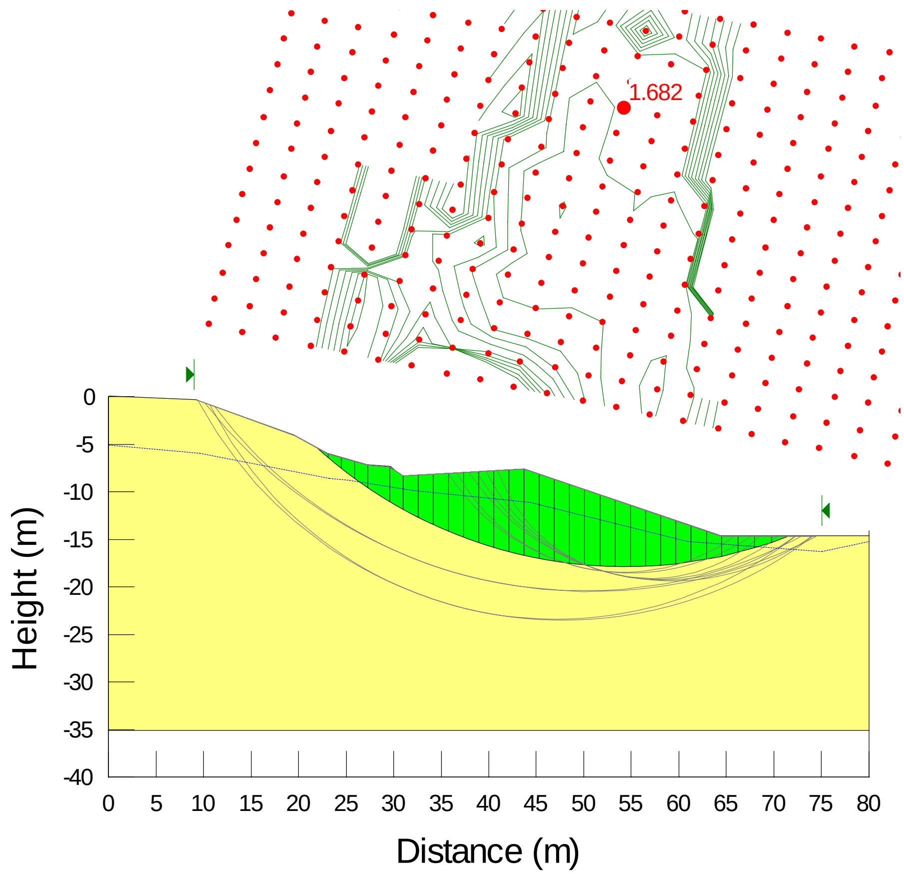


Figure 90. Slope stability analysis for slope 13

Table 37 Summary of basic property results for slope 13

BH	Depth (m)	Grain Size			Atterberg Limit			Classification		Water content (%)	Total density (kN/m^3)
		Sand (%)	Silt (%)	Clay (%)	LL (%)	PL (%)	PI (%)	USCS	AASHTO		
1	3.1	47	34	19	27	13	14	CL	A-6	13.1	18.3

Table 38. Summary of slope analysis results for slope 13

Analysis No.	Shear Strength			Factor of Safety	
	Source	ϕ' (deg.)	c' (kPa)	M-P	Bishop
1	BST	16	8	1.682	1.682
2	Ring Shear	26.0	0.1	1.925	1.923
3	Back-calculated	14.3	0.0	1.001	1.000

M-P: Morgenstern-Price method

Slope 14 (Hwy E57 Luther Slope D)

Site Conditions

Location

The slope is a fill slope and is located at the north side of Highway E57, 0.5 mile west Des Moines River, 4.5 miles west of Luther, Boone County (Figure75, see the section for Slope 11). This slope is on the opposite side of the road to Slopes 11 and 12.

History

The exact history of the development for the slope was not recorded. The air-photo did not appear to show apparent evidence of landslide (Figure 76, see the section for Slope 11). The scarp and the hump generally appeared old in August 2003 when the slope was first investigated. The bushes and vegetations were well grown on the slope surface (Figure 91).

Area Geology

According to the USDA (1981a) Soil Survey Report, the soils of Boone County formed in glacial till and sediment from glacial till, glacial outwash and alluvium, etc. Glacial till is the parent material of most of the soils. Most of the soils formed in glacial till deposited by the most recent, the Wisconsin Glaciations. Sandstone and shale are the oldest parent materials in the county, which were deposited during the Pennsylvanian and Permian Periods.

Field Investigations

Slope Geometry

The slope had an overall sloping angle of about 12 degrees (H:V = 3.0:1), a maximum length of 47 m and a maximum height of 10 m (Figure 92). The width of the slope with slide is about 30 m. It had multiple minor scarps on the surface of the slope with height ranging from 0.1 of 0.3 m. It also had a hump downside of the scarp near the toe of the slope. The toe of the slope is partially eroded by the creek.

Site Geology

One borehole was drilled manually on the mid of slope (Figure 92). The depth of the borehole was 2.7 m. The borehole showed that the slope was made of yellowish brown glacial till which was generally soft to medium stiff. Shale was found in some outcrops near the bank of the Des Moines River but was not seen in the slope. The boring log is shown in the Appendix (Figure A32).

Ground Water Level

Ground water level was not observed in the borehole after boring (Figure 92). The ground water level was estimated for the whole slope based on the water level in the creek.

Borehole Shear Test Results

BST was conducted near the bottom of the borehole to obtain the shear strength of the soil. The results are presented in Figure 93. The results show that $\phi' = 15^{o}$, and $c' = 10$ kPa.

Lab Investigations

Basic Properties

Basic properties for representative soil sample were investigated and the results are summarized in Table 39. The results showed that the glacial till sample comprised 49% of sand, 32% of silt and 19% of clay. Its liquid limit was only 27%. The soil was classified as low plasticity clay (CL) by USCS.

Ring Shear Test Results

Ring shear test was conducted for one soil sample and the result is presented in Figure 94. The result indicated that the glacial till has residual shear strength parameter values of $\phi_r' = 26.9^{o}$ with $c_r' = 4.2$ kPa.

Slope Analysis

Soil Properties

The field and lab test results show that the slope mainly consists of low plasticity glacial till. Based on the field visual inspections and BST results, the soil was assumed to be uniform for the slope stability analysis.

Method of Slope Analysis

In the slope analysis, the slip surface was assumed to be circular passing the main scarp (Figure 95). The estimated ground water table (GWT) condition was used. Back-calculation was also performed to determine the average shear strength parameter values of the soil giving unity factor of safety (FS).

Results and Discussions

Three slope analyses were performed and the results were given in Table 40. The results show that FS was larger that 1.0 using the shear strength parameter values obtained from BST. FS was also larger than 1.0 using shear strength parameter values obtained from ring shear test. The back-calculated shear strength parameter values have same ϕ' than that from BST but with a lower c' value.

All these results suggest that the slope could have failed under the conditions as in Analysis 3 (the back-calculation). The shear strength parameter values from back-calculation indicated the average mobilized shear strength during slope failure. The BST results in Table 40 represented the in-situ strength of the soil. The residual shear strength as obtained from the ring shear test was the ultimate shear strength corresponding to large displacement of the soil. This value was higher than the value obtained from BST and the value obtained from back-calculation, which may be due to the soil variability. The soil sample for the ring shear test may not be exactly the same as what BST has been performed on. It could also be attributed to the ductile or hardening behavior that the soil exhibited.

Conclusions

BST was used to characterize the slope. The shear strength parameter values obtained from BST were used for the slope analysis to investigate the possible cause of the failure. The slope could have failed with a circular slip surface passing the observed main scarp of the slope. The soil in the slope had an average in-situ strength values of $\phi' = 15^0$ and c' = 10 kPa as measured by BST. It probably had a mobilized shear strength values of $\phi' = 15^0$ and c' = 6.5 kPa during the failure. The residual shear strength parameter values were $\phi_r' = 26.9^0$ and c' = 4.2 kPa, which could be the upper bound shear strength value for the glacial till in the slope. The soil could have exhibited ductile or hardening response with respect to shearing.

Figure 91. Slope 14, looking southeast (photo taken by Yang, 10/10/04)

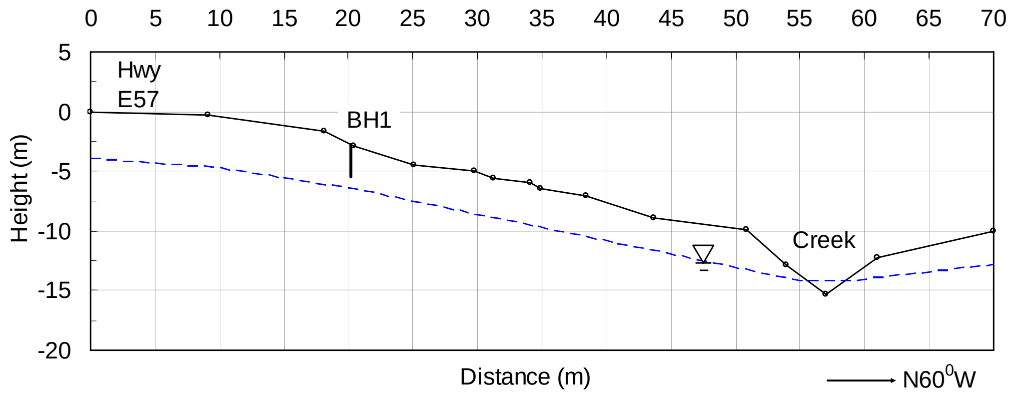


Figure 92. Cross-section for slope 14

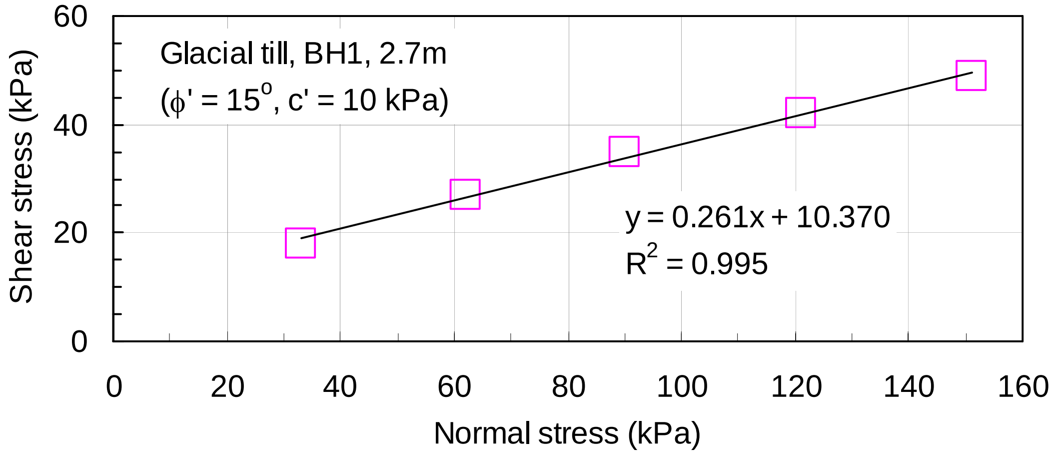


Figure 93. BST results for slope 14

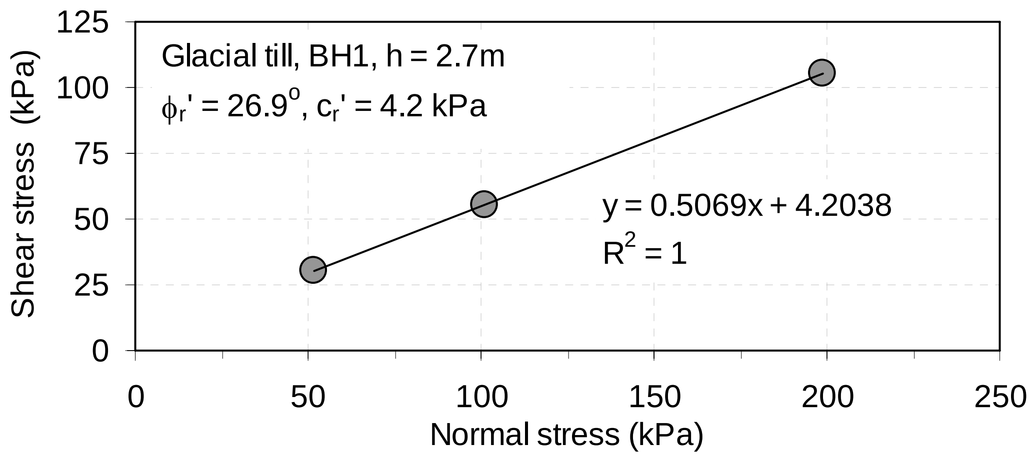


Figure 94. Ring shear test results for slope 14

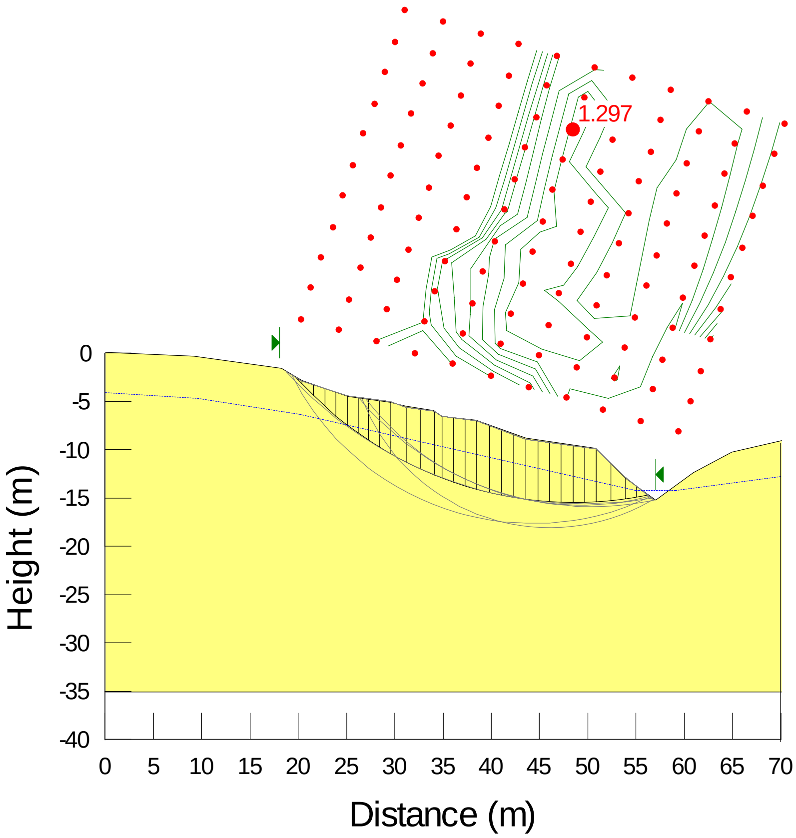


Figure 95. Slope stability analysis for slope 14

Table 39. Summary of basic property results for slope 14

BH	Depth (m)	Grain Size			Atterberg Limit			Classification		Water	Total
		Sand (%)	Silt (%)	Clay (%)	LL (%)	PL (%)	PI (%)	USCS	AASHTO	content (%)	density (kN/m^3)
1	2.7	49	32	19	27	14	13	CL	A-6	10.6	18.0

Table 40. Summary of slope analysis results for slope 14

Analysis No.	Shear Strength			Factor of Safety	
	Source	ϕ' (deg.)	c' (kPa)	M-P	Bishop
1	BST	15	10	1.297	1.298
2	Ring Shear	26.9	4.2	1.722	1.721
3	Back-calculated	15.0	6.5	1.000	1.000

M-P: Morgenstern-Price method

Slope 15 (Hwy63 Sugar Creek)

Site Conditions

Location

The project is located at the site of the proposed Highway 63 over Sugar Creek Bridges, Bridge Design No. 1001, Segment 3, Ottumwa Bypass, east of Ottumwa, Wapello County, Iowa (Figure 96). The project involves embankment fill slopes.

History

Approach embankment fills on both sides of the Sugar Creek with pile-supported abutments were designed previously to support the Highway US 63 over Sugar Creek Bridge in Wapello County, Iowa. Slope analyses indicated potential global instability for the slopes in front of the abutments with slip surface passing through the highly weathered shale, when assuming a shear strength parameter, effective cohesion (c') of 10 kPa, in accordance with IaDOT design guidelines. As a result, ground improvement alternatives and retaining wall alternatives were proposed by CH2MHill (2004) with the estimated costs ranging 3 to 5 million dollars.

In view of the high costs, a comprehensive supplemental subsurface exploration and testing program was developed by CH2MHill and executed at the site with the joint effort from Iowa State University (ISU) in August 2004. The purpose of the program was to verify that the shear strength parameters for the soils, especially for the highly weathered shale, used in the slope stability analyses were reasonable; and to possibly develop more realistic and site specific design parameters to optimize the design, justify and /or possibly reduce the estimated costs of any measures if required (CH2MHill 2005).

During the initial field investigation program for the project in April 2001, 16 bridge borings were drilled. Except for the information of soil stratification for the site, the investigation program only provided shear strength parameters for three samples of alluvial soils. No shear strength parameters for the shales were obtained (CH2MHill 2005). Therefore, in the investigation program of August 2004, extensive tests mainly including in-situ borehole shear tests (BSTs, performed by ISU) were conducted to obtain the shear strength parameters of the soils, especially for the shales. Based on the BST results, CH2MHill (2005) performed slope stability analyses for seven slope sections. As a parallel and independent study as presented in this report, ISU also performed slope stability analyses but through different approaches, which included parametric study and reliability study, to avoid repetition. The findings in the study will be used as verifications and supplements to those of CH2MHill (2005).

Area Geology

According to the USDA (1981b) Soil Survey Report, most of the soils in Wapello County formed in glacial till, loess and alluvium. Clayey shale is the oldest parent material forming the bedrock of the project site. The shale consists of a series of beds deposited during the Des Moines sedimentary cycle in the Pennsylvanian period. The bedrock surface closely parallels to the existing ground surface.

Field Investigations

Site Geology

A total of 16 borings and 10 borings were drilled during the investigation programs in the Sugar Creek project site in 2001 and 2004, respectively. The borings were distributed on both sides of the Sugar Creek. The site has gentle slope surface overall covered with well grown woods and vegetation (Figure 97). The boring layout for the 2004 program is shown in Figure 98, and the boring logs are presented in the Appendix B.

The subsurface of the project site can be generally grouped into 4 layers with increasing depth. The first layer consists of lean clay with sand and gravel (a thin layer of topsoil and fluvium or slope wash) underlain by clayey sand and silt (alluvium); or a mixture of clay, silt, sand and small amount of gravel. The actual compositions vary throughout the site. The layer is referred to as the Alluvium layer in this report for simplicity. The thickness of the Alluvium layer ranges from 2.7 to 8.4 m.

The Alluvium layer is underlain by three layers of highly weathered shale (H.W.Sh), moderately weathered shale (M.W.Sh), and slightly weathered shale (S.W.Sh) in order of increasing depth. The divisions of the three layers of the shale were based on the field visual inspection and strength tested using pocket penetrometer. In general, H.W.Sh has a relatively low strength, S.W.Sh has a relatively high strength, and M.W.Sh represents the transition between H.W.Sh and S.W.Sh.. The S.W.Sh was close to fresh rock and was identified by split spoon refusal or where the N-value in standard penetration test was below 50 blows or greater per 6-inch increment (CH2MHill 2005).

The surface of H.W.Sh generally appears to parallel the existing ground surface. It has a gentle slope ranging from 12.5:1 (H:V) to 10:1 (H:V) in the north side, and a relatively steeper slope of 3:1 (H;V) in the south side of Sugar Creek. The thickness of H.W.Sh ranges from 0 to 3.7 m in the north side and 0.5 to 4.7 m in the south side of Sugar Creek, and most of the H.W.Sh is less than 3 m thick. The H.W.Sh was underlain by M.W.Sh, which has a thickness ranging from 0 to 5.7 m in the north side and 1.5 to 8 m in the south side of Sugar Creek. M.W.Sh was underlain S.W.Sh, where the boreholes were terminated. In the south side of the creek, a nearly horizontal, 0.5-0.9 m thick limestone seam was also encountered and mainly located in the M.W.Sh. layer.

The boring results indicate that the spatial distributions of the soil layers are highly variable, and the section of slope changes from place to place. One of the typical sections of slope with the proposed embankment fill is shown in Figure 99.

Ground Water Table

Ground water levels (GWLs) were recorded at least 24 hours after the completion of drilling. In the north side, GWL was located in the Alluvium and approximately paralleled the surface of the H.W.Sh.. In the south side, GWL was measured only in BH CH1007 near the edge of the Sugar Creek. In general, GWL had gentle slope dipping towards the creek and connected with the water in the creek.

Borehole Shear Test Results

BSTs were the most important part of the 2004 investigation program. A total of 35 BSTs and Rock BSTs were performed at different layers in the 10 borings with emphasis given on the H.W.Sh.. The results of BST are summarized in Table 41. Examples of BST results tested in BH CH1009 are shown in Figure 100, and the results of 2 Rock BSTs and a BST with a high pressure plate tested in S.W.Sh are given in Figure 101. Other detailed results for the BSTs are presented in Appendix Figures A.33 to A.36. The results show that BSTs generally yielded satisfactory results as indicated by the large coefficient of correlation (R^2) for the plot of shear stress versus normal stress. It shows that shear strengths of the soils generally increase with the increasing depth. It is apparent that the S.W.Sh has relatively large cohesion.

The BST results indicate that the shear strengths of the soil layers are highly variable (Table 41). The variability is further indicated by the statistical results as summarized in Table 42. Despite of the variation of the shear strength parameter values for the soils, the general trend that the shear strengths increase following the increase in the depth of the shales is still apparent. It is also note-worthy that the average shear strength of H.W.Sh, which has average shear strength parameter values of $\phi' = 12.8^o$ and $c' = 33.2$ kPa, is much higher than that of $c' = 10$ kPa as assigned by IaDOT design guidelines.

Lab Investigations

Basic Properties

Basic properties for representative soil samples with emphasis on the shales were investigated and the results are summarized in Table 43. The Atterberg limit and clay fraction for the shales are also plotted in Figures 102 and 103. The results show that clay fraction for the shale ranges from 30-65%, liquid limit varies between 35% and 75%, and plastic limit varies from 15% to 45%. All the shales are classified as either low plasticity clay (CL) or high plasticity clay (CH) according to USCS.

Direct Shear Test Results

As a comparison to BST, laboratory direct shear tests were conducted on some of the soil samples, and the test results are summarized in Table 44. Examples of the test results for soil samples in BH CH1009 are also given Figure 104, and the complete test results are presented in Appendix Figures A.37 to A.40. It appears that the shear strength parameter values obtained from the direct shear tests are reasonably in agreement with those obtained from BST for the H.W.Sh..

Triaxial Test Results

A number of laboratory triaxial tests and unconfined compression tests were also conducted on some of the soil samples, and the test results are summarized in Table 45. An example of the test results for soil sample in BH CH1010 is given Figure 105. The shear strength parameter values obtained from the triaxial tests are reasonably close with those obtained from BSTs and direct shear tests for the H.W.Sh., though they are not exactly the same. The difference in the shear strength of the soils may be mainly due to the variation of the soil (the soil samples tested may not be exactly the same), and due to the different testing methods.

Ring Shear Test Results

Ring shear test were conducted for H.W.Sh. and M.W.Sh. samples from different boreings and the results are summarized in Table 46. An example of the test results for soil sample in BH CH1010 is given Figure 106, and the complete test results are presented in Appendix Figures A.41 and A.42. The results indicated that the residual shear strengths of the shales, which are the ultimate shear strength of the soils after large soil displacement, are generally low. These values are reasonable as the Atterberg limits of the shales are relatively large thus the soils have relatively low residual shear strengths.

Mineralogy and Morphology

The x-ray diffractograms (XRD) for the random oriented bulk shale samples from different borings are presented in Appendix Figures A43 to A52. The minerals identified are summarized at the bottom of each of the diffractograms. The minerals identified include quartz, montmorillonite, kaolinite, illite, calcite and cristobalite.

Slope Analysis

Methodology of the Slope Analysis

The boring results show that there is a dramatic spatial variation of the soil stratification in the project site, which is particularly reflected by the variation of the thickness of the soil layers from place to place. The in-situ BSTs and lab tests indicate that the shear strengths of the soils also vary significantly, even for the soils that are categorized as the same layer. In general, both the slope profiles and soil shear strength are highly variable.

