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Soil slope stability investigation and analysis in Iowa

Hong Yang
Iowa State University

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Soil slope stability investigation and analysis in Iowa

by

Hong Yang

A dissertation submitted to the graduate faculty
in partial fulfillment of the requirements for the degree of
DOCTOR OF PHILOSOPHY

Major: Civil Engineering (Geotechnical Engineering)

Program of Study Committee:
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Ames, Iowa

2005

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For the Major Program

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ABSTRACT

This study presents the results of investigations and analyses for two failed slopes and one proposed embankment slope involving clay shales in Iowa. The *in-situ* Borehole Shear Test (BST) was the primary test method for obtaining the soil shear strength parameter values in the investigations, which were supplemented with laboratory ring shear tests for the three slopes and other laboratory tests including direct shear tests for the embankment slope. The limit equilibrium method was used for the slope stability analyses. The major findings in the study include (1) the estimate of the mobilized shear strength parameter values for the slope failures could be improved by considering the BST measurements compared with empirical method of using “good engineering judgment or experience”; (2) geotechnical information including *in-situ* BST measurements was effective in the characterization of the weathered shales with emphasis on the different weathering grades for slope stability analyses; (3) the relatively large amount of *in-situ* shear strength parameter values was particularly useful for probabilistic slope stability analysis, which provided a check and comparison to the probabilistic analyses using conventional shear strength parameter values of indirect field measurements or laboratory measurements.

CHAPTER 1. INTRODUCTION

OVERVIEW

Soil slope instability continues to be a problem in Iowa. Failures occur in both cut slopes and earth embankment slopes. Lohnes and Kjartanson (2002) reported that at least 48 of the 99 counties in Iowa have experienced slope stability problems since 1993. A particular case was the Highway 330 slope failure in Jasper County, Iowa, which developed an approximate 35 m long head scarp (Fig. 1.1). Field borings conducted after the tension cracks 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.

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 AND SCOPE

The objectives and scope of the study are as follows:

(1) Develop and validate appropriate test procedures for quickly determining *in-situ* shear strength parameter values using the BST technique through the investigations of two slope failures in clay shales, and show the significance of the application of the BST in understanding the failure mechanisms;

(2) Classify and characterize the weathered shales associated with potential slope instability for a major embankment slope project using the BST, and demonstrate the usefulness of the BST in shale characterization with respect to different weathering grades;

(3) Illustrate the importance and effectiveness of using the relatively large amount of the *in-situ* shear strength parameter values for slope stability analysis through the probabilistic approach.

DISSERTATION ORGANIZATION

In this dissertation, Chapter 2 provides literature review relevant to the study, which includes (1) Borehole Shear Test; (2) Residual strength and ring shear test; (3) Limit equilibrium slope analysis; and (4) Probabilistic slope stability analysis.

Chapters 3, 4 and 5 comprise three independent, full papers submitted to major technical journals. Each paper appears as a dissertation chapter which includes introduction, references to literature reviewed, results, discussion, conclusions, acknowledgements and a list of references. The first paper presents the results of post-failure investigations of two clay shale slopes using Borehole Shear Tests and ring shear tests, and shows the significance of the application of the BST in understanding the failure mechanisms of slopes. The second paper describes the results of characterization and engineering properties of clay shales for an embankment slope, and demonstrates the usefulness of the BST in shale characterization with emphasis on different weathering grades. The third paper illustrates the importance of probabilistic analysis of embankment slope using the *in-situ* shear strength parameter values measured by the BST and the comparison with the use of the laboratory shear strength parameter values.

Chapter 6 provides general conclusions that summarize the significant research findings from each of the three papers.

The Appendix presents the test data or details that are not included in the dissertation chapters. Finally, the Vita provides a brief sketch about the author of the dissertation.

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Figure 1.1 Existing slope failure at Highway 330 in Jasper County, Iowa
(after White 2003)

CHAPTER 2. LITERATURE REVIEW

BOREHOLE SHEAR TEST

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 effective 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 Figures 2.1 and 2.2. Tests are conducted by expanding diametrically opposed contact shear plates into a borehole under a constant known normal stress, then allowing the soil to consolidate, and finally by pulling vertically and measuring the shear stress. Data points of BST are plotted on a Mohr-Coulomb shear envelope (Figure 2.3) 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, which is demonstrated by pore pressure measurements during test (Lutenegger and Tierney 1986). Complete descriptions of the test procedures for BST can be found in the literature (e.g. Lutenegger 1987).

Currently, two types of shear plates for the BST device are available (Figure 2.2), an ordinary pressure shear plate, used for testing soils with relatively low shear strengths (maximum normal stress of 440 kPa and shear stress of 350 kPa); and a high pressure shear plate, used for testing soils with relatively high shear strengths (maximum normal stress of 2.8 MPa and shear stress of 2.2 MPa) (Handy et al. 1976). The quality of the test may be directly verified by examining the shear plates at the end of the test. A well performed test is

supported by full attachment of soil to the shear plates (Figures 2.2(b) and 2.2(c)) unless the soil is washed away by water in the borehole. Results from the BST are found to be in reasonable agreement with those from laboratory tests such as triaxial test (e.g. Lambrechts and Rixer 1981).

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 moduli. In addition, the BST has also 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).

The Rock Borehole Shear Test (Rock BST) is also a portable direct shear device used to evaluate rock shear strength in-situ. The device was developed by Dr. Handy and his associates at the Iowa State University (Handy et al. 1976). The operation mechanism of the Rock BST is similar to that of the BST, except that the Rock BST is designed to cater for a much higher normal and shear stress. The maximum rock shear strength that may be measured is 45 MPa, and the range of applied normal stress is 0 to 86 MPa (Handy et al. 1976). The Rock BST device consists of three basic parts, i.e. the shear head assembly, the pulling jack, and the console (Figure 2.4). A number of authors have reported the successful uses of Rock BST in measuring the shear strength of rock (e.g. Higgins and Rockaway 1979, 1980; Pitt and Rohde 1984).

RESIDUAL STRENGTH AND RING SHEAR TEST

Skempton (1964, 1985) described the residual strength as the minimum strength of 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 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 clay particle reorientation parallel to the direction of shearing (Lambe and Whitman 1979; Bromhead 1979). While the 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 as 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; Stark and Eid 1994). Attempts to correlate ϕ_r' with soil mineralogical composition were also made by Tiwari and Marui (2005). Residual strength is often related to long-term stability problem and for areas with landslide history, bedding planes or folded strata (Skempton 1985). The drop into residual strength from peak strength may cause reactivation of old landslides.

Residual strength parameters are usually determined using a rotational ring shear test device. Various 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 2.5 and 2.6) has become widely used due to its simplicity in operation compared to other various models. 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. 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 D6467-99 (ASTM 2002) 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.

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 of 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:

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

which can be expressed as

$$F = \frac{c + \sigma \tan \phi}{\tau_{eq}} \quad (2)$$

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 are 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 the Borehole Shear Test, which can produce a large amount of soil shear strength data in a 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.

Limit Equilibrium Slope Analysis

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

(1) Summation of forces in a vertical direction for each slice, where the resulting 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 resulting 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 ;

(4) Summation of forces in a horizontal direction for all slices, giving rise to a force equilibrium factor of safety, F_f .

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) Fellenius (1936) ordinary method of slices; (2) Bishop (1955) 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) complete 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) provide comprehensive review and summary on these computational methods.

Among the ten methods that can be used to determine the factor of safety, the Bishop (1955) simplified method, the Janbu (1968) method and the 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 STABILITY 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, and 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 et al. 1993), which is often referred to as the Gaussian distribution function that is written as:

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

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 of Factors of Safety

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.

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. It is computed by integrating the area under the probability density function for factors of safety less than 1.0. 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. It provides a measure of how much confidence one can have in the computed value of FS and leads to an estimate of the probability of failure. 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):

$$\beta = \frac{|\mu - 1.0|}{\sigma} \quad (4)$$

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.

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).

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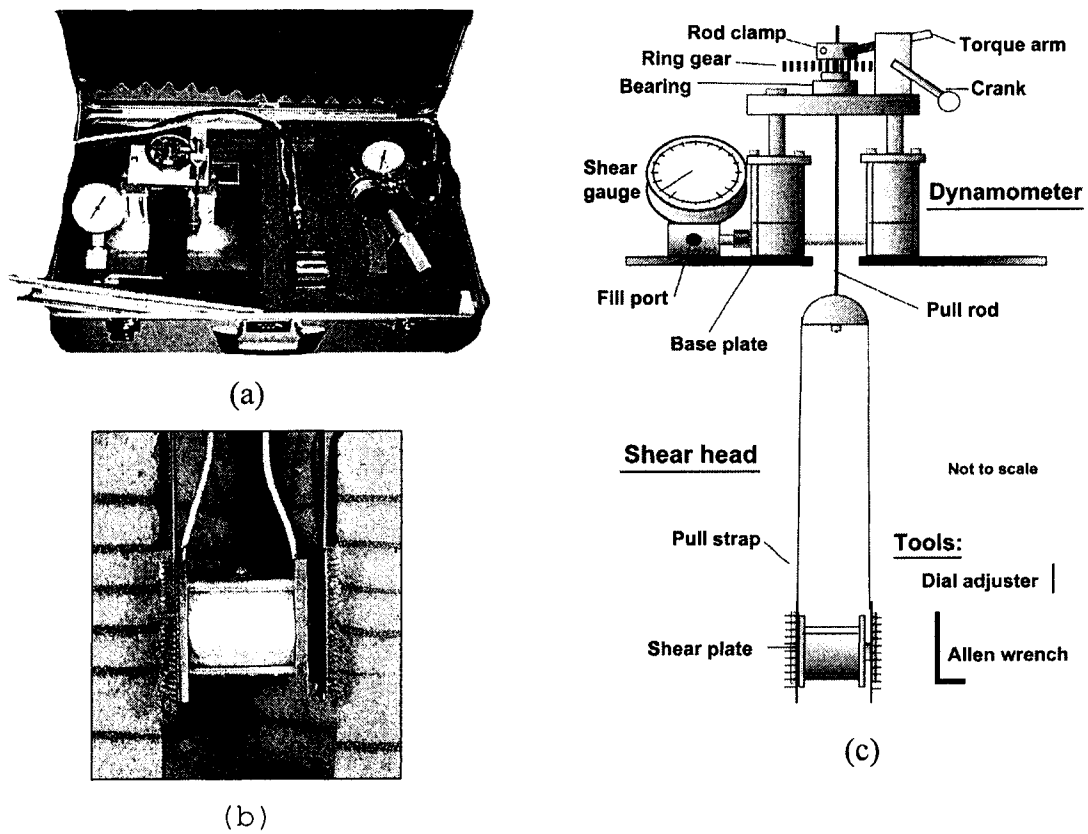


Figure 2. 1 Borehole Shear Test (BST) apparatus

(a) Pressure console and gauges; (b) Shear plates in the cross-section of a borehole; (c) Schematic diagram (Handy 2001)

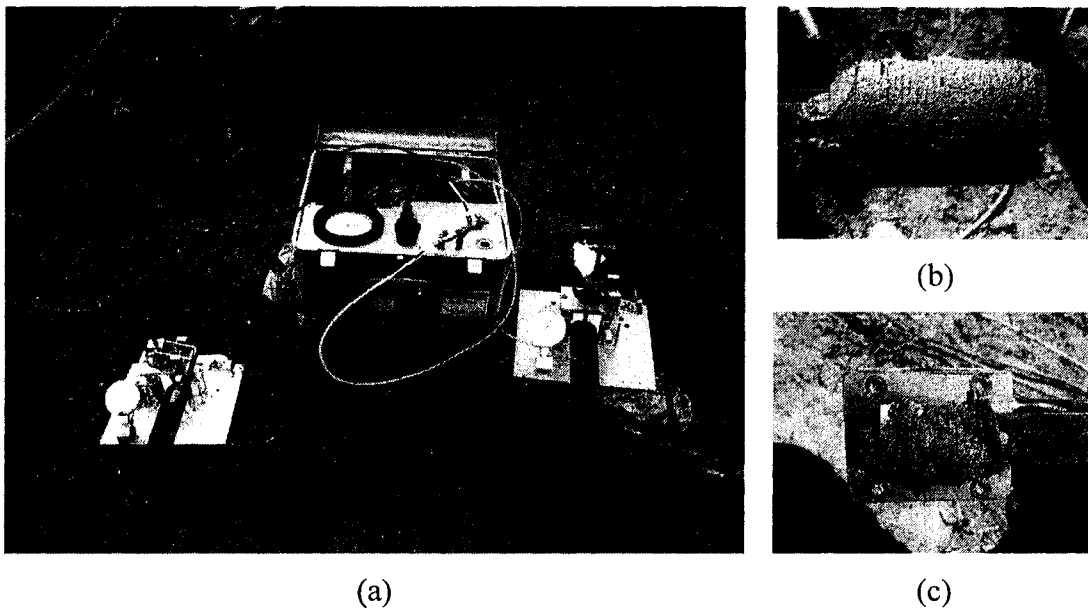


Figure 2. 2 Pictures of BST device

(a) Base plates and pressure console; (b) Ordinary pressure shear plates after shearing; (c) High pressure shear plates after shearing.

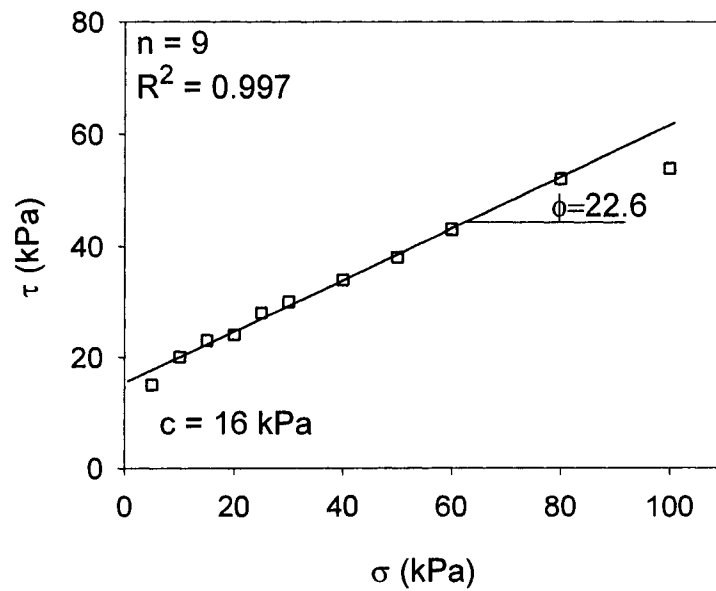


Figure 2. 3 Example of in-situ BST results (Handy 2001)

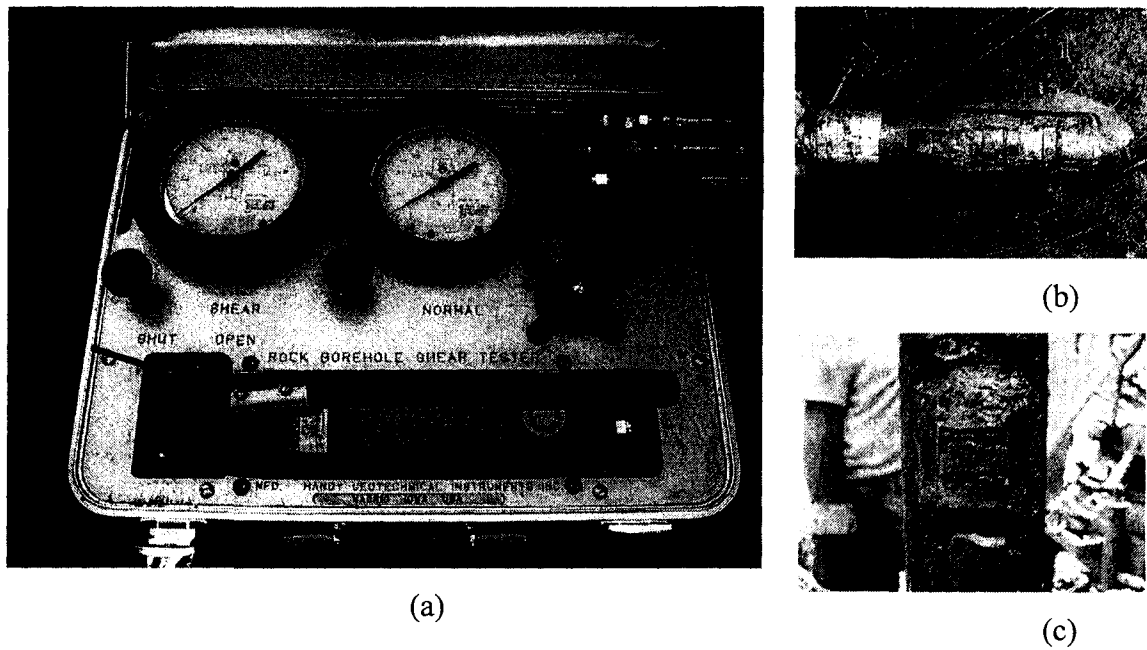


Figure 2. 4 Rock Borehole Shear Test device

(a) Pressure console; (b) Shear plate before shearing; (c) Shear plate after shearing.

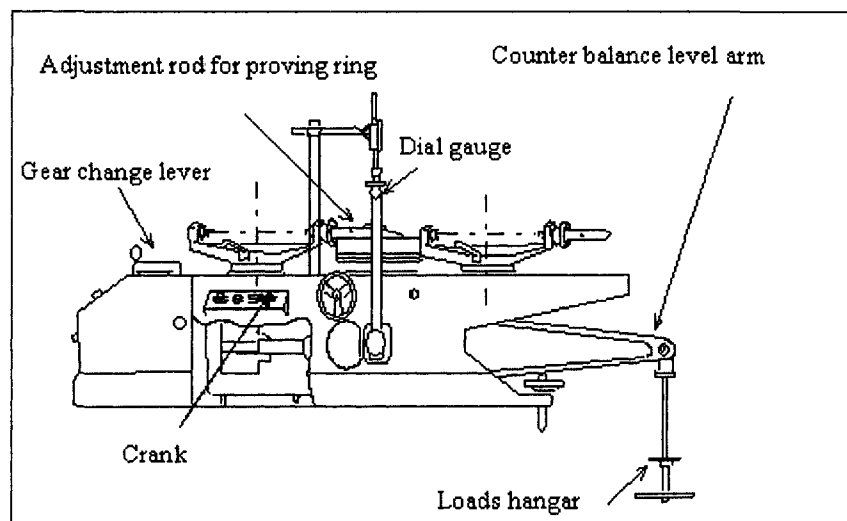


Figure 2. 5 Schematic diagram for the Bromhead (1979) ring shear apparatus

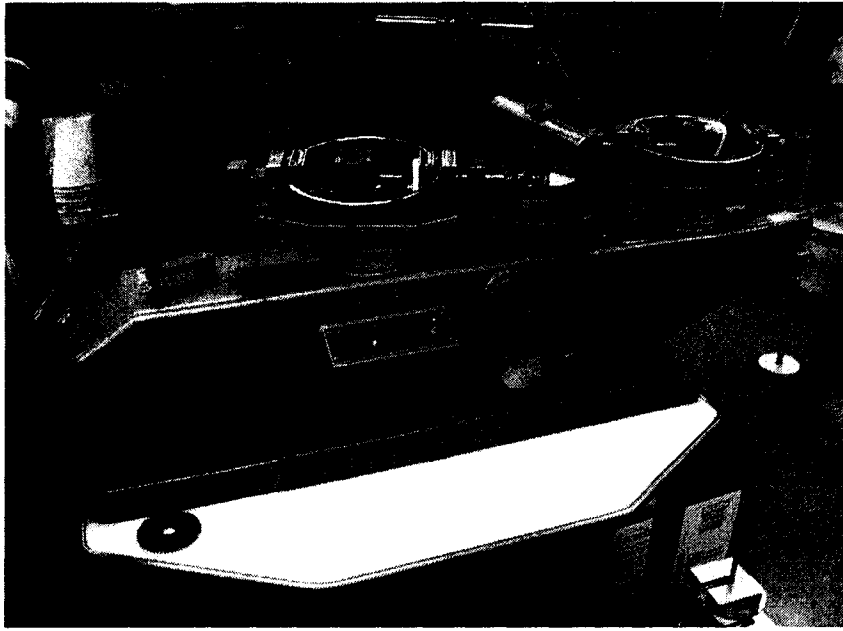


Figure 2. 6 Photograph for the Bromhead (1979) ring shear apparatus

CHAPTER 3. POST-FAILURE INVESTIGATIONS OF TWO CLAY SHALE SLOPES USING BOREHOLE SHEAR TESTS AND RING SHEAR TESTS

A paper submitted to *Canadian Geotechnical Journal*

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ABSTRACT

Investigations were conducted on two first-time slope failures in stiff clay shale with large movement using the *in-situ* Borehole Shear Test (BST) and a laboratory ring shear test. Limit equilibrium slope stability analyses and back calculations were performed on the observed slip surface for each slope. The BST measured the peak shear strength and partially softened shear strength, while the ring shear test measured the residual shear strength of the soils. A range of mobilized shear strengths at failure was obtained from back calculations due to the unknown ground water conditions at failure. The most probable mobilized shear strength at failure was estimated by considering the partially softened and residual shear strengths in the failure zone. The strength changes, or the “strength path”, due to the slope movement, can thus be fully established and used to examine the failure mechanisms of the slopes. The evaluated slope failures are attributed to progressive failures, and were likely triggered by high ground water tables. This paper represents an improvement compared to the empirical method of using “good engineering judgment or experience” to estimate the mobilized shear strength parameter values for slope failures.

Keywords: Borehole Shear Test; Ring shear test; *In-situ* test; Residual strength; Clay shale slope; slope stability.

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INTRODUCTION

Soil slope instability continues to be a problem in engineering practice. Many slope failures are attributed to difficulties in measuring or discovering the major factors that affect slope stability, especially the variations in shear strength parameter values of the soil. An effective means to acquire this information is through a combination of direct measurements using *in-situ* tests and laboratory tests. This paper describes such an approach for two case studies using the *in-situ* Borehole Shear Test (BST) and laboratory ring shear test.

It has been recognized that the shear strength of stiff clay in a slope can decrease from the peak value (prior to the slope movement) to the fully softened value (normally before or during the slope failure), and finally to the residual value (after a relatively large slope movement) (e.g., Skempton 1964; Bishop 1967). The peak strength and softened strength are determined by field tests either directly (e.g., field direct shear box test) or indirectly (e.g., pressuremeter tests or penetration tests); or by laboratory tests directly (such as direct shear test and triaxial compression test). Other methods for direct measurement of shear strength in slope include the *in-situ* Borehole Shear Test (BST) (Handy 1986), though its applications are not widely documented. The residual strength of the soil is usually determined by a laboratory ring shear test (e.g., Bromhead 1979).

The mobilized shear strength at the time of slope failure is normally estimated by back calculations. As unique values of the shear strength parameters, i.e. soil cohesion (c') and friction angle (ϕ'), cannot be determined by back calculation due to the single piece of information provided by the slope failure (i.e., factor of safety equals to unity), Duncan and Stark (1992) suggested that the best procedure is to assume the value of ϕ' using “good engineering judgment and experience”, then calculate the value of c' at slope failure. Skempton (1964 and 1985) recommended that fully softened shear strength is appropriate for first-time sliding; while Mesri and Shahien (2003) concluded that the lower bound for mobilized shear strength is the fully softened shear strength in first-time slope failures in homogeneous soft to stiff clays. The estimations of the mobilized shear strength parameter values can be difficult as the ground water conditions at failure are usually unknown.

In view of the difficulties and complications in obtaining the shear strengths, this study attempts to adopt an alternative approach for the complete evaluation of the shear

strength changes, with emphasis on the estimation of the mobilized shear strength at failure. The approach involves the use of the *in-situ* BST and the laboratory ring shear test, and is demonstrated by investigation of two first-time slope failures in stiff clay shale of large movement. The procedures involved in the study can be briefly illustrated in Figure 3.1. Basically, a preliminary field investigation (slope geometry, failure features, etc.) is conducted, followed by the *in-situ* BST. Based on the obtained information, the failure mode and slip surface of the slope can be identified. Peak strength and partially softened strength are measured from BST; and residual strength is determined from a ring shear test as conventionally. However, back-calculated, mobilized shear strength parameter values can be best estimated with the knowledge of the partially softened shear strength and residual shear strength. Through the proposed approach and procedures, the entire developments of the soil strengths, or the “strength path” as proposed in this study, can be fully established; and the failure mechanism of the slopes, which have been attributed to progressive failure for the two slopes, is also examined.

BACKGROUND INFORMATION

Borehole Shear Test

Among the various test equipment and apparatus to obtain the shear strength parameter values, the Borehole Shear Test (BST) is unique in that it gives a rapid, direct and accurate *in-situ* measurement of the shear strength parameters. The fundamental concept of the BST involves performing a series of direct shear tests on the inside of a borehole (Handy and Fox 1967; Wineland 1975) as shown in Figure 3.2. Tests are conducted by expanding diametrically opposed contact shear plates into a borehole under a constant known normal stress, then allowing the soil to consolidate, and finally by pulling vertically and measuring the shear stress. Points are plotted on a Mohr-Coulomb shear envelope by measuring the maximum shear resistance at successively higher increments of applied normal stresses. Depending on soil type, total testing time for a typical test with four to five data points is approximately 30 to 60 minutes (Lutenegger and Hallberg 1981). The BST is normally a

consolidated-drained test, which is demonstrated by pore pressure measurements during 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; Demartincourt and Bauer 1983); hard clays (Handy et al. 1985) and stiff soil (Lutenegger et al. 1978); and unsaturated soils (Miller et al. 1998). It was also used to study the preconsolidation pressures and soil moduli (White and Handy 2001). In addition, the BST has also been used to study a few landslide case histories (e.g., Tice and Sams 1974; Handy 1986). These 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) indicative of a partially softened to residual shear strength.

Residual Strength and Ring Shear Test

Skempton (1964, 1985) described 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 condition during shearing. The shear strength of the soil can drop from the 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 achievement of the residual strength results in the formation of a new fabric. The drop in strength is attributed to clay particle reorientation parallel to the direction of shearing (Lambe and Whitman 1979). While the cohesion provides much of the peak shear strength, the soil often has little cohesion once a shear surface is formed (Skempton 1964). Residual strength is frequently related to long-term stability problem and areas with landslide history, bedding planes or folded strata (Skempton 1985). The drop into residual strength from peak strength may also cause reactivation of relict landslides.

Residual strength parameters are often determined using a rotational ring shear test device. Different 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 has become widely used due to its simplicity in operation compared to the other models. 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. In this study, the test procedures (multistage test procedures) described in ASTM D6467-99 (ASTM 2002a) were adopted to determine the residual shear strengths of the soils. The soil specimen is pre-sheared at a relatively large displacement rate (0.89 mm/min) for one revolution (a displacement of 267 mm), followed by subsequent shearing under a smaller displacement rate (0.036 mm/min) under several different normal stresses. The plot of shear stress versus normal stress gives the Mohr-Coulomb failure envelope and the residual shear strength parameter values.

Slope Stability Analysis and Back Calculation

The calculation of global slope stability is normally expressed in terms of the factor of safety computed by means of limit equilibrium methods. Many methods are available to determine the factor of safety. In this study, the Morgenstern and Price (1965) method was adopted due to the non-circular nature of the failure planes. The method satisfies all conditions of equilibrium, and is applicable to any shape of slip surface. The computations of the slope analysis were performed using the computer program Slope/W (GEO-SLOPE 2004).

When a slope failure occurs, the shear strength of the soil is mobilized along the full length of the slip surface. The mobilized strength can be estimated by performing a back calculation. Knowing that the factor of safety is equal to unity, analyses are performed to determine what the mobilized soil shear strength must have been for the failure to have occurred. Duncan and Stark (1992) noted that back calculation has been found to be effective where conditions are simple, and attractive for determining soil strengths, because it avoids many of the problems associated with laboratory and *in-situ* tests. The principle limitation of back calculation is the fact that the stress conditions at the time of failure are not precisely known. Also, as a slope failure provides a single piece of data, assumptions are inevitably required to determine values of both c' and ϕ' , and these values are not unique.

CASE HISTORY I - ALBIA SLOPE

Site Description and Characterization

The Albia slope is located along Highway 34 MP171.7, three miles west of Albia, Monroe County, Iowa. In this area, shale residuum is the parent material. The shale consists of a series of beds deposited during the Des Moines sedimentary cycle in the Pennsylvanian period (286 to 320 million years ago) (Eicher et al. 1984). These beds include shale of different colors and textures, conglomerates, and a few organic coal layers (USDA 1984).

The exact time of the slope failure is unknown. Failure or deformation may have occurred prior to 2001 based on the accounts of a nearby resident. The failure features of scarp head and bulges of the slide (Figure 3.3) appeared to be at least a few years old based on vegetation growth when the slope was investigated in July 2004. The representative profile for the slope is showed in Figure 3.4. It shows that the original slope had an overall sloping angle of about 11 degrees down-dipping towards Highway 34, a maximum length of 40 m and a maximum height of 8 m. 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 (Figure 3.3). The width of the slope at the toe was about 70 m (along the highway). There were a few bulges at the surface of the slope.

A total of four boreholes were drilled manually along the slope profile at the locations shown in Figure 3.4. The boreholes showed that the slope was composed of brown to grey, highly weathered clay shales which were generally medium stiff to stiff. A thin layer of weaker soils appeared to exist at the lower portion of BH2, BH3 and BH4 where the borings were relatively easily advanced. All the boreholes were terminated when the borings reached very stiff, slightly weathered shale and could not be further preceded due to the limitation of the manual operation. Ground water table in boreholes BH2, BH3 and BH4 was observed and measured 24 hours after the boring. Ground water was not present in BH1.

Particle-size distribution analyses on three shale samples obtained from the lower portion of BH2 and the mid portion of BH4 indicated that the soils are composed of 49% clay, 48% silt and 3% sand on average. The soils were found to have natural moisture content

of 13 to 34%; and liquid limit of 61% and plasticity index of 35% on average. The total unit weight of the shale varied from 18.0 to 19.1 kN/m³. All samples are classified as high plasticity clay (CH) according to the Unified Soil Classification System (ASTM 2002b).

Borehole Shear Test Results

A total of nine BSTs were performed mainly in the lower portion of the boreholes (Figure 3.4), and the results are presented in Figure 3.5. The BST results appeared reliable as indicated by the values of coefficient of correlation (R^2), which were generally larger than 0.99. The results show that the effective internal friction angle (ϕ') for the shales ranged from 11° to 40°, and the effective cohesion intercept (c') varied from 5 to 22 kPa. The nine BST results had average ϕ' value of 23.5° and c' value of 12.4 kPa with coefficient of variation of 0.36 and 0.49, respectively.

It is noteworthy that the BST at BH2 (3.2m) gave a shear strength envelope that is significantly lower than BSTs at higher elevations in the same boreholes (i.e., BSTs at BH2 (2.0m) and BH2 (2.6m)) (Figure 3.5). The BST at BH3 (2.6m) and BST at BH4 (1.1m) also gave considerably lower strength envelopes as compared with other BSTs, especially those for BH1. Coincidentally, the elevations of these three BSTs with relatively low strength envelopes were the same as the elevations where borings were relatively easily advanced during the manual drilling. These observations suggest that there could exist a relatively thin, weak zone of soil along the elevations of BH2 (3.2m), BH3 (2.6m) and BH4 (1.1m); and the slope could have slipped along this weak zone as indicated by the slip surface in the slope profile (Figure 3.4). The slip surface passed through the observed scarp and the toe of the slope, and was supported by the main features of the slope failure, i.e. the depression of the surface between BH1 and BH2 and the bulge at BH3 and BH4 (Figure 3.4).

Ring Shear Test Results

A total of four ring shear tests (RST) were conducted on reconstituted shale samples and the results are presented in Figure 3.6. All the results had $R^2 > 0.998$, and the residual friction angle (ϕ_r') ranges from 5.9° to 6.8° with a small residual cohesion intercept (c_r'). The

residual shear strength parameter values are relatively low, which is consistent with the relatively high average liquid limit of 61% and average plasticity index of 35% of the shales (e.g., Mitchell 1993).

Results of Slope Stability Analyses

In order to investigate the possible conditions at pre-failure and end of failure of the slope, slope stability analyses and back calculations were performed considering different situations of slope geometry, ground water table location and shear strengths of the soil. As the geometry of the slope was significantly different between the pre-failure and end of failure conditions, and the location of the ground water table at pre-failure (or during the failure) of the slope was unknown, three situations (Table 3.1) were considered for slope analyses - (1) original ground surface assuming water table located on the original slope surface; (2) current ground surface assuming water table located on the current slope surface; and (3) current ground surface using current water table (observed during the slope investigation, see Figure 3.4). Assuming the water table is located on the slope surface represents the worst ground water conditions with respect to stability calculations.

The shear strength parameters considered in the slope analyses included (A) the average shear strength parameter values measured by BST; (B) the lowest set of shear strength parameter values measured by BST; (C) back-calculated shear strength parameter values giving unity factor of safety based on conditions (1) (Table 3.1); and (D) shear strength parameter values measured by ring shear test (one of the four results were used since they are essentially the same, Figure 3.6). Factors of safety using the Morgenstern-Price (1965) method were calculated for the observed slip surface. As a simplifying assumption, the soil unit weight and shear strength parameter values were assumed constant through the soil profile for each analysis.

The results of the slope stability analyses are summarized in Table 3.1. The results show that, the factors of safety for the original slope were much larger than 1.0 (2.52 and 2.02) when using the average and lowest shear strength parameter values as measured by BST, even under the worst ground water conditions. These results contradicted the fact that the slope has failed and had a factor of safety of 1.0 at failure. Therefore, the shear strength

that was developed at failure must have been lower than these BST measurements. The back-calculated shear strength parameter values ($\phi' = 11.0^\circ$ and $c' = 4.5$ kPa) gave factor of safety of 1.0, and indicated the possible shear strength that has been at the slope failure under the most unfavorable ground water conditions. Since a large displacement have occurred after the slope failure as indicated by the slope geometry (Figures 3.3 and 3.4), the soil should have reached the residual large strain state. At this case, the slope has a factor of safety of 1.12 under the current ground water table conditions. This indicates the current slope is stable, which agrees with the fact that it is standing. However, use of the residual strength under conditions (1) and (2) resulted in factors of safety much less than 1.0 (0.48 and 0.66) (Table 3.1), indicating that (a) shear strength must have been higher than the residual strength before or during the slope failure, as under conditions (1), i.e. shear strength had not dropped to residual value before or during the slope failure; or (b) the current slope will reactivate if the ground water table starts to rise before reaching the current slope surface, as under conditions (2).

A series of back calculations were further performed to estimate various shear strength parameter values of the soil assuming various ground water table conditions, and the ϕ' and c' values required to give factor of safety of 1.0 were obtained. The results are presented in Figure 3.7. It shows that the required ϕ' and c' values become smaller if the location of the ground water table becomes lower in the slope, as expected. For the slope failure under the most unfavorable ground water conditions, the c' value was estimated to be less than 4.5 kPa (point C1) if the ϕ' value of 11.0° , the lowest ϕ' value as measured by BST, is used.

The changes of shear strength are also presented in terms of a Mohr-Coulomb failure envelope in Figure 3.8. It shows that drop of the shear strength after the slope failure is significant. From these changes, the brittleness of the soil, characterized by the brittleness index, can be evaluated. The brittleness index was defined by Bishop (1976) as the difference between the peak and residual strengths over the peak strength. With an estimated normal stress of 50 kPa, the peak and residual shear strengths are 34 and 7.5 kPa, respectively, giving a brittleness index of 0.78.

CASE HISTORY II - WINTERSET SLOPE

Site Description and Characterization

The slope is located at the west side of Highway 169, about three miles north of Winterset, Madison County, Iowa. According to the USDA (1975), the soils in this area mainly formed in loess, glacial till and shales. The oldest parent material is a series of beds deposited during a sedimentary cycle in the Pennsylvanian period. The beds consist of limestone, shale of different colors and textures, and sandstones.

The slope started to move in 2003 and failed in 2004 (Kretlow 2004). The scarp of the slide appeared to be new when the slide was investigated in June 2004 as shown in Figure 3.9. A representative slope profile is shown in Figure 3.10. The slope had an overall sloping angle of about 13 degrees up-dipping towards the highway, a maximum length of 33 m and a maximum height of 7 m. There was a nearly straight, steep scarp near the top of the slope with a maximum height of 1.7 m. The scarp extended along the side of highway for about 70 m. Transverse cracks were present at the mid height of the slope.

A total of four boreholes were drilled during the field investigation along the representative slope profile (Figure 3.10). Borehole BH1 was drilled with a rotary drilling rig; and boreholes BH2, BH3 and BH4 were drilled manually with a hand auger. The borings indicated that the slope was mainly composed of weathered clay shales of multiple colors of brown, grey and reddish. The soils were generally medium stiff to stiff. Slightly weathered shale with traces of limestone was encountered at the lower portion of BH1. BH2, BH3 and BH4 were terminated when the borings reached relatively strong soil and could not be further advanced with manual augering. The soils were relatively easy to drill manually before the strong zone was reached in BH2 and BH4. The ground water table in the four boreholes was measured 24 to 48 hours after the boring. The wet surface near the toe of the slope indicated near surface ground water at the toe of the slope.

Particle-size distribution analyses on shale samples from lower portion of BH2 and BH3 indicated the soils were composed of 36% clay, 62% silt and 2% sand on average. Values of natural moisture content for the soils obtained from the boreholes were found to lie in the range of 16 to 31%, with a liquid limit of 55% and a plasticity index of 31% on

average. The total unit weight of the shale varied from 18.3 to 19.7 kN/m³. All the shales were classified as high plasticity clay (CH) according to the Unified Soil Classification System (ASTM 2002b, D2487-00).

Borehole Shear Test Results

A total of nine BSTs were performed for the Winterset slope and the results are presented in Figure 3.11. The results generally had an $R^2 > 0.98$; and gave ϕ' values of 18° to 35° and c' values of 11 to 45 kPa. The nine BST results also had average ϕ' value of 24.4° and average c' value of 24.1 kPa with coefficient of variation of 0.27 and 0.46, respectively.

BSTs at BH2 (2.7m) and BH4 (1.7m) gave shear strength envelopes that were much lower than BSTs in other locations of the slope (Figures 3.10 and 3.11). This strongly suggests that the slip surface passed through the zone near BH2 (2.7m) and BH4 (1.7m), as indicated in the slope profile (Figure 3.10). The slip surface was consistent with the experience of the manual boring, and was supported by the topographic features of the slope including the graben at BH2 and the surface crack between BH3 and BH4.

Ring Shear Test Results

A total of four ring shear tests were conducted on reconstituted shale samples, and the results are presented in Figure 3.12. The results, having $R^2 > 0.998$, indicate that the residual frictional angles of the shales (ϕ_r') vary from 12.0 to 16.3° with a small residual cohesion intercept (c_r') ranging from 1.9 to 3.5 kPa.

The shales in the Winterset slope have higher residual friction angles than those in the Albia slope (Figure 3.6), while they have lower clay content and lower values of liquid limit and plasticity index than the shales in Albia slope. These observations are consistent with those reported in the literature, i.e. lower clay content and lower values of liquid limit and plasticity index generally correspond to higher residual friction angle (e.g., Mitchell 1993).

Results of Slope Stability Analyses

Slope stability analyses were performed for the Winterset slope in a fashion similar to that for the Albia slope, and the results are summarized in Table 3.2. The factors of safety were much larger than 1.0 (3.42 and 2.32) for the original slope when using the average and lowest shear strength parameter values as measured by BST under the worst ground water table conditions. Since the slope had a factor of safety of 1.0 at the time of failure, the shear strength that was developed at failure should have been lower in the failure zone than determined from the BST measurements. One set of the back-calculated shear strength parameter values yields $\phi' = 18.0^\circ$ and $c' = 3.2$ kPa for $FS = 1.0$. The current slope likely reached a residual state due to the large movement after the slope failure (Figures 3.9 and 3.10). Thus the use of the residual shear strength is applicable. In this case, the slope has a calculated factor of safety of 1.22 under the field observed ground water table conditions. This agrees with the fact the current slope is safe and standing. On the other hand, use of the residual strength under conditions (1) and (2) resulted in FS value less than 1.0 (0.80 and 0.96) (Table 3.2), indicating that the shear strength must have been higher than the residual strength during the slope failure if the water table was located on the slope surface (conditions (1)); or the current slope will reactivate and fail if the water table rises towards the current slope surface (conditions (2)).

Various shear strength parameter values of the soil were also obtained by back analyses assuming different ground water table conditions. The ϕ' and c' values required to give factor of safety of 1.0 are presented in Figure 3.13. It shows that the required c' values at the time of the slope failure was estimated to be less than 3.2 kPa (point C) with water table located on the slope surface, if the ϕ' value of 18.0° , the lowest value of BST measurements, was used. The different shear strengths of the soil as used in the slope stability analyses are also presented in the form of strength envelopes in Figure 3.14. Similarly, the change of the shear strength was considerable before and after the slope failure.

DISCUSSION

Strength Changes of the Soil

In the previous stability analyses for the two slopes, several different shear strengths have been involved, which were obtained from BSTs, ring shear test and back-calculation. These shear strengths reflect the strength changes associated with the slope movement. The average strength values measured by BST (after averaging out the soil variation) can represent the peak strength, since the soils tested were essentially intact. The lowest strength values measured by BST can represent the partially softened strength, because the values are measured within or close to the failure zone where soils may have undergone some small displacement, as discussed previously. Partially softened strength was used to differentiate from the fully softened strength. The ring shear test measured the residual or the ultimate strength of the soil.

The most uncertain part of the shear strength is the mobilized strength at the slope failure and its relationship with the fully softened shear strength. This issue has been extensively studied and reported by numerous researchers since 1960s (e.g., Skempton 1964; Bjerrum 1967). For the first-time slope failures in stiff shales as in this study, the fully softened shear strength is the lower bound for mobilized shear strength, as concluded by Mesri and Shahien (2003) after having reanalyzed 99 case histories of slope failures. In this study, a wide range of the possible mobilized shear strength was obtained from back-calculations (Figures 3.7 and 3.13) due to lack of the knowledge of the ground water table conditions at failure. However, the range was greatly narrowed by the knowledge of the partially softened strength and the residual strength, as the magnitude of the mobilized shear strength must be limited between the partially softened strength and the residual strength. In addition, it has been generally recognized that there is a large drop-off of cohesion from the peak strength to the fully softened strength, and the cohesion for the fully softened strength is small or close to zero (Skempton 1964). Thus, the mobilized (fully softened) strength most probably has the ϕ' value similar to the partially softened strength with cohesion near the residual strength, as indicated by points C1, C2 or C in Figures 3.7 and 3.13. The mobilized strength further decreases to the residual strength after a large displacement. Hence, the

entire change of the shear strength is established for the soil in the slope as indicated by the strength paths in Figures 3.7 and 3.13 and the failure envelopes in Figures 3.8 and 3.14. In addition, the approach to estimate the mobilized shear strength could be considered as an improvement as opposed to use “engineering judgment or experience” (Duncan and Stark 1992).

Corresponding to the most probable mobilized shear strength as shown in Figures 3.7 and 3.13, the most probable ground water table at the time of the slope failure can also be estimated. The ground water table at failure was estimated to be located within the elevation of 0 to -6.0 m for the Albia slope, and -1.5 m to -4.0 m for the Winterset slope. Apparently, these water tables are much higher than the measured water table (Figures 3.4 and 3.10). This suggests that high ground water table could have triggered the slope failure.

Possible Failure Mechanisms of the Slopes

The results of the slope stability analyses show that the original slopes would be stable when the shear strength parameter values obtained from BST are used under the worst ground water conditions (Tables 3.1 and 3.2). However, the calculated results contradict the fact that slope failures occurred. The reason for this contradiction could be attributed to progressive failure, which has been widely reported for stiff clays and shales in the literature.

Progressive failure refers to the non-uniform mobilization of shear strength along a potential slip surface, and its mechanisms became better understood in the context of overconsolidated clays and clay shales in the 1960s (e.g., Skempton 1964; Bjerrum 1967; Bishop 1967). Basically, if the shear stress exceeds the available strength in a small zone along the slip surface of a slope, the excess loading will have to be transferred to adjacent zones. However, if the soils exhibit brittle or strain-softening behavior, the stress transfer is likely to lead to failure in adjacent zones as well. If equilibrium cannot be obtained at that time, the process will continue until failure conditions extend along the entire slip surface. Thus, failure, having been initiated at a single point, generally propagates resulting in the ultimate failure of the slope until it reaches a new equilibrium or stable condition.

A major factor in the development of progressive failure is brittleness, which can be characterized by the brittleness index. The index is defined as the difference between the

peak and residual strengths over the peak strength (Bishop 1967). Apparently, the larger the difference between the peak and the residual strength is, the larger the value of the brittleness, or the more brittle the soil. Consequently, the possibility of progressive failure can be directly related to the value of the brittle index, with higher brittle index values being indicative of more potential problems. In this study, the difference between the peak strength and the residual strength for both slopes are larger, as can be seen from the strength changes (Figures 3.8 and 3.14). For example, with an estimated effective normal stress of 50 kPa for the two slopes, the shear strength can drop from a peak value of 34 kPa to a residual value of 7.5 kPa for the Albia slope based on Table 3.1, giving a brittleness index of 0.78; and drop from a peak value of 47 kPa to a residual value of 14 kPa for the Albia slope based on Table 3.2, giving a brittleness index of 0.70. The large brittleness index values strongly suggest that progressive failure have contributed to the two slope failures described.

SUMMARY AND CONCLUSIONS

Post-failure investigations were conducted for two similar stiff clay shale slopes of large movement using *in-situ* Borehole Shear Tests (BST) and laboratory ring shear test. The BST measured the peak strength and partially softened strength, and the ring shear test measured the residual strength of the soils. Slope stability analyses and back calculations were performed on the observed slip surface of each slope. A range of mobilized shear strength at failure, with the fully soften strength being its lower bound as reported in literature, were obtained due to the unknown ground water conditions at failure. The most probable mobilized shear strength at failure was estimated based on the knowledge of the partially softened strength and residual strength, which could be considered as an improvement as compared to the empirical method of using “engineering judgment or experience”. Based on the approach in the study, the change of the strengths, or the “strength path”, due to the slope movement, can be best established. The information, together with the information on the failure zones and the failure mechanism of the slopes, will be useful in slope repair design. The failures of the slopes were attributed to progressive failure, and likely triggered by high ground water tables.

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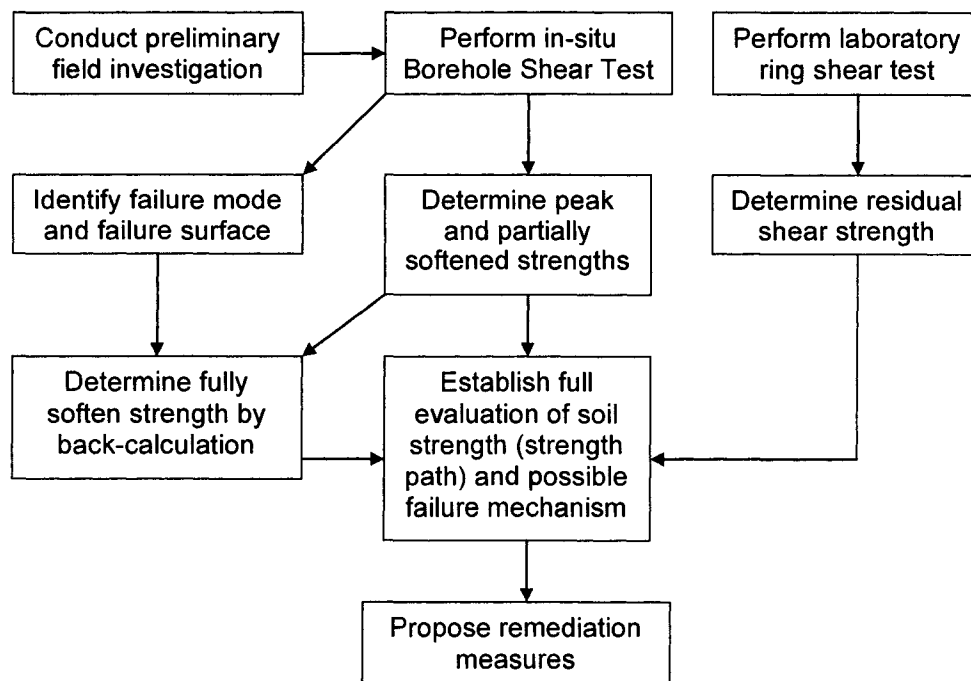


Figure 3. 1 Post-failure slope investigation processes involving BST and ring shear test

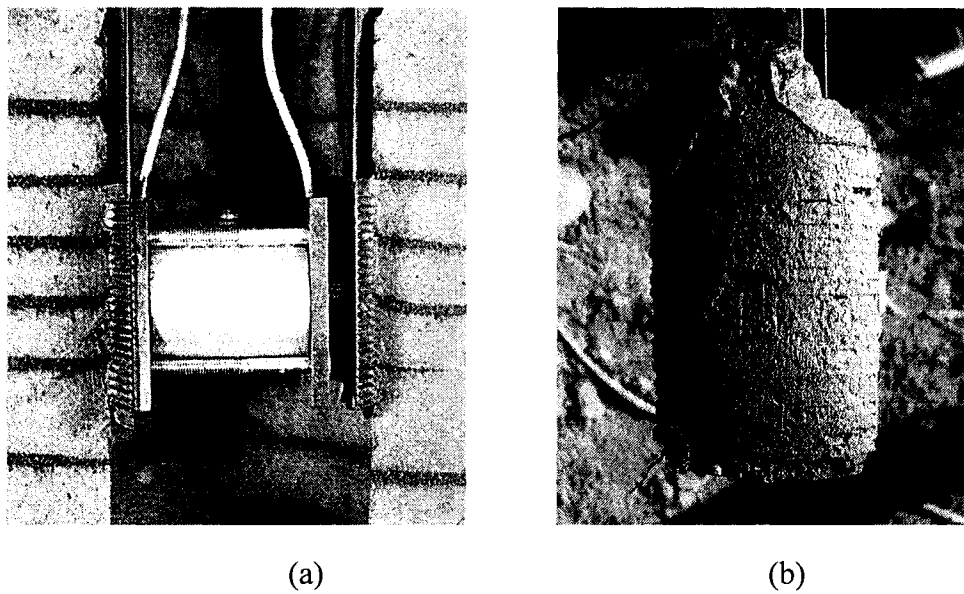


Figure 3. 2 Shear plates of a Borehole Shear Test device

(a) Cross-section of BST in a borehole before vertical shearing (Handy 2001);

(b) Shear plate covered with soil after testing.