CH2MHill (2005) produced seven representative cross-sections at different locations of the project site based on the boring information, and performed slope analysis for each of the sections. These seven sections may be of sufficient amount based on their engineering judgment. To avoid repetition, only one of these seven sections was adopted in this study, which was the north abutment (SBL) section (Figure 99) and was referred to as the Real Section. The Real Section is among the slope profiles which comprise relatively thick H.W.Sh at large depth and give relatively low factor of safety (FS) as compared with other sections (CH2MHill 2005). To account for the great variety of slope sections, an Idealized Section was also produced for slope analysis in this study. In the Idealized Section (Figure 107), the soil layers were all assumed horizontal, which was in line with the fact that the slopes of the soil layers in the site were generally gentle, especially for those with deep H.W.Sh.. The depth of H.W.Sh was assumed to be at the elevations of 8 to 11 m, which was the largest depth observed from the borings. The thickness of H.W.Sh was assumed to be 3.0 m, because most of the H.W.Sh had thickness ranging from 1 to 3 m. In general, the Idealized Section was intended to standardize the various slope sections and to accommodate the "worst" situation.

The Idealized section was used to perform systematic parametric study of slope analysis to investigate the effect of various factors that affect FS of the slope. The parametric study was focused on the H.W.Sh., which include its ϕ', c', unit weight (γ) and its depth, since H.W.Sh. has the lowest shear strength and is the critical layer in the project site. Other factors affecting FS were also considered in the parametric study, which included the water table in the creek and the ground water table (GWT) within the slope. Besides the parametric study, a probabilistic slope analysis was also carried out on both the Real section and the Idealized section to account for the large variation of the soil shear strength and the variation of GWT.

In the slope analyses, the soil shear strength parameter values as obtained from BST were adopted (Table 47). This is because BST gave in-situ shear strength of the undisturbed soil, which was more reliable and realistic. The average shear strength parameter values and unit weight for the embankment compacted fill were recommended by IaDOT. The standard deviations of ϕ' and c' value for the fill and those of the unit weight of all the soils are assumed considering their variability in engineering practice. The maximum water table level for the creek was assumed to be the highest water level according to the estimated 500 years flood event (CH2M 2005); and the minimum water table level was assumed to be the GWT measured in the soils during the boring investigations. To simplify the GWT conditions, the GWT was assumed to be flat since the slope for measured GWT was gentle.

In the slope stability analysis, both Morgenstern-Price method and Bishop simplified method were adopted. Methods of both circular slip surface search and block slip surface search were used to obtain the potential slip surface and locate the critical slip surface which has minimum FS in a deterministic stability analysis. Back-calculations were also performed to determine the average shear strength of the soil giving unity factor of safety (FS =1.00).

Results of the Parametric Slope Analyses and Discussions

Parametric study of slope analysis on the Idealized slope section (Figure 107) were perform to investigate the sensitivity of FS with respect to ϕ', c' and γ of the H.W.Sh.. The mean values of the shear strength parameters of the other four soil layers and mean GWT were used (Table 47). The results of the analyses are presented in Figures 108, 109 and 110. The results show that ϕ' and c' values have major effect on FS, and γ has minor effect on FS of the slope. FS of the slope can vary significantly with the variation of the values of ϕ' and c' but does not change much with the change in γ. The effect of the ϕ' and c' values were expected. It also indicated that the effect of γ in FS can be neglected.

The sensitivity of FS with respect to the variation of the water table level in the creek was studied and the result is shown Figure 111. In the analysis, mean values for all the parameters were taken; and the GWT in the slope was assumed to be the same as that of the water level in the river, which is in line with the observations that the GWT within the soils was generally flat during the filed borings. The result indicates that within the range of the variation of the water level in the creek, the variation of FS for the slope is insignificant. The difference between the maximum and minimum FS corresponding to the lowest (-1.6 m) and highest (+1.6 m) water level in the creek is only 0.13 for various analyses assuming different slip surfaces.

In the previous analyses, GWT in the slope was assumed to be the same as the water level in the creek. However, it is also possible that GWT conditions within the slope soils be altered considerably after the embankment is constructed, ant the GWT can be significant different from the water table level in the creek. The exact future GWT depends on many factors such as the hydraulic properties of the fill and the existing soils, the local hydrogeological conditions and climatic conditions (precipitation and evaporation); and its evaluation requires complex analysis coupled with groundwater seepage analysis. Such study is beyond the scope of this report. Nevertheless, a simplified parametric slope analysis was performed to evaluate the effect of future GWTs on the slope stability. The GWT was idealized by assuming it being flat within the slope, extending to the edge of the slope, following the slope surface and then connecting to the mean water table of the creek (elevation of 14.6 m). Such a GWT and the slope analysis results are presented in Figure 112. It shows that when GWT in the slope was raised from the lowest elevation of 14.6 m to the top of the embankment at elevation of 30.0 m, FS value dropped by about 0.38 for different potential slip surfaces using different analysis methods. The results indicate that the GWT within the slope has significant effect on slope stability. However, as the FS value of about 1.20 was still larger than utility under the extremely unfavorable condition with GWT located at the top of the embankment, the effect of GWT within the slope can be released if further study on this factor is not required.

Considering H.W.Sh was found at various elevations at different locations of the project site, the effect of the elevation of H.W.Sh. on slope stability was also studied. The H.W.Sh was assumed to be horizontal and located at various depths in a slope section which had similar configuration with the Idealized section of Figure 107. Thicknesses of both H.W.Sh and M.W.Sh were maintained as 3.0 m, and the geometry of the slope remained the same. When the elevations of the shales were “raised up”, the overlying Alluvium and the compacted fill might have to be partially or totally “cut off” to maintain the geometry of the slope. Also, GWT was assumed to be at the mean level of elevation 14.6 m. Soil parameters were assumed to be of their mean values. The results of the analysis are presented in Figure 113. It indicates that FS value significantly increased following the increase in the elevation of the H.W.Sh. A slope section comprising shallow H.W.Sh. has a considerably larger FS value. When the top of H.W.Sh was at elevation of 20 m or above, the potential slip surface will pass through the Alluvium or compacted fill, i.e. the location of H.W.Sh has no more effect on the slope stability. In addition, under the extremely unfavorable condition assumed with H.W.Sh located at elevation of 5.0 - 8.0, which was 3.0 m lower than that in the Idealized section, the FS for the slope is still larger than 1.42. These results suggest that slope comprising relatively shallow H.W.Sh. can be released for the stability concern.

Results of the Probabilistic Analyses and Discussions

Probabilistic slope analyses were performed on both the Real section (Figure 99) and the Idealized section (Figure 107) using exactly the same soil parameter values and water table information (Table 47). The slope profiles, the most critical slip surfaces using circular slip surface search and block slip surface search, and the minimum FS values corresponding to the deterministic analysis, are shown in Figures 114 to 117. The results show that the critical slip surface passed through the relatively weak layer of H.W.Sh. under all situations. The locations of the critical slip surface were also very close to each other under different situations.

The probability density of FS and the probability distribution of FS are presented in Figures 118 and 119, respectively, and also summarized in Table 48. The peak FS values in the probability density curves indicate the mean FS values, which are also the same corresponding to the 50% of probability of FS in the probability distribution curves. The probability of failure, or P(FS<1.00), can be obtained from the curves in Figure 119 where FS=1.00, which has the maximum value of about 6% for the 8 different cases. From the details of the results as summarized in Table 48, it can be seen that the maximum value of P(FS<1.00) is 5.8% for the 8 different cases. The mean FS values range from 1.571 to 1.789 assuming different slip surface for the two different sections (Idealized and Real sections). These results suggest that the slopes analyzed are generally safe, with probability of failure less than 6%.

The probability of failure as analyzed by CH2MHill (2005) was generally smaller than 0.1%, which was significantly lower than the results of this study. This is mainly due to the reason that the shear strength parameter values adopted by CH2MHill were generally larger than those adopted in this study. CH2MHill proposed to excavate and replace the Alluvium and H.W.Sh. with compacted fill for the south abutment area, and the BST results tested in the soils to be excavated were not used for the statistical calculation and the input for the probabilistic analysis. For example, CH2MHill obtained average $\phi' = 16^o$ with a standard deviation of 5^o, and average $c' = 42$ kPa with a standard deviation of 17 kPa. for the H.W.Sh. layer. These values are considerably higher than those used in this study (Table 47). In this study, no excavation of the weak soils is considered. Nevertheless, the mean FS values are still comparable with those of CH2MHill, in which mean values of FS of 1.8 and 1.9 were reported with similar slope section.

Results of the Back-Analysis for FS =1.0

Back-analyses of slope stability were performed on the Idealized section to determine the shear strength of the H.W.Sh required for yielding FS=1.00. The soil parameter values for other soil layers and water table were assigned to be their mean values as in Table 47. Back-analyses were also performed on a slope section with the same geometry and GWT, but assuming the slope is homogeneous comprising H.W.Sh. only. This represents the extremely unfavorable soil conditions for the slope. The different shear strength parameter values of the H.W.Sh. giving FS = 1.00 for the slope together with the actual values as obtained from BSTs are presented in Figure 120. The results show that the shear strength giving FS =1.00 by assuming circular slip surface is the lowest; the shear strength giving FS =1.00 by assuming block slip surface is significantly higher than that by assuming circular slip surface. These results suggest that it is safer by assuming “block” slip surface than assuming circular slip surface for the slope comprising underlying weak layer as in the cases here. This is in agreement with the reality that many slopes comprising multiple layer soils failed with “block” slip surface rather than circular slip surface. In the extreme case of the slope comprising purely H.W.Sh., the shear strength required for giving FS =1.00 is much higher than the slope comprising actual soils of multiple layers. This indicates that it may be over-conservative when such extreme soil condition is assumed for the slope section. However, all the values of the in-situ shear strength parameter values of H.W.Sh. as measured by BST are located above the curves of c’-ϕ’ plot giving FS =1.00 for the Idealized section, indicating the H.W.Sh has sufficient strength for the slope to remain safe (FS>1.0).

Summary and Conclusions

Geotechnical investigation and characterization was conducted on the Sugar Creek project site with the emphasis on the determination of the shear strengths of the soils, particularly those of the relatively weak layer of the highly weathered shale, as there was potential global slope instability for the proposed embankment slopes with slip surface passing through the highly weathered shale. A substantial number of tests were performed for the soil or soil samples at different elevations of different borings. The tests included in-situ borehole shear test (BST), and laboratory direct shear, triaxial compression and ring shear test. The shear strength parameter values as obtained from BST, which gave the shear strength of in-situ, undisturbed soils, were used for slope stability analysis.

Slope analyses were performed basically on two sections, the Real section and the Idealized section. The Real section was the section comprising highly weathered shale at the lowest elevation with relatively large thickness. This section was representative of the most unfavorable slope section. The Idealized section assumed horizontal soil layers and idealized soil configurations in order to accommodate the great variety of soil stratifications, and was also used for parametric slope analysis. The parametric analysis was performed to investigate the sensitivity of factor of safety (FS) with respect to the changes of various factors affecting FS of the slope, particularly in line with the dramatic variation of the shear strength parameter values of the soils. Probabilistic slope analyses were also performed using the statistical results of the shear strength parameter values and the different water table conditions.

The study shows that the shear strength of the highly weathered shale has the most effect on FS of the slope. Variation of soil unit weight does affect the FS of slope appreciably. Variation of water table level in the creek slightly affects the FS, and variation of water table level within the slope moderately affects the FS of slope. In addition, the elevation of highly weathered shale significantly affects FS of the slope. All the analyses using mean values of the various soil parameter and water table level resulted in FS values larger than unity indicating the slopes are generally safe.

The probabilistic analyses show that the results for the Real section and the Idealized section are very close to each other for all cases using analysis methods of Morgenstern-Price Method and Bishop Simplified Method by assuming both circular slip surface and block slip surface. The values of probability of failure ranges from 2.77% to 5.80%, and the mean FS values range from 1.57 to 1.79 for the 8 cases analyzed.

Different combinations of ϕ' and c' values for the highly weathered shale required for giving FS =1.00 for the slope were obtained by back-calculations. The results were compared with the shear strength parameter values of the highly weathered shale as measured from Borehole Shear Tests. It is found that all the measured shear strength parameter values are located above the curves of c'-ϕ' plot for giving FS =1.00 for the Idealized slope section, indicating the H.W.Sh has sufficient strength for the slope to remain safe (FS>1.00).

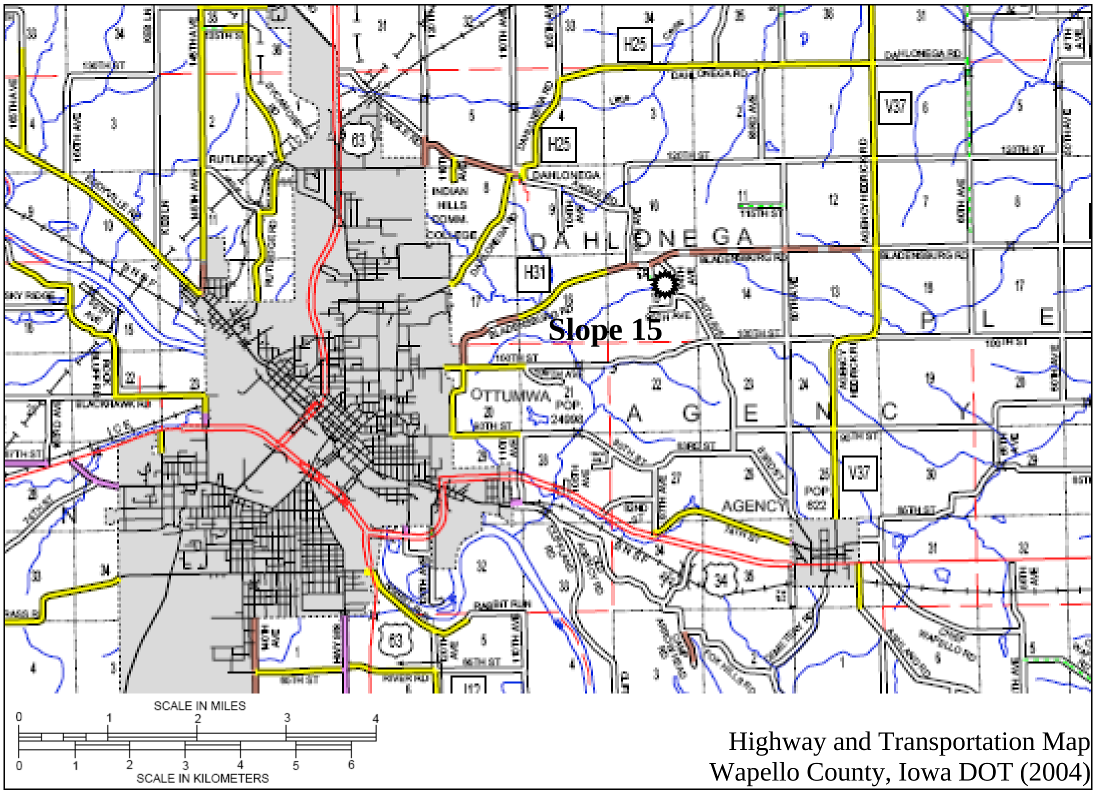


Figure 96. Location of slope 15 (Sugar Creek Project, Ottumwa, Wapello Co.)

(a) Looking north, overview of the north side of the site (photo taken by Thompson, 07/27/04)

(b) Looking north, investigation of the north side of the site (photo taken by Thompson, 07/26/04)

Figure 97. Photographs for slope 15

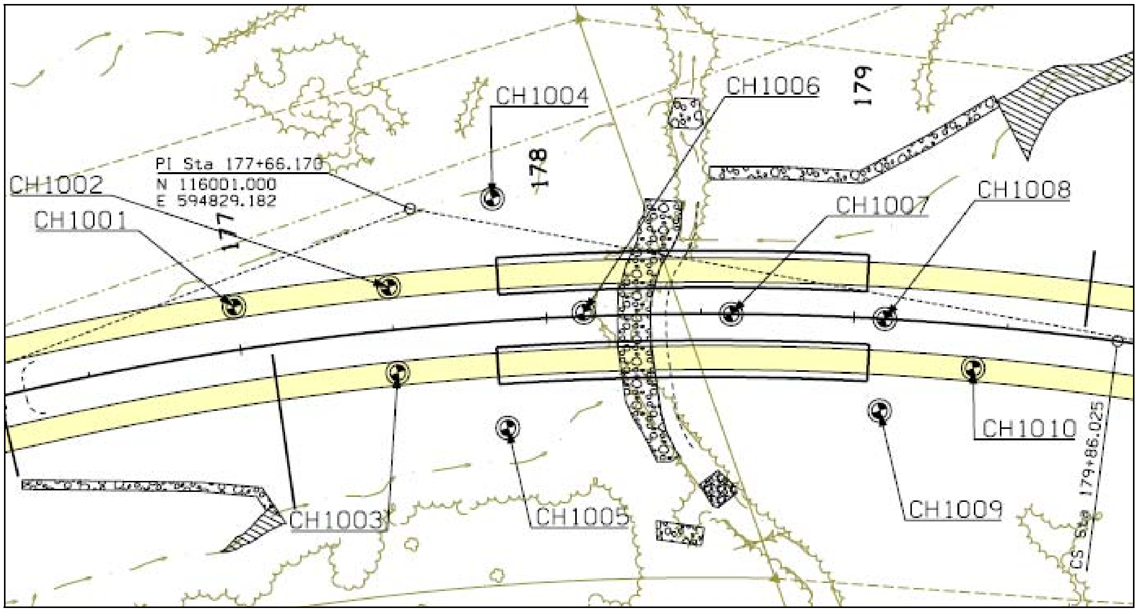


Figure 98. Boring layout for slope 15 (CH2MHill 2005)