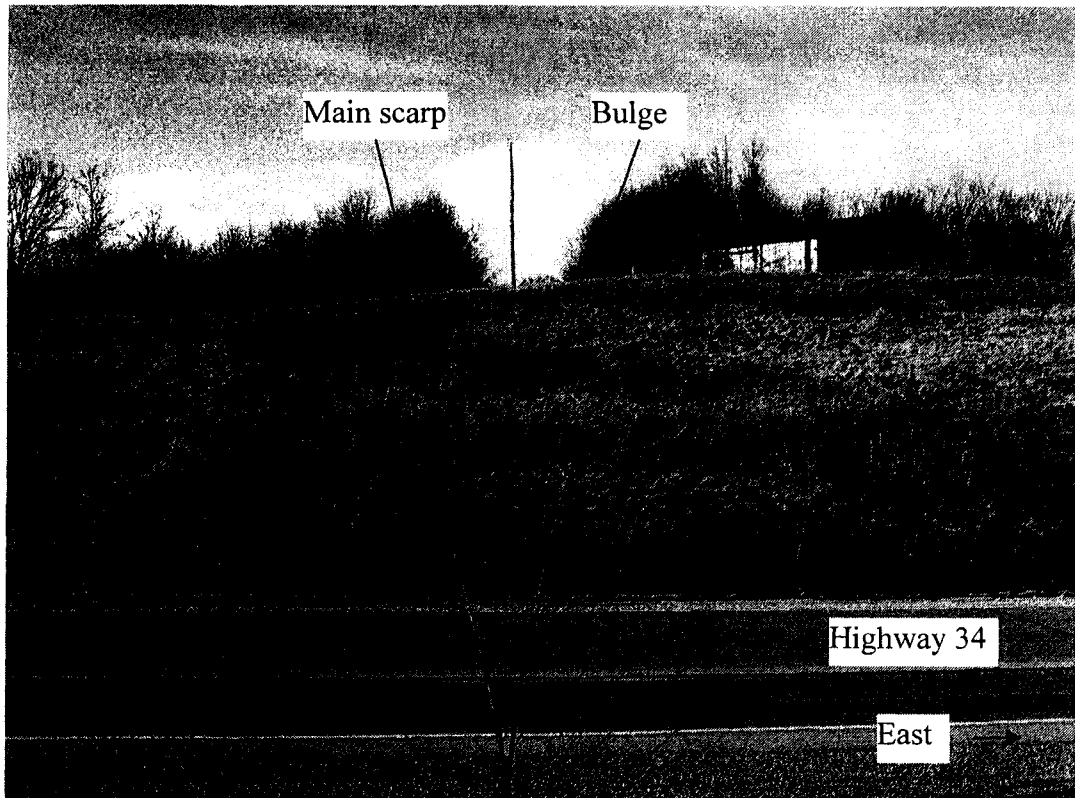


Figure 3. 3 Overview of Albia slope

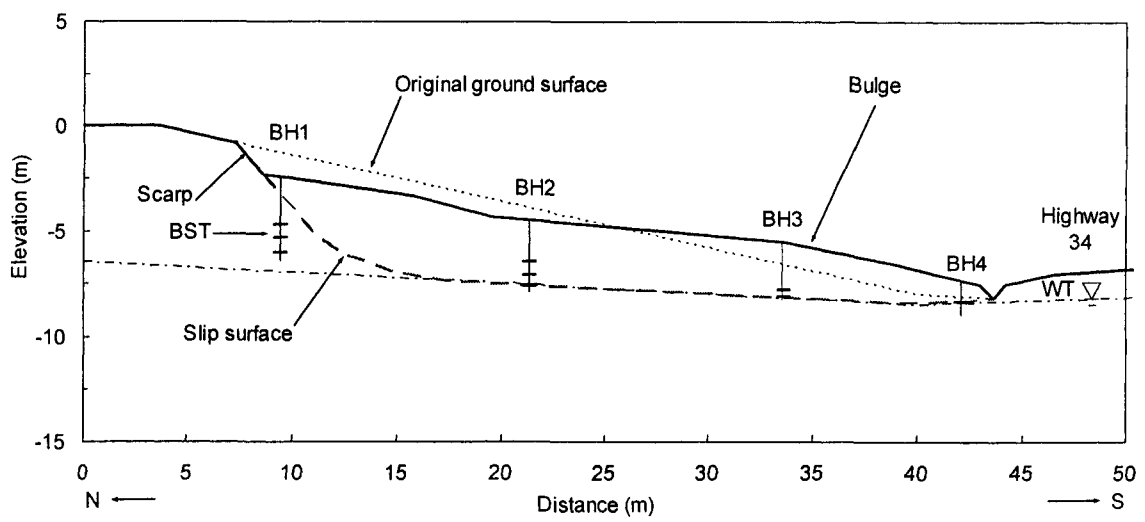


Figure 3. 4 Profile of Albia slope

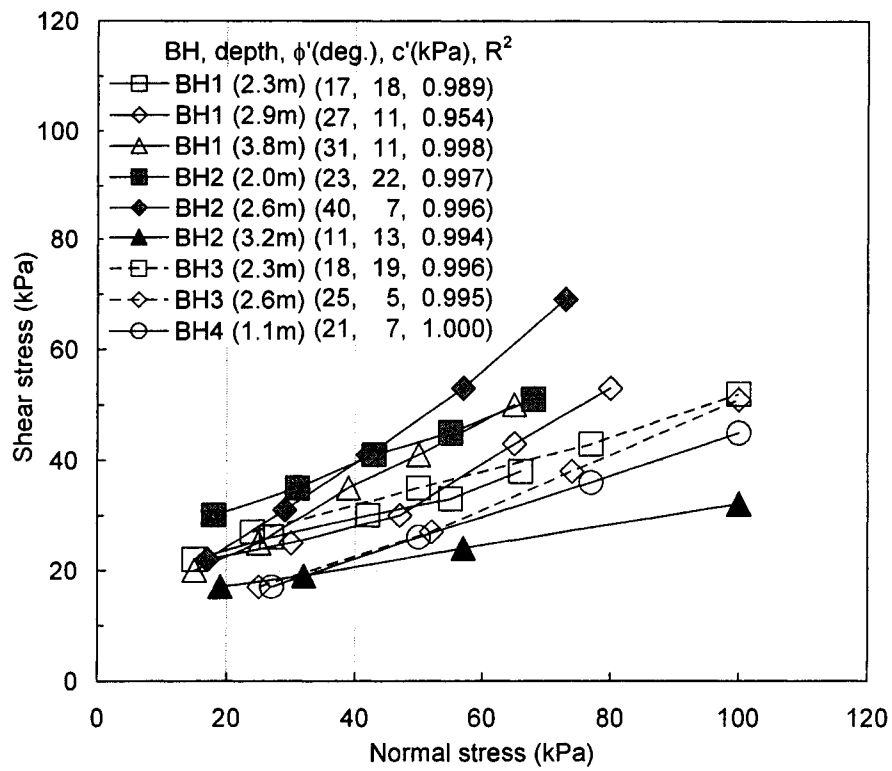


Figure 3. 5 Borehole Shear Test results for Albia slope

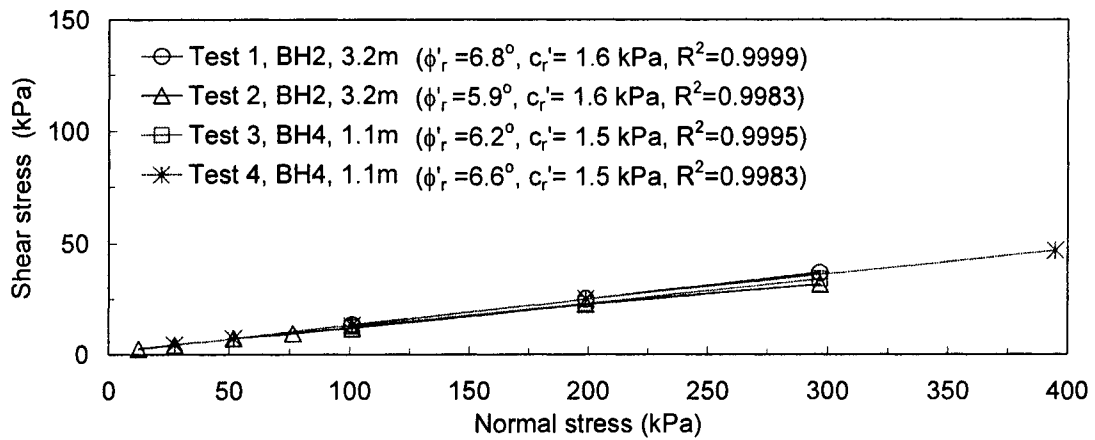


Figure 3. 6 Ring shear test results for Albia slope

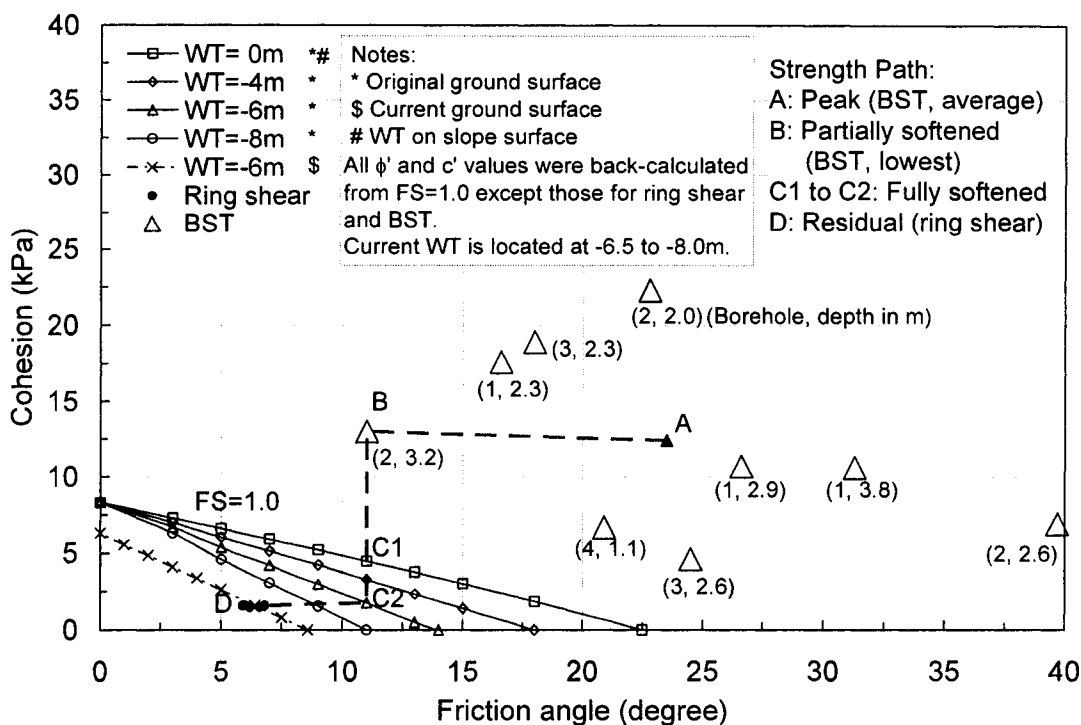


Figure 3. 7 Strength path for the shale at Albia slope

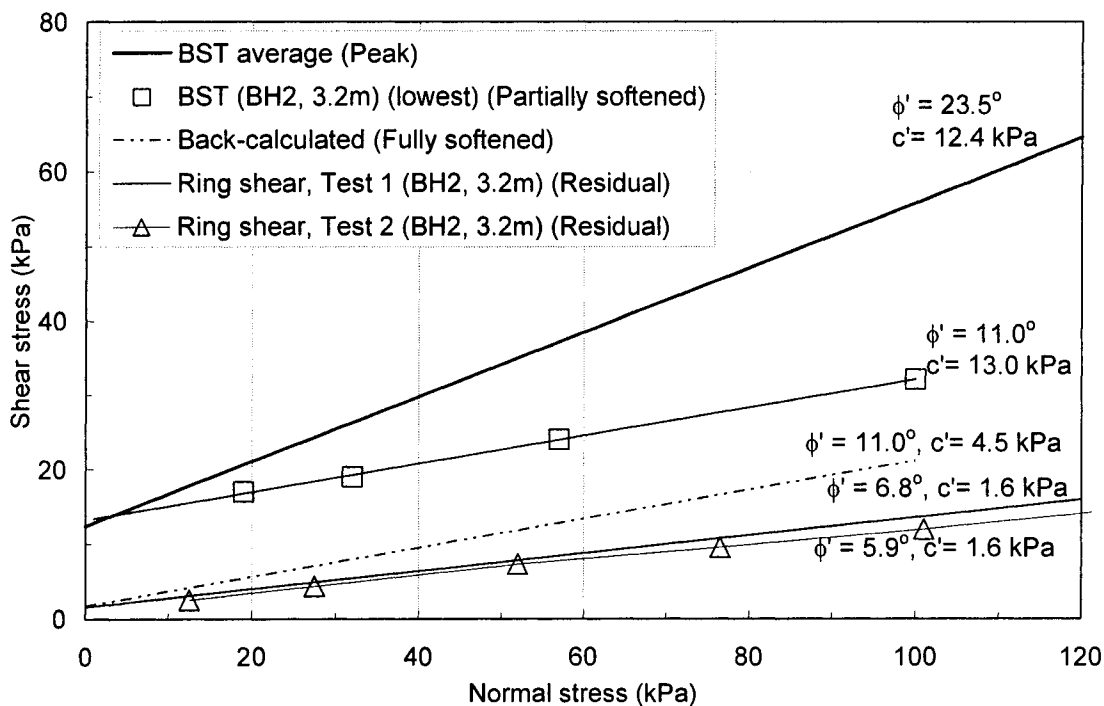


Figure 3. 8 Different shear strength envelopes for the shale at Albia slope

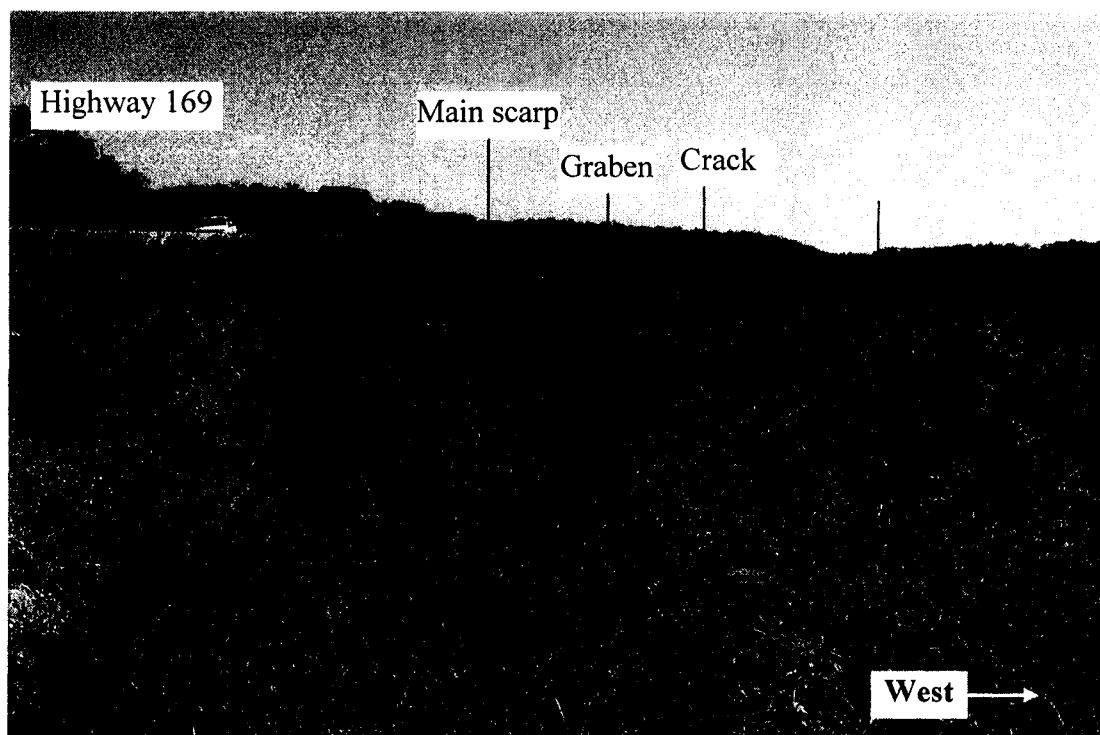


Figure 3. 9 Overview of Winterset slope

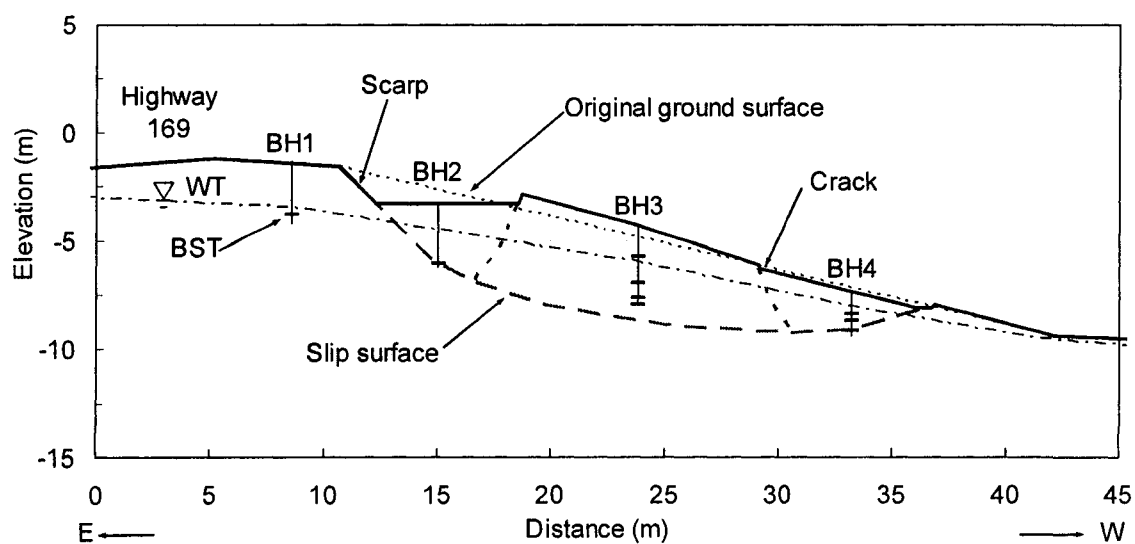


Figure 3. 10 Profile of Winterset slope

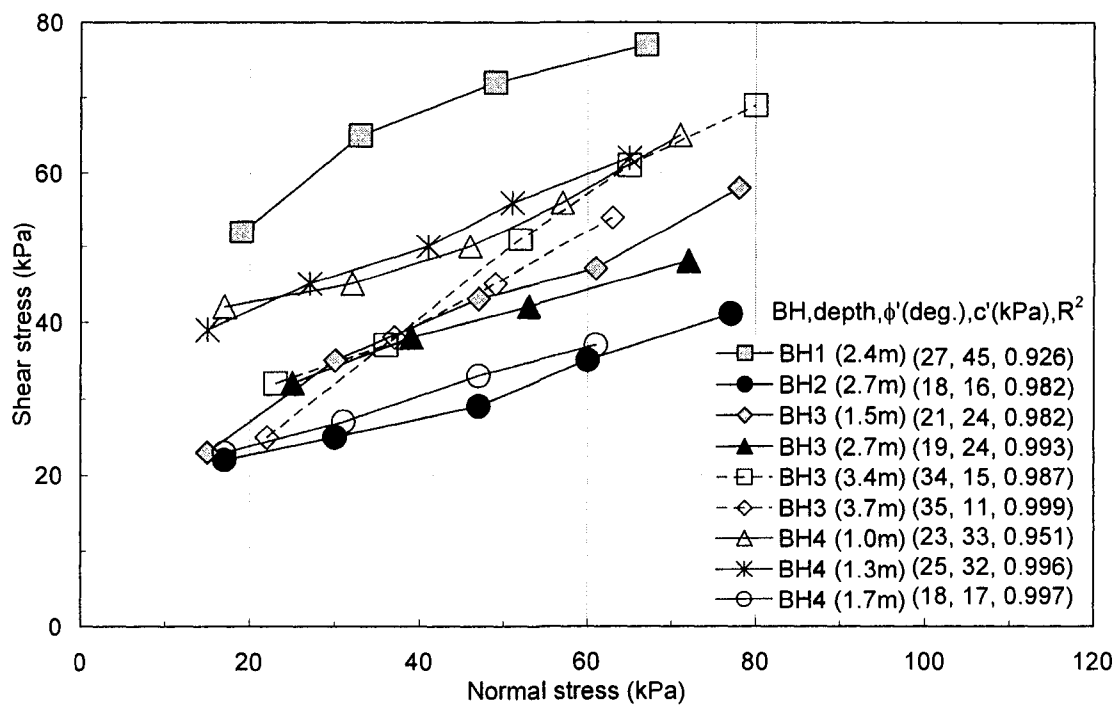


Figure 3. 11 Borehole Shear Test results for Winterset slope

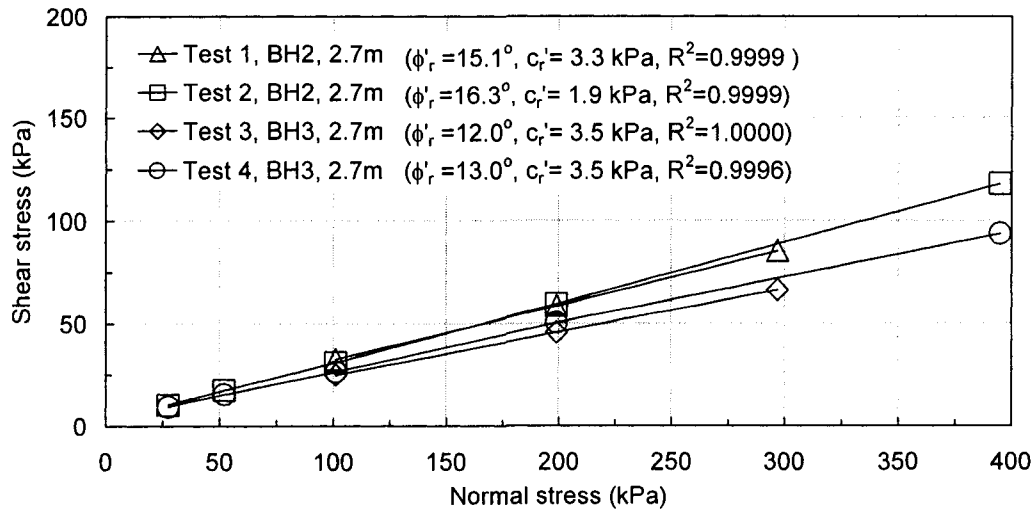


Figure 3. 12 Ring shear test results for Winterset slope

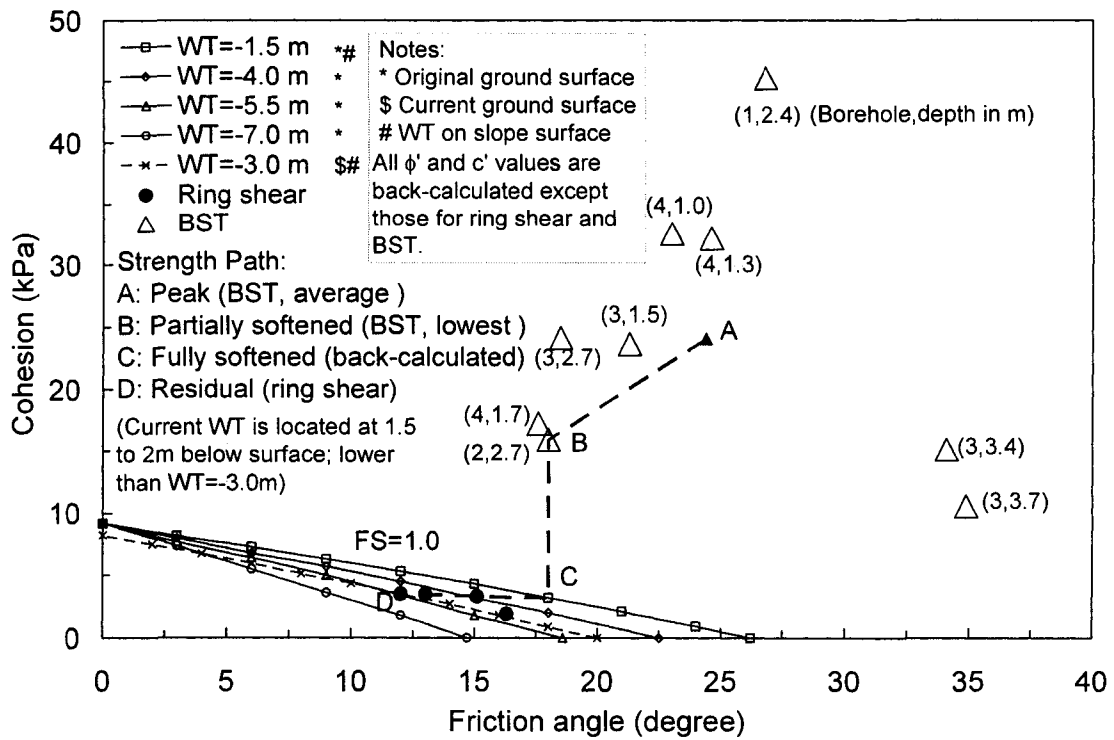


Figure 3. 13 Strength path for shale at Winterset slope

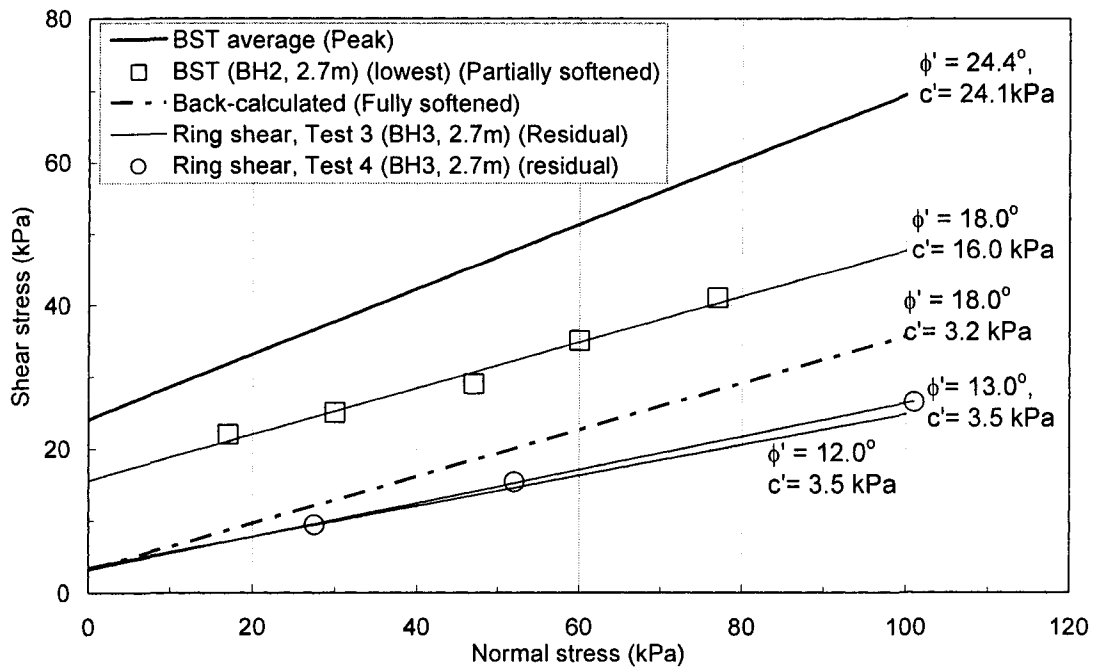


Figure 3. 14 Different shear strength envelopes for the shale at Winterset slope

Table 3. 1 Summary of the Slope Analysis Results for Albia Slope

| Shear Strength Parameter | | | | | Factor of Safety ** | | |
|--------------------------|------------------|--------------------|-------------------|---------------|---------------------|------|------|
| No. | Source | Description | ϕ' (deg.) | c' (kPa) | (1) | (2) | (3) |
| A | BST, average | Peak strength | 23.5 | 12.4 | 2.52 | 3.44 | 5.12 |
| B | BST, lowest set | Partially softened | 11.0 | 13.0 | 2.02 | 2.72 | 3.47 |
| C | Back-calculated* | Fully softened | 11.0 | 4.5 | 1.00 | 1.37 | 2.12 |
| D | Ring shear | Residual strength | 6.8 | 1.6 | 0.48 | 0.66 | 1.12 |

Notes: (1) - original ground surface with water table on the original slope surface;

(2) - current ground surface with water table on the current slope surface;

(3) - current ground surface with current water table as measured;

* Based on Conditions (1);

** On the specified slip surface using Morgenstern-Price (1965) method.

Table 3. 2 Summary of the Slope Analysis Results for Winterset Slope

| Shear Strength Parameter | | | | | Factor of Safety ** | | |
|--------------------------|------------------|--------------------|-------------------|---------------|---------------------|------|------|
| No. | Source | Description | ϕ' (deg.) | c' (kPa) | (1) | (2) | (3) |
| A | BST, average | Peak strength | 24.4 | 24.1 | 3.42 | 3.95 | 4.49 |
| B | BST, lowest set | Partially softened | 18.0 | 16.0 | 2.32 | 2.69 | 3.15 |
| C | Back-calculated* | Fully softened | 18.0 | 3.2 | 1.00 | 1.23 | 1.62 |
| D | Ring shear | Residual strength | 12.0 | 3.5 | 0.80 | 0.96 | 1.22 |

Notes: same as Table 3. 1.

CHAPTER 4. CHARACTERIZATION AND ENGINEERING PROPERTIES OF CLAY SHALES FOR AN EMBANKMENT SLOPE IN IOWA

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ABSTRACT

The Sugar Creek embankment slope project contains highly weathered, overconsolidated clay shales predicted to involve global slope instability when an empirical, conservative shear strength value was used in the initial slope stability analyses. The initial analyses motivated a comprehensive geotechnical investigation including extensive basic property tests, shear strength testing including the *in-situ* Borehole Shear Test (BST), laboratory direct shear tests, triaxial compression tests and ring shear tests. The primary objective of the investigation was to better characterize the shales according to weathering classification and to obtain realistic, specific shear strength parameter values for more detailed slope stability analyses. The study suggests that classification of weathering of the shales correlates well with the peak shear strength values of the shales, i.e. a higher weathering degree consistently corresponds to lower shear strength values; but does not correlate well with residual shear strength values or other soil index properties. The large database of shear strength parameter values obtained from the *in-situ* tests reasonably agrees with the results of the laboratory tests. The paper represents a detailed case study for using geotechnical information including *in-situ* BST measurements to characterize weathered shale materials for slope stability analyses.

Keywords: Characterization; Clay shale; Weathering; Shear strength; Residual strength; Slope; *In-situ*.

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INTRODUCTION

The safe and economical design of slopes and structures requires the best possible assessment of subsurface ground conditions, particularly the strength of the soils encountered, and the best evaluation of the design strength parameters. Soils normally have wide ranges of geotechnical properties due to their composition and complex geological processes and histories (Mitchell 1993). Further complications arise as a consequence of the differences among the results obtained from various *in-situ* and laboratory test methods. Among the many types of earth materials, shales probably create more potential slope instability problems than any other material (Yagiz 2001). One of the major problems engineers encounter in shales with respect to slope analysis is the determination and selection of the shear strength parameter values. Lack of information on shear strength values specific to a site may result in an over-conservative design. On the other hand, proper *in-situ* and laboratory tests can provide shear strength values that are specific and realistic to the site, and consequently result in both safe and economical design.

This paper describes the characterization of clay shales based on weathering classification for a proposed embankment slope project in Iowa, and presents the results of shear strength values of the soils obtained from various test methods. The tests include *in-situ* Borehole Shear Test (BST), and laboratory direct shear test, triaxial compression test, unconfined compression test and ring shear test. The weathering classification of shales was related to basic properties and shear strengths. The shear strengths of the soils, particularly the highly weathered shale, obtained from different test methods are compared, and the possible reasons for the differences of the shear strengths are discussed. A preliminary engineering evaluation of the shales with respect to slope stability is also described.

PROJECT BACKGROUND

The Sugar Creek embankment slope project is located in Wapello County, Iowa (Figure 4.1). Prior to the detailed site investigation, the approach embankment fills on both sides of Sugar Creek were to be designed using pile-supported abutments to support the

highway bridge across the creek. Slope analyses indicated that, based on the preliminary design guidelines of Iowa Department of Transportation (IaDOT), there was potential global instability for the slopes in front of the abutments with the slip surfaces passing through the highly weathered sloping shale interface using a friction angle equal to zero and cohesion of 10 kPa. As a result, ground improvement and retaining wall alternatives were proposed with estimated costs ranging from 3 to 5 million dollars (Farouz et al. 2005). In view of the high costs, a comprehensive supplemental subsurface exploration and test program was developed and performed in 2004 at a small cost to supplement the preliminary investigation conducted in 2001. 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 improvement measures, if required.

During the initial field investigation program for the project in 2001, a total of 16 boreholes were drilled. Though much information was obtained on the soil stratification for the site, no shear strength parameter values for the shales were measured. Therefore, in the investigation program of 2004, an additional ten boreholes were drilled and extensive *in-situ* and laboratory tests were conducted to obtain the soil shear strength parameter values of the project. Based on the measured shear strength of the soils obtained, the stabilities of the slopes were re-evaluated and a substantial savings resulting from the proposed ground improvement measures is expected to be realized.

GEOLOGY AND SITE CHARACTERIZATION

Area Geology

Iowa is commonly divided into seven regions based on the various landforms found in each region (Prior 1976; Prior 1991), and Wapello County is located on the Southern Iowa Drift Plain (Figure 4.1). The landscapes are characterized by gently rolling hills and valleys, which have been formed by hundreds of thousands of years of erosion and stream development. 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.

According to the USDA (1981) Soil Survey Report, most of the soils in Wapello County formed in glacial till, loess and alluvium. Clay shale is the oldest parent material consisting of a series of beds deposited during the Des Moines sedimentary cycle in the Pennsylvanian period (286 to 320 million years ago) (Eicher et al. 1984). Thin beds of sandstone and coal are encountered between the shale layers in places. In the project site, loess-derived and till-derived soils are present. The bedrock primarily consists of clay shale, with sandstone and thin limestone layers. Loess and till were not encountered in the project site. These soils have probably been subjected to surface erosion which occurred commonly in river valleys in this area (Prior 1976; USDA 1981).

Site Geology and Characterization of the Shales

A total of 26 mechanically drilled boreholes was distributed on the two sides of Sugar Creek to cover an area of about 200x50 m² for the project site. The subsurface soils of the project site can be roughly divided into two apparently distinguishable groups, i.e. the alluvium layer and the underlying shales. The alluvium layer is brown or light grey, and mainly consists of lean clay with sand and small amounts of gravel underlain by clayey sand and silt; or a mixture of clay, silt, sand and small amounts of gravel. The compositions of this layer vary throughout the site depending on the locations relative to the river and the floodplain. Some of the soils were loose and likely unconsolidated. The thickness of the alluvium ranges from 2.7 to 8.4 m.

The shales underlying the alluvium layer have multiple colors of brown, grey and black, etc., and vary spatially and with depths. The shales have relatively low shear strength in the upper portion, which behaves like soil; but become stronger with increasing depth, and behaves like rock. In fact, shales have been appreciated as the boundary materials between soil and rock (Yagiz 2001). It was observed at the project site that the shear strength of the shales can be associated with the weathering degree, which is directly related to the depths of the shales. Therefore, it is desirable to classify the shales and to obtain their shear strength

values for the project based on the weathering degree, so that it will be helpful for design and construction purposes.

Weathering classification for the characterization of rocks has been proposed by various researchers (e.g., Dearman 1976; Turraul and Gurpinar 1997). Various classification systems for shales have also been developed based on a geological scheme (e.g., Picard 1971; Spears 1980), and based on an engineering scheme (e.g. Underwood 1967; Morgenstern and Eigenbrod 1974; Wood and Doe 1975). However, a weathering classification with respect to the characterization of shales appears lacking in the literature. This could be due to the fact that the term shale is used to designate all argillaceous, fine-grained varieties of sedimentary rock that formed from consolidation of clay, silt and mud; thus the weathering classification may be too complex and unpractical to be generalized. On the other hand, it is desirable to know the engineering properties of the shales and the extent of the shales of different weathering degrees for the site, so that an economical design of slopes and structures and subsequent construction can proceed, as noted previously. Therefore, attempts were made to carry out weathering classifications of shales for their characterizations specific to the site. Instead of using six weathering grades for rocks, such as those in Dearman (1976) (i.e., residual soil; completely, highly, moderately and slightly weathered; and fresh), three weathering grades (i.e., highly, moderately, and slightly weathered) were adopted for the project for the purpose of simplicity and convenience, while maintaining sufficient detail to delineate the layers for slope stability analyses.

The classification of the shales using the three weathering grades resulted in three layers of shales for the site, i.e. highly weathered shale (H.W.Sh), moderately weathered shale (M.W.Sh), and slightly weathered shale (S.W.Sh). The distinction of the three weathering grades or layers was essentially a combination of the classifications using the geological scheme and the engineering scheme. The geological aspects mainly included color, texture and structure of the shales as in the field visual observations; and the engineering aspects mainly included soil consistency, and were supplemented with the field testing results of pocket penetrometer and standard penetration tests, and the split spoon refusal for S.W.Sh. In general, H.W.Sh was relatively soft and weak, S.W.Sh was relatively hard and strong, and M.W.Sh was the transition zone between H.W.Sh and S.W.Sh. 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 150-mm increment. Based on the classification of the three shale layers, *in-situ* and laboratory tests were performed to obtain the engineering properties for each layer.

The surface of H.W.Sh generally parallels the existing ground surface. It has a gentle slope ranging from 12.5:1 (H:V) to 10:1 (H:V) on the north side of the creek, and a relatively steeper slope of 3:1 (H:V) on the south side of the creek. The thickness of H.W.Sh ranges from 0 to 3.7 m on the north side and 0.5 to 4.7 m on the south side of the 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 on the north side and 1.5 to 8 m on the south side of the creek. M.W.Sh was underlain S.W.Sh, where the boreholes were terminated. On the south side of the creek, a nearly horizontal, 0.5-0.9 m thick limestone seam was encountered in the M.W.Sh layer. The boring results indicated that the spatial distributions of the shales were highly variable both vertically and laterally. The 24-hour ground water table was gently sloping towards the creek. A typical slope section showing the soil profiles and water table is presented in Figure 4.2.

BASIC AND INDEX PROPERTIES OF THE SHALES

Grain Size Distributions

Basic properties for representative soil samples with emphasis on the shales were investigated. Grain size distribution analyses of the shales were performed for 11 samples of H.W.Sh and M.W.Sh from different boreholes in accordance with ASTM D422-63 (ASTM 2002a). The results show that the shales have 31 to 61% clay fraction ($<0.002\text{mm}$), 39 to 67% silt fraction and less than 9% sand ($>0.075\text{ mm}$), except for one H.W.Sh sample having 17% sand.

The clay contents generally increase with the decrease in depth (Figure 4.3(a)), or with the increase in weathering degree of the shale (since weathering degree is higher near the surface). This suggested that the higher degree of weathering resulted in the formation of more clay minerals. However, the classification of the weathering degree itself does not show

a definite relationship with the depth as indicated by the scattering of the M.W.Sh and S.W.Sh at different depth in Figure 4.3(a). This observation suggests that classification of weathering can not depend on the results of the grain size analysis or the clay fraction only, but may be related to other factors such as mineralogy of the soils.

Atterberg Limits

Atterberg limits for the shale samples were determined following the descriptions of ASTM D4318-00 (ASTM 2002b), and the results are presented in Figures 4.3(b) and 4.3(c). The results show that liquid limits (LL) of the shales range from 37 to 72%, and plasticity indexes (PI) range from 16 to 44%. LL and PI of the shales have good correlation with the clay fraction (CF) of the shales. The regression result between PI and CF indicates an activity (as defined by Skempton 1953) of 0.66 for the shales. All the shales are located between the A-line and the U-line in the Casagrande's plasticity chart, thus classified as either low plasticity clay (CL) (LL<50%) or high plasticity clay (CH) (LL>50%) according to the Unified Soil Classification System (ASTM D2487-00, ASTM 2002c).

The results also show that Atterberg limits do not exhibit a clear relationship with the weathering degree of shales, i.e., the M.W.Sh and the S.W.Sh does not necessarily have low CF or LL. This observation suggests that the classification of the weathering cannot be purely based on the index properties of the soils.

Mineralogy of the Shale

X-ray diffraction (XRD) analysis on random oriented bulk soil samples were performed for ten shale samples including H.W.Sh, M.W.Sh and S.W.Sh, from different depths of different borings. One of the typical XRDs for the H.W.Sh is presented in Figure 4.4. The minerals identified are summarized at the bottom of the diffractogram. In general, quartz, kaolinite and illite were found in the XRDs for all the shale samples. Montmorillonite was discovered in H.W.Sh only; and cristobalite and pyrite were observed in two of the four H.W.Sh samples. These results suggest that the montmorillonite may have been resulted from the complete weathering of the shale, while the less weathered shale only produced

kaolinite and illite. In addition, the presence of the montmorillonite is also consistent with the relatively high LL of the shales as observed from the Atterberg limits tests.

SHEAR STRENGTHS OF THE SOILS

Results of In-situ Borehole Shear Tests

Various *in-situ* test methods are available for geotechnical field investigations. Borehole Shear Test (BST) and Rock BST are probably the unique ones as they provide direct measurements of *in-situ* shear strength parameter values of soil and rock (Handy and Fox 1967; Handy et al. 1976). Basically, a series of direct shear tests are performed on the inside of a borehole. Normal stress is applied to the wall of the borehole through a pair of shear plates, and the peak shear stress is measured *in-situ* separately and concurrently. Thus, the internal friction angle (ϕ') and cohesion intercept (c') of the soil and rock are determined from the Mohr-Coulomb failure envelope. Total testing time for a typical test with 4 to 5 data points is approximately 30 to 60 minutes depending on soil type (Handy 1986). The BST is normally a consolidated-drained test as demonstrated by the pore pressure measurements during the test (Demartinecourt and Bauer 1983). The rate of the shear head displacement is generally 0.05 mm/s (Wineland 1975).

In this project, a total of 33 BSTs and 2 Rock BSTs were performed at different layers in the 10 borings of 75 mm in diameter, with emphasis on the H.W.Sh. All the results show that the tests were well performed as revealed by the large values of coefficient of correlation (R^2) between the shear stresses and the normal stresses, which are generally larger than 0.99. Examples of the test results are presented in Figure 4.5(a), which shows an increased shear strength of the shales with increased depths. Figure 4.5(a) also shows that reduced shear strength was obtained when a “residual” BST or a repeated BST was performed on the same elevation of the borehole. The test results for all the H.W.Sh are shown in Figure 4.5(b), which indicates that the shear strength parameter values of the H.W.Sh are highly variable. The overall average strength parameter values are $\phi' = 13.4^\circ$ and $c' = 30$ kPa; the interpreted upper bound values are $\phi' = 16.7^\circ$ and $c' = 55$ kPa, and the interpreted lower bound values are $\phi' = 9.6^\circ$ and $c' = 5$ kPa.

The variations of the shear strength values are further illustrated by the $c'-\phi'$ plot for all the shales in Figure 4.5(c), and by the summary of statistical results in Table 4.1. Despite the variation of the shear strength values, the general trend that the shear strength values increase with the decrease in weathering degree is apparent. H.W.Sh generally have low shear strength values; and S.W.Sh generally have high shear strength values, mainly exhibited by the much higher cohesions. M.W.Sh have shear strength values between H.W.Sh and S.W.Sh indicating a transition layer. These observations suggest that the shear strength values of the shales are well correlated with the weathering classification, indicating the weathering classification scheme is valid. It is also note-worthy that the average shear strength values of H.W.Sh, which are $\phi' = 12.8^\circ$ and $c' = 33.2$ kPa, are much higher than that of $c' = 10$ kPa as assigned by the IaDOT design guidelines prior to the tests.

Results of Direct Shear Tests

A total of 20 consolidated drained direct shear tests (DS) were performed in the laboratory, which included four tests on the alluvium soils, ten tests on H.W.Sh, and six tests on the M.W.Sh. The tests were performed on undisturbed Shelby tube soil samples following the procedures of ASTM D3080-98 (ASTM 2002d). Each test comprised three to five specimens. The size of the specimens was typically 63.5 mm in diameter and 20.1 mm in height. The loading rate of the shear force was 0.025 mm/min.

The test results show that R^2 values are generally larger than 0.99 indicating the effectiveness of the tests, though R^2 values as low as 0.91 were also observed for a few tests, which may be due the soil sample variability. As an example, DS results for a H.W.Sh are presented in Figures 4.6(a) and 4.6(b). The peak strength for the drained DS specimens occurred after a displacement of about 1 to 5 mm (correspond to 1.6 to 7.9% of strain). With further displacement the specimens softened, and there was little difficulty in distinguishing peak strength from fully softened strength (as defined by Skempton 1985). The failure envelopes gave the peak and fully softened strength values as shown in Figure 4.6(b). It is of interest to note that the stress displacement curves (Figure 4.6(a)) do not show significant decrease in shear strength with increasing displacement. This indicates that the displacement required to reach residual strength should be significantly larger than what was obtained in

the direct shear apparatus (7 mm). Skempton (1985) stated that an intact clay sample requires a minimum displacement of 100 to 500 mm to reach a final steady residual value in a ring shear test. The ring shear test results are presented in a later section.

The DS test results for all the H.W.Sh are presented in Figure 4.6(c), which shows the large variation of the shear strengths of the soil. The apparent average strength parameter values were $\phi' = 19.5^\circ$ and $c' = 26.6$ kPa, and the suggested lower bound and upper bound values were $0.5\phi'$ and $1.5\phi'$ (with the same c' value), respectively. The variability of the shear strengths of the soils were similarly illustrated by the statistical results as summarized in Table 4.2. The results show that the H.W.Sh had average shear strength parameter values of $\phi' = 21.4^\circ$ and $c' = 20.4$ kPa, which are quite close to the apparent average values as shown in Figure 4.6(c).

The relatively high cohesion intercept of the peak strength failure envelope is typical of an overconsolidated clayey soil (e.g. Skempton 1964). This is because the overconsolidated soil, having once been subjected to a higher preconsolidation pressure, is in a dense state with respect to the current confining pressure. As a result, a cohesion intercept of the failure envelope is exhibited after shearing (Anderson and Sitar 1995). Overconsolidation could be the result of erosion and the subsequent removal of overburden stress on the indurated shale. The high cohesion intercept could also be the result of soil structure imparted by cemented bonds within the soil, i.e. the soil has structure as a result of cementation and bonding of the soil fabric (Mitchell 1993; Anderson and Sitar 1995).

Results of Triaxial Compression Tests

Two consolidated drained triaxial tests (CD) were performed on Shelby tube samples of H.W.Sh from borehole CH1010 following the procedures of ASTM D4767-95 (ASTM 2002e). The size of the samples was 72 mm in diameter and 150 mm in height. Stress paths for one of the tests are presented in Figure 4.7(a), which indicates peak shear strength parameter values of $\phi' = 27.6^\circ$ and $c' = 20.6$ kPa. The other CD test gave peak strength values of $\phi' = 33.7^\circ$ and $c' = 0$ kPa. The stress-strain curves of the tests indicated that the tests exhibited stress softening behavior with the peak strengths at 3 to 4% of axial strain. The

curves of the volume change versus axial strain showed that the volume changes were first negative then became positive after 6 to 8% of axial strain, indicating that the shales were over-consolidated.

Consolidated undrained triaxial tests (CU) were also performed on Shelby tube samples, with four tests on H.W.Sh samples of different boreholes and two tests on the silty clay of the alluvium layer. The tests followed the procedures as described in ASTM D4767-95 (2002e). Stress paths of the CU test on H.W.Sh from borehole CH1010 are presented in Figure 4.7(b). Tests ended at 16% of axial strain for the three specimens. The results show that at the relatively low confining pressure (specimens A and B), negative pore pressures were induced at the end of the tests, which were indicated by the total stress being lower than the effective stress. These observations indicate that the shales were over-consolidated. However, at a relatively high confining pressure (specimen C), positive pore pressure was induced throughout the test, indicating that the preconsolidation pressure must be lower than the confining pressure as applied for the specimen C, which was about 400 kPa. From the failure envelope, the peak shear strength parameter values were found to be $\phi' = 27.5^\circ$ and $c' = 4.0$ kPa, which agreed with the ϕ' value obtained from the CD test, but with a smaller c' value.

To show the overall CU test results on H.W.Sh, the effective principal stresses at failures for all the tests were plotted in a p' - q diagram as shown in Figure 4.7(c). The plots show considerable scatter owing to the shale sample variations. Regression analysis resulted in overall shear strength parameter values of $\phi' = 25.5^\circ$ and $c' = 10.0$ kPa, which are close to the average values of $\phi' = 26.6^\circ$ and $c' = 5.7$ kPa obtained from the statistical results of the four CU tests. The statistical results of CU tests are summarized in Table 4.3.

Results of Unconfined Compression Tests

A total of four unconfined compression tests were performed on cored samples of the S.W.Sh to determine their unconfined compressive strengths (q_u). Tests were performed following ASTM D2166 (ASTM 2002f). The size of the specimens was 50.8 mm in diameter and 95.3 mm in height. The loading rate was 2% of axial strain per minute. The test results

are presented in Figure 4.8. The results show that the required axial strain for the shale to reach peak strength was about 6 to 7%. The q_u values ranged from 360 to 640 kPa, which indicate that the shales were hard (Terzaghi et al. 1996). The results also show in general that the deeper the shale (thus the lesser the weathering), the higher the q_u value, which was an indication of deeper shale being less weathered.

Results of Ring Shear Tests

The shear strength of the soil can drop from peak value to the residual value after large displacement, and the drop can be significant for materials with large amount of clay minerals, particularly platy minerals (Skempton 1964, 1985). The drop in strength is attributed to the clay particle reorientation parallel to the direction of shearing (Lambe and Whitman 1979; Bromhead and Dixon 1986). The residual strength is often related to long-term stability problem and areas with landslide history, bedding planes or folded strata, and reactivation of old landslides (Skempton 1985). Thus, it is desirable to investigate the residual strength of the shales for the project.

The Bromhead (1979) ring shear apparatus was used in the study due to its simplicity in operation and its availability. In the apparatus, the ring shaped specimen has an internal diameter of 70 mm, an external diameter of 100 mm and a thickness of 5.0 mm. Drainage is provided by two porous bronze stones fixed to the upper platen and to the bottom of the container. Multistage test procedures as described in ASTM (D6467-99) (2002g) were adopted. The specimen was prepared on shale passing US sieve No. 50 (0.30 mm) and remolded at a water content near the plastic limit to minimize the sample settlement during the consolidation. At the end of the primary consolidation, the test specimen was pre-sheared at a relatively high displacement rate (0.89 mm/min) for one revolution (a displacement of 267 mm) and followed by subsequent shearing with a lower displacement rate (0.036 mm/min) under different normal stresses (typically 100, 200 and 400 kPa). The plot of the shear stress versus the normal stress gave the Mohr-Coulomb failure envelope and the effective residual shear strength parameter values.

A total of 14 ring shear tests were performed on shale samples obtained from different depths of different boreholes, which corresponded to the shale samples for the index

property tests. Typical test results are presented in Figures 4.9 (a) and 4.9(b). It can be seen that the shear stress became constant under a specific normal stress once the shear surface has been formed after a relatively large displacement (normally larger than 200 mm). The subsequent displacement under a new normal stress is normally 10 mm to achieve constant shear stress. The correlation between the shear stresses and the normal stresses was very high, generally having an R^2 value larger than 0.999 since the tests are performed on exactly the same soil.