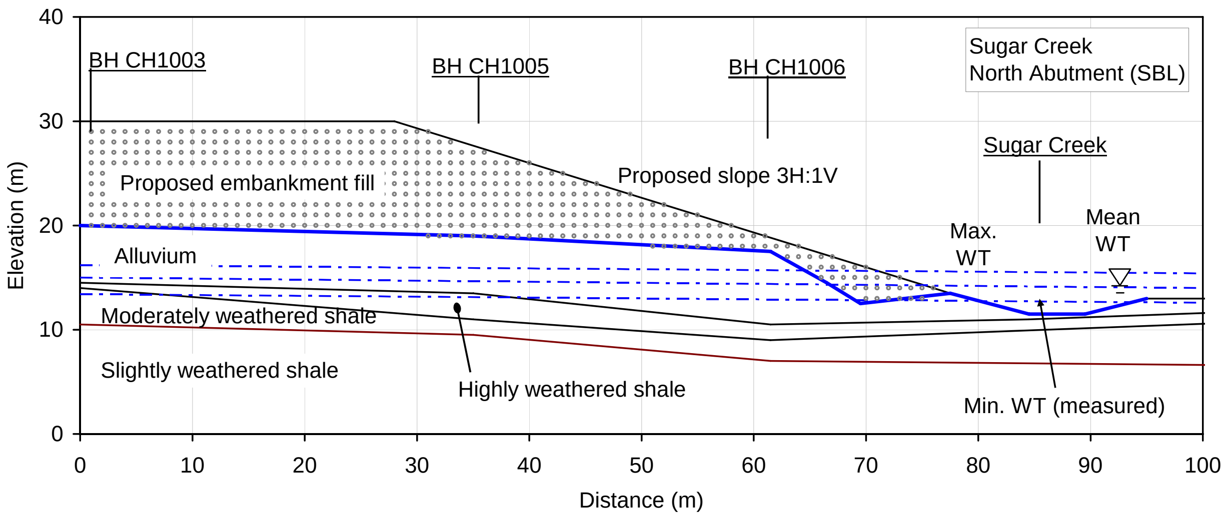


Figure 99. Cross-section at north abutment (SBL) for slope 15

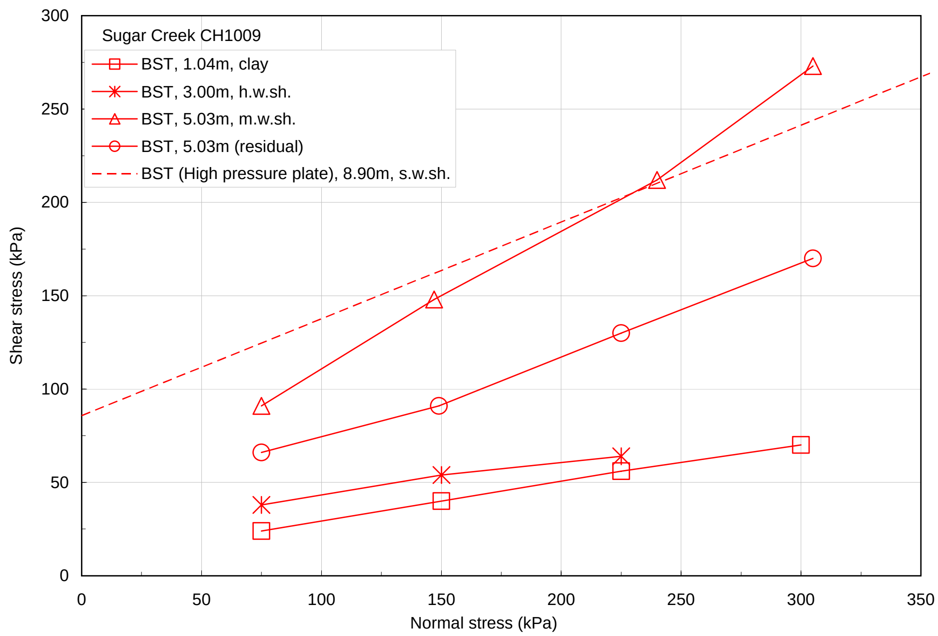


Figure 100. BST results in boring CH1009 at slope 15

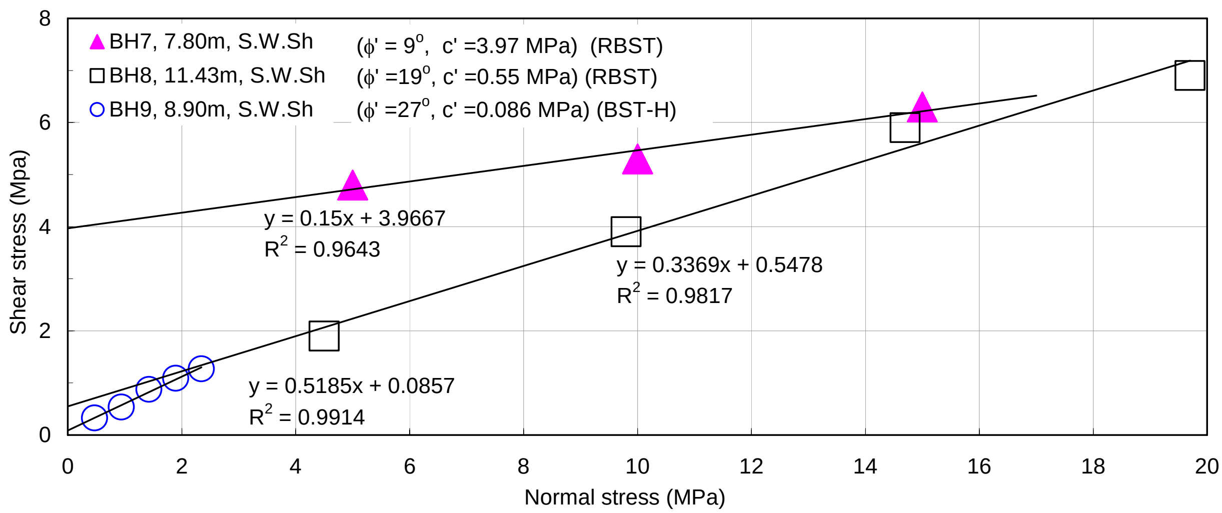


Figure 101. RBST and BST-H results for slope 15

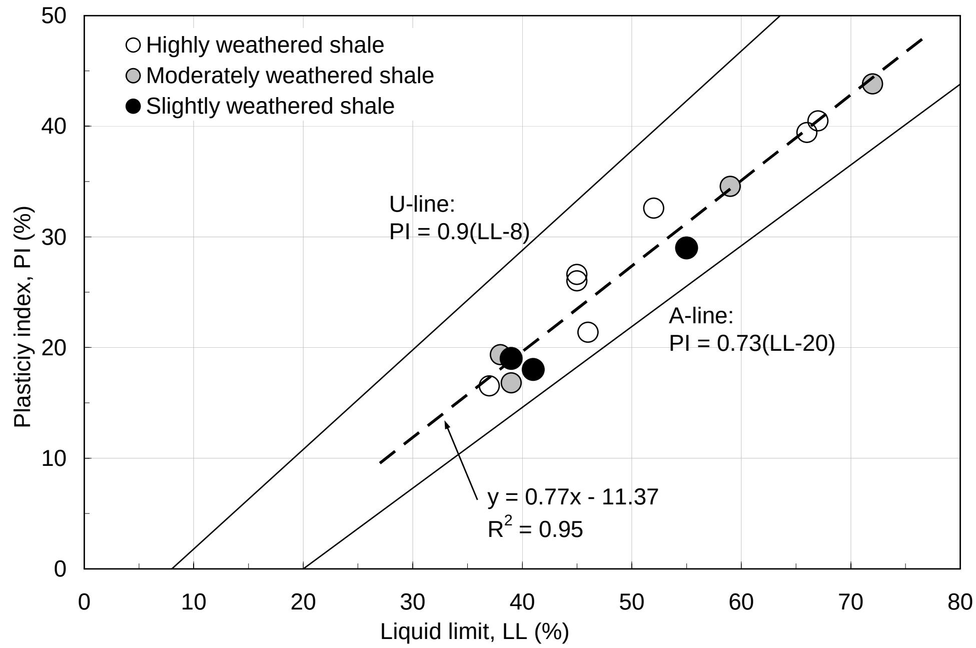


Figure 102. Plastic limit versus liquid limit for the shales at slope 15

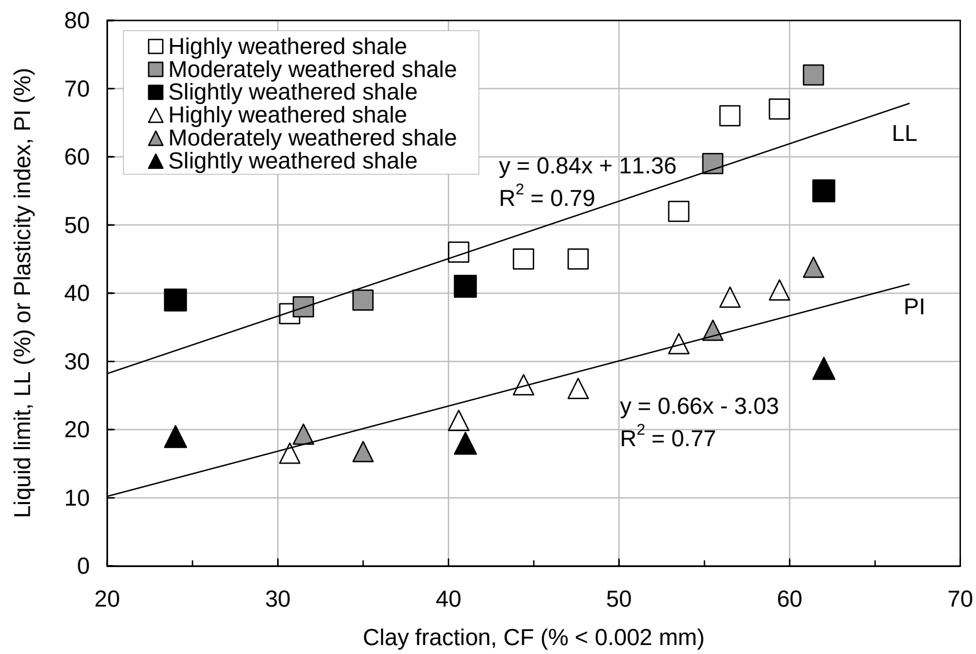


Figure 103. Atterberg limits versus clay fraction for the shales at slope 15

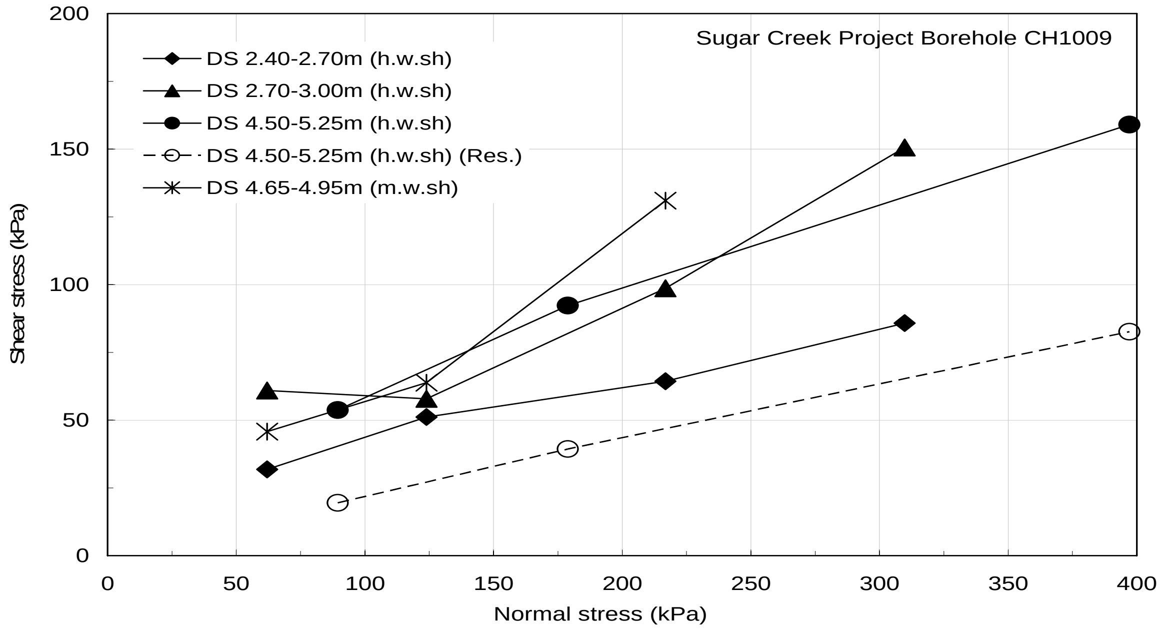


Figure 104. Direct shear test results for samples of boring CH1009 at slope 15

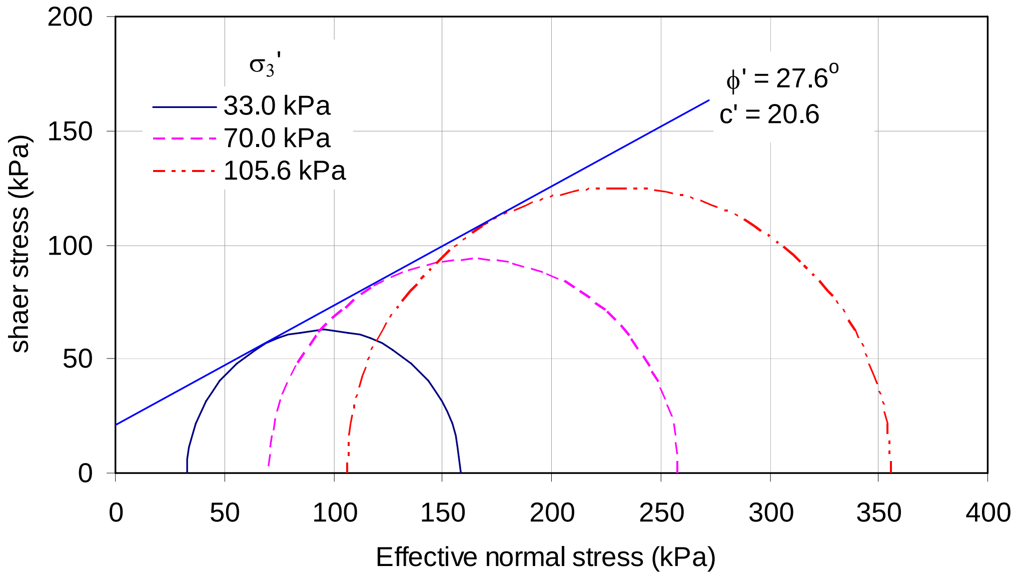


Figure 105. Consolidated drained triaxial test for highly weathered shale at 0.6-1.2m in boring CH1010 at slope 15

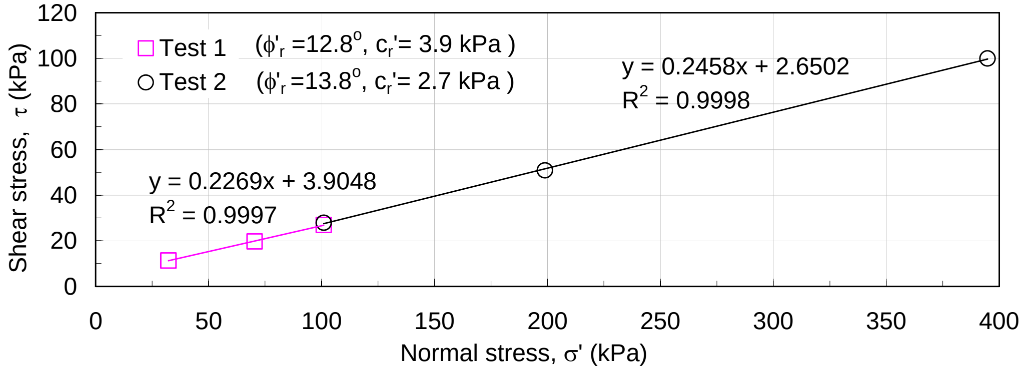


Figure 106. Results of ring shear test for the highly weathered shale at 0.6-1.2m in boring CH1010 at slope 15

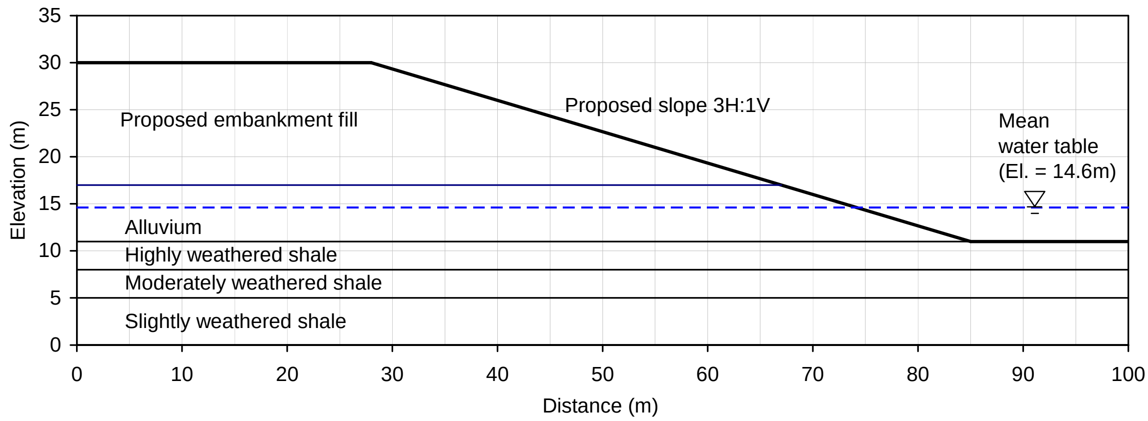


Figure 107. Idealized section for slope stability analysis for slope 15

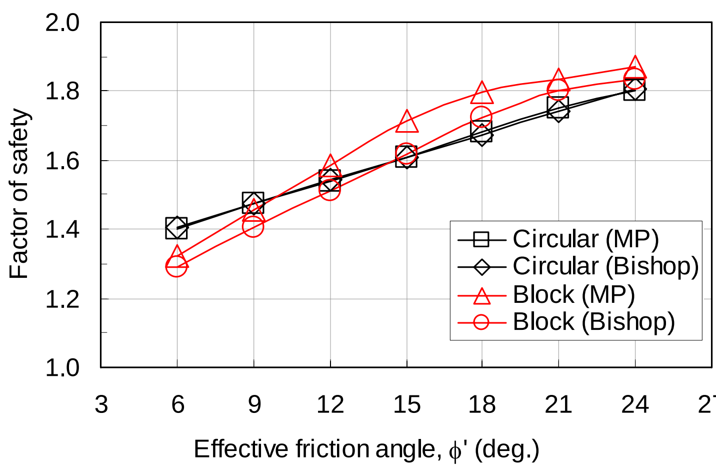


Figure 108. Sensitivity of FS on effective friction angle of the highly weathered shale for the Idealized section for slope 15

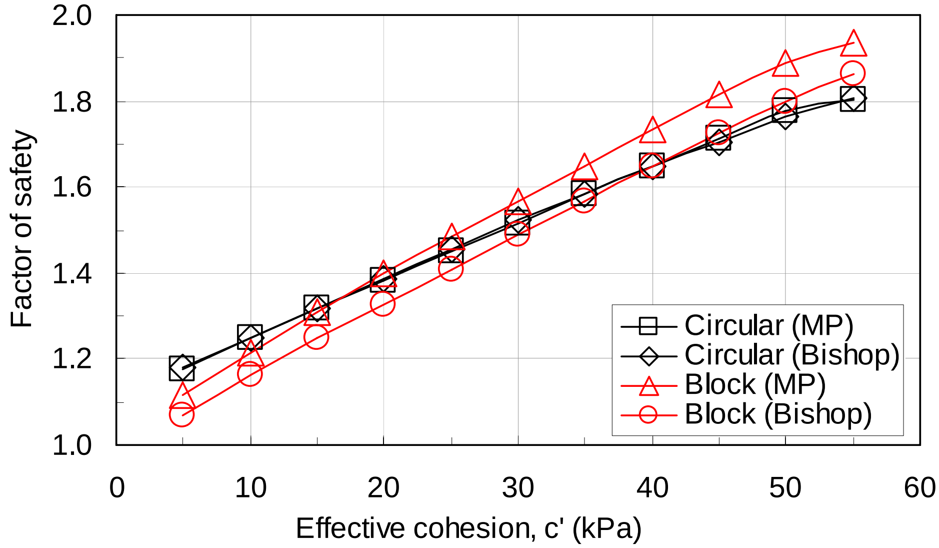


Figure 109. Sensitivity of FS on effective cohesion of the highly weathered shale for the Idealized section for slope 15

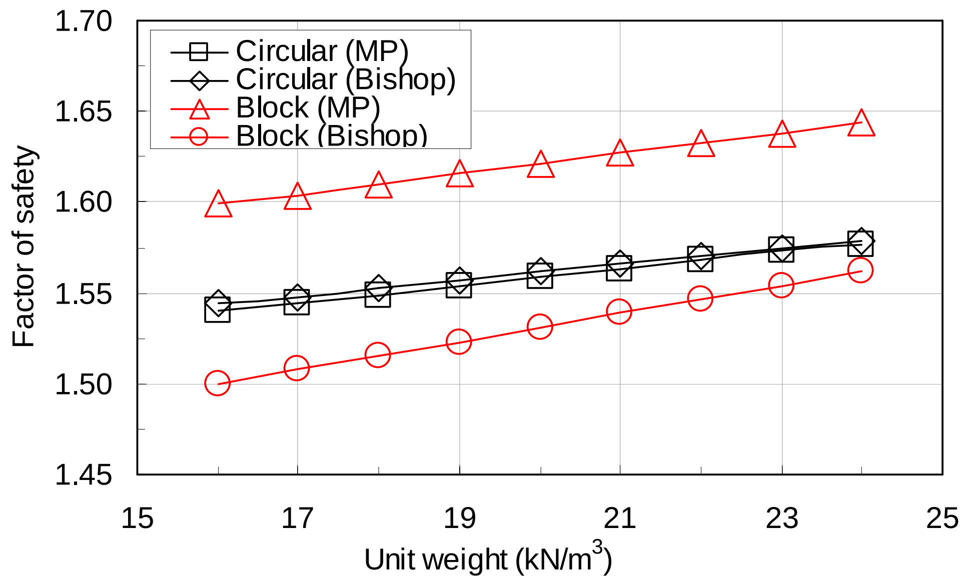


Figure 110. Sensitivity of FS on unit weight of the highly weathered shale for the Idealized section for slope 15

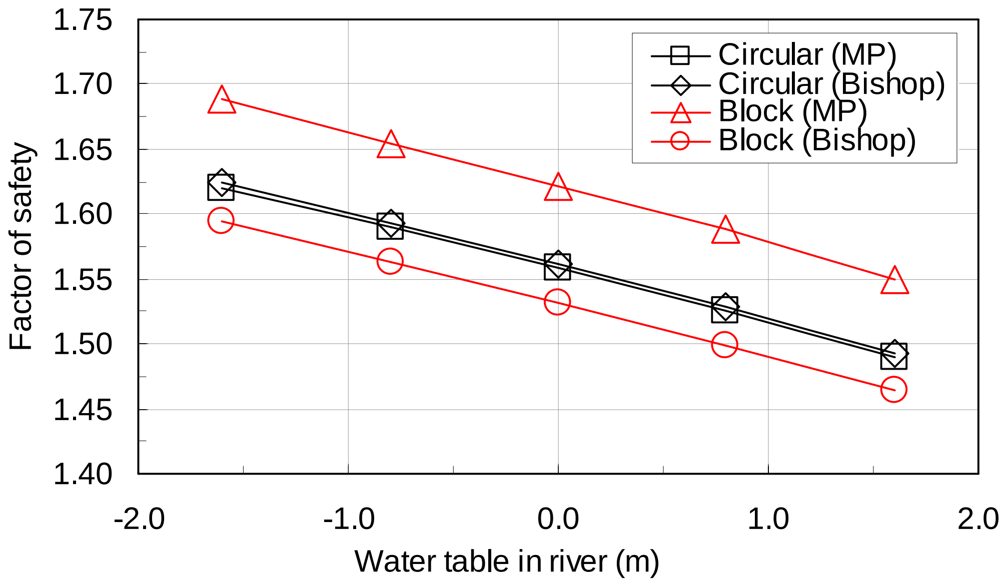


Figure 111. Sensitivity of FS on water table level in the river for the Idealized section for slope 15

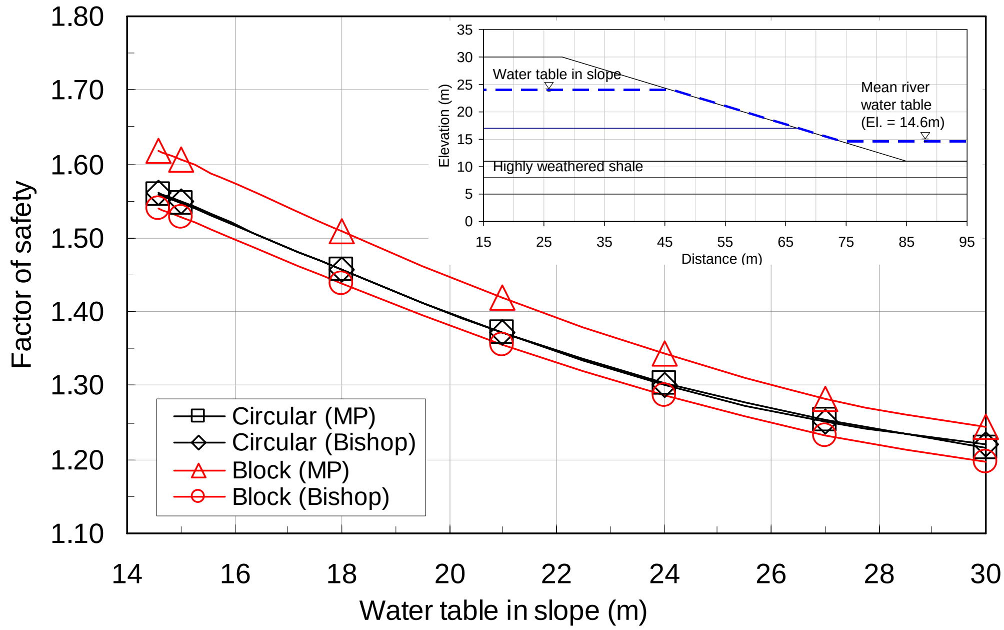


Figure 112. Sensitivity of FS on the water table level within the slope for the Idealized section for slope 15

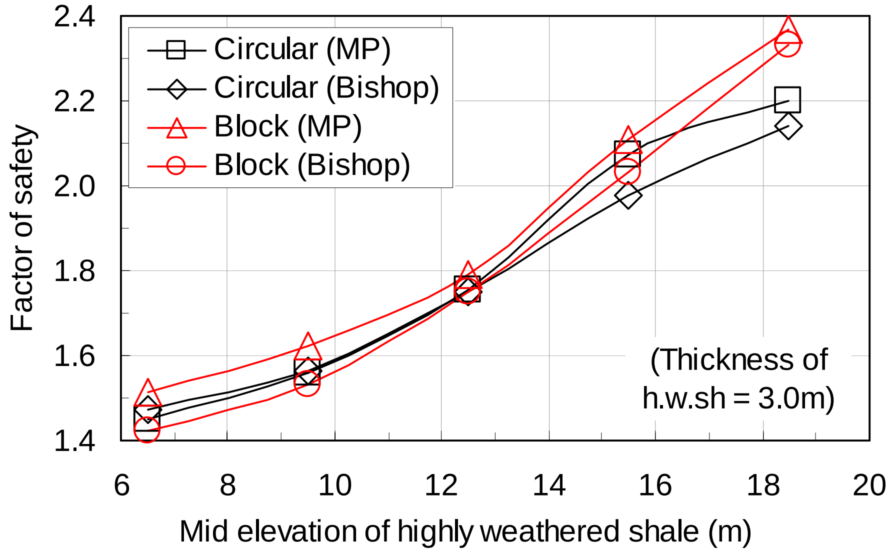


Figure 113. Sensitivity of FS on the elevation of the highly weathered shale for the Idealized section for slope 15

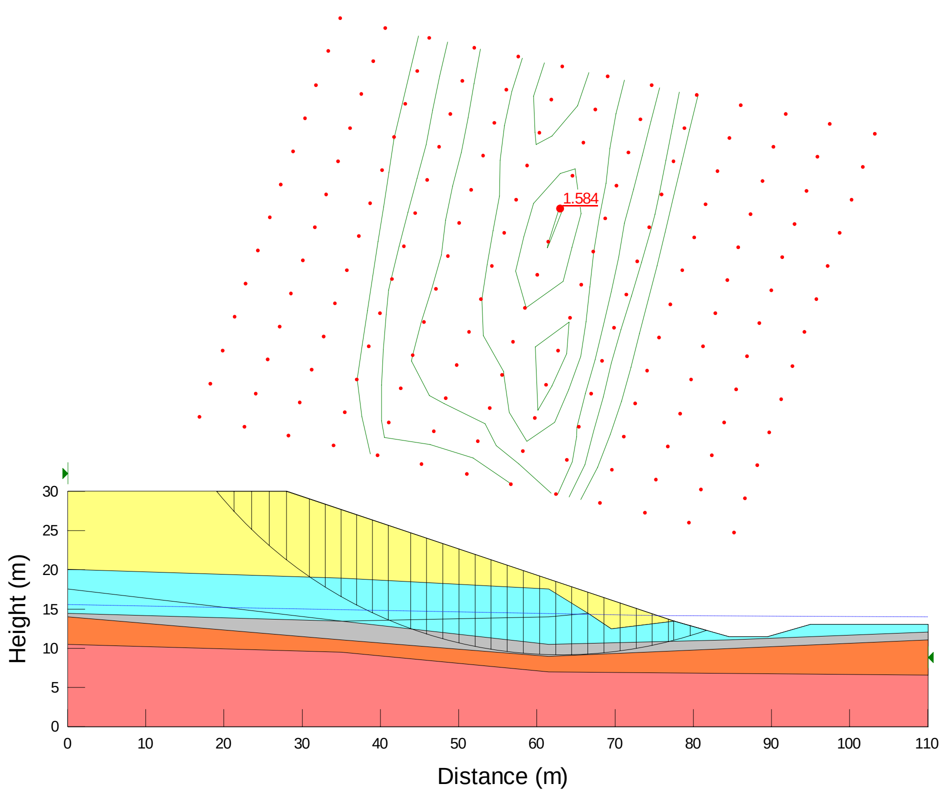


Figure 114. Slope analysis assuming circular slip surface for the Real section for slope 15