The residual strength depends on the percentage of the clay particles that can be reoriented during the shearing and the ability of these particles to be reoriented. Therefore, many attempts have been made to correlate the residual strength parameter value, ϕ_r' , with the soil index properties (e.g., Voight 1973; Kanji 1974; Lupini et al. 1981; Mesri and Cepeda-Diaz 1986; Collotta et al. 1989; Stark and Eid 1994). Recently, a correlation of ϕ_r' with soil mineralogical composition was also made by Tiwari and Marui (2005). Generally, the correlations with index properties proposed by the different researchers vary significantly, indicating “the correlations of residual friction angles with the soil index properties cannot be general” as noted by Lupini et al. (1981); and the correlation for clay shale in one geologic formation can not be necessarily used in another geologic formation (Beene 1969). In this study, the ϕ_r' values for the 14 tests on the shale samples were plotted against the plasticity index (Figure 4.9(c)). The plot shows that ϕ_r' generally decreased with the increase in the plasticity index, which agreed with those reported in the literature. A correlation that was similar to the form proposed by Kanji (1974) was also obtained as shown in the figure. This correlation may be used to estimate the residual shear strength values of the shales for the site. On the other hand, Figure 4.9 (c) did not show a clear correlation between the weathering degrees and the residual friction angles of the shales.

DISCUSSION OF THE SHEAR STENGTH VALUES

The test results show that the shear strength values obtained from BST, DS and CU for each soil layer do not match exactly, though they are comparable and in reasonably

agreement (Tables 4.1, 4.2, and 4.3). Several reasons may contribute to the discrepancies, which include the experimental errors, the soil variability and soil inherent properties.

For the concern on the experimental error in BST, Lutenegger and Timian (1987) investigated the reproducibility of BST results in soft and medium consistency marine clays. They found that there was no difference in the test results between an experienced operator and an inexperienced operator; and test results obtained by ten inexperienced operators generally fell within 95% confidence limits. Their findings confirmed that BST is simple to operate, and the variability of soil strength values caused by BST testing method could be essentially eliminated. As for DS and CU tests, since the tests are under well-controlled conditions and have become routine operations in laboratory, their effects on the soil strength variability can also be neglected. In fact, DS tests and CU (or CD) tests should give essentially same results for the same soil, as reported by Lambe and Whitman (1979) that comparisons between the value of ϕ' , from triaxial and direct shear tests, after averaging out experimental errors in the determination of the values, yield results that differ generally by no more than two degrees. All these observations suggest that the experimental errors should not have been the major contributors to the differences between the soil strength values from different test methods.

Soil variability could have contributed to the discrepancy of the results among different test methods. The soils tested by BST could not be exactly identical with those tested by DS and CU, even the *in-situ* BST locations have been intended to be the same or very close to the locations where undisturbed soil samples were collected and tested in laboratory. This may be especially true for the M.W.Sh, for which the c' values from BST (Table 4.1) are much higher than the c' values from DS tests (Table 4.2).

Another reason responsible for the difference between the shear strength values from different tests could be due to the inherent characteristics of the soils. This can be illustrated by comparing the strength values from BST and DS (and CU), especially the average shear strength values. It can be seen that the ϕ' values from BST are generally lower than those from DS and CU, while the c' values from BST are generally higher than those from DS and CU, for both the alluvium and H.W.Sh, as reflected by the average values in Tables 4.1 and 4.2. These observations are also illustrated in Figure 4.10, showing the plot of c' versus ϕ' of

the H.W.Sh from BST, DS and CU; and in Figure 4.11, showing the average strength envelopes for both the alluvium and the H.W.Sh from BST, DS and CU test methods. The comparison should be promising at least on the results of H.W.Sh as obtained from BST and DS, since they contained the same, relatively large number of tests (i.e. 10 tests, Figure 4.11). These average shear strength values should have reflected the general behavior of the soils with respect to shear strengths of the H.W.Sh.

Since a BST yields a vertical failure surface, and a DS test yields a horizontal failure surface in the soil, the test results reflect the shear strength of the soil in two orthogonal directions. Thus, the different shear strengths as obtained in two different directions may reflect the strength anisotropy of the soil. Strength anisotropy has long been realized by researchers such as Skempton and Hutchinson (1969), who reported the anisotropy of undrained strength for London Clay and found that in terms of effective stress, the results of tests where shearing occurs in horizontal directions are 12 to 25% lower than of tests where shearing occurs at vertical directions. For the H.W.Sh in this study, the results show that the shear strength in horizontal direction as tested by DS was lower than that in vertical direction as tested by BST, when the normal stress was lower than about 80 kPa (the cross-over normal stress) (Figure 4.11). The situation was reversed when normal stress was larger than the cross-over normal stress. These observations were the indications of strength anisotropy of the shales. However, it should be noted that the magnitude of the normal stress with respect to the horizontal and vertical directions will be different even for the same location in the ground when the strength anisotropy is considered, because the normal stress in the horizontal direction (or the lateral stress) is generally a function of K_0 (coefficient of earth pressure at rest) and OCR (overconsolidation ratio) of soil. Therefore, the strength of the soil is complicated by the anisotropic behavior.

Shear strength anisotropy can be inherent due to depositional directional differences (Mitchell 1993), or the presence of discontinuities such as joints and fissures which may exhibit some degree of preferred orientation. Shear strength anisotropy can also be the result of an anisotropic stress state, i.e. the change in orientation of the principal stress (Hansen and Gibson 1949), thus causing the induced anisotropy (Chowdhury 1978). Generally, it is not easy to separate the two types of strength anisotropy.

PRELIMINARY EVALUATION OF THE SHALES AND SLOPE STABILITY

A preliminary engineering evaluation of the characteristics of shales can be carried out following the descriptions by Underwood (1967), based on the physical properties of the shales. Montmorillonite has been recognized as the most unfavorable clay minerals in soils. Since montmorillonite was found in the H.W.Sh, this soil was potentially problematic, which agreed with the general knowledge. Other unfavorable conditions for shales include cohesive strength of 30 to 700 kPa and friction angle of 10 to 20 degrees (Underwood 1967). The test results (Tables 4.1, 4.2 and 4.3) suggested that the H.W.Sh belongs to these categories. In general, the H.W.Sh in the site has unfavorable engineering properties.

The stability of the slopes can be evaluated based on the measured shear strength parameter values. The BST measurements gave relatively lower shear strength values than the DS and CU measurements at the estimated stress level of 100 to 200 kPa for the project (Figure 4.11), thus should be relatively conservative and safe when used for stability analysis. Slope stability analyses based on the measured shear strength values have shown small probability of failure (Farouz et al. 2005; Yang et al. 2005). Therefore, use of the realistic shear strength values that is specific to the site is expected to be cost effective and safe for the project. The use of *in situ* testing was a cost-effective way of optimizing slope design. Roughly, for every \$1 spent on *in situ* testing, the estimated construction cost savings are between \$30 and \$50 (Farouz et al. 2005). For the H.W.Sh having an apparently very low shear strength as revealed by the field investigation, excavation has also been proposed as a part of the remediation measures.

SUMMARY AND CONCLUSIONS

The site for the Sugar Creek embankment slope project was characterized, and engineering properties for soils, mainly the shear strength for the shales, were investigated. Both the stratification and shear strength values of the soil were highly variable. The classification of weathering of the shales that was specific to the site was proposed as an aid in characterizing the slope instability. It was found that the classification of weathering could

not be purely relied on the index properties of the shales, but it was consistently correlated with shear strength values of the shales. The shear strength values obtained from different methods did not exactly match, but were comparable and showed reasonable agreement, considering the variable nature of the soil. The internal friction angles obtained from BST were generally lower than those obtained from direct shear tests, while the cohesion intercepts obtained from BST were generally larger than those from direct shear tests, for both the alluvium and the highly weathered shale. This observation could be mainly attributed to soil variability, test methods and shear strength anisotropy. The use of the weathering classification and the measured shear strength values are expected to be economical and safe for the slope design and ground improvement measures.

ACKNOWLEDGEMENTS

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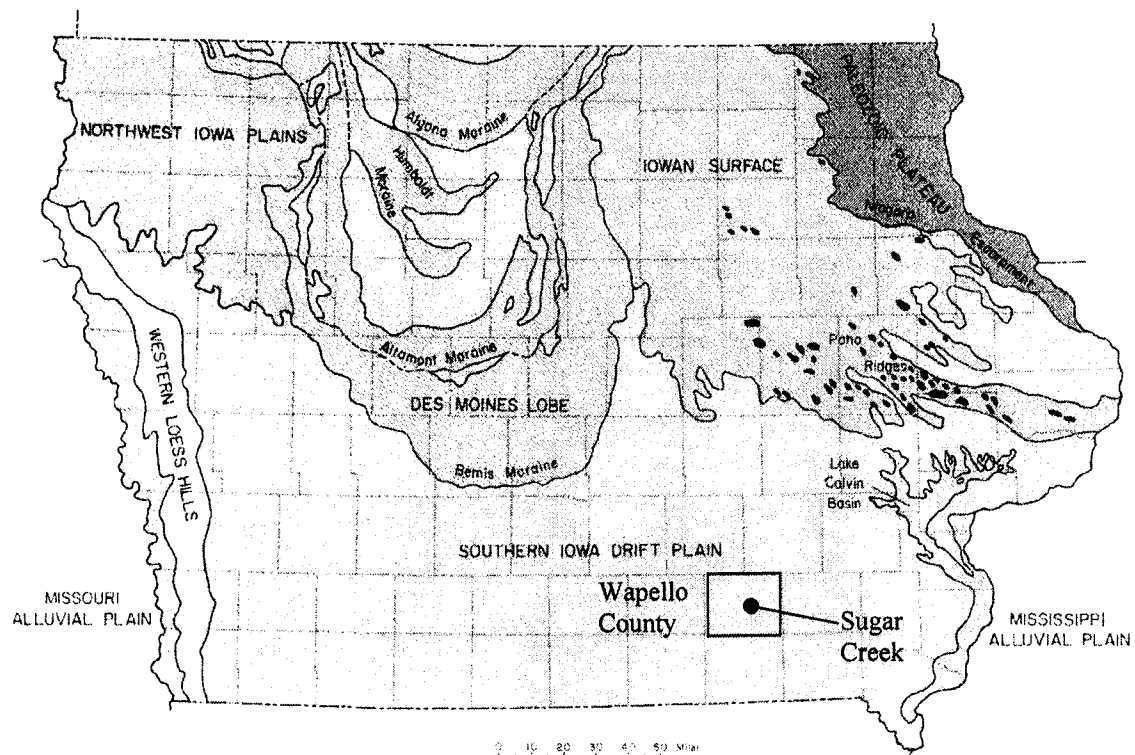


Figure 4. 1 Landform regions of Iowa (Prior 1976) and location of Sugar Creek project

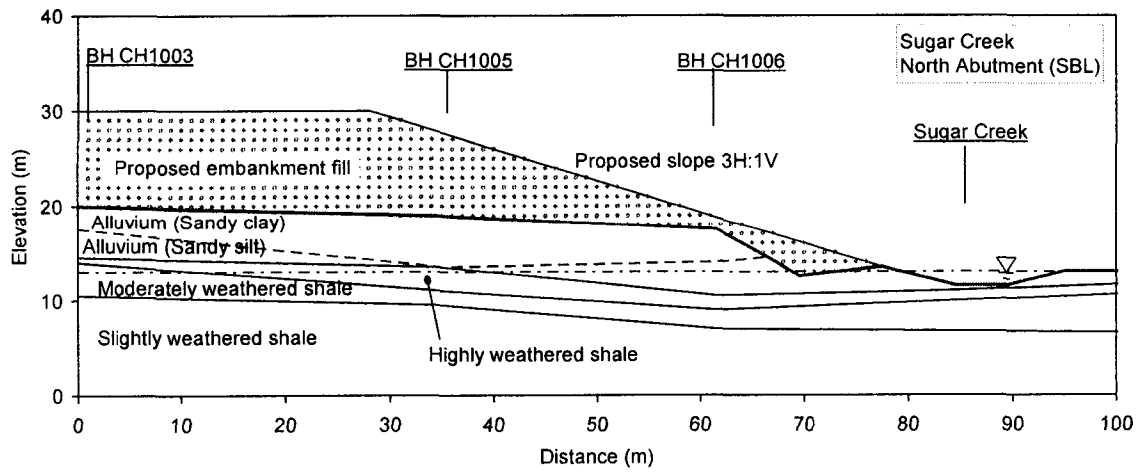
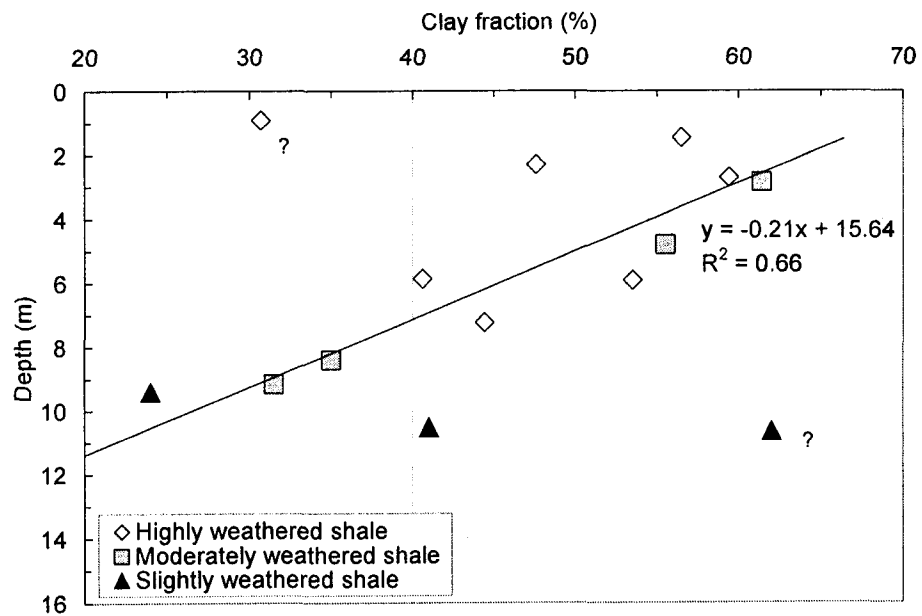
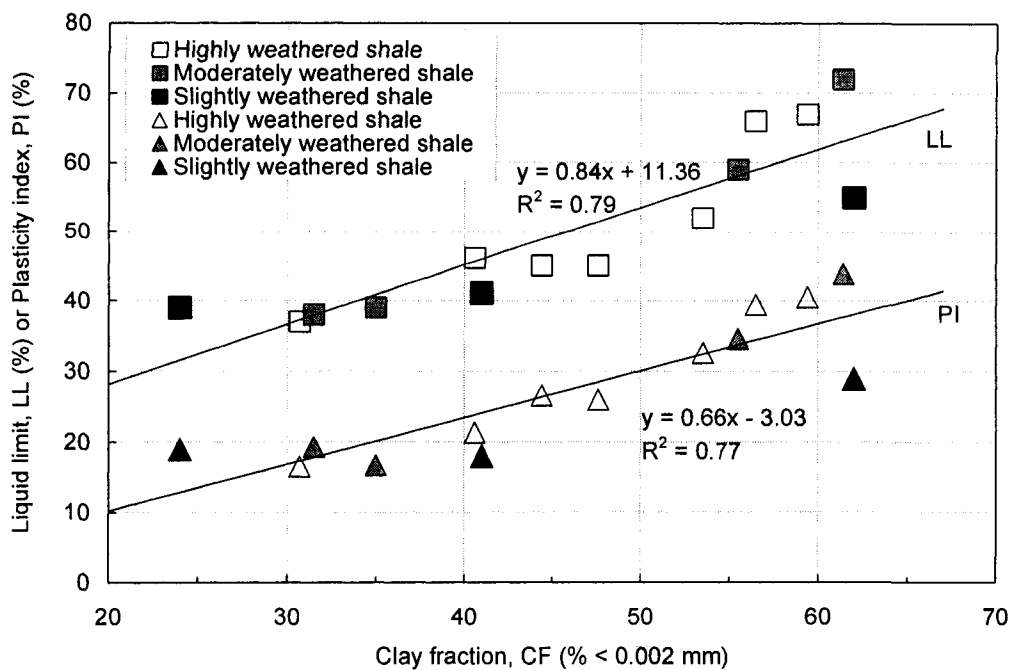


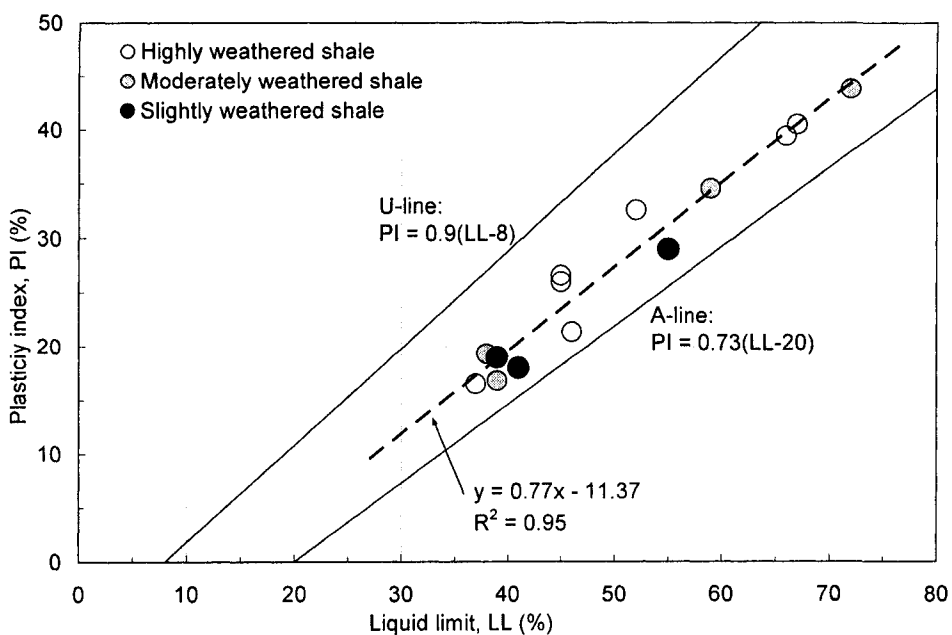
Figure 4. 2 Soil profile for the embankment slope



(a)



(b)



(c)

Figure 4. 3 Index property and classification data for the shales.

(a) Clay fraction versus depth; (b) Atterberg limits versus clay fraction; (c) Classification.

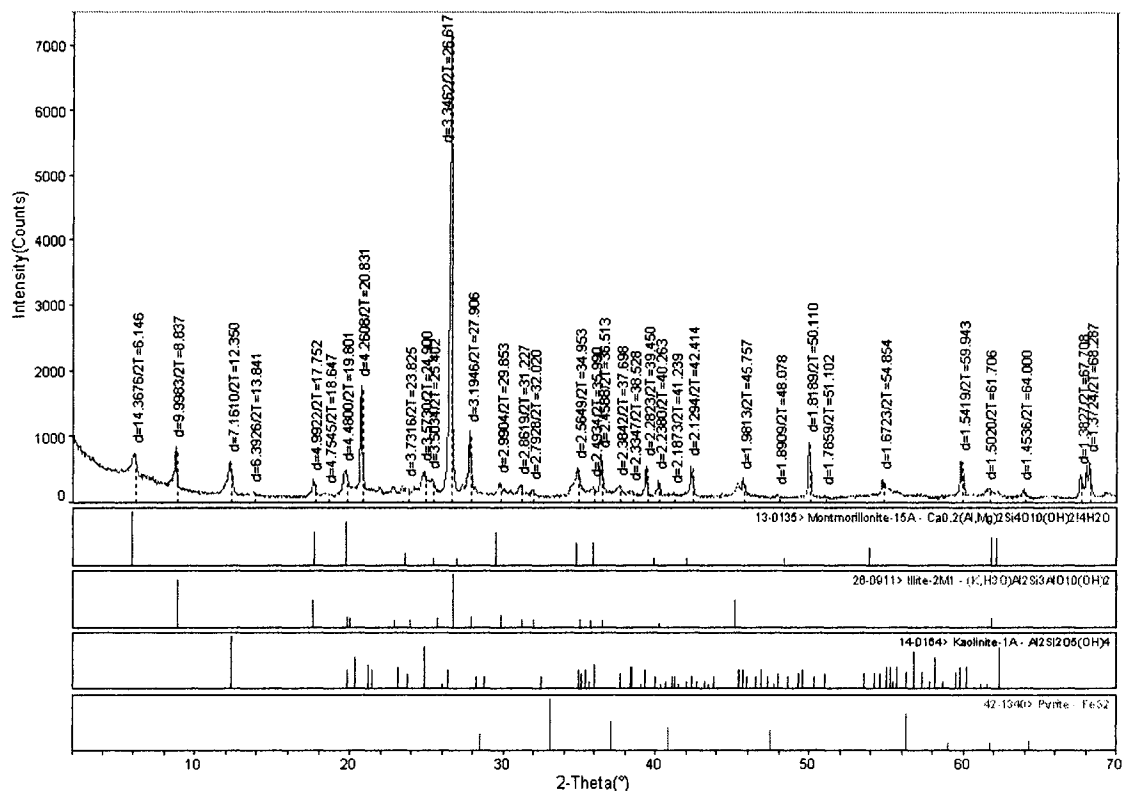
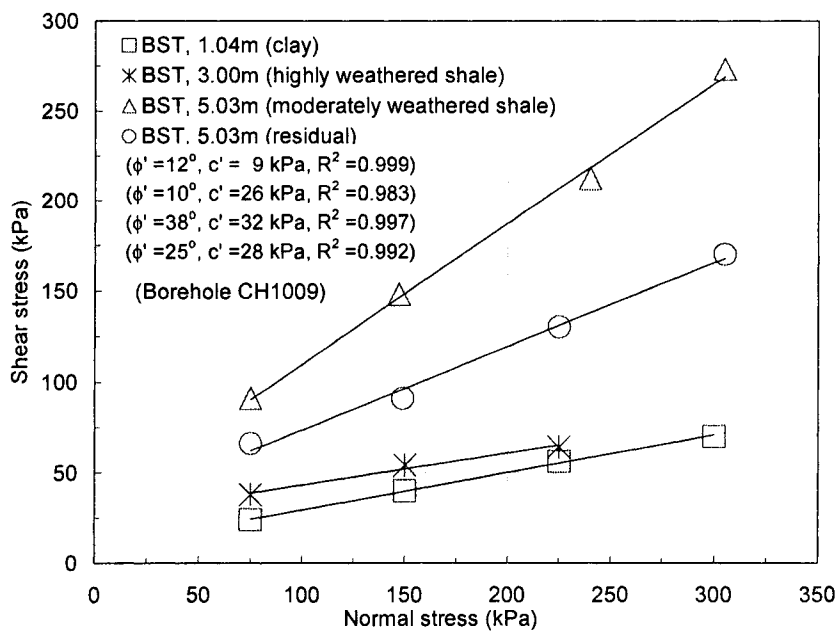
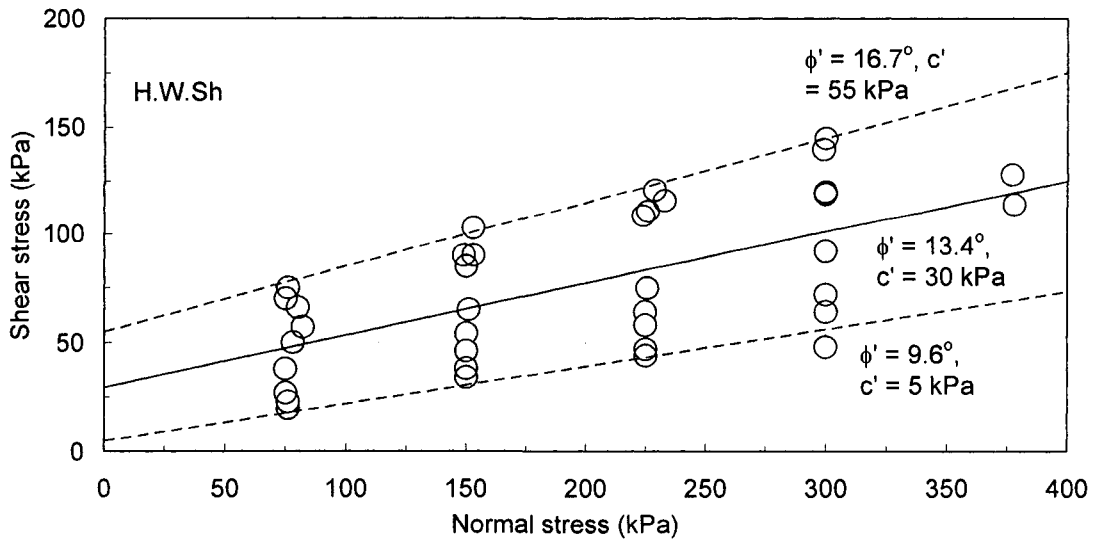


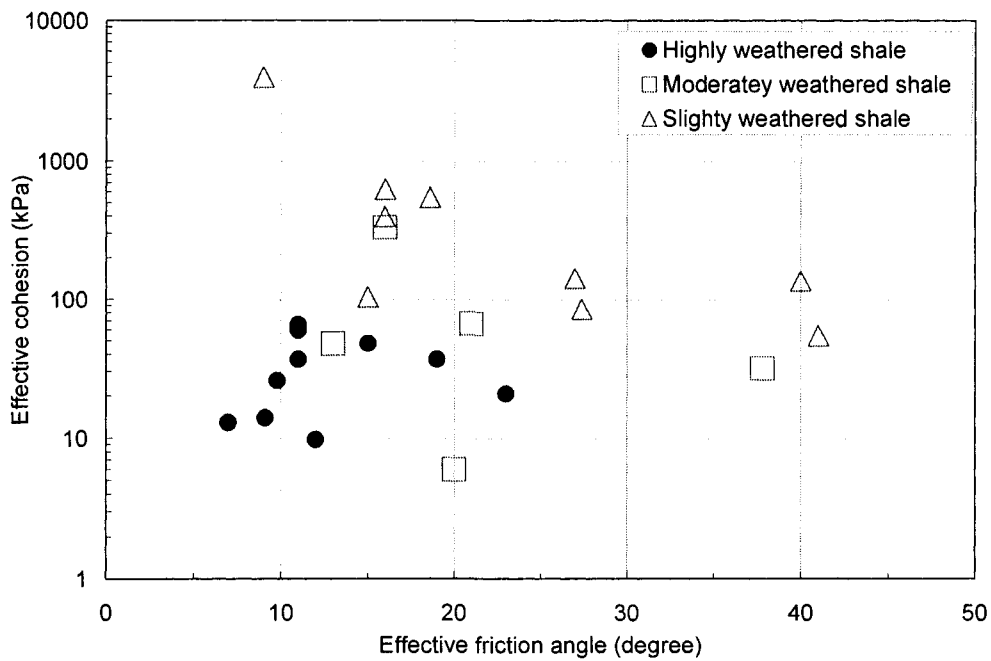
Figure 4. 4 X-ray diffraction for H.W.Sh from depth of 1.2 m in borehole CH1010