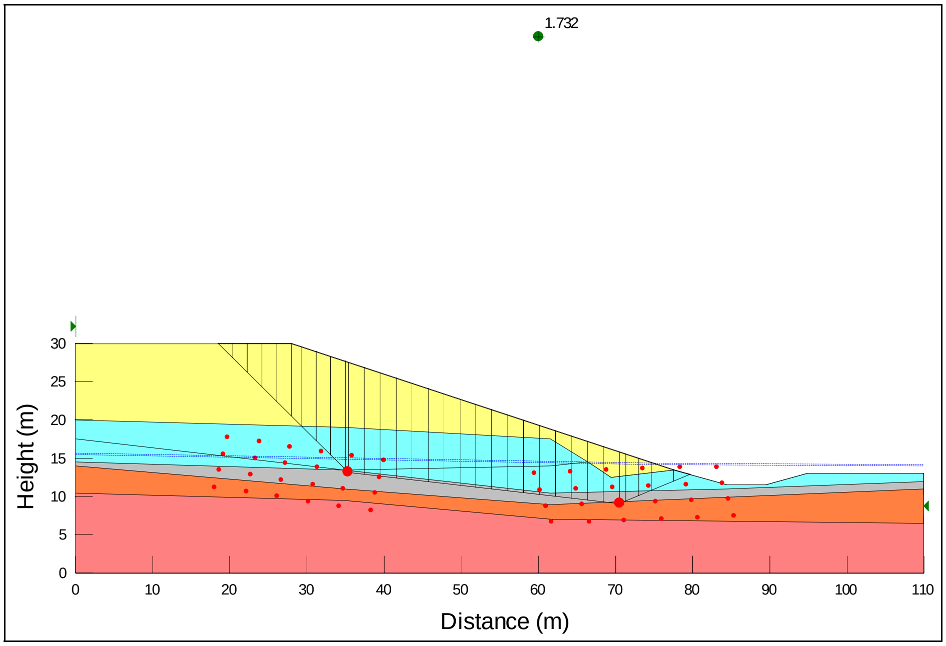


Figure 115. Slope analysis assuming block slip surface for the Real section for slope 15

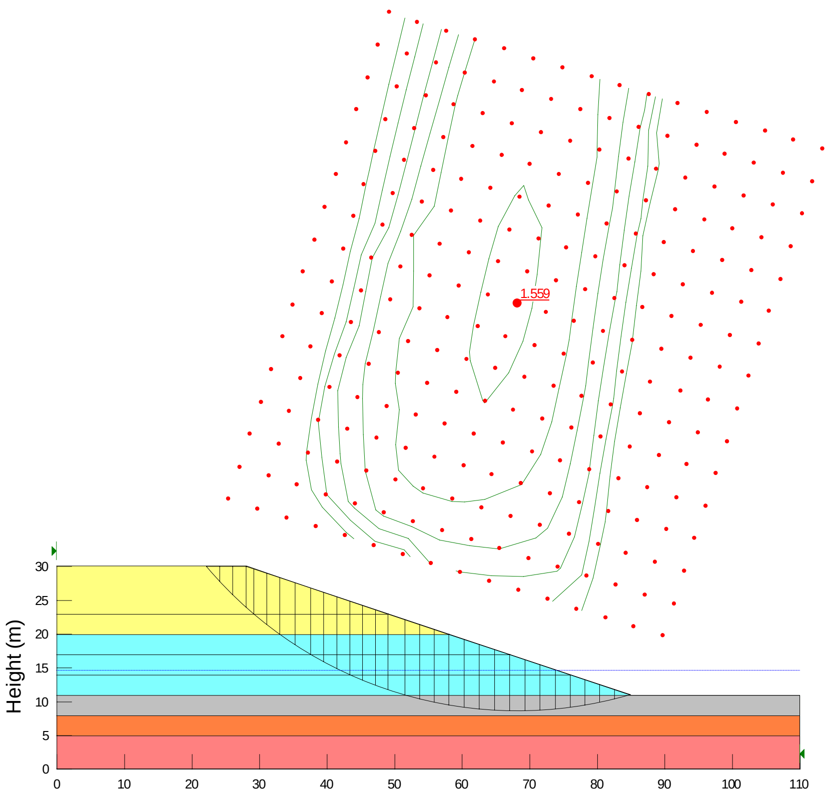


Figure 116. Slope analysis assuming circular slip surface for Idealized section for slope 15

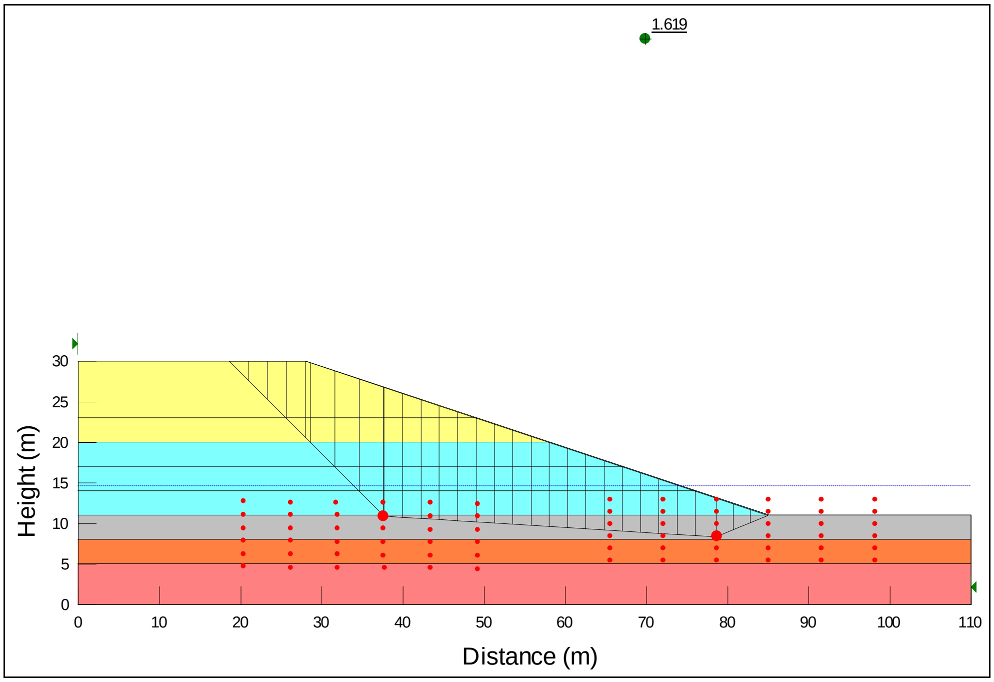


Figure 117. Slope analysis assuming block slip surface for the Idealized section for slope 15

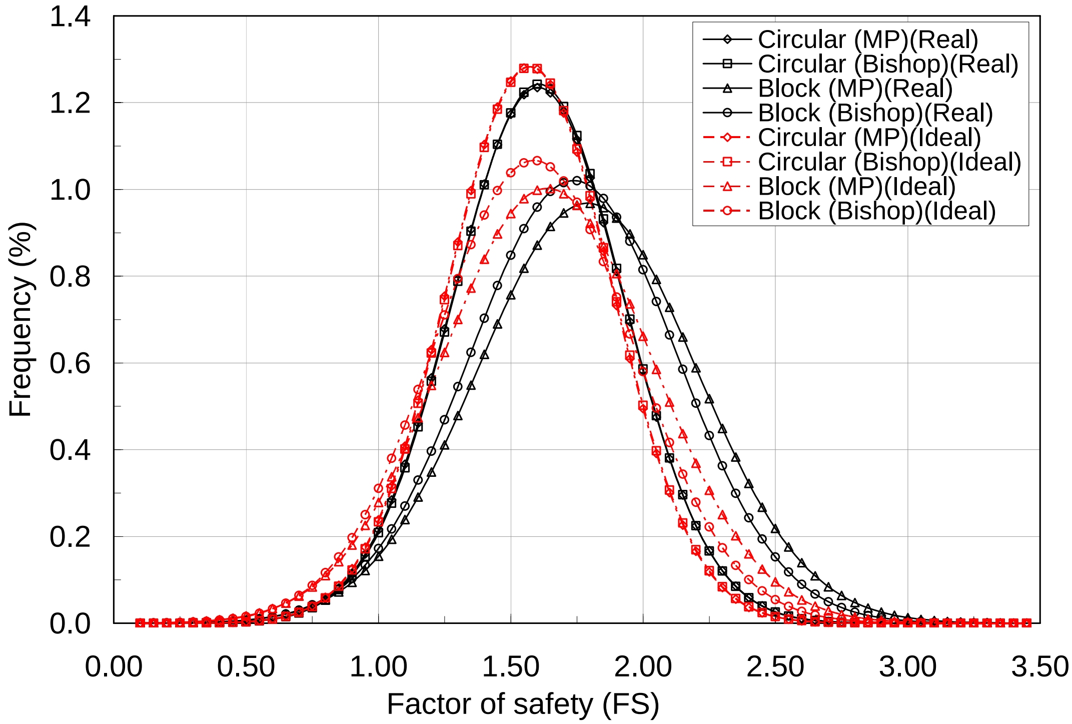


Figure 118. Probability density of FS for slope 15

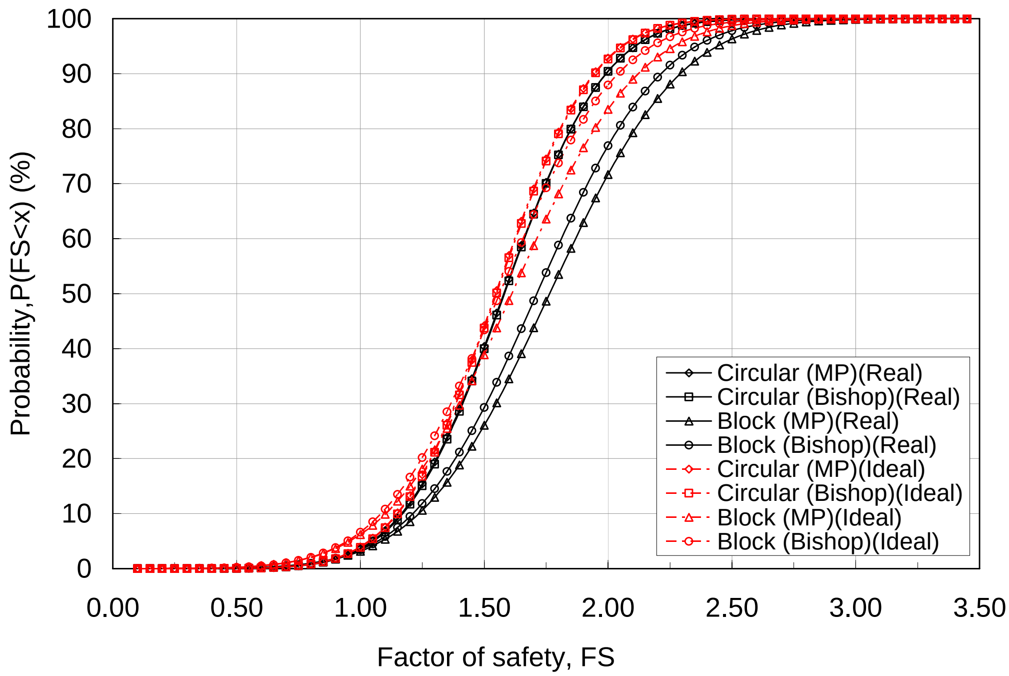


Figure 119. Probability distribution of FS for slope 15

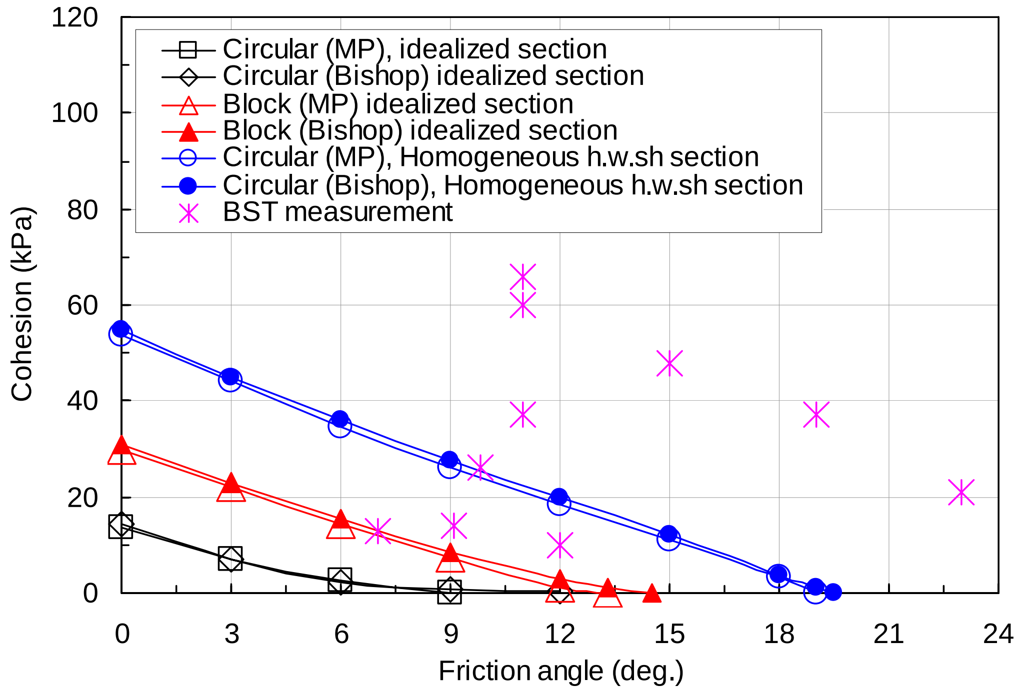


Figure 120. Shear strength parameter values required for giving FS of unity for slope 15

Table 41. Summary of BST results for slope 15

S.No.	Borehole	Type of BST	Depth (m)	Soil	ϕ' (deg.)	c' (kPa)	R^2	Data Points	Test Date	Remarks
1	CH1001	O	6.80	clay	21	23	0.994	4	7.28	
2	CH1001	H	10.35	h.w.sh	15	48	0.961	4	7.28	
3	CH1001	H	10.50	m.w.sh	16	334	0.967	4	7.28	
4	CH1001	H	12.50	s.w.sh	41	55	0.999	4	7.28	
5	CH1002	O	1.52	clay	16	64	0.929	4	7.27	
6	CH1002	H	10.71	s.w.sh	15	104	0.994	4	7.27	
7	CH1003	O	5.89	h.w.sh	19	37	0.994	4	7.26	
8	CH1003	H	12.75	s.w.sh	40	137	0.998	4	7.27	Use 3 of 4 points for regression.
9	CH1004	O	3.86	clay/sand	17	36	0.964	4	7.27	
10	CH1004	O	6.00	h.w.sh	23	21	0.990	4	7.27	
11	CH1004	H	8.65	m.w.sh	13	48	0.993	4	7.28	
12	CH1004	H	11.22	s.w.sh	16	399	0.992	4	7.28	Use 3 of 4 points for regression.
13	CH1005	O	6.05	h.w.sh	11	66	0.899	4	7.27	
14	CH1005	O	7.13	h.w.sh	11	37	0.964	5	7.27	
15	CH1005	H	9.50	s.w.sh	27	143	0.956	4	7.27	
16	CH1006	O	2.74	clay	17	33	0.940	4	7.26	
17	CH1006	O	9.30	m.w.sh	20	6	1.000	4	7.26	
18	CH1006	H	10.52	s.w.sh	16	629	0.968	4	7.26	
19	CH1007	O	2.44	h.w.sh	11	60	0.964	5	7.26	
20	CH1007	O	2.44	h.w.sh	11	30	0.904	6	7.26	Residual test. Use 4 of 6 points for regression.
21	CH1007	O	3.33	m.w.sh	21	67	0.964	5	7.26	
22	CH1007	O	3.33	m.w.sh	11	57	0.995	6	7.26	Residual test. Use 4 of 6 points for regression.
23	CH1007	R	7.80	s.w.sh	9	3970	0.964	4	7.26	Use 3 of 4 points for regression.
24	CH1008	O	1.12	h.w.sh	7	13	0.945	4	7.24	
25	CH1008	O	2.92	lime stone /shale	21	0	0.986	4	7.24	Assume c' = 0.
26	CH1008	R	11.43	s.w.sh	19	550	0.982	4	7.24	
27	CH1009	O	1.04	clay	12	9	0.999	4	7.24	
28	CH1009	O	3.00	h.w.sh	10	26	0.983	4	7.24	Use 3 of 4 points for regression.
29	CH1009	O	5.03	m.w.sh	38	32	0.997	4	7.24	
30	CH1009	O	5.03	m.w.sh	25	28	0.992	4	7.24	Residual test.
31	CH1009	H	8.90	s.w.sh	27	86	0.991	4	7.24	
32	CH1010	O	0.75	clay/shale	-	-	-	4	7.23	Data scattered.
33	CH1010	O	1.07	h.w.sh	12	10	0.977	4	7.23	
34	CH1010	O	1.07	h.w.sh	11	12	0.985	4	7.23	Residual test.
35	CH1010	O	1.27	h.w.sh	9	14	0.981	4	7.23	

BST	= Borehole Shear Test;	h.w.sh	= highly weathered shale;
O	= BST with ordinary pressure plate;	m.w.sh	= moderately weathered shale;
H	= BST with high pressure plate;	s.w.sh	= slightly weathered shale.
R	= Rock BST.		

Table 42. Statistics of BST results for slope 15

Soil	Total No. of Test	Friction angle, ϕ' (deg.)				Cohesion, c' (kPa)			
		Max.	Min.	Ave.	S.D.	Max.	Min.	Ave.	S.D.
Alluvium	5	21	12	16.5	3.4	64	9	33.0	20.3
Highly weathered shale	10	23	7	12.8	4.9	66	10	33.2	19.9
Moderately weathered shale	5	38	13	21.6	9.6	334	6	97	134
Slightly weathered shale	9	41	9	23.3	11.3	3970	55	675	1254

Note: Residual BST results are not included
Max. = maximum value *Ave. = Average value*
Min. = Minimum value *S.D. = Standard deviation*

Table 43. Summary of basic property results for slope 15

BH	Depth (m)	Soil	Gran Size			Atterberg Limit			Classification		Water content (%)	Total density (kN/m^3)	Dry density (kN/m^3)
			Sand (%)	Silt (%)	Clay (%)	LL	PL	PI	USCS	AASHTO			
1	6.5-7.1	s.clay									21.7	19.6	16.1
2	1.2-1.8	s.clay									23.2	19.5	15.8
3	5.6-5.9	h.w.sh	5	55	41	46	25	21	CL	A-7-6	27.4	19.9	15.6
4	3.6-4.2	s.clay									22.6	17.5	14.3
4	5.8-6.4	h.w.sh									<u>20.9</u>		<u>16.8</u>
4	7.7-8.2	m.w.sh									<u>15.0</u>		<u>18.5</u>
4	8.2-8.65	m.w.sh	5	91	4	39	22	17	CL	A-6			
5	4.8-5.4	s.clay				<u>45</u>	<u>14</u>	<u>31</u>					<u>15.1</u>
5	5.5-6.1	h.w.sh	2	45	54	52	19	33	CH	A-7-6	15.8	21.0	18.1
5	7.15-7.3	h.w.sh	17	39	44	45	18	27	CL	A-7-6			
6	8.85-9.45	m.w.sh	2	67	32	38	19	19	CL	A-6			
7	2.0-2.6	h.w.sh	9	44	48	45	19	26	CL	A-7-6	24.9	20.2	16.3
7	3.2-3.6	m.w.sh									<u>12.3</u>		<u>19.9</u>
8	1.2-1.8	h.w.sh											<u>14.5</u>
8	2.7-3.0	m.w.sh	4	35	61	72	28	44	CH	A-7-6			
9	0.9-1.4	s.clay									27.9	16.5	12.9
9	1.4-1.5	h.w.sh	1	43	57	66	27	39	CH	A-7-6	20.1		15.9
9	2.7-3.0	h.w.sh	0	41	59	67	27	40	CH	A-7-6			
9	4.65-4.95	m.w.sh	4	41	56	59	24	35	CH	A-7-6			
10	0.6-1.2	h.w.sh	6	63	31	37	20	17	CL	A-6	19.8	18.9	15.8

Note: Results with underline are tested by CH2M (2005).

Table 44. Summary of direct shear test results for slope 15

BH	Depth (m)	Soil	DS				DS (residual)			
			ϕ' (deg.)	c' (kPa)	R^2	Data points	ϕ' (deg.)	c' (kPa)	R^2	Data points
1	6.5-7.1	s.clay	23	27	0.993	4				
2	1.2-1.8	s.clay	24	18	0.999	4				
3	5.6-5.9	h.w.sh	21	18	0.991	4				
4	3.6-4.2	s.clay	28	17	0.994	5				
4	7.7-8.2	m.w.sh	<u>22</u>	<u>12</u>	<u>0.990</u>	<u>3</u>	<u>21</u>	<u>0</u>	<u>1.000</u>	<u>3</u>
4	8.2-8.65	m.w.sh	18	22	0.969	3				
5	5.5-6.1	h.w.sh	22	0	0.994	4				
5	7.15-7.3	h.w.sh	18	38	0.955	4				
6	8.85-9.45	m.w.sh	14	43	0.959	3				
7	2.0-2.6	h.w.sh	22	23	0.923	4				
7	3.2-3.6	m.w.sh	<u>18</u>	<u>19</u>	<u>1.000</u>	<u>3</u>	<u>15</u>	<u>2</u>	<u>0.988</u>	<u>3</u>
8	2.7-3.0	m.w.sh	15	41	1.000	3				
9	0.9-1.4	s.clay	31	14	0.983	5				
9	2.4-2.7	h.w.sh	12	21	0.983	4				
9	2.7-3.0	h.w.sh	21	24	0.912	4				
9	4.5-5.2	h.w.sh	<u>19</u>	<u>27</u>	<u>0.993</u>	<u>3</u>	<u>12</u>	<u>2</u>	<u>0.999</u>	<u>3</u>
9	4.65-4.95	m.w.sh	29	5	0.958	3				
10	0.6-1.2	h.w.sh	26	28	0.954	7				
10	0.6-1.2	h.w.sh	26	15	0.996	4				
10	0.6-1.2	h.w.sh	<u>28</u>	<u>11</u>	<u>0.999</u>	<u>3</u>	<u>21</u>	<u>2</u>	<u>0.994</u>	<u>3</u>
10	0.6-1.2	h.w.sh	*27*	*98*	*0.996*	*7*				

Note: Results with underline are tested by CH2M (2005)

Table 45. Summary of triaxial and unconfined compression test results for slope 15

BH	Depth (m)	Soil	CU		CD		UC
			ϕ' (deg.)	c' (kPa)	ϕ' (deg.)	c' (kPa)	s_u (kPa)
4	5.8-6.4	h.w.sh	<u>24</u>	<u>7</u>			
5	4.8-5.4	s.clay	<u>20</u>	<u>13</u>			
5	9.2-10.7	s.w.sh					<u>321</u>
6	9.9-11.4	s.w.sh					<u>239</u>
7	7.4-8.9	s.w.sh					<u>215</u>
8	1.2-1.8	h.w.sh	<u>21</u>	<u>1</u>			
9	0.9-1.4	s.clay	<u>30</u>	<u>0</u>			
9	7.5-9.0	s.w.sh					<u>181</u>
10	0.6-1.2	h.w.sh			34	0	
10	0.6-1.2	h.w.sh			28	21	
10	0.6-1.2	h.w.sh	<u>28</u>	<u>1</u>			
10	0.6-1.2	h.w.sh	<u>34</u>	<u>10</u>			

CU = Consolidated undrained triaxial; CD = Consolidated drained triaxial;
UC = Unconfined compression. Results with underline are tested by CH2M (2005).