(a)



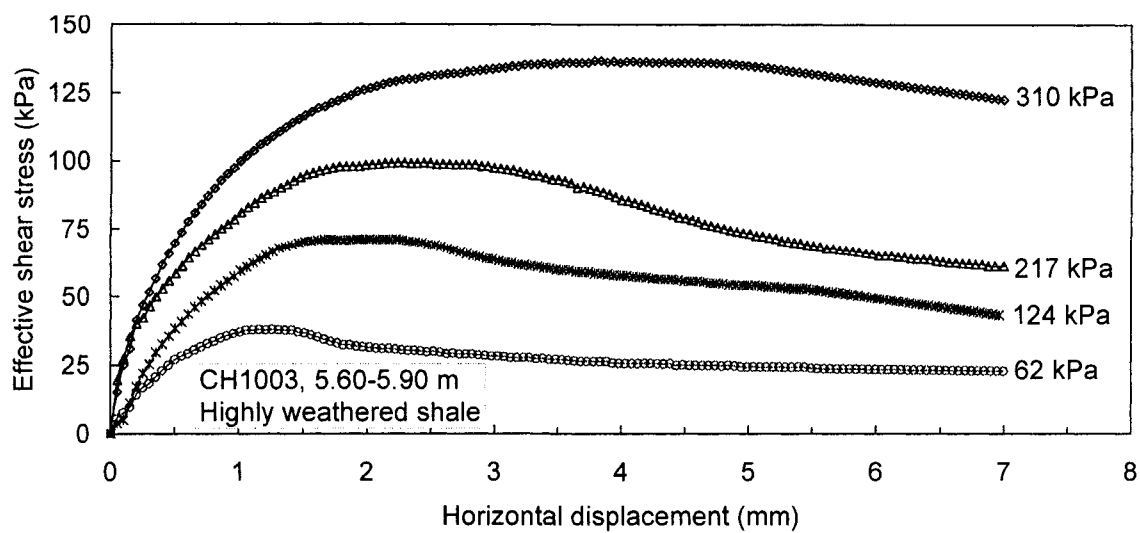
(b)



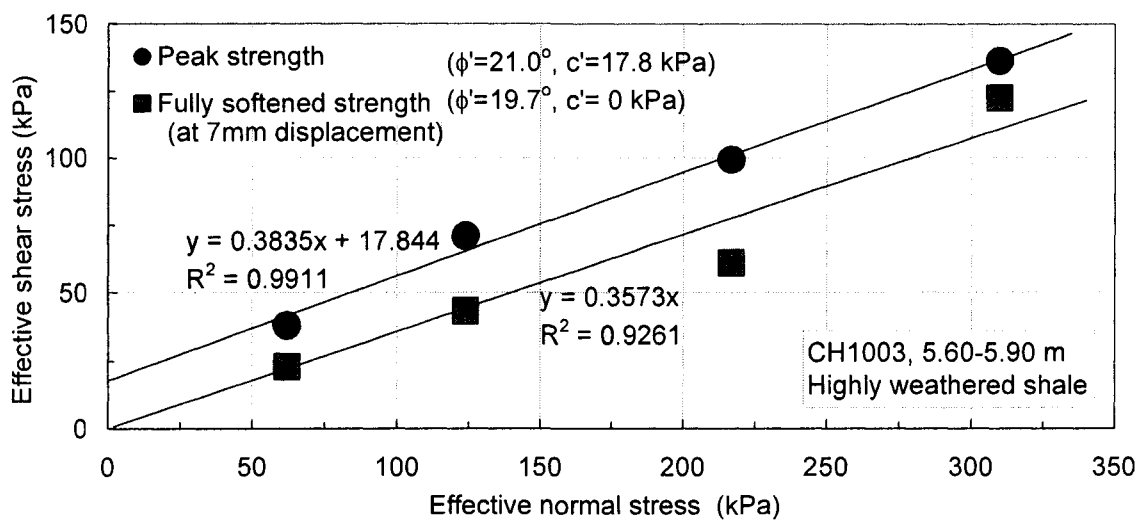
(c)

Figure 4. 5 Borehole Shear Test results

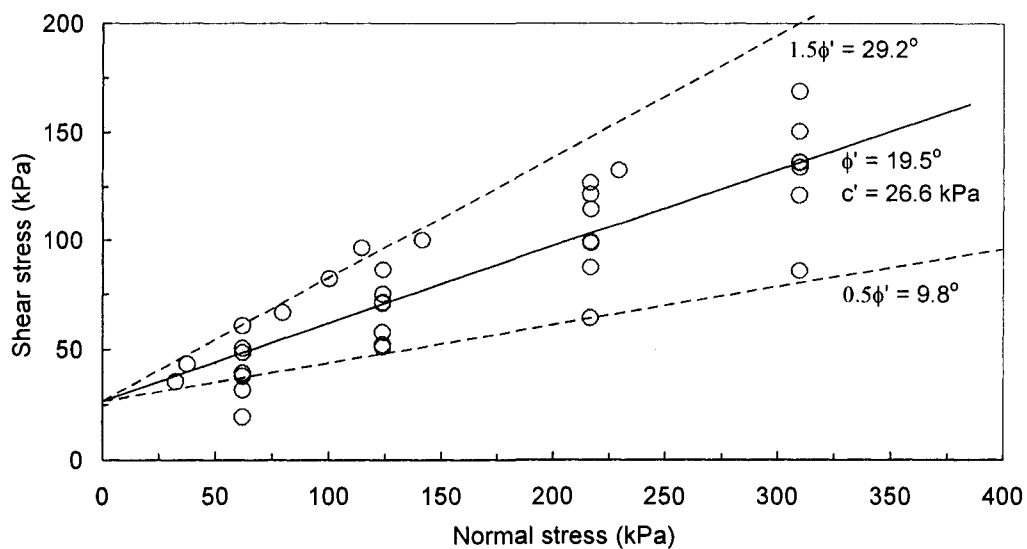
(a) Examples of BST results; (b) BST results for all the highly weathered shales; (c) BST shear strength parameter values for all the shales.



(a)

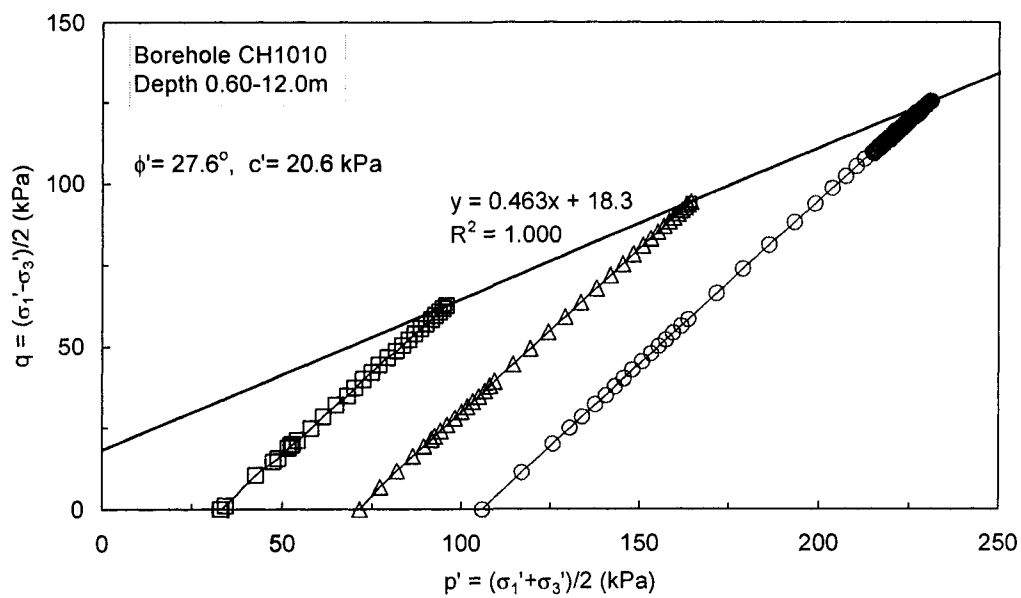


(b)

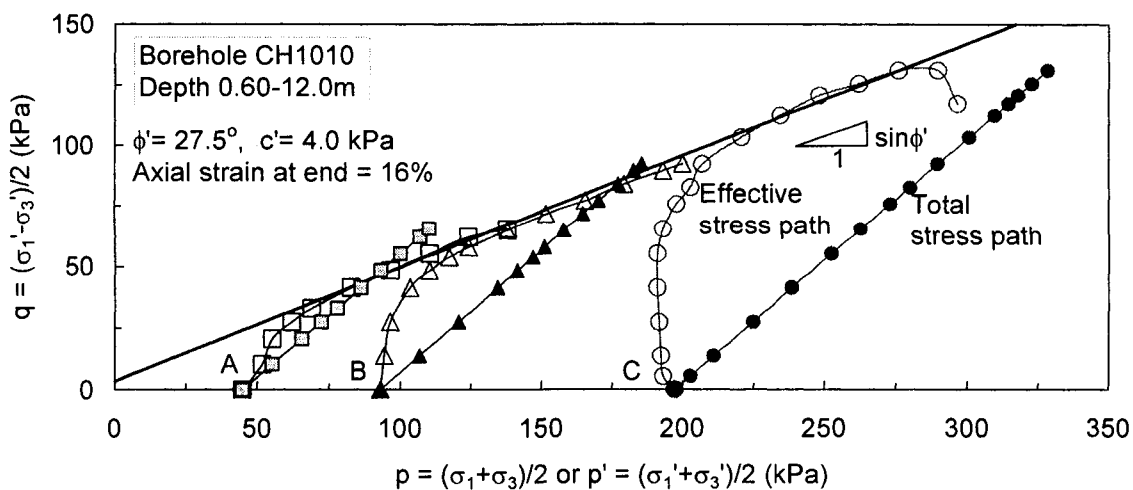


(c)

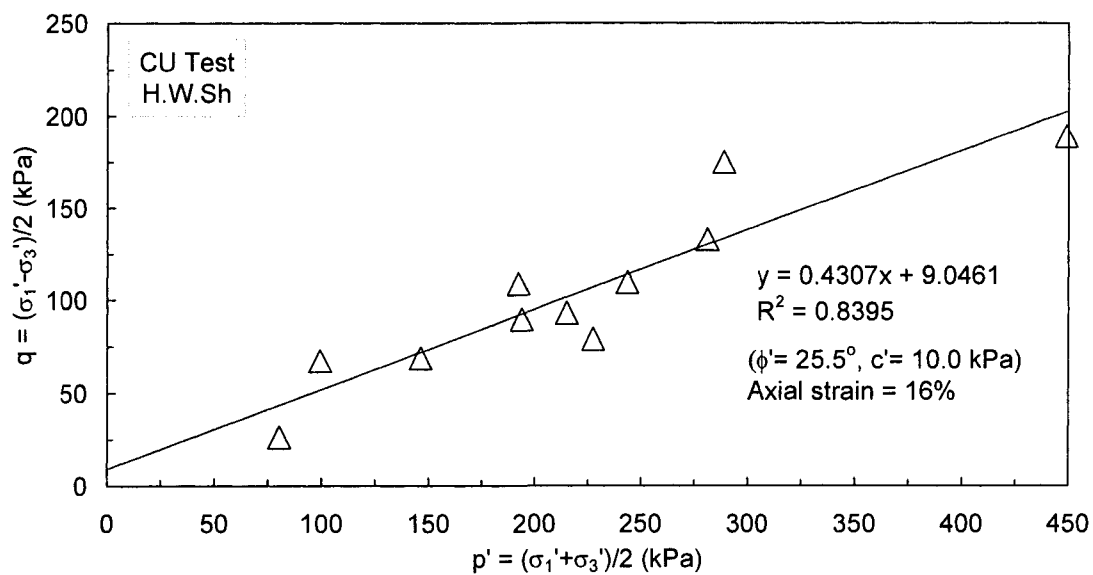
Figure 4. 6 Direct shear test results for the highly weathered shale
 (a) Typical curves of shear stress versus displacement; (b) Strength envelopes; (c)
 Results of direct shear tests for all the highly weathered shales.



(a)



(b)



(c)

Figure 4. 7 Results of triaxial compression tests for the highly weathered shale
(a) Effective stress paths in consolidated drained test (CD); (b) Effective and total stress paths in consolidated undrained test (CU); (c) p' - q plot for all the CU tests.

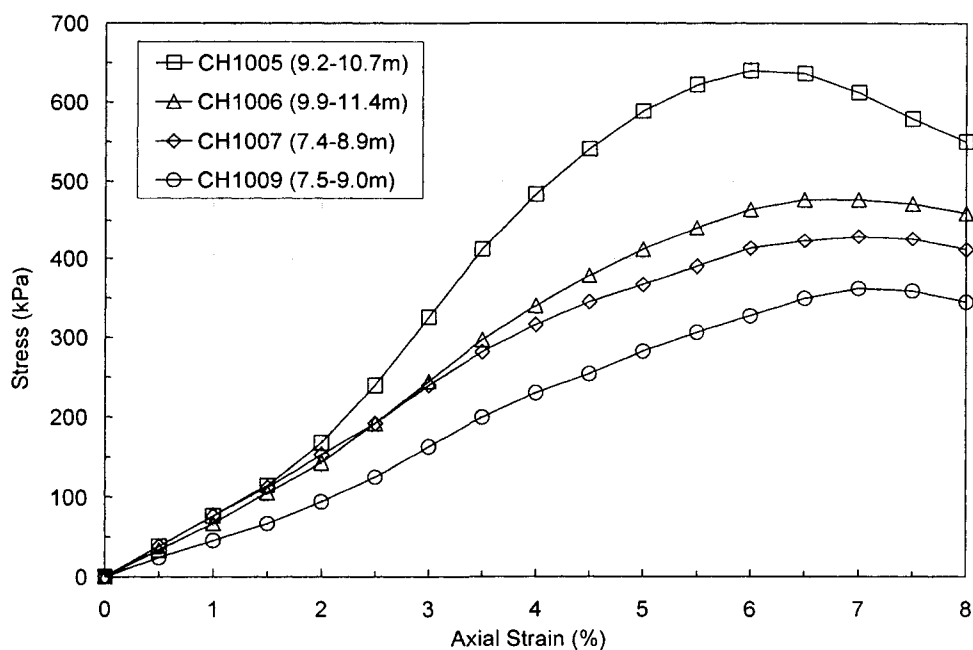
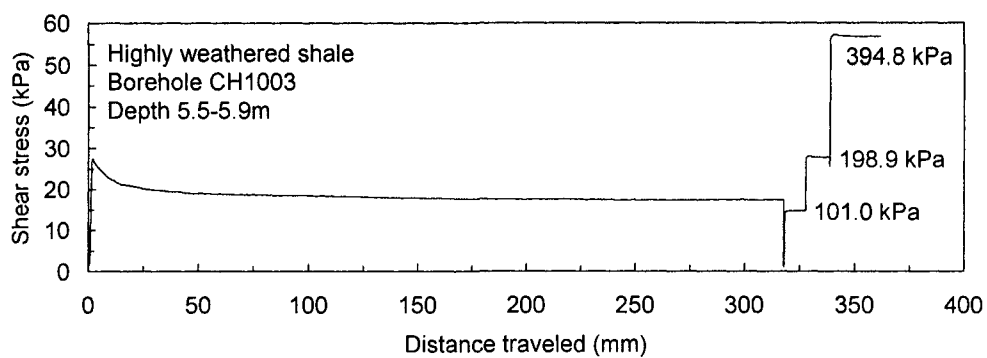
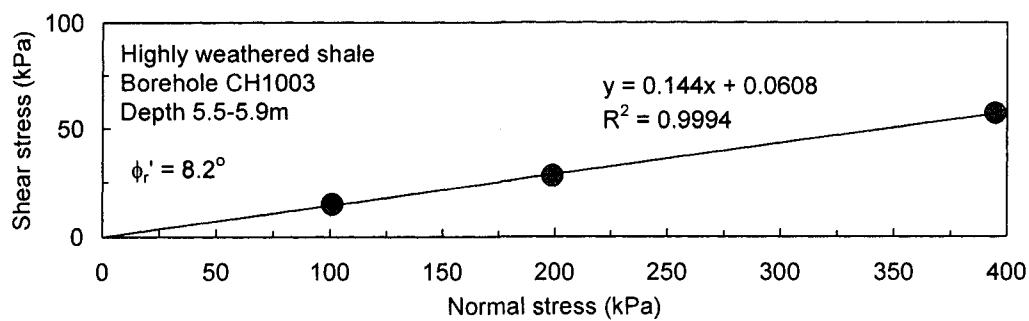


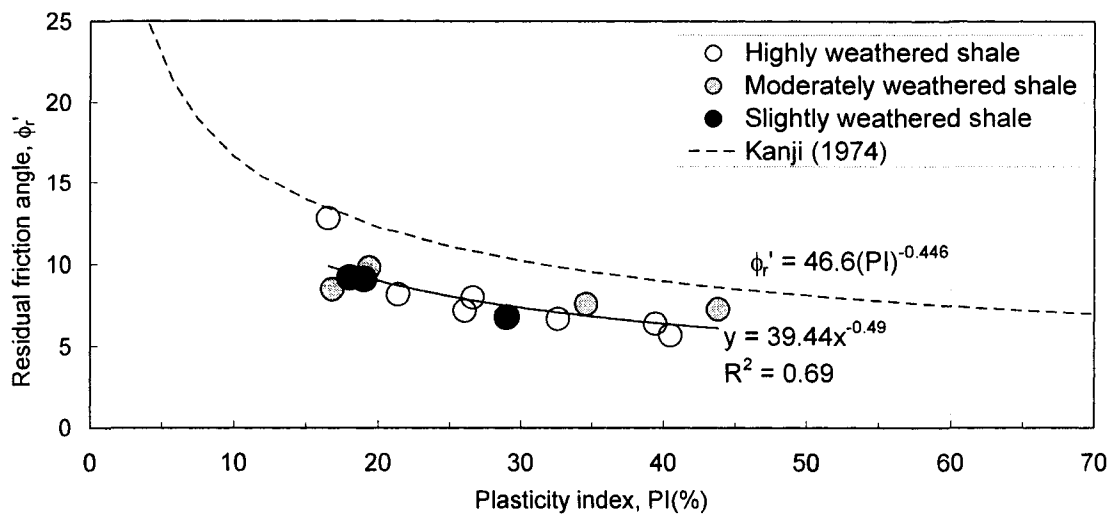
Figure 4. 8 Unconfined compression test results for the slightly weathered shales



(a)



(b)



(c)

Figure 4. 9 Ring shear test results for the highly and moderately weathered shales
 (a) Shear stress versus displacement; (b) Failure envelope; (c) residual friction angle versus PI for all the weathered shale samples.

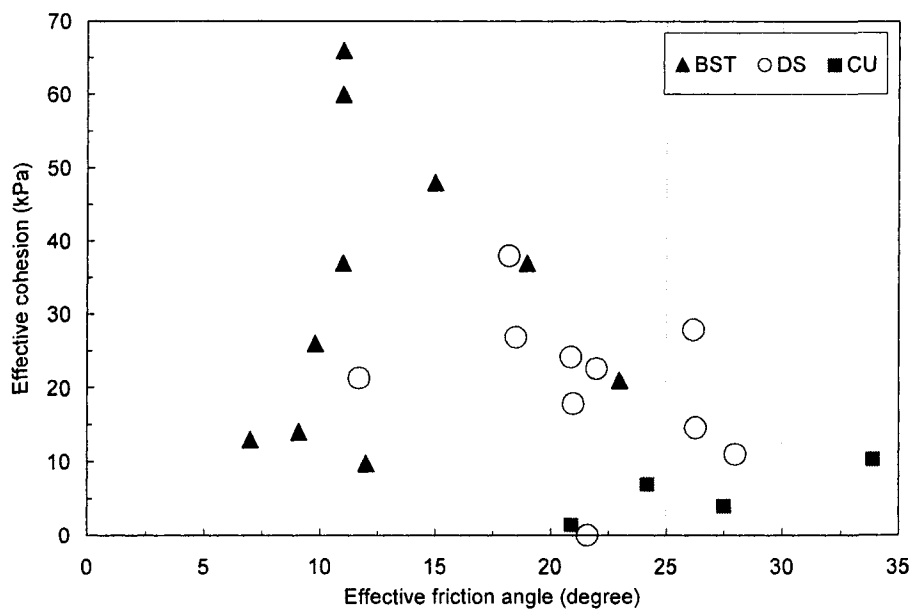


Figure 4. 10 Comparison of shear strength parameter values for the highly weathered shales

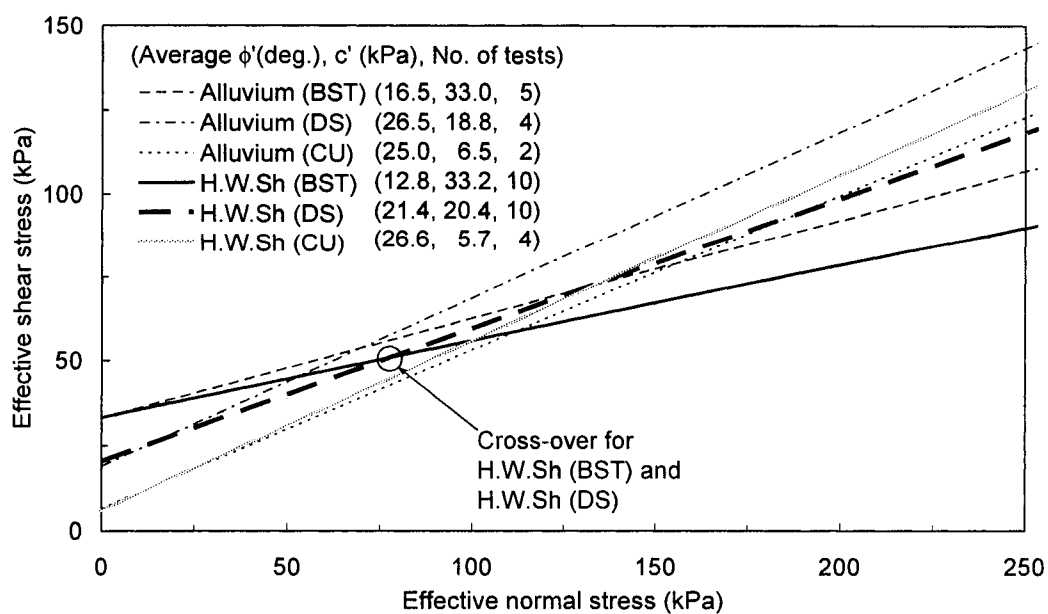


Figure 4. 11 Comparison of the average shear strength envelopes of different test methods

Table 4. 1 Statistics of Borehole Shear Test (BST) Results

| Soil | Total No. of Test | ϕ' (deg.) | | | | 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 |

Max. = maximum value

Ave. = Average value

Min. = Minimum value

S.D. = Standard deviation

Table 4. 2 Statistics of Direct Shear Test (DS) Results

| Soil | Total No. of Test | Friction angle, ϕ (deg.) | | | | Cohesion, c (kPa) | | | |
|----------------------------|-------------------------|-------------------------------|------|------|------|---------------------|------|------|------|
| | | Max. | Min. | Ave. | S.D. | Max. | Min. | Ave. | S.D. |
| Alluvium | 4 | 31 | 23 | 26.5 | 3.8 | 27 | 14 | 18.8 | 5.8 |
| Highly weathered shale | 10 | 28 | 12 | 21.4 | 4.8 | 38 | 0 | 20.4 | 10.4 |
| Moderately weathered shale | 6 | 29 | 14 | 19.4 | 5.7 | 43 | 5 | 23.6 | 15.6 |

Max. = maximum value Ave. = Average value
Min. = Minimum value S.D. = Standard deviation

Table 4. 3 Statistics of Consolidated Undrained Triaxial Test (CU) Results

| Soil | Total No. of Test | Friction angle, ϕ (deg.) | | | | Cohesion, c (kPa) | | | |
|------------------------|-------------------------|-------------------------------|------|------|------|---------------------|------|------|------|
| | | Max. | Min. | Ave. | S.D. | Max. | Min. | Ave. | S.D. |
| Alluvium | 2 | 30 | 20 | 25.0 | 7.1 | 13 | 0 | 6.5 | 9.2 |
| Highly weathered shale | 4 | 34 | 21 | 26.6 | 5.5 | 10 | 1 | 5.7 | 3.8 |

Max. = maximum value Ave. = Average value
Min. = Minimum value S.D. = Standard deviation

CHAPTER 5. PROBABILISTIC ANALYSIS OF A CLAY SHALE EMBANKMENT SLOPE USING IN-SITU AND LABORATORY STRENGTH PARAMETER VALUES

A paper submitted to *Geotechnical and Geological Engineering*

Hong Yang¹, David J. White², and Vernon R. Schaefer³

ABSTRACT

Probabilistic slope stability analyses were performed on the Sugar Creek embankment slope project in Iowa. Shear strength parameter values from two independent sources of *in-situ* Borehole Shear Test (BST) and laboratory direct shear test (DST) were used in the analyses assuming normal distributions. Both circular and non-circular slip surfaces following the Morgenstern-Price method and the Bishop simplified method were used for comparisons. The results show that the location of the critical slip surface using the BST shear strength parameter values is different from that using the DST values. The calculated factors of safety against slope instability are slightly smaller and the probability of failure is higher, when using the BST values compared to the DST values. The difference in results is due to BST measurements providing a lower mean value but larger variability than the DST measurements. With respect to the assumed slip surfaces, the non-circular critical slip surfaces gave lower factors of safety, but the circular “critical” slip surfaces gave higher probability of failure, indicating inconsistency in the location of the “critical” slip surface resulted from the variability in the input parameters. The use of the two independent sources of shear strength parameter values provided a comparison and check for the evaluation of the slope stability and probability of failure. The paper represents the first detailed analyses and application of *in-situ* BST results in a probabilistic slope stability analysis.