Table 46. Summary of ring shear test results for slope 15

BH	Depth (m)	Soil	ϕ_r' (deg.)	c_r' (kPa)	R^2	Data points	ϕ_r' (c'=0)
3	5.6-5.9	h.w.sh	8.2	0	0.9994	3	8.2
4	8.2-8.65	m.w.sh	8.5	0	1.0000	3	8.5
5	5.5-6.1	h.w.sh	6.7	2.1	0.9997	3	7.1
5	7.15-7.3	h.w.sh	8.0	1.7	1.0000	3	8.4
6	8.85-9.45	m.w.sh	9.8	0.9	1.0000	3	10.0
7	2.0-2.6	h.w.sh	7.2	1.2	1.0000	3	7.4
8	2.7-3.0	m.w.sh	7.3	1.9	0.9999	3	8.0
9	1.4-1.5	h.w.sh	6.4	1.0	0.9995	3	6.6
9	2.7-3.0	h.w.sh	5.7	2.9	0.9999	3	6.4
9	4.65-4.95	m.w.sh	7.6	1.0	0.9991	3	8.0
10	0.6-1.2	h.w.sh	12.8	3.9	0.9997	3	15.4

Table 47. Parameters used for the slope analysis for slope 15

Layer	Soil	ϕ' (deg.)		c' (kPa)		γ' (kN/m^3)	
		Ave.	S.D.	Ave.	S.D.	Ave.	S.D.
1	Compacted Fill	12.0	*2.4*	29	*6.3*	20.4	*1.6*
2	Alluvium	16.5	3.4	33.0	20.3	19.0	*2.0*
3	Highly weathered shale	12.8	4.9	33.2	19.9	20.0	*2.0*
4	Moderatey weathered shale	21.6	9.6	97	134	20.0	*2.0*
5	Slighty weathered shale	23.3	11.3	675	1254	21.0	*2.0*
	WT (water table)	Max.	Min.	Ave.	S.D.		
	Position	1.6	-1.6	0	1.6		
	Elevation (m)	16.2	13.0	14.6	-		

Table 48. Summary of the results of probabilistic analysis for the Real section and the Idealized section for slope 15

Real Section	**Circular (MP)**	**Circular (Bishop)**	**Block (MP)**	**Block (Bishop)**
Mean FS	1.604	1.606	1.789	1.738
Reliability Index	1.87	1.89	1.91	1.89
P (Failure) (%)	3.06	2.95	2.77	2.94
Standard Dev.	0.323	0.321	0.412	0.391
Min FS	0.583	0.583	0.592	0.599
Max FS	2.610	2.584	3.266	3.102
# of Trials	5000	5000	5000	5000
Idealized Section	**Circular (MP)**	**Circular (Bishop)**	**Block (MP)**	**Block (Bishop)**
Mean FS	1.571	1.574	1.637	1.587
Reliability Index	1.83	1.85	1.60	1.57
P (Failure) (%)	3.33	3.24	5.46	5.80
Standard Dev.	0.311	0.311	0.398	0.374
Min FS	0.454	0.446	0.366	0.307
Max FS	2.784	2.788	3.160	2.959
# of Trials	5000	5000	5000	5000

Additional Discussion

Basic Properties and Shear Strengths of the Soils from a Regional Scale

The basic properties of the glacial tills and the clay shales have revealed some tendency from a regional scale. This can be shown from Figures 121 and 122. The figures show that the glacial tills generally have lower clay fraction and lower plasticity index (PI) than the clay shales. Accordingly, the residual friction angles for the glacial tills are also generally higher than those for the clay shales (Figure 123). These results may be a reflection of the different origins and formations of the tills and shales. Corresponding to the basic properties, all the shales are classified as low plasticity clay (CL) by USCS; while most of the shales are classified as high plasticity clay (CH), and a few of the shales are classified as CL.

For the clay shales in Slope 15 (Sugar Creek slope), the residual frictions were also generally low (Figure 124), ranging from 6 to 13°. The spatial variation of the residual friction angle was not promising, as no tendency or correlation between the residual friction angles and the depths of the shales (weathering degree) could be found. However, the residual friction angles were generally decreased following the increase in PI of the shales.

For the in-situ shear strength parameter values of the soils as measured from the BSTs, no clear tendency was noted on a regional basis. However, statistics of the BST measurements on the glacial tills and clay shales for Slopes 1 to 14 (Table 49) show that, the glacial tills have a similar average value of friction angle with the clay shales, but have a considerably lower average value of cohesion. These results suggest that glacial till slopes may be generally less stable than clay shale slopes on a regional scale under similar conditions.

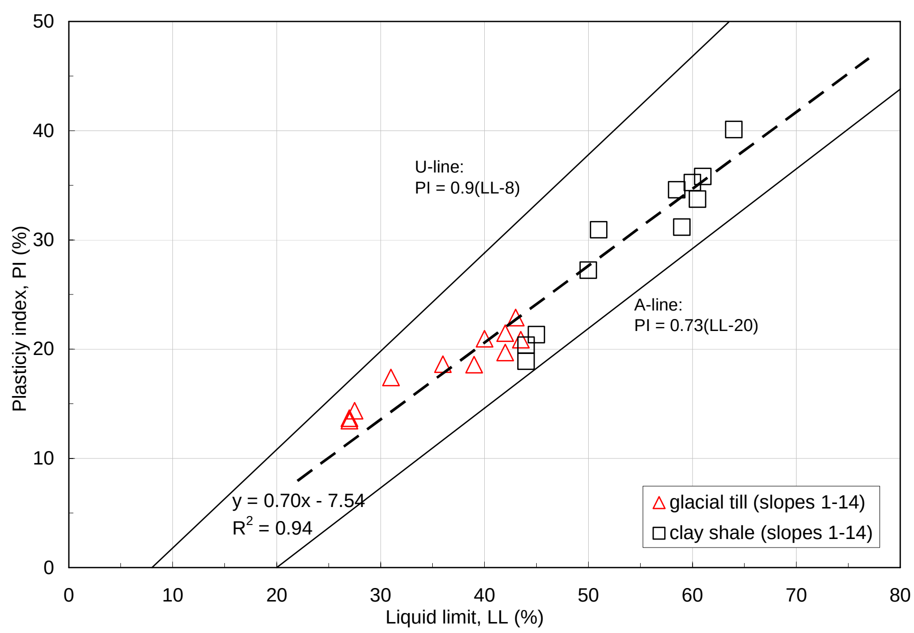


Figure 121. Atterberg limits for all the soils in slopes 1 to 14

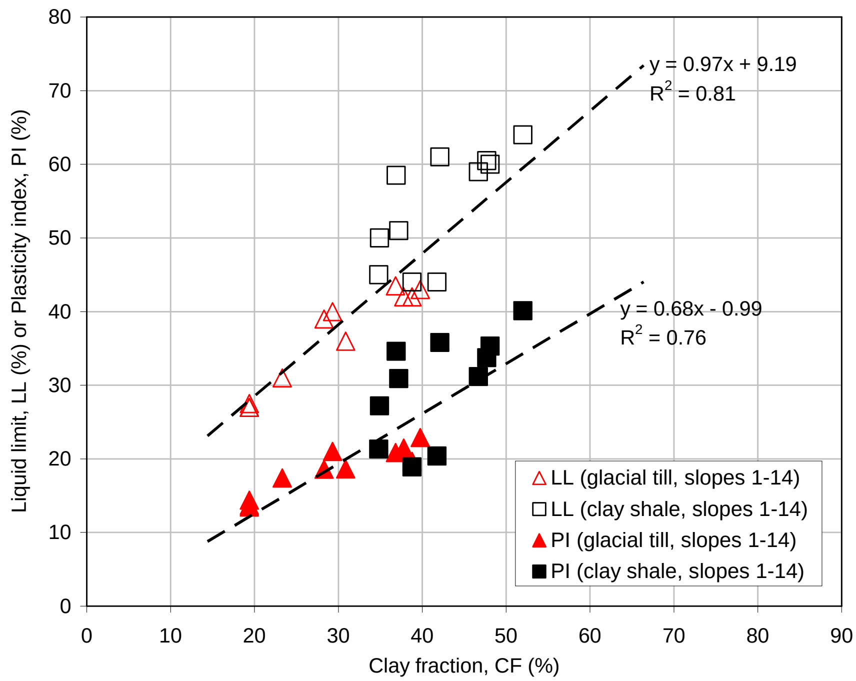


Figure 122. Atterberg limits versus clay fraction for all the soils in slopes 1 to 14

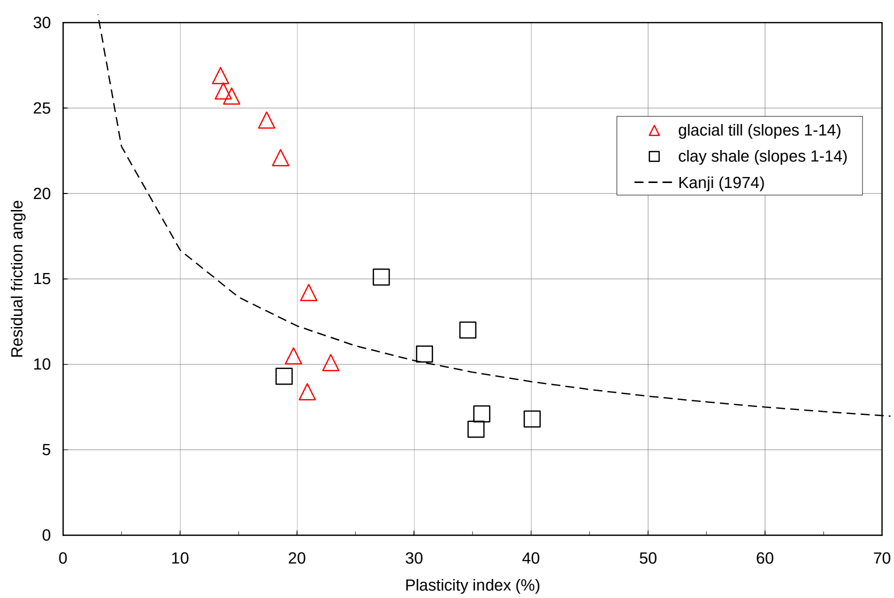


Figure 123. Residual friction angel versus plasticity for soils in slopes 1 to 14

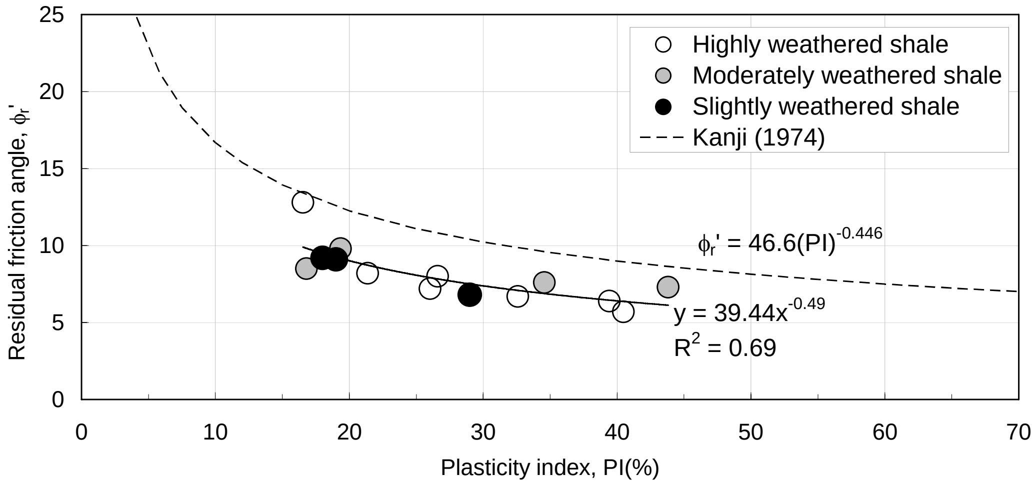


Figure 124. Residual friction angel versus plasticity for soils in slope 15

Table 49. Statistics of the shear strength parameter values from the BST for slopes 1 to 14

Soil	Total No. of Test	ϕ' (deg.)				c' (kPa)			
		Max.	Min.	Ave.	S.D.	Max.	Min.	Ave.	S.D.
Glacial till	23	39	12	22.5	7.2	35	1	11.6	7.6
Clay shale	23	40	10	22.1	7.7	45	5	17.7	9.8

SUMMARY AND CONCLUSIONS

Summary

A total of 15 slopes along highways in Iowa were investigated, which included 13 slides (failed slopes), one unfailed slope and one proposed embankment slope (the Sugar Creek Project). The slopes are mainly comprised of either clay shale or glacial till, which are commonly encountered in Iowa. The slopes are generally gentle and of small scale with slope angle ranging from 11^{o} to 23^{o} and height ranging from 6 to 23m

Extensive field investigations and laboratory tests were performed for the slopes. Field investigations included survey of slope geometry, borehole drilling, soil sampling, in-situ Borehole Shear Testing (BST) and ground water table measurement. Laboratory investigations mainly comprised of ring shear tests, soil basic property tests (grain size analysis and Atterberg limits test), mineralogy analyses, soil classifications, natural water contents and density measurements on the representative soil samples from each slope. Extensive direct shear tests and a few triaxial compression tests, unconfined compression tests were also performed on undisturbed soil samples for the Sugar Creek Project.

Based on the results of field and lab investigations, slope stability analysis was performed on each of the slopes to determine the possible factors resulting in the slope failures, or to evaluate the potential slope instabilities using limit equilibrium methods. Deterministic slope analyses were performed for all the slopes. Probabilistic slope analysis and sensitivity study were also performed for the slope of Sugar Creek Project.

Conclusions

BSTs are competent to characterize the slopes, especially to obtain the soil shear strength parameter values that are essential for the slope stability analysis. The shear strength parameter values obtained from BSTs have the advantages in that they gave direct, in-situ measurements of soil shear strength in a relatively quick manner.

BSTs appeared to have measured the peak shear strength parameter values of the soils for almost all the cases, and the slopes have factor of safety (FS) larger than one all the time indicating the slopes were stable under the conditions (especially the ground water table conditions) when they were investigated.

The ring shear tests gave the residual shear strength parameter values of the soils. These values were normally lower than those values obtained from BSTs since they corresponded to larger displacement of the soil and represented the ultimate shear strength of the soils. FS based on the soil residual shear strength are generally smaller than one, indicating the soils in the slopes may not have reached the residual state.

The ring shear tests occasionally gave shear strength parameter values that were larger than those obtained from BST. The reason may be due to the soil variability, i.e. the ring shear test and the BST may not have tested exactly the same soils. In this case, the residual shear strength parameter values may indicate the upper bound of the soil shear strength. Another reason could be that the soils exhibited ductile or hardening response to shearing.

The back-calculated shear strength of the soils for the slope to give unity FS were generally between the shear strength measured by BST and by ring shear test, indicating that the slopes failed or could fail when the soil shear strength become softened. This situation occurred or will occur once the slope movement was initialized. Most of the slope failures may have been associated with relatively high ground water table conditions.

The slope analysis assuming slip surface passing through the observed scarp or failure zone on the slope surface indicated the most probable slip surface of failure for the slope. The determination of the slip surface together with the soil shear strength will be useful for slope remediation design.

For a site involving great variability of both soil stratification and soil shear strength and various ground water table conditions as the case of Sugar Creek Project, sensitivity study of slope analysis and probabilistic slope analysis were proven to be useful and effective. Sensitivity analysis showed that shear strength of the soil is the most sensitive parameter affecting FS. Effect of unit weight on FS is negligible. Water table in the slope has significant effect on FS, while water table in the river has moderate effect on FS.

Probabilistic slope analysis was useful when a relatively large amount of input parameters are available, such as the shear strength parameter values as obtained from BST for the Sugar Creek Project. Probability of failure for the slope was evaluated based on the statistical distribution of the soil shear strength. The results are useful for further evaluation of the slope design.

RECOMMENDATIONS

Recommendations for Future Studies

Borehole shear testing can be performed more on the shearing zone of a failed slope or the potential slip zone of the proposed slope as long as the site investigation program is permitted. This may give better information to determine or to predict the controlling factors resulting in the slope failure or the potential instability. The shearing zone can be estimated by trial slope analysis in conjunction with the failure features such as the scarp for a failed slope.

To establish long term slope monitoring, including ground water table variation and slope deformation, for some selected slopes in order to collect relatively complete information, which will result in improved slope analysis together with the soil shear strengths obtained from Borehole Shear Test. This may be especially suitable for those newly constructed slopes susceptible to slope instability.

To establish detailed landslide inventory for the state as long as the resources is available. This will be helpful to overview the slope instability problems from a regional prospect.

Provide pore water pressure measurement for the Borehole Shear Test (BST) so that the measurement of the effective stress can be monitored and verified, especially for clayey soils due to their low permeability. This may improve the BST measurements.

Perform quantitative mineralogical analysis for the weathered shales to investigate the possible correlation of the mineralogical compositions with the weathering grades.

Recommendations for Implementations

The research findings are expected to benefit civil and geotechnical engineers of government transportation agencies, consultants, and contractors dealing with slope stability, slope remediation, and geotechnical testing in Iowa. In-situ BST measurements provide reliable, site-specific soil parameters for design applications which can lead to substantial cost savings over using empirical estimations for critical soil properties.

As the BST is an alternative to expensive and time-consuming laboratory testing, the device is particularly useful in obtaining relatively large amounts of data necessary for probabilistic analyses. Procedures for incorporating Borehole Shear tests into practice are documented in Volume 2 of this report. Nevertheless, some training may be required for effective and appropriate use.

The BST is intended to test soils such as clays, silts, and sands. The device can produce erroneous results in gravelly soils. Additionally, the quality of boreholes affects test results, and disturbance to borehole walls should be minimized before test performance. A final limitation of widespread Borehole Shear testing may be its limited availability, as only five test devices are currently being used in Iowa.

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APPENDIX

Boring Log

Slope 1
Location: Hwy34 MP169.3
Boring No. BH1

Date: 8.12.04
Drilling method: Hand auger
Driller: HY

Depth (ft)	Depth (m)	Soil Descriptions	Profile Symbol	Borehole Shear Test No	Sample No.	Liquid limit (%)	Plasticity Index (%)	Moisture Content (%)	Total density (kN/m3)	Ground Water Level / Date
0	0.0	Fill material: yellowish brown clay (glacial till) mixed with light grey clay (shale), soft to medium stiff, with trace of fine gravel								
1	0.3				1	45	21	28.0	17.0	
2	0.6									
4	1.2									
6	1.8									
7	2.1			1						
8	2.4				2	39	19	25.0	18.0	
10	3.0									8.12.04

(End of borehole)

Symbol: Sand, Clay

Figure A1. Borehole log for BH1 at slope 1

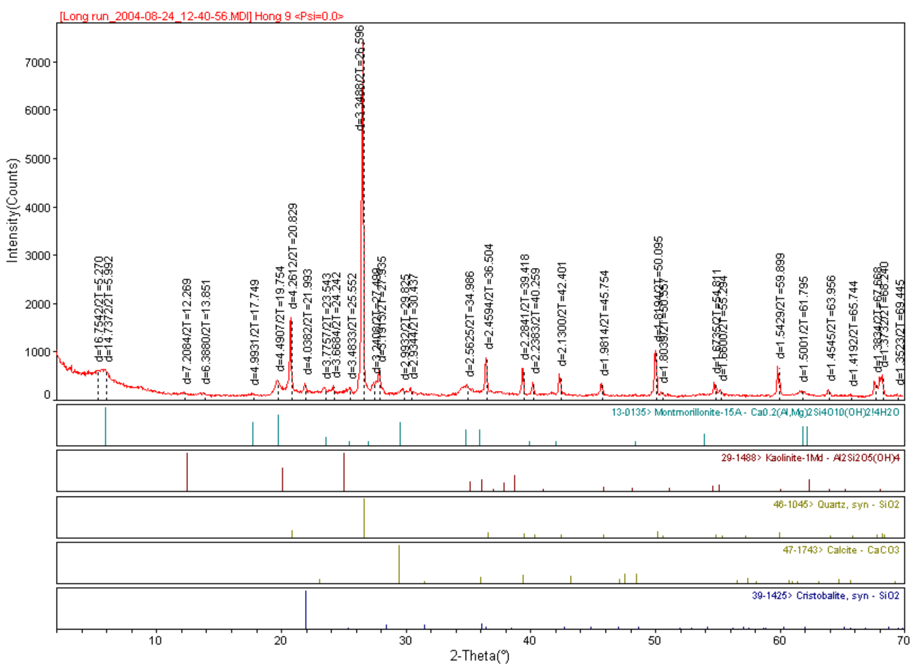

Figure A2. X-Ray Diffraction results for the shale at depth of 0.3 m for slope 1

Boring Log

Slope 2
Location: Hwy34 MP171.7
Boring No.: BH1

Date: 7.20.04
Drilling method: Hand auger
Driller: HY

Depth (ft)	Depth (m)	Soil Descriptions	Profile Symbol	Borehole Shear Test	Sample No.	Liquid limit (%)	Plasticity Index (%)	Moisture Content (%)	Total density (kN/m3)	Ground Water Level / Date
0	0.0	Light brown silty clay (shale) with trace of sand covered with 6 inches of topsoil								
2	0.6									
4	1.2	Dark brown or grey silty clay (highly weathered shale), wet, medium stiff to stiff								
6	1.8									8.12.04
7.5	2.3			1	1			13.2		
9.5	2.9			2	2			19.3		
12.5	3.8			3	3			26.1	19.1	
13	3.9	Grey clay (highly weathered shale), soft to medium stiff								

(End of borehole)

Symbol: Sand, Clay, Soft to medium stiff clay

Figure A3. Borehole log for BH1 at slope 2

Boring Log

Slope 2
Location: Hwy34 MP171.7
Boring No.: BH2

Date: 8.12.04
Drilling method: Hand auger
Driller: HY

Depth (ft)	Depth (m)	Soil Descriptions	Profile Symbol	Borehole Shear Test	Sample No.	Liquid limit (%)	Plasticity Index (%)	Moisture Content (%)	Total density (kN/m3)	Ground Water Level / Date
0	0.0	Light brown silty clay (shale) with trace of sand covered with 6 inches of topsoil								
2	0.6									8.12.04
4	1.2	Dark brown or grey silty clay (highly weathered shale), wet, medium stiff to stiff								
6.5	2.0			1	1			20.1		
8.5	2.6			2	2			25.0		
9.5	2.9									
10.5	3.2	Grey clay (highly weathered shale), soft to medium stiff		3	3	64	40	25.2	19.0	
11										

(End of borehole)

Symbol: Sand, Clay, Soft to medium stiff clay

Figure A4. Borehole log for BH2 at slope 2

Boring Log

Slope 2
Location: Hwy34 MP171.7
Boring No.: BH3

Date: 8.12.04
Drilling method: Hand auger
Driller: HY

Depth (ft)	Depth (m)	Soil Descriptions	Profile Symbol	Borehole Shear Test	Sample No.	Liquid limit (%)	Plasticity Index (%)	Moisture Content (%)	Total density (kN/m3)	Ground Water Level / Date
0	0.0	Light brown silty clay (shale) with trace of sand covered with 6 inches of topsoil								
2	0.6									
4	1.2	Dark brown or grey silty clay (highly weathered shale), wet, medium stiff to stiff								8.12.04
6	1.8									
7.5	2.3			1						
8.5	2.6			2	1			23.5	18.9	
9	2.7	Grey clay (highly weathered shale), soft to medium stiff								

(End of borehole)

Symbol: Sand; Clay; Soft to medium stiff clay

Figure A5. Borehole log for BH3 at slope 2

Boring Log

Slope 2
Location: Hwy34 MP171.7
Boring No.: BH4

Date: 8.12.04
Drilling method: Hand auger
Driller: HY

Depth (ft)	Depth (m)	Soil Descriptions	Profile Symbol	Borehole Shear Test	Sample No.	Liquid limit (%)	Plasticity Index (%)	Moisture Content (%)	Total density (kN/m3)	Ground Water Level / Date
0	0.0	Dark grey shale mixed with coal								
1	0.3									
2	0.6	Dark brown or grey silty clay (highly weathered shale), wet, medium stiff to stiff			1	59	31	30.9		8.12.04
3	0.9									
3.5	1.1			1	2	60	35	33.6	18.0	
5	1.5	Grey clay (highly weathered shale), soft to medium stiff								
5.2	1.6	Mederately weathered shale								

(End of borehole)

Symbol: Clay; Soft to medium stiff clay; Mederately weathered shale

Figure A6. Borehole log for BH4 at slope 2

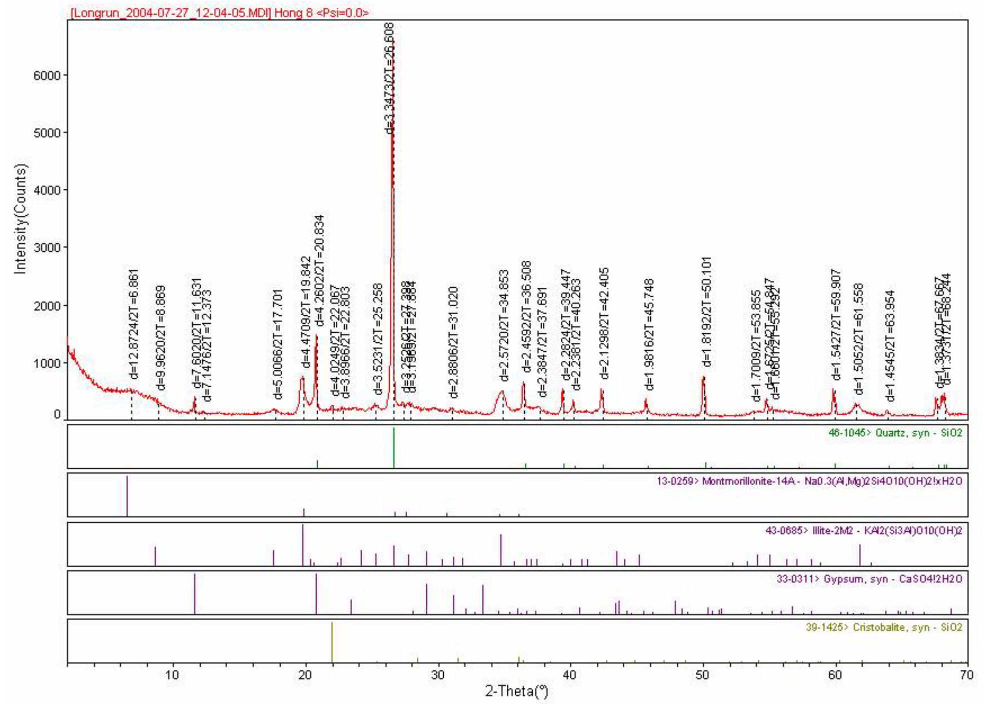

Figure A7. X-Ray Diffraction results for the shale at depth of 0.6m in BH4 for slope 2