Keywords: Probability; Reliability; Slope stability; Borehole Shear Test; Strength parameter; Slip surface.

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INTRODUCTION

Slope stability assessment is generally a difficult geotechnical problem because of the uncertainties involved. In fact, slope engineering is perhaps the geotechnical subject that is most dominated by uncertainties (El-Ramly et al. 2002). The sources of uncertainty involved in slopes include spatial uncertainties (such as site stratigraphy and variability) and data uncertainties (such as soil engineering properties) (Abramson et al. 2002), and “the uncertainty in the values of the soil properties is a major, in most cases the major, contributor to the uncertainty in the stability of slopes and embankments” (Christian et al. 1992). Conventional approaches of deterministic slope stability analysis do not account for quantifying the uncertainties in an explicit manner. Conventional approaches rely on the use of the factor of safety as defined by the ratio of the strength available over the strength mobilized to maintain equilibrium. Due to the recognition of the uncertainties involved in determining the available strength of the soil, the selection of a suitable value of factor of safety, which is used to offset the risk raised from the uncertainties, has been based on engineering judgment and local experience; and conservative parameters and designs are often used to manage the uncertain conditions. The impact of such subjective conservatism cannot be evaluated, and past experience indicates that apparently conservative designs do not always warrant safety against failure. Furthermore, the use of a single value of factor of safety can be misleading, as it is known that the slope calculated to have the same value of factor of safety may indeed pose different risk levels, depending on the degree of variability of the shear strength parameters.

The evaluation of the role of the uncertainties in soil parameters gave rise to the application of the probabilistic concepts and methods. Using probabilistic analyses, uncertainties can be quantified and included in the design process in a rational manner, and the probability of failure and the reliability index of the design can be assessed accordingly. Since 1970, when probabilistic slope stability analysis was first introduced into slope engineering (e.g., Alonso 1976; Tang et al. 1976), probabilistic concepts and principles have been well established in the literature. Some representative publications include Li and Lump (1987), Christian et al. (1994) and Hassan and Wolff (1999), among others. However, it appears that most of the probabilistic slope analyses in the literature utilize a single source of

shear strength parameters, i.e. the strength parameters are obtained from either *in-situ* tests or laboratory tests. For the *in-situ* tests, BST is the only method that provides direct measurements of the shear strength parameter values, i.e., the internal friction angle (ϕ') and cohesion intercept (c') (Handy 1986). Furthermore, most published studies consider only one of the few available deterministic models and commonly consider only circular slip surfaces (Hassan and Wolff 2000).

In view of the aforementioned inadequacy in probabilistic slope stability analysis in the literature, this study investigates the use of *in-situ* BST shear strength parameter values and laboratory direct shear test (DST) strength parameter values, the deterministic methods, and the locations of the critical slip surface, on the probability of failure and reliability index of the slope. The study was based on a recently proposed embankment slope project in Iowa, which involved multiple soil layers with a weak layer of highly weathered shale. For comparison to the *in-situ* BSTs, laboratory DSTs were performed on essentially the same soils for the project site. Thus, two independent sets of shear strength parameter values from different sources were obtained, and their statistical results assuming normal distribution were used for the probabilistic slope stability analyses. Two deterministic slope stability methods, i.e. the Bishop (1955) simplified method and the Morgenstern-Price (1965) method, were used; and both circular and non-circular slip surfaces were considered. The Monte Carlo approach (Tobutt 1982) was used for the probabilistic procedures, and the computations were executed using the computer program Slope/W (GEO-SLOPE 2004). Though Bishop simplified method was developed based on a circular slip surface, the method was implemented for non-circular slip surface in the Slope/W program. Different analyses were performed on the same slope section, and their results are discussed and compared.

THEORY OF PROBABILISTIC SLOPE ANALYSIS

Probabilistic slope stability analysis quantifies the probability of failure of a slope. The input parameters in a probabilistic analysis are the mean values of the parameters (such as ϕ' , c' and unit weight of the soil); and the variability of these parameters are represented by the standard derivations. The analysis results include the probability of failure and the reliability index.

Probability Density Function

Since soils are naturally formed materials, and their physical properties vary from point to point. The variability of soil properties is a major source of the uncertainty. 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 et al. 1993; and Baecher and Christian 2003), which is written as:

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

where $f(x)$ = relative frequency; σ = standard deviation; and μ = mean value. Eq. (1) is also known as the probability density function (PDF) for normal distribution. This distribution is symmetric about the mean. Theoretically, the normal curve will never touch the x axis, since the relative frequency, $f(x)$, will be non-zero over the entire range. This is a limitation of the model since the input parameters have finite values. However, for practical purposes, the relative frequency can be neglected after $\pm 5\sigma$ away from the mean value (GEO-SLOPE 2004).

Statistical Analysis of Factor of Safety

The component input parameters in a slope stability analysis are modeled as random variables, and used to estimate the PDF of factor of safety (FS). This PDF is characterized by its mean value and standard deviation. It is usually assumed that the PDF of FSs to be either normally or lognormally distributed. Statistical analysis can be conducted to determine the mean, standard deviation, the PDF and the probability distribution function of the slope stability problem. The equations used in the statistical analysis for the normal distribution case are summarized as follows (Lapin 1983):

$$\text{Mean factor of safety, } \mu: \quad \mu = \frac{1}{n} \sum_{i=0}^n F_i \quad (2)$$

$$\text{Standard deviation, } \sigma: \quad \sigma = \sqrt{\frac{1}{n} \sum_{i=0}^n (F_i - \mu)^2} \quad (3)$$

Probability density function:
$$f(F) = \frac{1}{\sigma\sqrt{2\pi}} \exp\left[-\frac{1}{2}\left(\frac{F-\mu}{\sigma}\right)^2\right] \quad (4)$$

Probability distribution function:

$$f(F) = P[X \leq F] = \int_{-\infty}^F \left\{ \frac{1}{\sigma\sqrt{2\pi}} \exp\left[-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2\right] \right\} dx \quad (5)$$

where F_i = the trial factor of safety; n = number of trial factors of safety; and F = factor of safety.

Probability of Failure and Reliability Index

Two indices - probability of failure (PF) and reliability index (RI) - are available to quantify the stability or the risk level of a slope with the probabilistic slope analysis (Christian et al. 1994). The probability of failure is the probability of obtaining a FS less than 1.0. It is computed by integrating the area under the PDF for FSs less than 1.0. The PF 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 PF is a good index showing the actual level of stability of a slope.

The RI may provide a more meaningful measure of stability than the FS. The RI provides a measure of how much confidence one can have in the computed value of FS and leads to an estimate of the PF. The RI (β) is defined in terms of the mean (μ) and the standard deviation (σ) of the trial FS (Christian et al. 1994). For a normal distribution of PDS of FS, β is given by:

$$\beta = \frac{|\mu - 1.0|}{\sigma} \quad (6)$$

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

Probabilistic Slope Analysis Procedures

Probabilistic procedures for slope stability analysis can vary in assumptions, limitations, capability to handle complex problems, and mathematical complexity. Most of the procedures, however, can be categorized as either the approximate method (such as the Taylor series method; the First Order Second Moment method; the point estimate method, etc.); or the Monte Carlo simulation (El-Ramly et al. 2002). Approximate methods make simplifying assumptions that often limit their application to specific classes of problems. They allow the estimation of the mean but do not provide any information about the shape of the PDF of FS.

Monte Carlo simulation in slope analysis requires extensive computational efforts to solve slope stability problems, but the rapid developments in software provide increased use of this method. The computational procedures are described by Tobutt (1982), Hammond et al. (1992) and Chandler (1996) among others. The method 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 FSs many times; (4) the determination of some statistics of the computed FS, the probability density and the probability distribution of the problem (GEO-SLOPE 2004).

In the Slope/W program (GEO-SLOPE 2004), 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. During each Monte Carlo trial, the input parameters are updated based on a normalized random number. FSs are then computed based on these updated input parameters. By assuming that the FSs are also normally distributed, the mean and the standard deviations of the FSs are determined. The PDF 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 PF. The number of required trials increases as the number of variable inputs increases or the expected PF becomes smaller. It is not unusual to

calculate thousands of trials to achieve an acceptable level of confidence in a Monte Carlo probabilistic slope stability analysis (Mostyn and Li 1993). According to Harr (1987), to achieve results not differing by more than 1% from the estimated value with 99% confidence, the Monte Carlo procedure requires more than 16,000 trials. Giasi et al. (2003) performed Monte Carlo simulation by increasing the Monte Carlo trials from 10 to 20,000 and showed that Monte Carlo solution became stable after the number of trials was larger than 10,000. Thus, it has been assumed that 20,000 Monte Carlo trials for each simulation would be sufficient in this study.

CASE STUDY OF PROBABILISTIC SLOPE ANALYSIS

Project Background and Geological Conditions

Approach embankment fills with pile-supported abutments were designed prior to this study to support the proposed new bridge of Highway US 63 over Sugar Creek in Wapello County, Iowa, USA. Preliminary slope analyses indicated potential global instability for the slopes in front of the abutments, with a slip surface passing through the highly weathered shale, when using a cohesion of 10 kPa assumed by the design engineers. This approach resulted in high cost implications for the proposed ground improvement measures. To achieve an economical design, a comprehensive subsurface exploration and testing program was executed in 2004 at a small cost to supplement the preliminary investigation. The program comprised extensive *in-situ* Borehole Shear Tests (BST), laboratory direct shear tests (DST) on undisturbed samples, as well as other tests. Based on the actual shear strength of the soils obtained, the stabilities of the slopes were re-evaluated and substantial savings from the proposed ground improvement measures are expected to be realized.

The geotechnical investigation showed that the subsurface of the project site can be generally grouped into four layers. The first layer mainly consists of Alluvium soils of silt and clay or their mixtures with small amounts of sand and gravel. It 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 three layers of shale were distinguished based on the results of field visual examination and the variation of the soil

strength tested in the field and laboratory. In general, H.W.Sh had a relatively low strength, S.W.Sh had a relatively high strength; and M.W.Sh represented the transition between H.W.Sh and S.W.Sh. The boring results indicated that the spatial distributions of the soil layers were highly variable. Ground water table gently dipped towards and connected with the creek. A typical slope section showing the soil profile is presented in Figure 5.1.

Basic properties for representative soil samples were investigated with emphasis on the shales. The results show that clay fraction for the H.W.Sh and M.W.Sh ranged from 30 to 65%, liquid limit varied between 35 and 75%, and plastic limit varied from 15 to 45%. All the shales were classified as either low plasticity clay (CL) or high plasticity clay (CH) according to Unified Soil Classification System (ASTM 2002a). The complete details of the geotechnical investigation results have been reported by Schaefer et al. (2005).

In-Situ Borehole Shear Test Results

The fundamental consideration for the Borehole Shear Test (BST) is to perform a series of direct shear tests on the inside of a borehole (Handy 1986), and provide direct measurements of shear strength parameter values, which is unique among various *in-situ* testing methods. The BST device is shown in Figure 5.2. Basically, tests are conducted by expanding diametrically opposed contact shear plates into a borehole under a constant known normal stress, then allowing the soil to consolidate, and finally by pulling vertically and measuring the shear stress *in-situ* separately and concurrently (Handy 1986). The maximum shear resistances are measured at successively higher increments of applied normal stress. The shear strength parameter values of the soil are then obtained from the Mohr-Coulomb failure envelope by plotting the shear stresses versus the normal stresses. Complete descriptions of the test procedures for the BST can be found in the literature (e.g. Lutenecker 1987). The BST is normally representative of a consolidated-drained test, which has been demonstrated by pore pressure measurements during the test (e.g. Demartinecourt and Bauer 1983). The rate of the shear head displacement is generally 0.05 mm/s (Wineland 1975). The BST has also been proved simple to operate. Lutenecker and Timian (1987) investigated the reproducibility of BST results in soft and medium consistency marine clays. They found that there was no difference in the test results between an experienced operator and an

inexperienced operator; and test results obtained by ten inexperienced operators generally fell within 95% confidence limits.

In this project, a total of 29 BSTs were performed in different soil layers in different boreholes, with emphasis on the H.W.Sh. All the results show that the tests were well performed, as revealed by the large values of coefficient of correlation (R^2) between the shear stresses and the normal stresses, which were generally larger than 0.99. The plots of shear stress versus normal stress of BST for all the H.W.Sh are shown in Figure 5.3. The results indicate a significant variation of the shear strength parameter values of the soil. Mohr-Coulomb failure envelopes were provided by using a constant friction angle and varying the cohesion intercept (c'). The value of c' was assumed to be zero when it was negative after fitting with normal distribution.

The histograms of the shear strength parameter values are presented in Figure 5.4, together with the fit curves showing the normality of the results. A relatively large difference between the measured and the fit curve of the cumulative distributions can be noted for the friction angles, while such difference is relatively small for the cohesion. The difference between the measured and fit values can be attributed to the small number of test data, i.e. ten data points in this case. This inadequacy can only be overcome by obtaining sufficiently large amount of data points, though there is always a limitation on the available amount of data in engineering practice. Nevertheless, the general agreement of the trend of the cumulative distribution between the measured and fit curves is still a good indication on the validity of the normality. These normal distributions are used as input parameters for the probabilistic slope analyses, which are summarized in Table 5.1 based on the statistics of the test results. From Table 5.1, it can be seen that the general trend that the shear strength values increase with the increase in depth is apparent, despite the variation of the shear strength parameters values. H.W.Sh generally has low shear strength values; and S.W.Sh generally have high shear strength values, mainly exhibited by the much higher cohesions. M.W.Sh have shear strength values between H.W.Sh and S.W.Sh indicating a transition layer.

Laboratory Direct Shear Test Results

The direct shear test (DST) has become a routine method to obtain soil strength parameters due to its simplicity. The DST strength parameters can be different from those obtained from more elaborate tests such as triaxial compression, but the difference is normally small (Lambe and Whitman 1979). In fact, DST and triaxial compression tests should give essentially same results for the same soil. This was affirmed by Lambe and Whitman (1979), who reported that comparisons between the value of ϕ' , from triaxial and direct shear tests, after averaging out experimental errors in the determination of the values, yield results that differ generally by no more than two degrees. Therefore, DST strength parameters were also used for the slope analyses in the study as a comparison to the BST strength parameters. The DST was performed on undisturbed Shelby tube soil samples following the procedures of ASTM D3080 (ASTM 2002b). Each test was performed on three to five specimens. The size of the specimens was typically 63.5 mm in diameter and 20.1 mm in height. The loading rate of the shear force was normally 0.025 mm/min.

A total of 20 consolidated drained DSTs were performed which included four tests on the alluvium soils, ten tests on H.W.Sh, and six tests on the M.W.Sh. No DST was performed on S.W.Sh as the soil was too stiff to obtain undisturbed Shelby tube samples. The test results show that R^2 values are generally larger than 0.99 indicating the effectiveness of the tests, though R^2 values as low as 0.91 were also observed for a few tests, which are attributed to inherent soil sample variability. The DST results for all the H.W.Sh are presented in Figure 5.5, which show the variation of the soil shear strength measurements and Mohr-Coulomb failure envelopes assuming a constant friction angle. Histogram plots of the shear strength parameter values for the H.W.Sh and the fit cumulative normal distribution curves are shown in Figure 5.6. Similar to those for the BST measurements, differences are observed between the measured and the fit cumulative distribution curves. However, the general agreements on the trends of the curves are apparent and indicate the validity of the normality of the test results. The variability of the shear strengths of the soils are also illustrated by the statistical results summarized in Table 5.2.

It can be seen that the numbers of BST and DST are similar for the Alluvium, H.W.Sh and M.W.Sh (Tables 5.1 and 5.2). This provides the basis to compare the statistical

results of the two different types of tests and the associated slope stability analysis results. The statistical results show that the strength parameter values from the two types of tests are quite different. This difference can be attributed to differences in test methods and inherent soil variability. The BST shears a single soil sample vertically, while DST shears multiple samples horizontally and averages three or more soil samples.

Slope Stability Analysis Results and Discussions

Input Parameters for the Analyses

The two sets of the strength parameters, i.e. those from the BST (Table 5.1) and those from the DST (Table 5.2), were used for the slope stability analyses separately. Since no DST strength parameters were available for the layer of S.W.Sh, they were assumed to be the same as the BST strength parameters for the layer. This assumption was reasonable as the strength of the S.W.Sh was much higher than the overlying soils; and the S.W.Sh essentially had no effect on the slope stability analysis, which is supported by the fact that the critical slip surface does not pass through the layer as shown in the following analyses. For the compacted fill of the embankment soil, the mean values of strength parameters of ϕ' of 12° and c' of 29 kPa, as recommended by Iowa Department of Transportation (IaDOT), were adopted for all analyses. The standard deviations were assumed to be 0.4° for ϕ' and 2.0 kPa for c' , which were considered reasonable, since the quality of the compaction can be relatively well controlled during construction. In addition, the soil unit weight were 20.4, 19.0, 20.0, 20.0 and 21.0 kN/m³ for the five soil layers with increasing depth, which were either recommended by IaDOT for the compacted fill or measured for the Alluvium and the three shale layers. The standard deviation of unit weight was assumed to be 0.3 kN/m³ for all five layers. For the water table in the slope, the highest water table level was assumed to be the maximum water level in the creek according to the estimated 500 years flood event; the lowest water table level was assumed to be the ground water table as measured during the field investigation; and the average water level was assumed to be the mean water level (Figure 5.1). The difference between the highest and the lowest water level was about 3.2 m.

Circular and Non-circular Slip Surface

Two types of slip surfaces, namely circular and non-circular slip surface, were considered in the slope stability analyses in the study. In the Slope/W program (GEO-SLOPE 2004), the circular slip surface is generated by assigning a series of trial centers of circles and radiuses, and the factor of safety (FS) is computed on each of the trial circular slip surfaces to search for the critical slip surface which has the lowest FS. For the non-circular slip surface, the “auto-locate slip surface” method that is built-in in Slope/W was adopted. A range of trial slip surfaces is analyzed over the problem extents, and the slip surface shape is optimized using an efficient statistical random walk procedure to obtain the lowest FS.

The locations of both the circular and non-circular critical slip surfaces are dependent on the soil strength parameter values, as can be seen in Figure 5.7. The figure indicates that the critical slip surfaces corresponding to BST strength parameters (BST slip surfaces) pass through the bottom of the H.W.Sh, while the DST slip surfaces pass through the bottom of M.W.Sh. DST slip surfaces were deeper than the BST slip surfaces. This is because the difference of the BST strength parameter values between H.W.Sh and M.W.Sh is significant (Table 5.1); while the difference of DST strength parameter values between the H.W.Sh and M.W.Sh are relatively small (Table 5.2).

Deterministic Slope Stability Analyses

Deterministic slope stability analyses were first performed to obtain the lowest FS (or the deterministic FS). The analysis was performed based on the mean values for all the soil properties and the mean ground water table level. Eight analyses in total were performed considering two sets of strength parameter values, two types of slip surfaces (Figure 5.7) and two analysis methods; and the results are presented in Table 5.3 (under the column D.FS).

A few observations can be made from the results. Firstly, FS values range from 1.521 to 1.587 when using BST strength parameters; and the FS values range from 1.599 to 1.624 when using DST strength parameters. These results demonstrate the influence of the lower mean values for the BST strength parameters compared to the mean values for the DST strength parameters, which resulted in lower FS values. Secondly, FS values for non-circular slip surface are consistently lower than FS values for circular slip surface, for the cases of using both BST and DST strength parameters. This suggests that the non-circular slip surface

is more critical than the circular slip surface for the slope involving layered soils. Thirdly, the difference of FS values are small when comparing the MP method and the Bishop method for the circular slip surface (i.e., 1.576 and 1.587 when using BST parameters; 1.620 and 1.624 when using DST strength parameters), but the difference of FS values are relatively large between the MP method and the Bishop method for non-circular slip surface (i.e., 1.540 and 1.521 when using BST parameters; 1.599 and 1.610 when using DST strength parameters). This could be due to the fact that the MP method satisfies all conditions of equilibrium; while the Bishop method satisfies vertical equilibrium and overall moment equilibrium only, it does not satisfy horizontal equilibrium. The Bishop method is normally recommended for circular slip surface (Abramson et al. 2002). Therefore, for the non-circular slip surface, MP method should be more accurate. Nevertheless, the results from the Bishop method still provide a comparison.

Probabilistic Slope Stability Analyses

A total of eight probabilistic slope stability analyses were performed corresponding to the deterministic slope stability analyses. All the soil properties and ground water levels were assumed to be of normal distributions; and the standard deviations as shown in Tables 5.1 and 5.2 were used. The probability density functions (PDFs) of FS from the analyses are presented in Figure 5.8, and the cumulative distribution functions of FS are presented in Figure 5.9. The results are also summarized in Table 5.3 (under the column Probabilistic Analysis I).

A few observations can be made from the results of the analyses. Firstly, use of the BST strength parameters generally resulted in lower mean FS values than the use of the DST strength parameters, as indicated by the FS values at the peaks of PDF curves (Figure 5.8) and mean FS values in Table 5.3 (shown as M.FS, which are 1.524 to 1.601 versus 1.615 to 1.660). These FS values are consistent with the deterministic FS values, as the mean FS values are mainly dependent on the mean values of the input parameters. The discrepancy of corresponding FS values was attributed to the analysis procedures involving Monte Carlo simulations. Secondly, the use of BST strength parameters generally gave smaller reliability indexes than the use of DST strength parameters (Table 5.3, i.e. 1.422 to 1.733 versus 2.163

to 2.479), which are also indicated by the flatter PDF curves (Figure 5.8). The flatter PDF curves are due to the larger standard deviations of FS, which resulted in smaller reliability index values (see Equation 6). These results reflect the fact that the overall variability of BST strength parameters are larger than those of DST strength parameters. Thirdly, corresponding to the smaller reliability index values, the use of the BST strength parameters generally resulted in higher probability of failure (PF) (Table 5.3, i.e. 3.68 to 7.78% versus 0.40 to 1.45%), which are also indicated by the magnitude of cumulative probability having FS values of 1.0 (Figure 5.9).

The probabilistic analysis results also show the effect of the slip surface and the analysis method. For the same slip surface, either circular or non-circular, PF obtained by the Bishop method (Analyses 2, 4, 6 and 8, Table 5.3) is always larger than those obtained by the MP method (Analyses 1, 3, 5 and 7). However, the PF values for the circular slip surface are relatively close when using the MP method and the Bishop method (e.g., Analyses 1 and 2; Analyses 5 and 6); the PF values for the non-circular slip surface are relatively different when using the MP method and Bishop method (e.g., Analyses 3 and 4; Analyses 7 and 8). The difference of PF values on the non-circular slip surface between the use of MP method and the Bishop method, especially that for Analyses 3 and 4, could be again due to the limitation of the Bishop method. Consequently, the results for the non-circular slip surface obtained from the MP method are considered to be more accurate.

Probabilistic Slope Stability Analyses with Emphasis on H.W.Sh

In order to estimate the contribution of the variability of the strength parameters of the H.W.Sh to the RI, another set of probabilistic analyses was performed by considering the variation of the H.W.Sh only. Parameters for the remaining soil layers and ground water table level were set to fixed values (the mean values). The results of the analyses are summarized in Table 5.3 (under columns of Probabilistic Analysis II). It can be seen that the results using BST strength parameters (Analyses 1 to 4) are quite different from those using DST strength parameters (Analyses 5 to 8) with respect to the RI values. For the cases of using BST strength parameters, the RI values increase slightly as compared with the corresponding RI values in Probability Analyses I. For example, RI for the Analysis 1

increased from 1.733 to 1.985. On the other hand, when using DST strength parameters, the RI values increased significantly. For example, RI for the Analysis 5 increased from 2.280 to 11.711. These observations indicate that H.W.Sh played a dominant role when BST strength parameters were used. The role became less important when DST strength parameters were used. This further illustrates the importance of the proper selection of strength parameter values.

Critical Situations in Slope Stability Analyses

In slope analysis, the lowest factor of safety (FS) and the highest probability of failure (PF) are the critical situations. However, it is noteworthy that the critical slip surface (i.e., the slip surface having the lowest deterministic FS value) does not necessarily correspond to the highest PF value. This is the case for Analyses 1 and 3, and Analyses 5 and 7, for which the same analysis method of MP method were use, thus allowing for comparison (Table 5.3). The non-circular slip surface has a lower FS value indicating that it is more critical than the circular clip surface. On the other hand, the non-circular slip surface has a lower PF value indicating that it is less important than the circular slip surface. The inconsistency is due to the effects of the uncertainties or variations in the input parameters on the PF. It shows FS was not a sufficient indicator of safety margin. Similar observations were also reported by other authors (e.g., Hassan and Wolff 1999). El-Ramly (2002) hence noted that an essential part of the probabilistic analysis is to consider all the possibly hazardous slip surfaces which include the deterministic critical slip surface, the minimum reliability index slip surface (or the maximum PF slip surface) and surface through weak layers. Bhattacharya et al. (2003) proposed a numerical procedure for locating the slip surface of minimum reliability index for slope, but the algorithm has not been implemented in Slope/W (GEO-SLOPE 2004). Nevertheless, it has been generally accepted that the probability estimated from the most critical slip surface is a reasonable estimate of slope reliability (Alonso 1976; El-Ramly 2002).