Boring Log

Slope 3
Location: Hwy34 MP175.3
Boring No BH1

Date: 7.18.04
Drilling method: Hand auger
Driller: HY

Depth (ft)	Depth (m)	Soil Descriptions	Profile Symbol	Borehole Shear Test No.	Sample No.	Liquid limit (%)	Plasticity Index (%)	Moisture Content (%)	Total density (kN/m3)	Ground Water Level / Date
0	0.0	Yellowish brown glacial till, soft to medium stiff, with trace of fine gravel								
2	0.6			1	1	42	20	23.1	19.0	
4	1.2			2	2					
6	1.8			3	3					(7.19.05)
7	2.1									
8	2.4			4	4					

(End of borehole)

Symbol Sand Clay

Figure A8. Borehole log for BH1 at slope 3

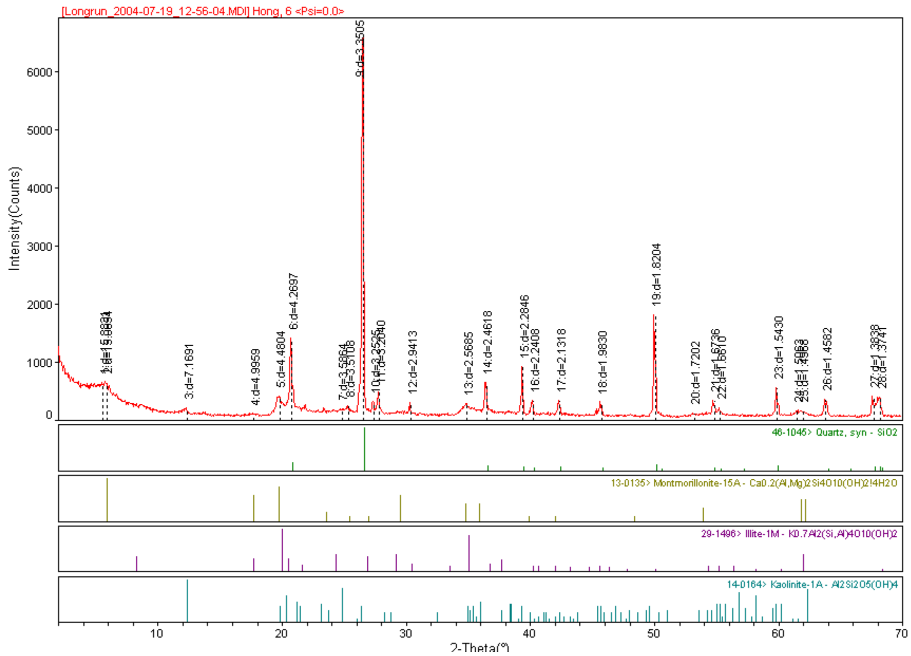

Figure A9. X-Ray Diffraction results for the till at depth of 0.6m in BH2 for slope 3

Boring Log

Slope 4
Location: Hwy34 MP175.5
Boring No BH1

Date: 7.18.04
Drilling method: Hand auger
Driller: HY

Depth (ft)	Depth (m)	Soil Descriptions	Profile Symbol	Borehole Shear Test	Sample No.	Liquid limit (%)	Plasticity Index (%)	Moisture Content (%)	Total density (kN/m3)	Ground Water Level / Date
0	0.0	Yellowish brown glacial till, soft to medium stiff, with trace of fine gravel								
1	0.3				1	42	21	24.7		
2	0.6									
4	1.2									
5	1.5			1	2	43	23	27.9	18.2	
6	1.8									∇
7	2.1									(7.19.05)

(End of borehole)

Symbol Sand Clay

Figure A10. Borehole log for BH1 at slope 4

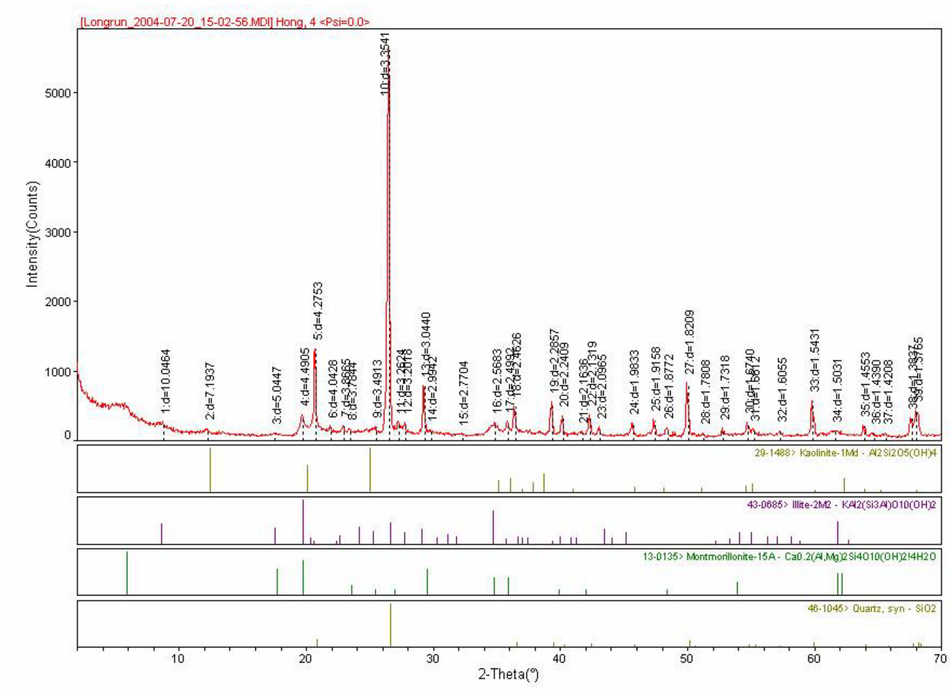

Figure A11. X-Ray Diffraction results for the till at depth of 0.6m in BH2 for slope 4

Boring Log

Slope 5
Location: Hwy34 MP178.3 (North)
Boring No. BH1

Date: 7.15.04
Drilling method: Hand auger
Driller: HY

Depth (ft)	Depth (m)	Soil Descriptions	Profile Symbol	Borehole Shear Test No.	Sample No.	Liquid limit (%)	Plasticity Index (%)	Moisture Content (%)	Total density (kN/m3)	Ground Water Level / Date
0	0.0	Yellowish brown glacial till, soft to medium stiff, with trace of fine gravel								
2	0.6			1	1	44	21	24.1	18.4	
4	1.2			2	2					
6	1.8			3	3					∇ (7.19.05)
8	2.4			4	4				17.9	

(End of borehole)

Symbol Sand Clay

Figure A12. Borehole log for BH1 at slope 5

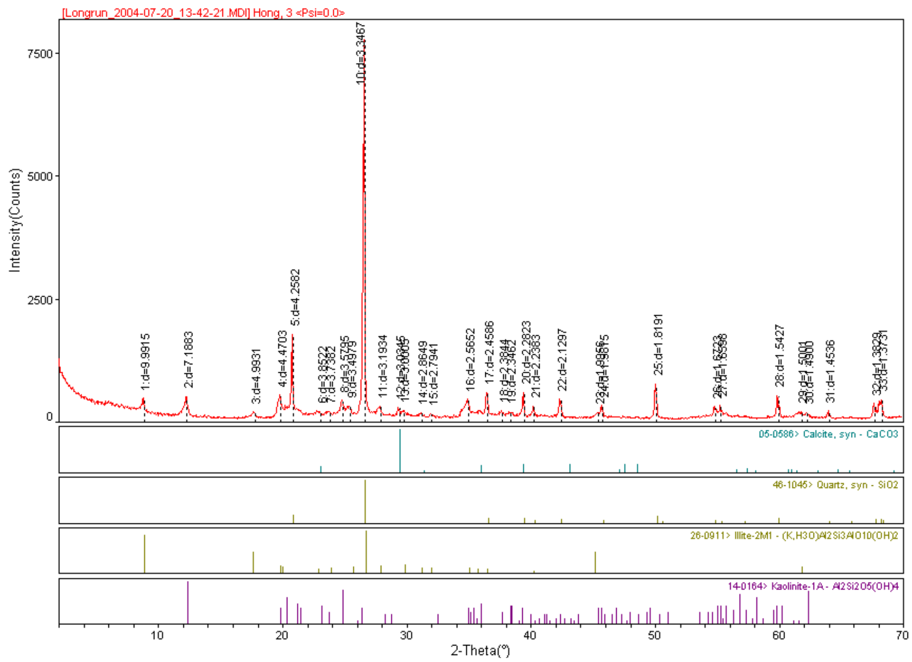

Figure A13. X-Ray diffraction results for the till at depth of 0.6m in BH2 for slope 5

Boring Log

Slope 6
Location: Hwy34 MP178.3 (South)
Boring No. BH1

Date: 7.14.04
Drilling method: Hand auger
Driller: HY

Depth (ft)	Depth (m)	Soil Descriptions	Profile Symbol	Borehole Shear Test No	Sample No.	Liquid limit (%)	Plasticity Index (%)	Moisture Content (%)	Total density (kN/m3)	Ground Water Level / Date
0	0.0	Brown and grey shale, soft to medium stiff								
1	0.3				1	44	19	25.3		
2	0.6									
4	1.2									
6	1.8			1	2			27.0	18.0	
7	2.1									(7.15.05)

(End of borehole)

Symbol Sand Clay

Figure A14. Borehole log for BH1 at slope 6

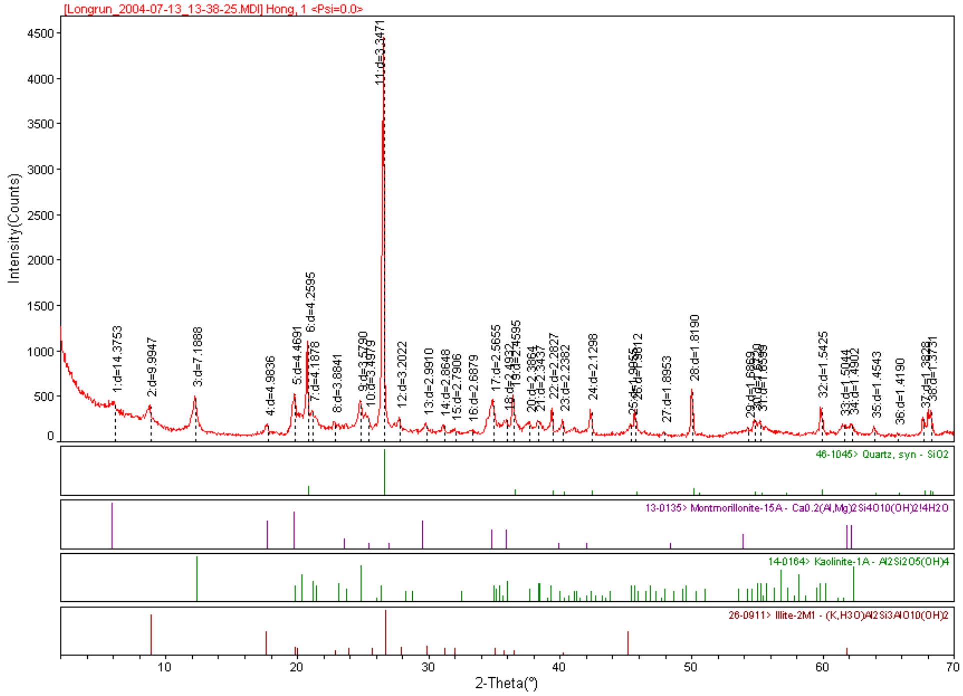

Figure A15. X-Ray diffraction results for the shale at depth of 0.3m for slope 6

Boring Log

Slope	7	Date:	7.16.04
Location:	Hwy169 Winterset	Drilling method:	rotary drillrig
Boring No.	BH1	Driller:	DJW, MS

Depth (ft)	Depth (m)	Soil Descriptions	Profile Symbol	Borehole Shear Test	Sample No.	Liquid limit (%)	Plasticity Index (%)	Moisture Content (%)	Total density (kN/m3)	Ground Water Level / Date
0.0	0.0	Grey and brown mixed silty clay, wet, medium stiff.								
2.0	0.6									
4.0	1.2									
5.5	1.7				1			19.1	18.3	
7.0	2.1									∇
8.0	2.4			1	2			17.0	18.6	
8.5	2.6									(7.19.04)
10.0	3.0	Brown clay shale, wet, soft to medium stiff								
11.5	3.5									
14.0	4.2	Redish brown clay shale, wet, medium stiff to stiff. Occasionally seen limestone pieces.								
16.0	4.8									
18.0	5.4									
19.0	5.7									
20.0	6.0	Brown clay shale, wet, stiff								

(End of borehole)

Note: Borelog below 11.5ft was projected from adjacent borehole drilled by IaDOT.

Symbol: Sand, Clay, Stiff clay

Figure A16. Boring log for BH1 at slope 7

Boring Log

Slope 7
Location: Hwy169 Winterset
Boring No BH2

Date: 7.16.04
Drilling method: rotary drillrig
Driller: DJW, MS

Depth (ft)	Depth (m)	Soil Descriptions	Profile Symbol	Borehole Shear Test	Sample No.	Liquid limit (%)	Plasticity Index (%)	Moisture Content (%)	Total density (kN/m3)	Ground Water Level / Date
0.0	0.0	Grey and brown mixed silty clay, wet, medium stiff.	Clay							
2.0	0.6									
										7.19.04 ∇
4.0	1.2									
6.0	1.8									
7.0	2.1				1			16.7	19.0	
9.0	2.7	Brown clay shale, wet, soft to medium stiff	Clay	1	2	50	27	16.2	18.9	

(End of borehole)

Symbol: Sand, Clay, Stiff clay

Figure A 17. Boring log for BH2 at slope 7

Boring Log

Slope 7
Location: Hwy169 Winterset
Boring No. BH3

Date: 7.19.04
Drilling method: Hand auger
Driller: HY

Depth (ft)	Depth (m)	Soil Descriptions	Profile Symbol	Borehole Shear Test	Sample No.	Liquid limit (%)	Plasticity Index (%)	Moisture Content (%)	Total density (kN/m3)	Ground Water Level / Date
0.0	0.0	Grey and brown mixed silty clay, wet, medium stiff.	Clay							
2.0	0.6									
4.0	1.2									
5.0	1.5			1	1					7.19.04 ∇
6.0	1.8									
7.0	2.1									
8.0	2.4	Brown clay shale, wet, soft to medium stiff	Clay							
9.0	2.7			2	2	59	35	29.0	19.2	
10.5	3.2									
11.0	3.3	Redish brown clay shale, wet, medium to stiff.	Stiff clay	3	3					
12.0	3.6			4	4					

(End of borehole)

Symbol: Sand, Clay, Stiff clay

Figure A18. Boring log for BH3 at slope 7

Boring Log

Slope 7
Location: Hwy169 Winterset
Boring No. BH4

Date: 7.19.04
Drilling method: Hand auger
Driller: HY

Depth (ft)	Depth (m)	Soil Descriptions	Profile Symbol	Borehole Shear Test	Sample No.	Liquid limit (%)	Plasticity Index (%)	Moisture Content (%)	Total density (kN/m3)	Ground Water Level / Date
0.0 2.0 3.7	0.0 0.6	Brown clay shale, wet, soft to medium stiff								7.19.04
3.4	1.0	Redish brown clay shale, wet, medium stiff to stiff.		1	1			20.3	19.3	
4.4	1.3			2	2			22.2		
5.7	1.7									

(End of borehole)

Symbol Sand Clay Stiff clay

Figure A19. Boring log for BH4 at slope 7

Boring Log

Slope 8 (East side)
Location: Hwy169 Afton (2 mi South)
Boring No BH1

Date: 8.11.04
Drilling method: Hand auger
Driller: HY

Depth (ft)	Depth (m)	Soil Descriptions	Profile Symbol	Borehole Shear Test No.	Sample No.	Liquid limit (%)	Plasticity Index (%)	Moisture Content (%)	Total density (kN/m3)	Ground Water Level / Date
0	0.0	Yellowish brown glacial till, soft to medium stiff, with trace of fine gravel								
2	0.6									
4	1.2									(8.12.04)
6	1.8			1	1	40	21	19.0	18.1	

(End of borehole)

Symbol Sand Clay

Figure A20. Borehole log for BH1 at slope 8

Boring Log

Slope 8 (East side) Date: 8.11.04
Location: Hwy169 Afton (2 mile South) Drilling method: Hand auger
Boring No. BH2 Driller: HY

Depth (ft)	Depth (m)	Soil Descriptions	Profile Symbol	Borehole Shear Test No.	Sample No.	Liquid limit (%)	Plasticity Index (%)	Moisture Content (%)	Total density (kN/m3)	Ground Water Level / Date
0	0.0	Yellowish brown glacial till, soft to medium stiff, with trace of fine gravel								(8.12.04)
2	0.6									
4	1.2			1	1			16.7		
6	1.8			2	2			16.7		
8	2.4			3	3			16.9	19.2	

(End of borehole)

Symbol Sand Clay

Figure A21. Borehole log for BH2 at slope 8

Boring Log

Slope 9 (West side) Date: 8.11.04
Location: Hwy169 Afton (2 mile South) Drilling method: Hand auger
Boring No. BH1 Driller: HY

Depth (ft)	Depth (m)	Soil Descriptions	Profile Symbol	Borehole Shear Test No.	Sample No.	Liquid limit (%)	Plasticity Index (%)	Moisture Content (%)	Total density (kN/m3)	Ground Water Level / Date
1	0.3	Brown glacial till, medium stiff, trace limestone			1	36	19	18.3		
2	0.6				2	61	34	29.2		
4	1.2	Grey clay shale, wet, medium stiff to stiff. Occasional limestone pieces.								(8.12.04)
6	1.8			1	3	61	36	26.0	18.5	
8	2.4			2	4			21.7		

(End of borehole)

Symbol Sand Clay

Figure A22. Borehole log for BH1 at slope 9

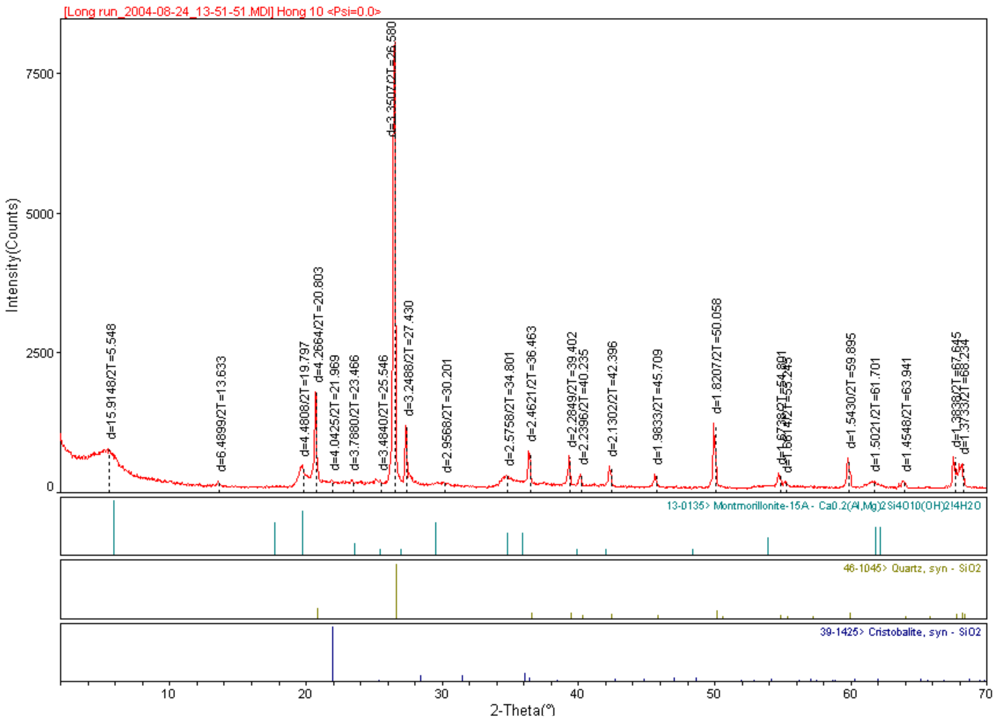

Figure A23. X-Ray Diffraction results for the shale at depth of 0.6m for slope 9

Boring Log

Slope 10 (Eest side) Date: 8.11.04
Location: Hwy169 Afton (4 mile South) Drilling method: Hand auger
Boring No. BH1 Driller: HY

Depth (ft)	Depth (m)	Soil Descriptions	Profile Symbol	Borehole Shear Test No.	Sample No.	Liquid limit (%)	Plasticity Index (%)	Moisture Content (%)	Total density (kN/m3)	Ground Water Level / Date
		Grey clay shale, wet, medium stiff to stiff. Occasional limestone pieces.								
2	0.6									
4	1.2									
6	1.8									(8.12.04)
8	2.4			1	1	51	31	23.8	18.0	

(End of borehole)

Symbol Sand Clay

Figure A24. Borehole log for BH1 at slope 10

Boring Log

Slope 11
Location: HwyE57 Luther (Slope A)
Boring No. BH1

Date: 10.09.04
Drilling method: Hand auger
Driller: HY

Depth (ft)	Depth (m)	Soil Descriptions	Profile Symbol	Borehole Shear Test No.	Sample No.	Liquid limit (%)	Plasticity Index (%)	Moisture Content (%)	Total density (kN/m3)	Ground Water Level / Date
0	0.0	Brown glacial till, soft to medium stiff, with trace of fine gravel								
2	0.6									
4	1.2									
6	1.8									
8	2.4									
10	3.0									
12	3.6			1	1	28	14	17.8	19.2	(10.10.04)
13.5	4.1			2						

(End of borehole)

Symbol: Sand, Clay

Figure A25. Borehole log for BH1 at slope 11

Boring Log

Slope 11
Location: HwyE57 Luther (Slope A)
Boring No. BH2

Date: 10.09.04
Drilling method: Hand auger
Driller: HY

Depth (ft)	Depth (m)	Soil Descriptions	Profile Symbol	Borehole Shear Test No.	Sample No.	Liquid limit (%)	Plasticity Index (%)	Moisture Content (%)	Total density (kN/m3)	Ground Water Level / Date
0	0.0	Brown glacial till, soft to medium stiff, with trace of fine gravel								
2	0.6									
4	1.2									
6	1.8									
8	2.4									
10	3.0									(10.10.04)
12.5	3.8			1	1			13.1		

(End of borehole)

Symbol: Sand, Clay

Figure A26. Borehole log for BH2 at slope 11

Boring Log

Slope 11 | Date: 10.09.04
Location: HwyE57 Luther (Slope A) | Drilling method: Hand auger
Boring No. BH3 | Driller: HY

Depth (ft)	Depth (m)	Soil Descriptions	Profile Symbol	Borehole Shear Test No.	Sample No.	Liquid limit (%)	Plasticity Index (%)	Moisture Content (%)	Total density (kN/m3)	Ground Water Level / Date
0	0.0	Brown glacial till, soft								
2	0.6	to medium stiff, with								
		trace of fine gravel								
4	1.2									
6	1.8									
8	2.4									(10.10.04)
9	2.7			1	1			15.0	19.4	

(End of borehole)

Symbol Sand Clay

Figure A27. Borehole log for BH3 at slope 11

Boring Log

Slope 12 | Date: 10.09.04
Location: HwyE57 Luther (Slope B) | Drilling method: Hand auger
Boring No. BH1 | Driller: HY

Depth (ft)	Depth (m)	Soil Descriptions	Profile Symbol	Borehole Shear Test No.	Sample No.	Liquid limit (%)	Plasticity Index (%)	Moisture Content (%)	Total density (kN/m3)	Ground Water Level / Date
0	0.0	Brown glacial till, soft								
2	0.6	to medium stiff, with								
		trace of fine gravel								
4	1.2									
6	1.8									
8	2.4									
10	3.0									
12.5	3.8			1	1			21.0	19.6	(10.10.04)
13.5	4.1			2	2					

(End of borehole)

Symbol Sand Clay

Figure A28. Borehole log for BH1 at slope 12

Boring Log

Slope 12
Location: HwyE57 Luther (Slope B)
Boring No. BH2

Date: 10.09.04
Drilling method: Hand auger
Driller: HY

Depth (ft)	Depth (m)	Soil Descriptions	Profile Symbol	Borehole Shear Test No.	Sample No.	Liquid limit (%)	Plasticity Index (%)	Moisture Content (%)	Total density (kN/m3)	Ground Water Level / Date
0	0.0	Brown glacial till, soft to medium stiff, with trace of fine gravel	Clay							
2	0.6									
4	1.2									
6	1.8									
8	2.4									
10	3.0									
12.5	3.8			1	1			15.0		

(End of borehole) *GWT not observed on 10.10.04*

Symbol: Sand, Clay

Figure A29. Borehole log for BH2 at slope 12

Boring Log

Slope 12
Location: HwyE57 Luther (Slope B)
Boring No. BH3

Date: 10.09.04
Drilling method: Hand auger
Driller: HY

Depth (ft)	Depth (m)	Soil Descriptions	Profile Symbol	Borehole Shear Test No.	Sample No.	Liquid limit (%)	Plasticity Index (%)	Moisture Content (%)	Total density (kN/m3)	Ground Water Level / Date
0	0.0	Brown glacial till, soft to medium stiff, with trace of fine gravel	Clay							
2	0.6									(10.10.04)
4	1.2									∇
6	1.8									
8	2.4									
10	3.0									
11	3.3			1	1	31	17	15.0		

(End of borehole)

Symbol: Sand, Clay

Figure A30. Borehole log for BH3 at Slope 12

Boring Log

Slope 13 — Date: 10.10.04
Location: HwyE57 Luther (Slope C) — Drilling method: Hand auger
Boring No. BH1 — Driller: HY

Depth (ft)	Depth (m)	Soil Descriptions	Profile Symbol	Borehole Shear Test No.	Sample No.	Liquid limit (%)	Plasticity Index (%)	Moisture Content (%)	Total density (kN/m3)	Ground Water Level / Date
0	0.0	Brown glacial till, soft								
2	0.6	to medium stiff, with								
		trace of fine gravel								
4	1.2									
6	1.8									
8	2.4									(10.11.04)
10	3.0			1	1	27	14	13.1	18.3	

(End of borehole)

<u>Symbol</u> Sand Clay

Figure A31. Borehole log for BH1 at slope 13

Boring Log

Slope 14 — Date: 10.10.04
Location: HwyE57 Luther (Slope D) — Drilling method: Hand auger
Boring No. BH1 — Driller: HY

Depth (ft)	Depth (m)	Soil Descriptions	Profile Symbol	Borehole Shear Test No.	Sample No.	Liquid limit (%)	Plasticity Index (%)	Moisture Content (%)	Total density (kN/m3)	Ground Water Level / Date
0	0.0	Brown glacial till, soft								
2	0.6	to medium stiff, with								
		trace of fine gravel								
4	1.2									
6	1.8									
8	2.4									
9	2.7			1	1	27	14	10.6	18.0	

(End of borehole) *GWT not observed on 10.11.04*

<u>Symbol</u> Sand Clay

Figure A32. Borehole log for BH1 at slope 14

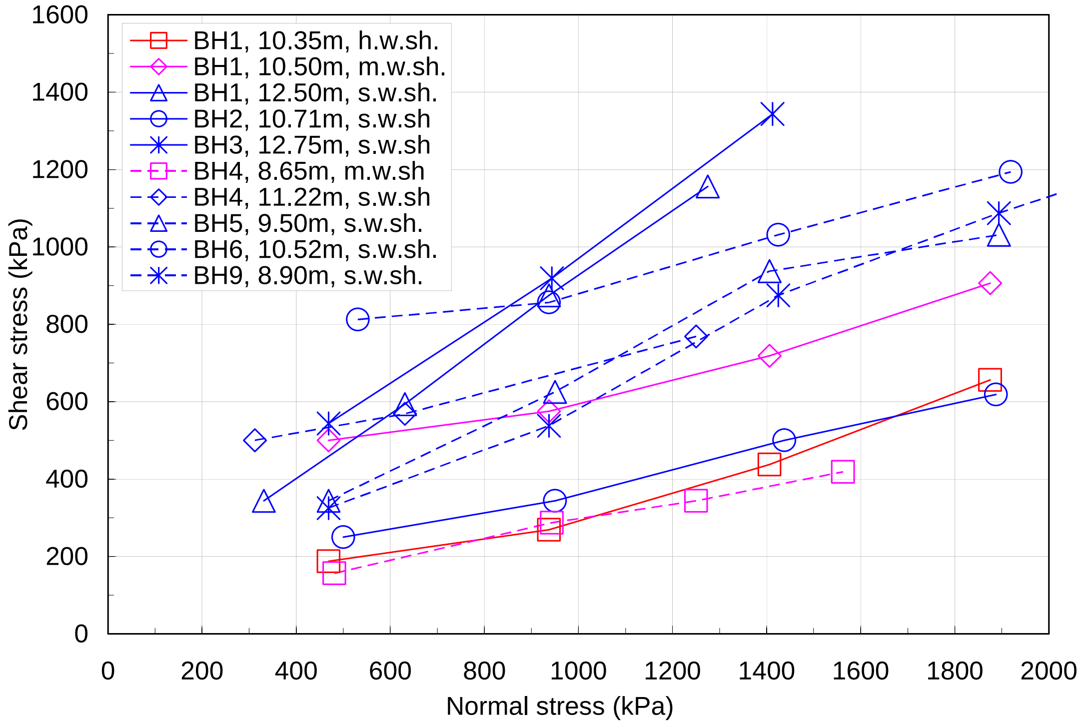

Figure A33. BST results for slope 15 (Part 1 of 4)

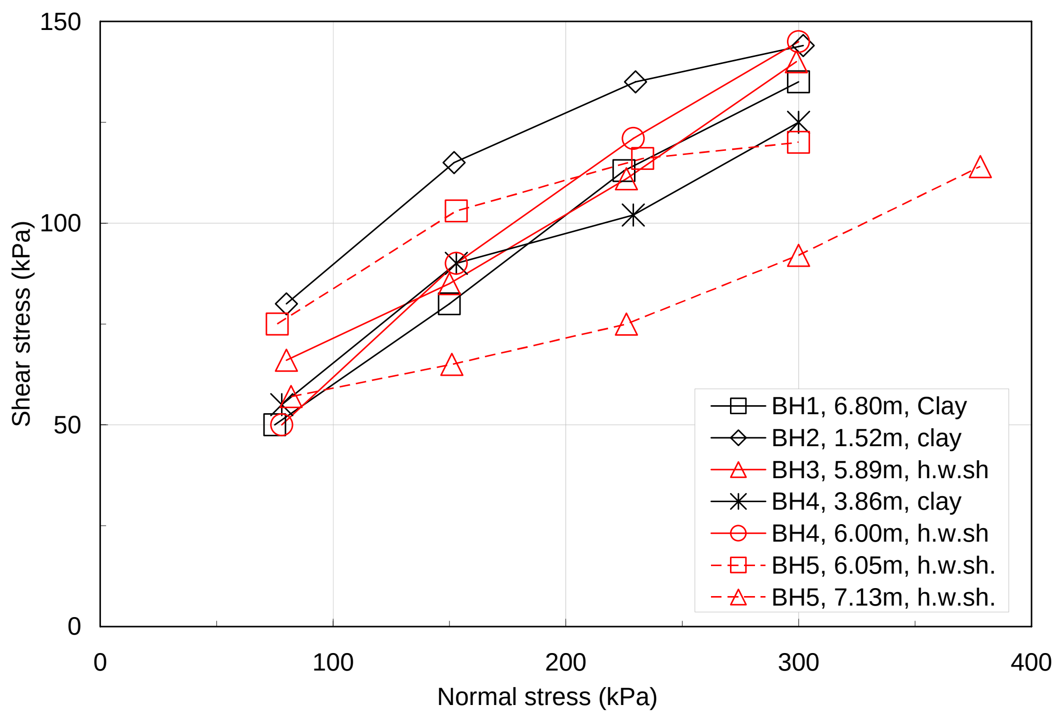

Figure A34. BST results for slope 15 (Part 2 of 4)

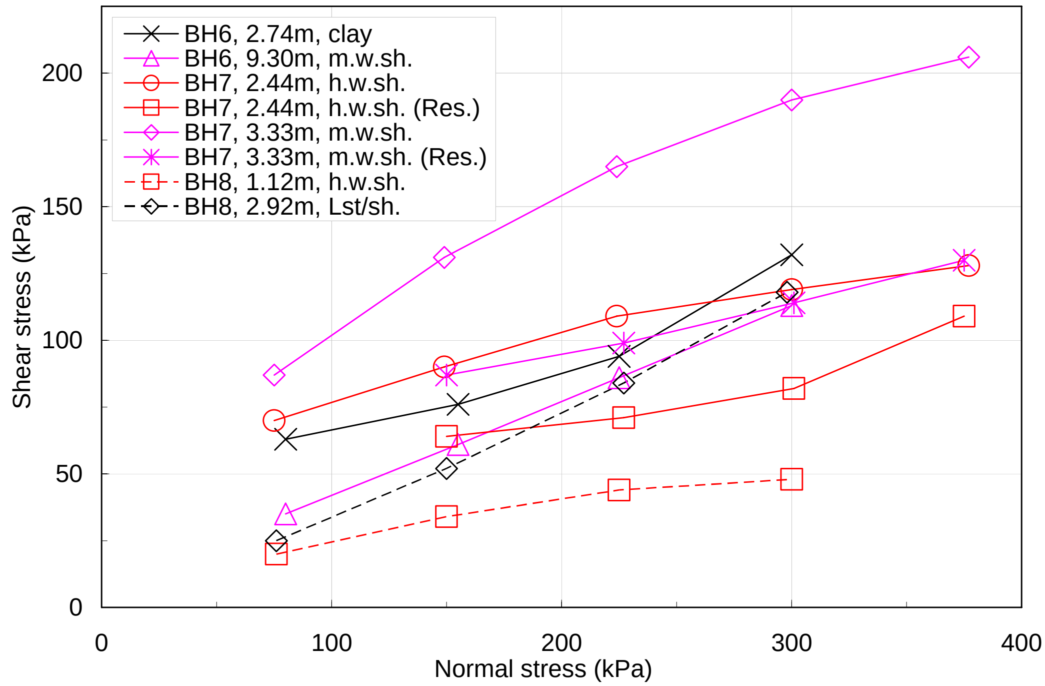

Figure A35. BST results for slope 15 (Part 3 of 4)

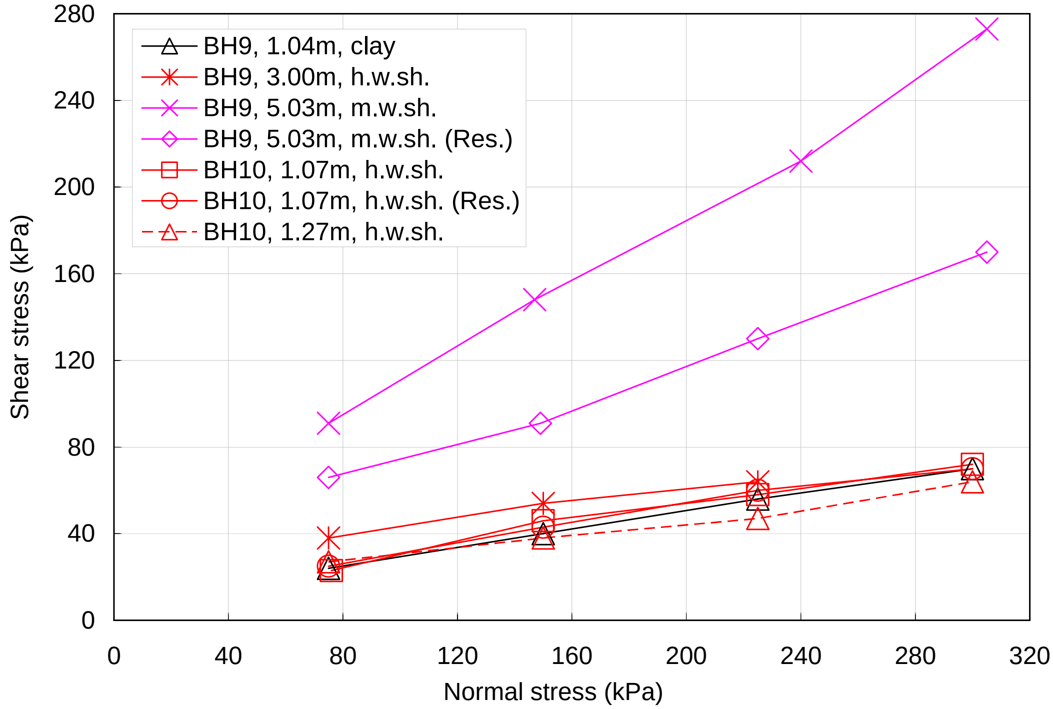

Figure A36. BST results for slope 15 (Part 4 of 4)

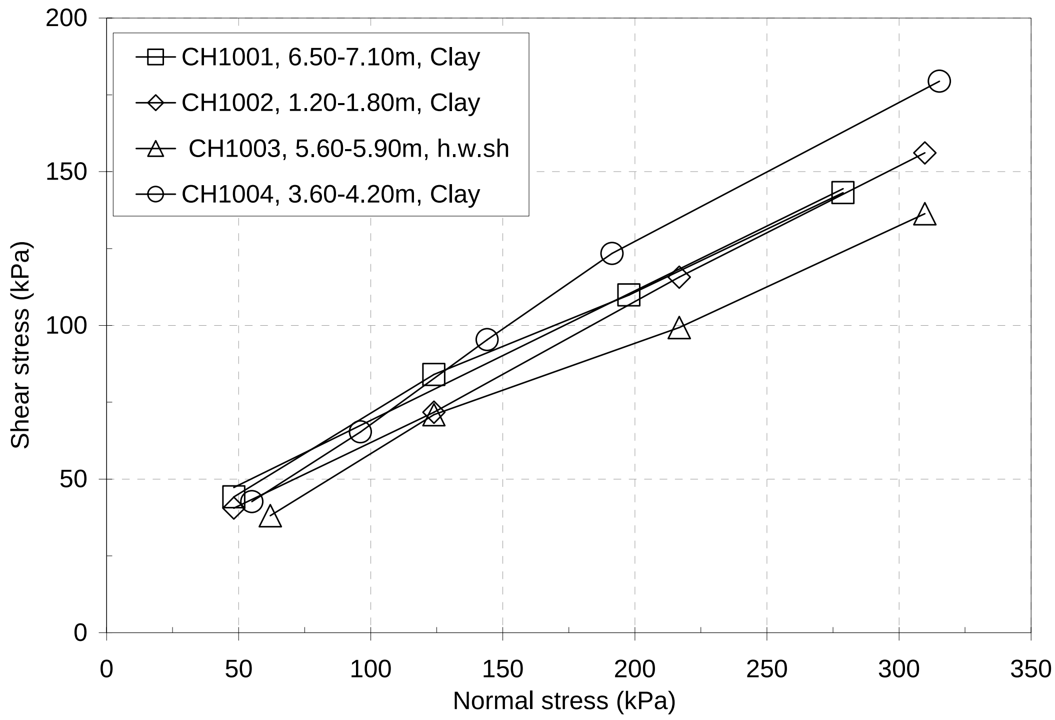

Figure A37. Direct shear test results for slope 15 (part 1 of 4)

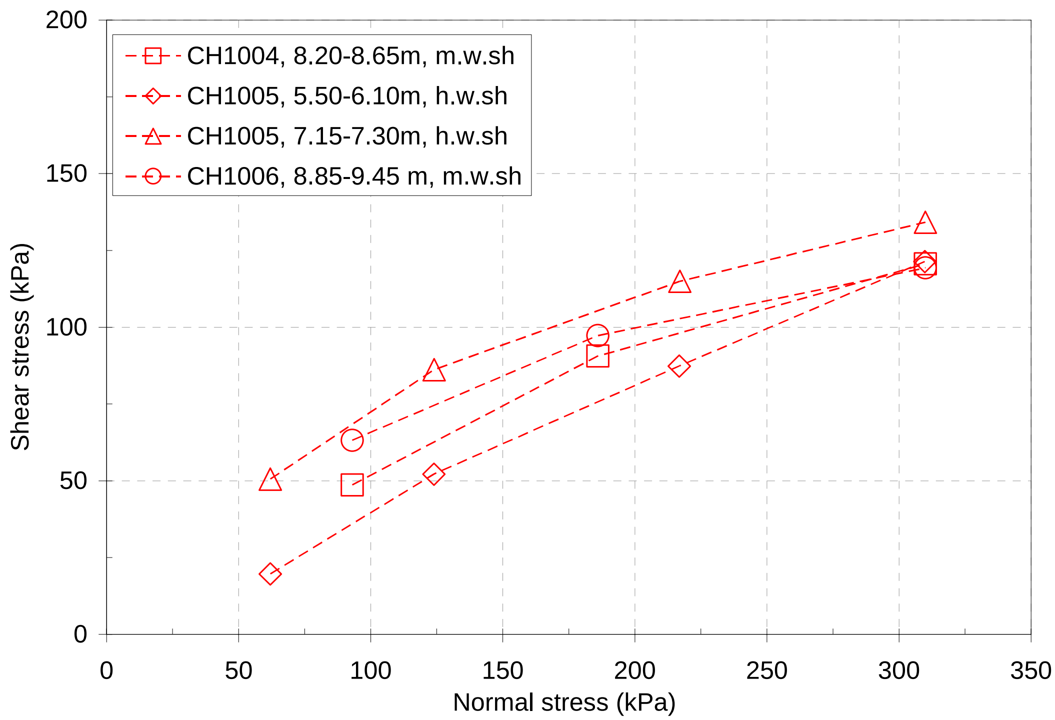

Figure A38. Direct shear test results for slope 15 (part 2 of 4)

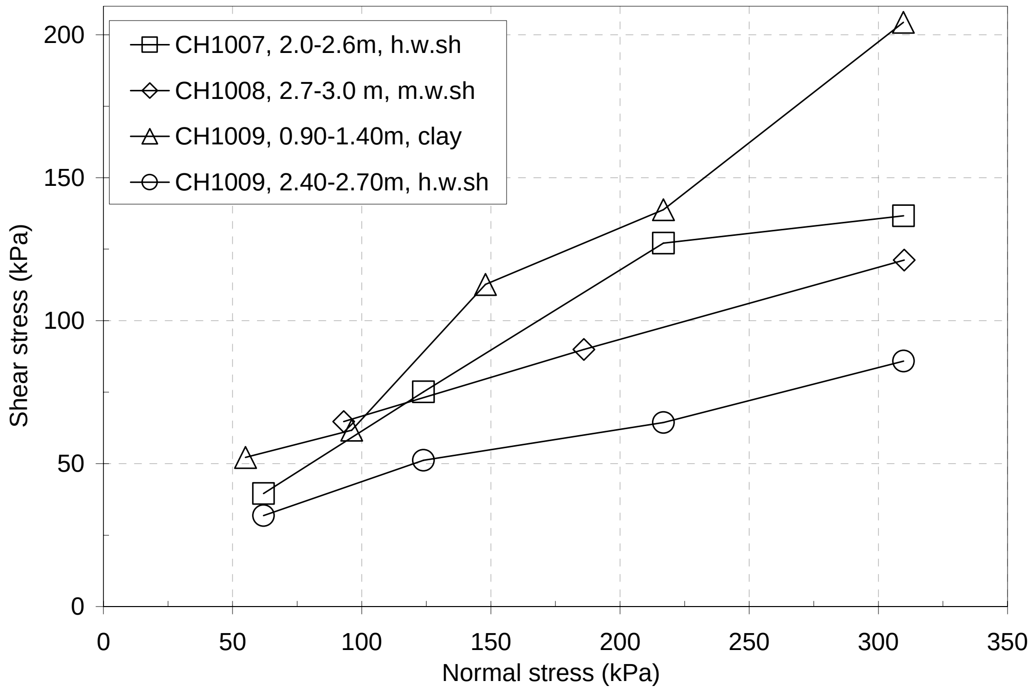

Figure A39. Direct shear test results for slope 15 (part 3 of 4)

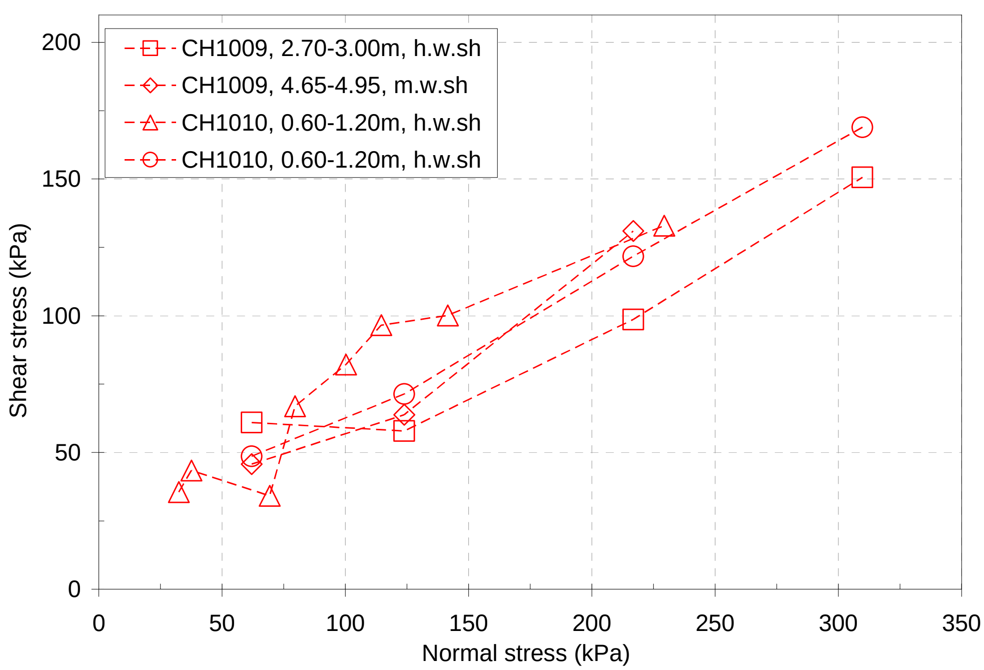

Figure A40. Direct shear test results for slope 15 (part 4 of 4)

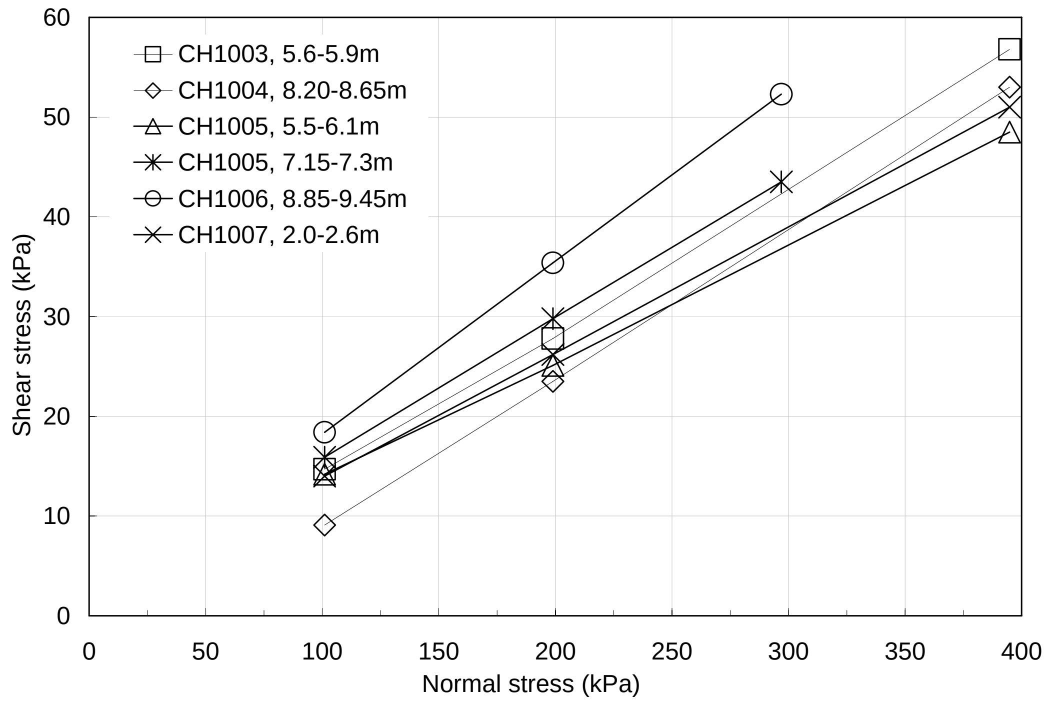

Figure A41. Ring shear test results for the shales at slope 15 (part 1 of 2)

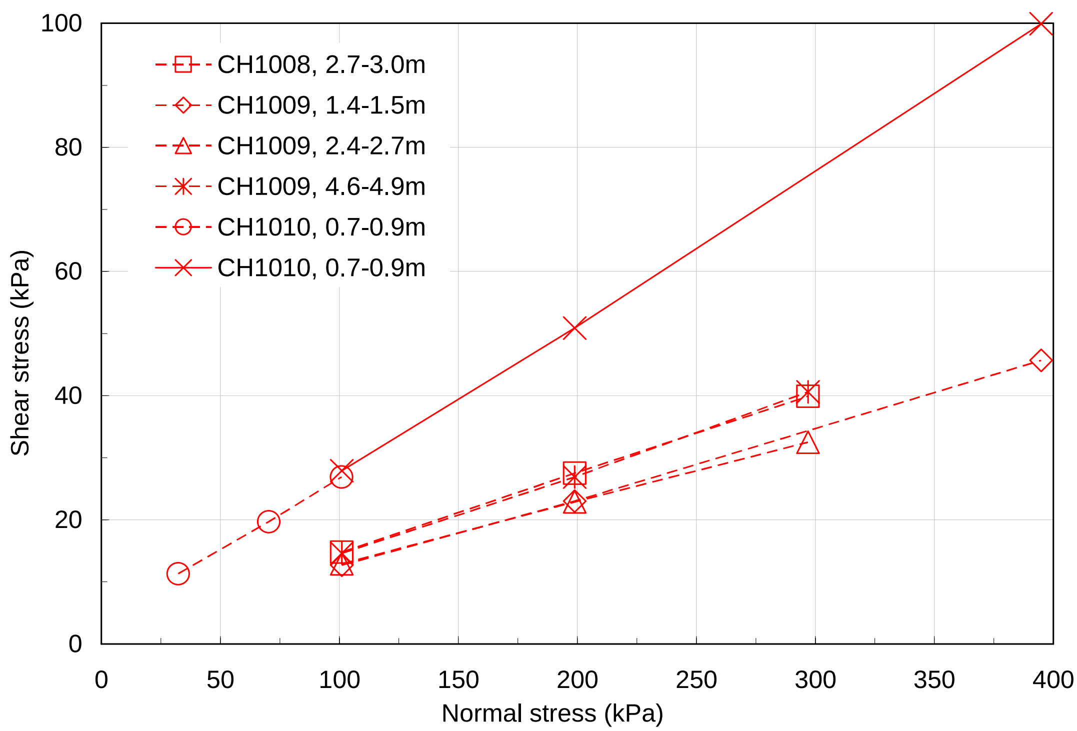

Figure A42. Ring shear test results for the shales at slope 15 (part 2 of 2)

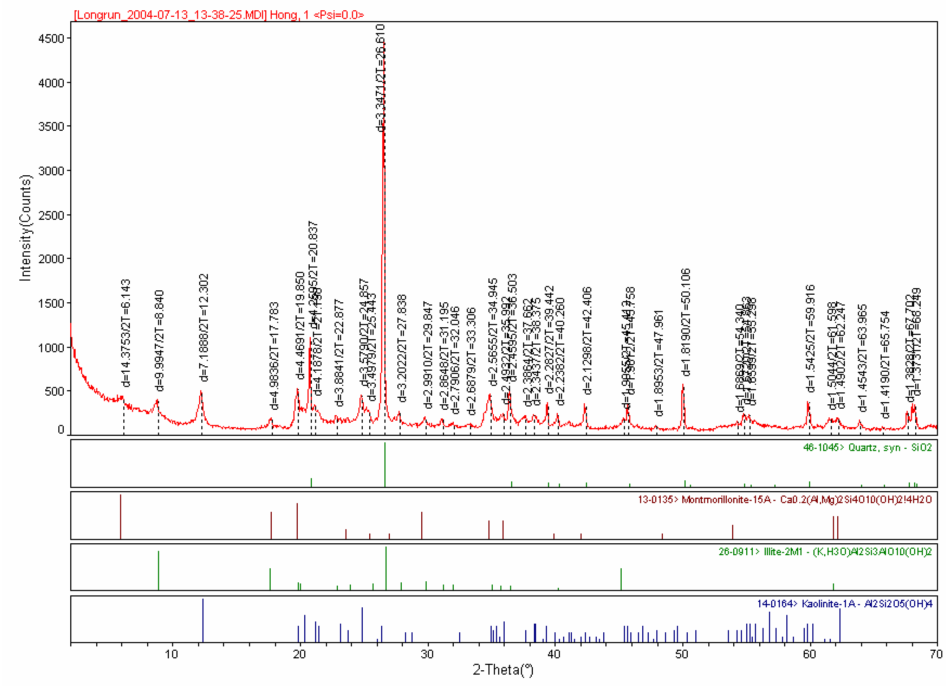

Figure A43. XRD result (1 of 10) (CH1003, 5.6-5.9m, highly weathered shale)

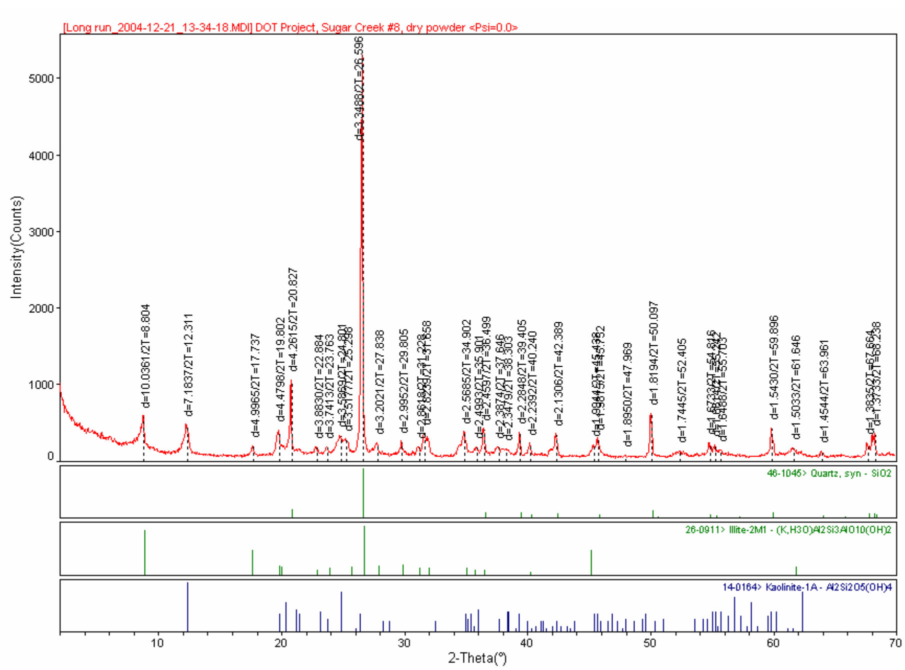

Figure A44. XRD result (2 of 10) (CH1003, 12.75m, slightly weathered shale)

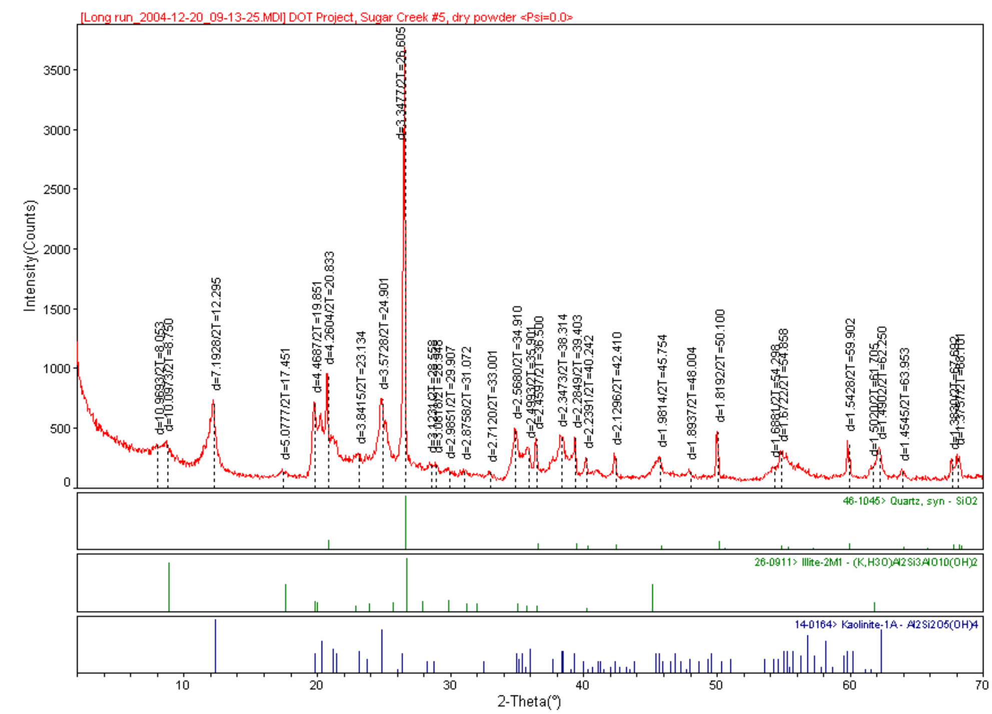

Figure A45. XRD result (3 of 10) (CH1004, 8.2-8.65m, moderately weathered shale)

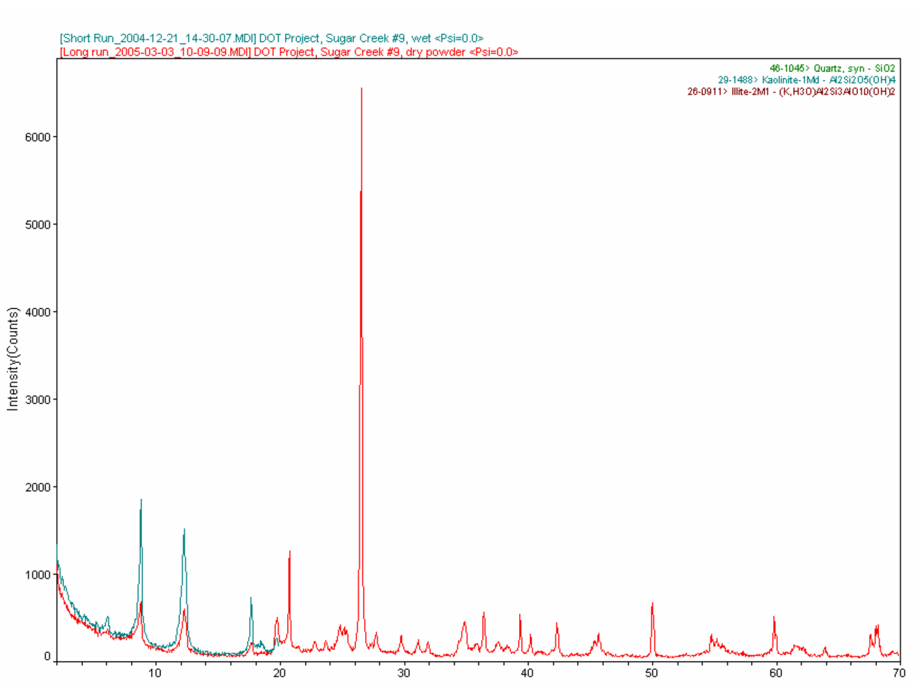

Figure A46. XRD result (4 of 10) (CH1004, 11.22m, slightly weathered shale)

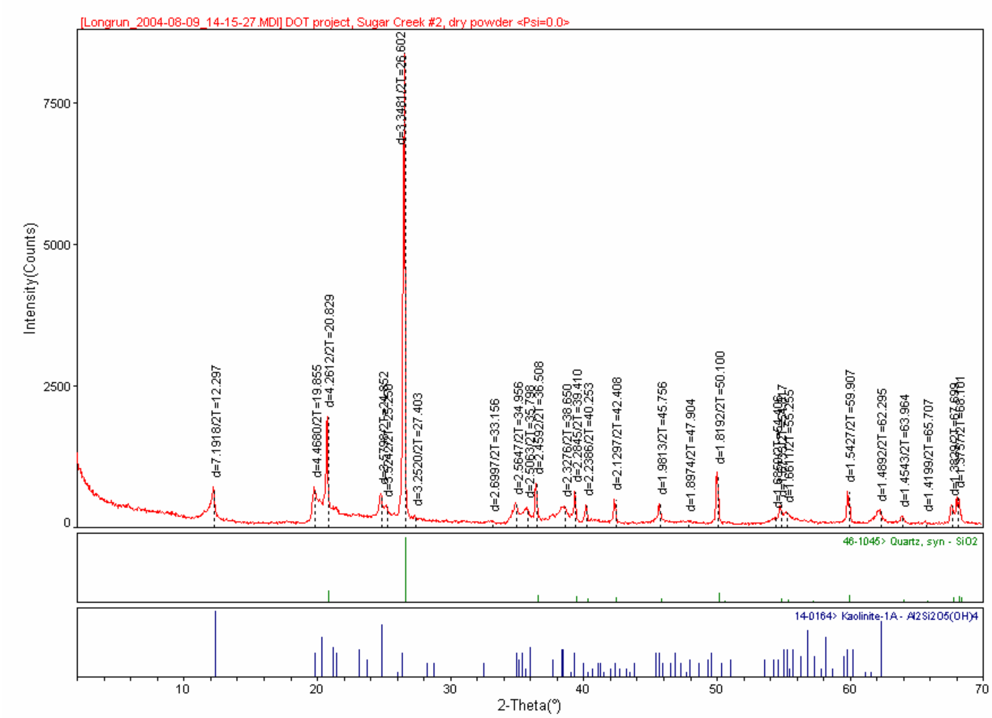

Figure A47. XRD result (5 of 10) (CH1005, 5.5-6.1m, highly weathered shale)

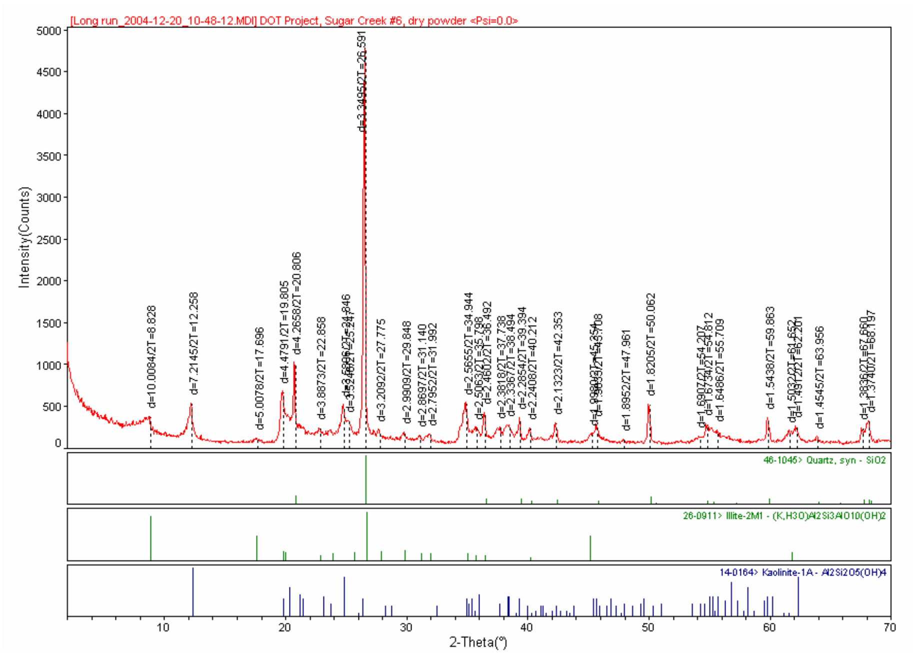

Figure A48. XRD result (6 of 10) (CH1005, 7.15-7.3m, highly weathered shale)

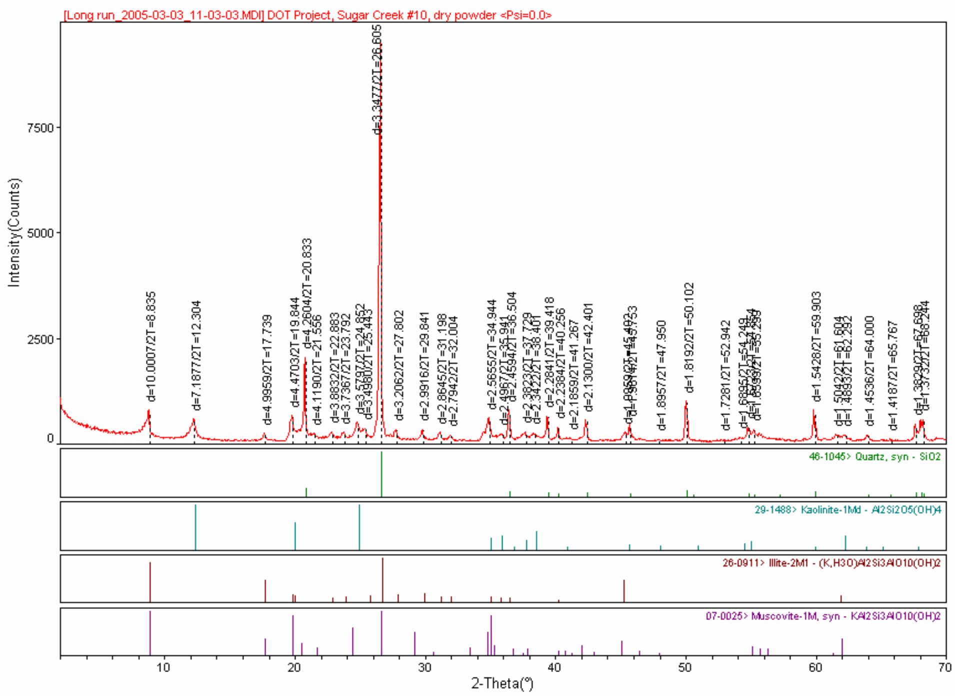

Figure A49. XRD result (7 of 10) (CH1005, 9.2-10.7m, slightly weathered shale)

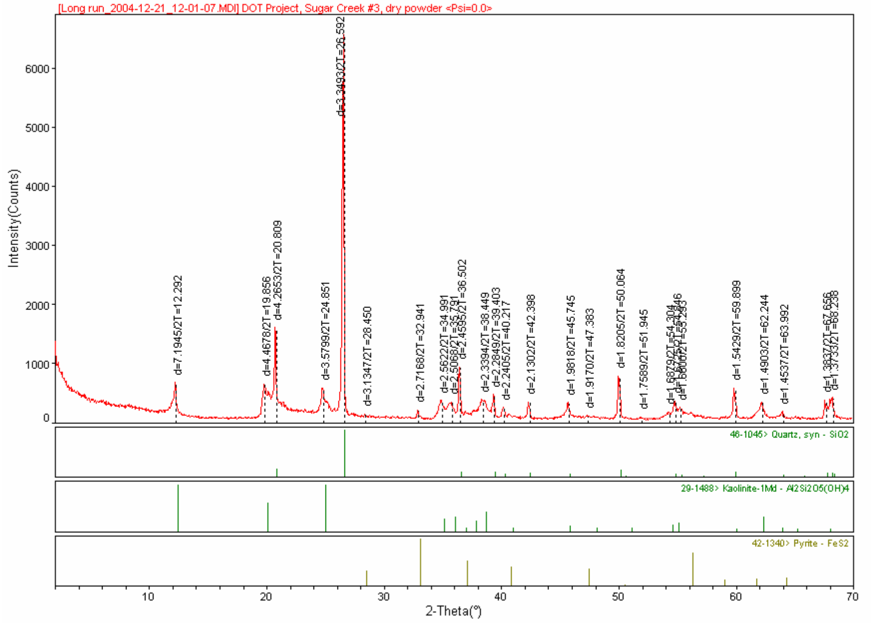

Figure A50. XRD result (8 of 10) (CH1007, 2.0-2.6m, highly weathered shale)

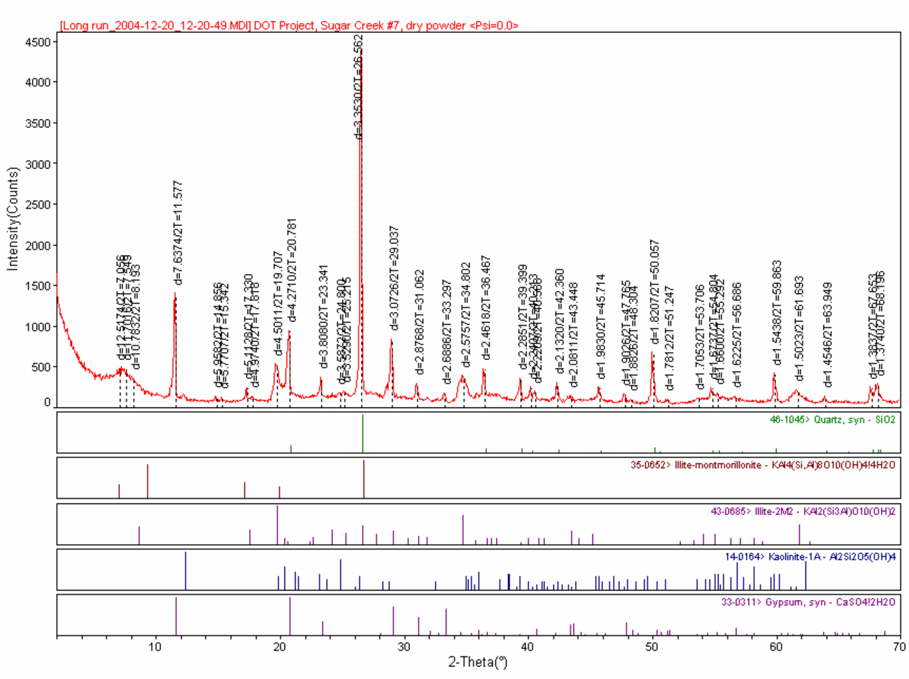

Figure A51. XRD result (9 of 10) (CH1009, 2.4-2.7m, highly weathered shale)

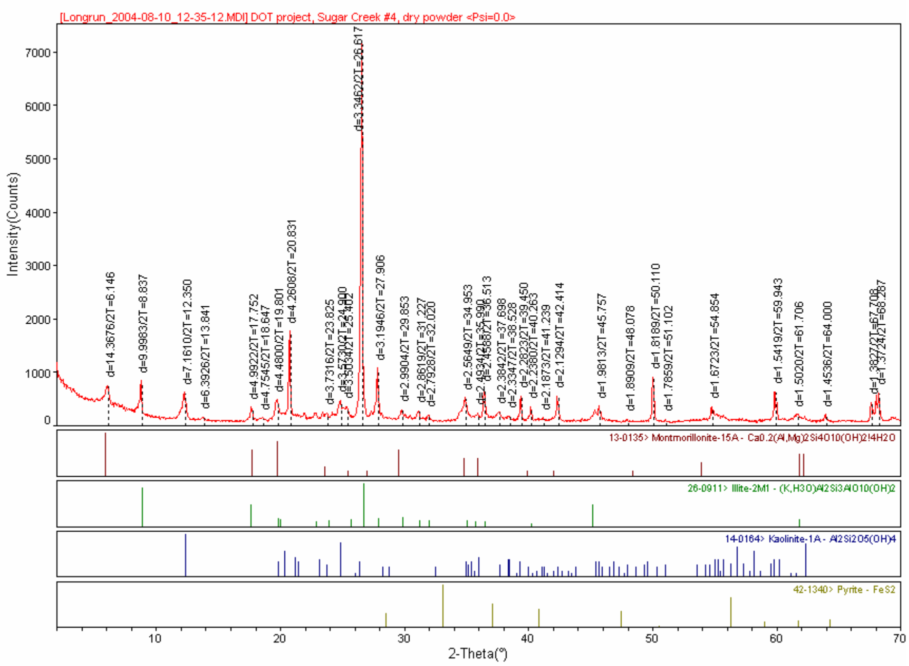

Figure A52. XRD result (10 of 10) (CH1010, 0.6-1.2m, highly weathered shale)