SUMMARY AND CONCLUSIONS

Probabilistic slope stability analyses were performed on the proposed Sugar Creek embankment slope project in Iowa. Shear strength parameter values from two independent sources of *in-situ* Borehole Shear Test (BST) and laboratory direct shear test (DST) were used in the analyses assuming normal distributions. Both circular and non-circular slip surfaces were considered, and the Morgenstern-Price method and the Bishop simplified method were used for comparisons.

The following conclusions can be drawn from the analyses:

(1) The locations of the critical slip surfaces using the BST shear strength parameter values are different from those using the DST values due to the difference in the two sets of the shear strength parameter values.

(2) The calculated factors of safety against slope instability are slightly smaller and the probability of failure is higher when using the BST values compared to the DST values. The difference in results is due to the fact that BST measurements have lower mean values but more variability than the DST measurements. The higher variation in BST measurements may be a result of testing on the same soil, while DST averages three or more soil samples.

(3) The highly weathered shale contributes much more to the overall probability of failure when using the BST measurements compared to the DST measurements.

(4) With respect to the assumed slip surfaces, the non-circular critical slip surfaces gave lower factors of safety, but the circular “critical” slip surfaces gave higher probability of failure, indicating the inconsistency on the locations of the “critical” slip surfaces resulted from the uncertainties of the input parameters.

(5) Morgenstern-Price method and Bishop simplified method gave very close results on circular slip surface, but gave considerably different results on non-circular slip surface, especially with respect to the probability of failure. This may be due to the limitations of Bishop simplified method on non-circular slip surface.

(6) The use of the two independent sources of shear strength parameter values provided comparison and check for the evaluation of the slope stability and probability of failure.

(7) This paper demonstrates the first detailed application of BST in probabilistic slope stability analyses.

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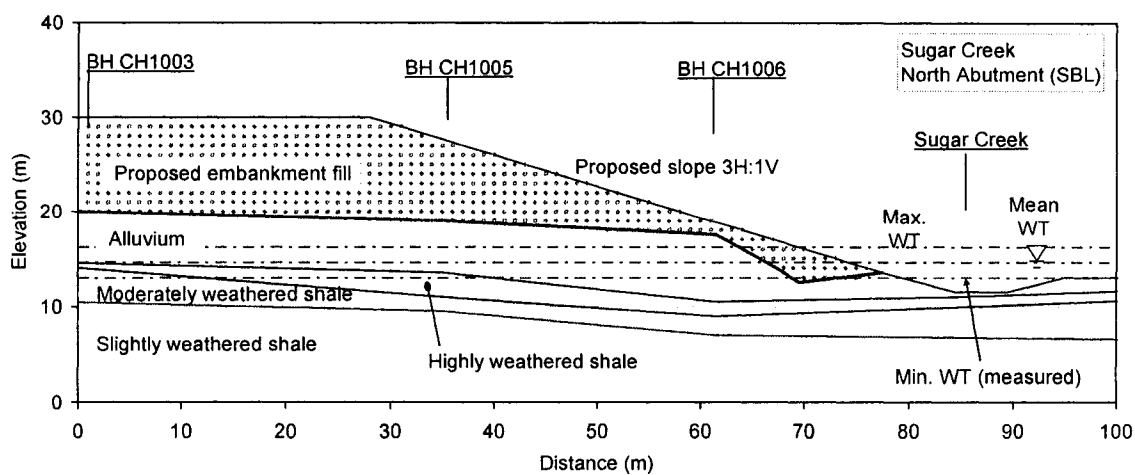
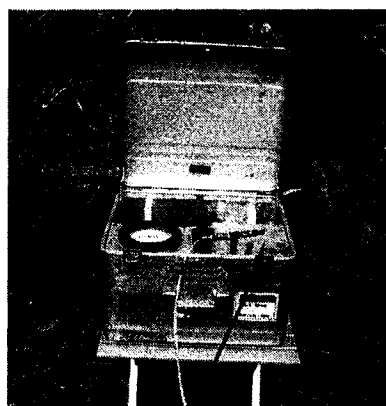
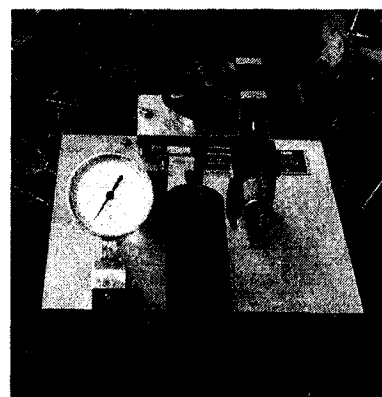


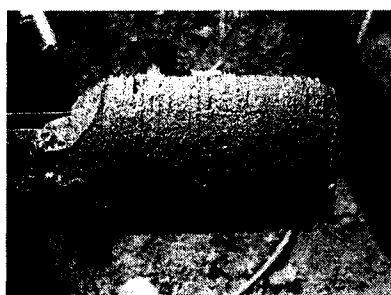
Figure 5. 1 Cross-section of a typical slope



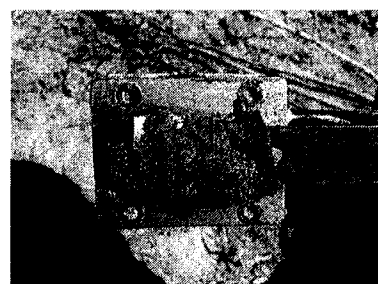
(a)



(b)



(c)



(d)

Figure 5. 2 The Borehole Shear Test (BST) device.

(a) Pressure console; (b) Pressure plate;
 (c) Ordinary pressure shear plates; (d) High pressure shear plates.

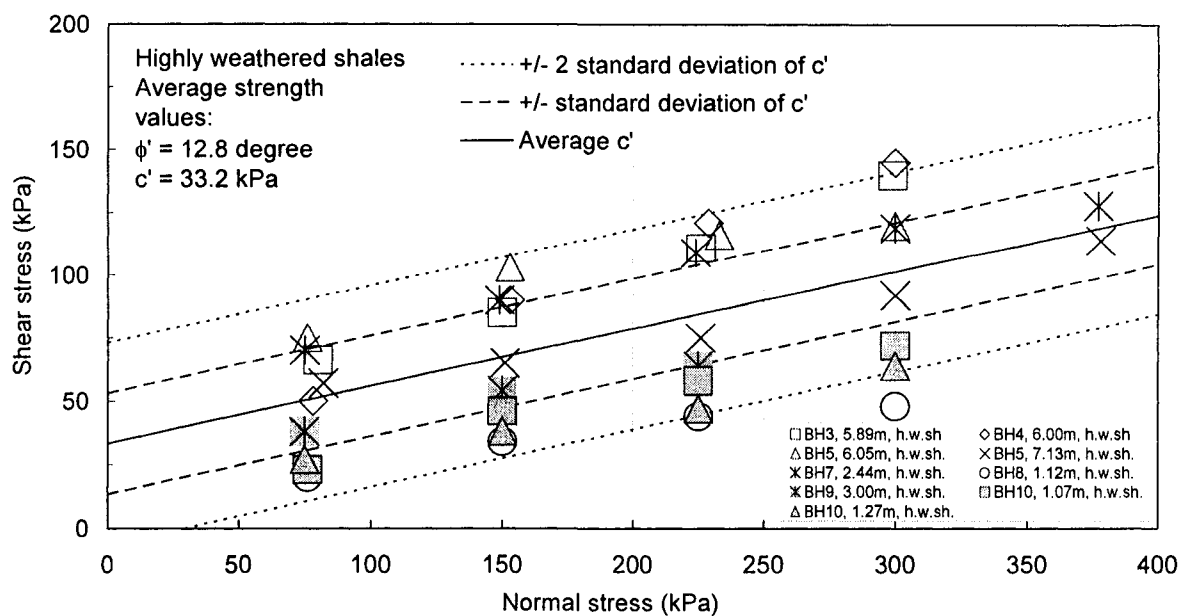
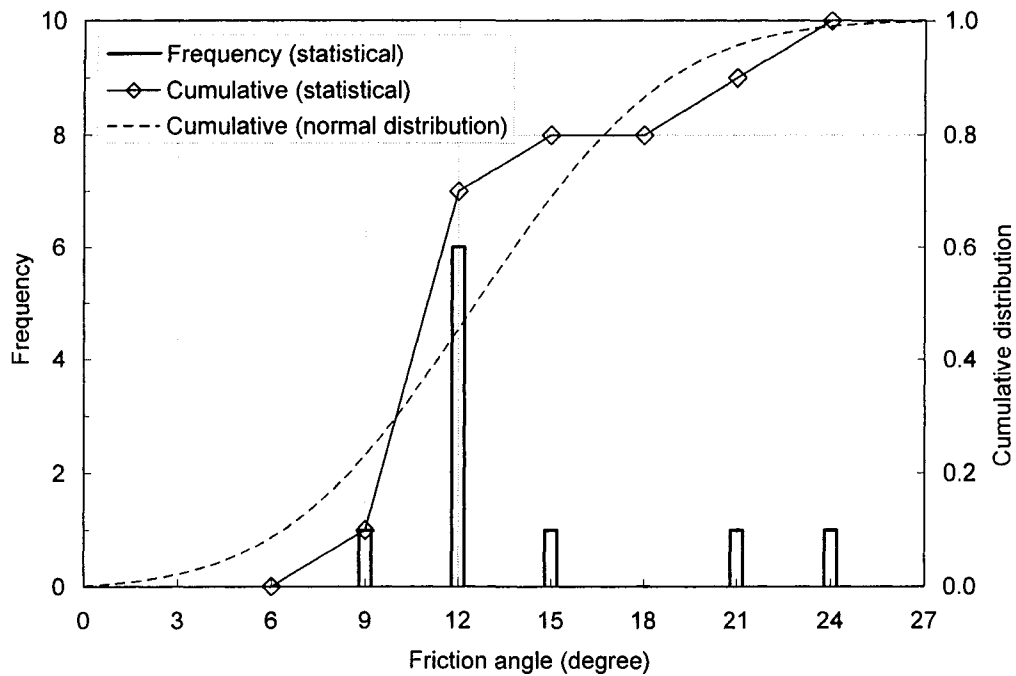
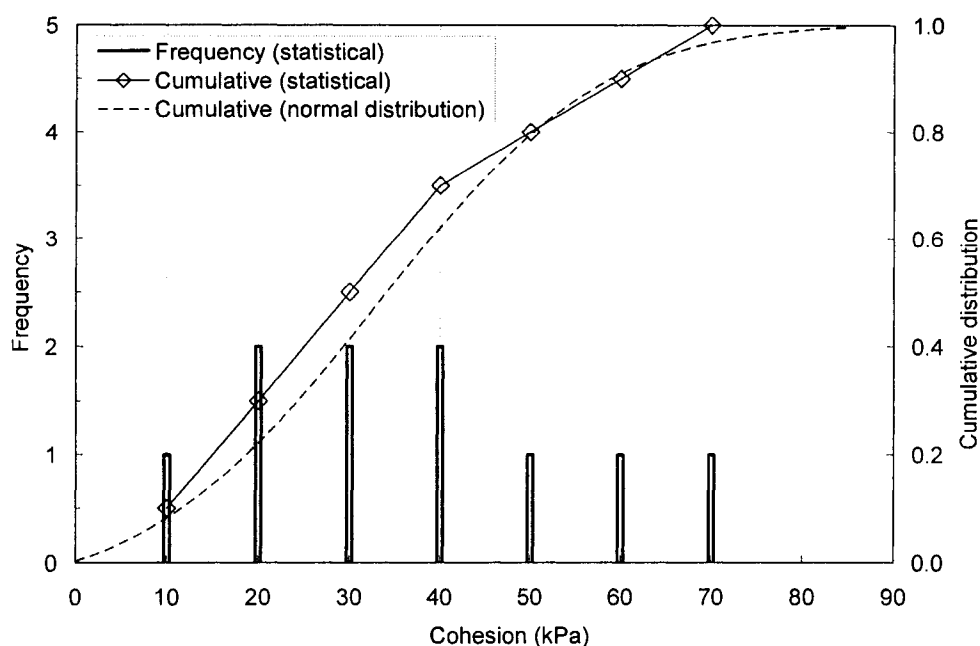


Figure 5. 3 Plot of normal stress versus shear stress in all the Borehole Shear Tests (BSTs) for the highly weathered shales



(a)



(b)

Figure 5. 4 Histograms and fitting curves of normal distribution for the shear strength parameter values obtained from the borehole shear tests (BSTs) for the highly weathered shales. (a) Friction angle; (b) Cohesion.

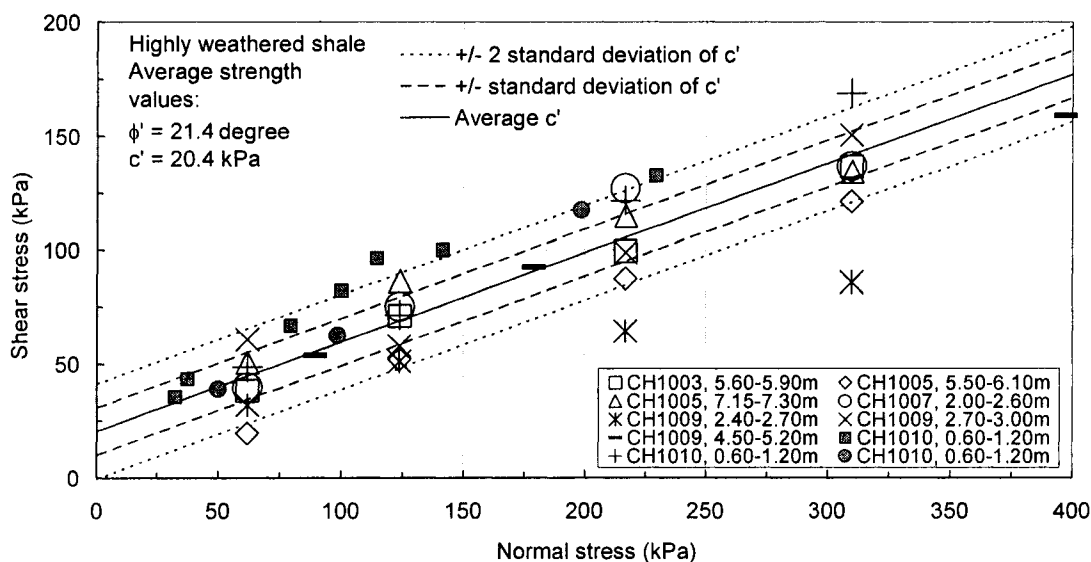
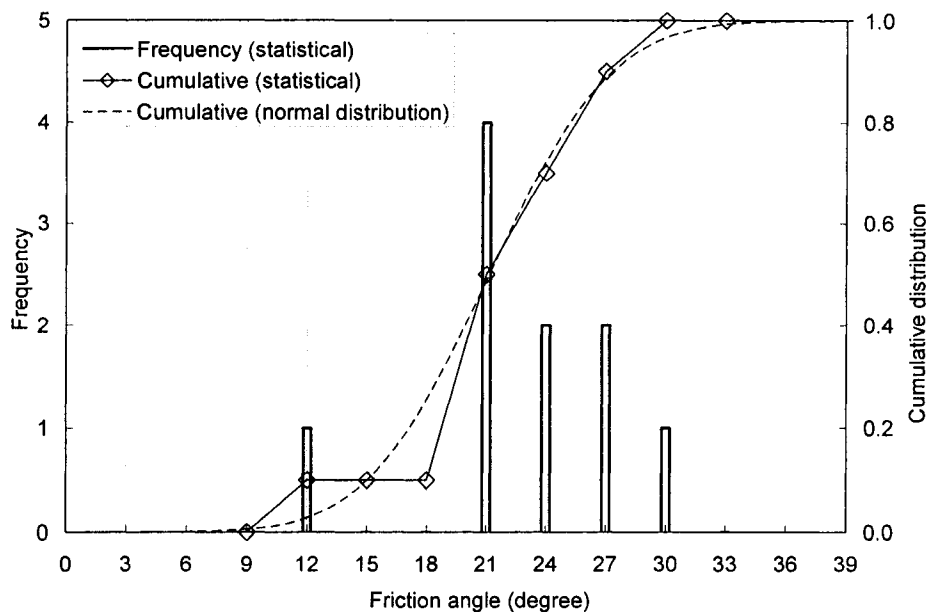
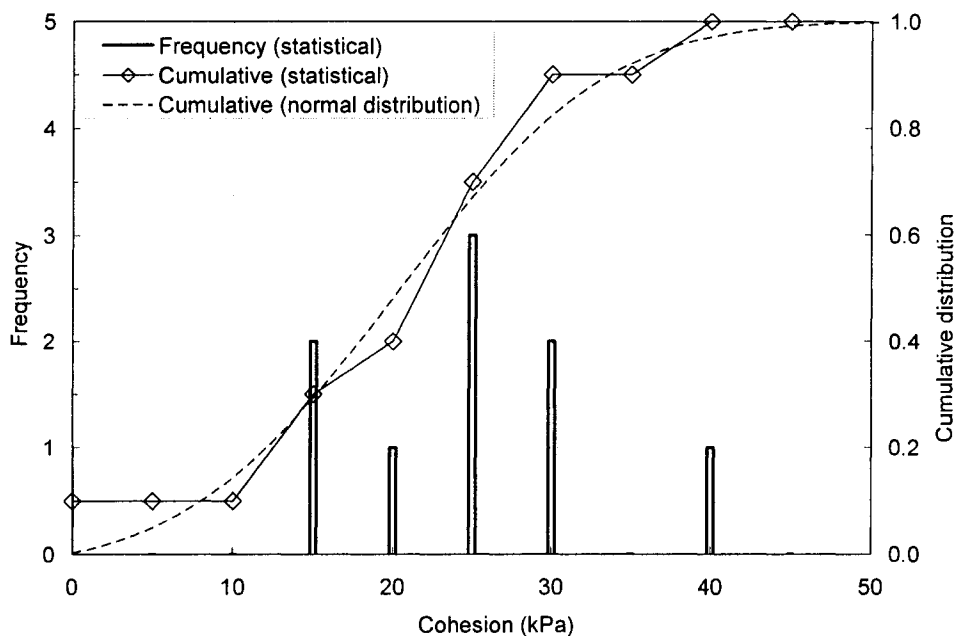


Figure 5. 5 Plot of normal stress versus shear stress in all the direct shear tests (DSTs) for the highly weathered shales



(a)



(b)

Figure 5. 6 Histograms and fitting curves of normal distribution for the shear strength parameter values obtained from direct shear tests (DSTs) for the highly weathered shales. (a) Friction angle; (b) Cohesion.

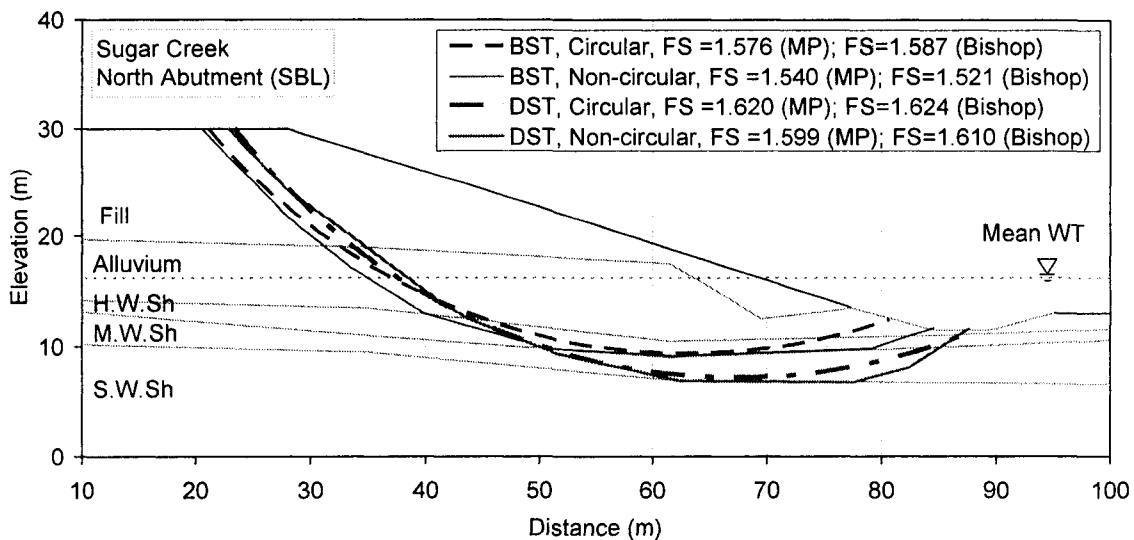
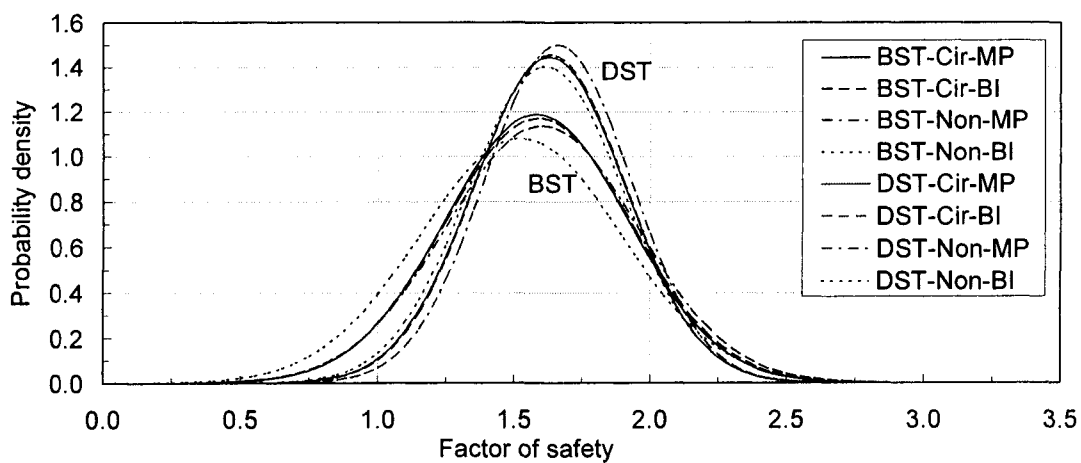
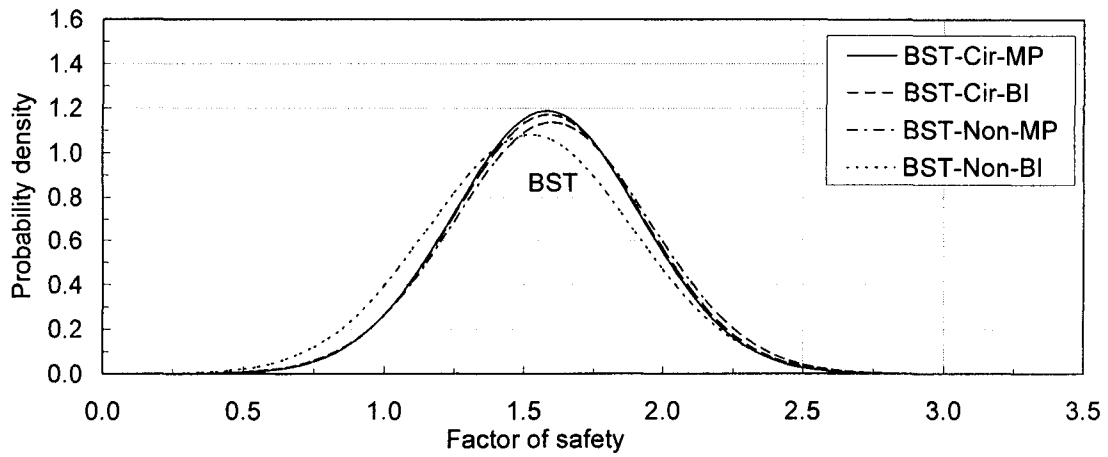


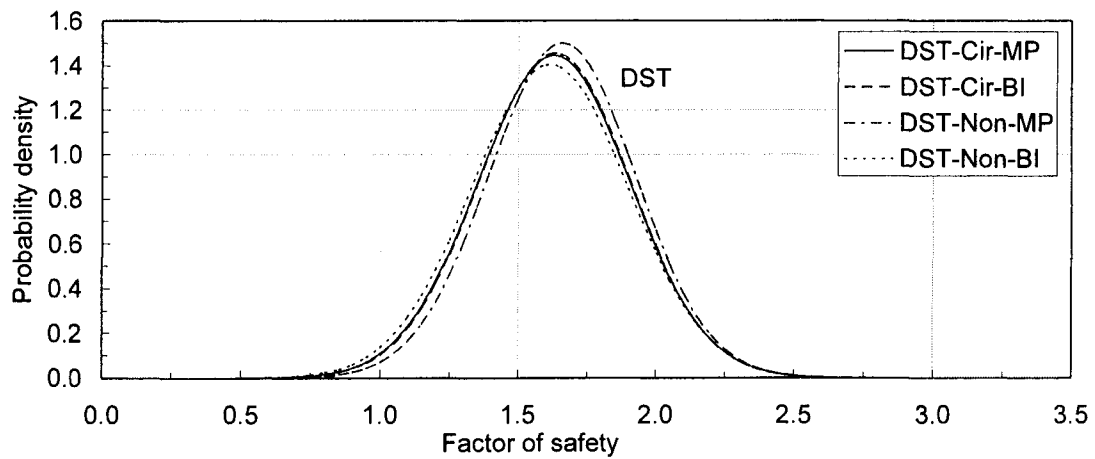
Figure 5. 7 The circular and non-circular critical slip surfaces corresponding to the different shear strength parameter values obtained from BST and DST



(a)



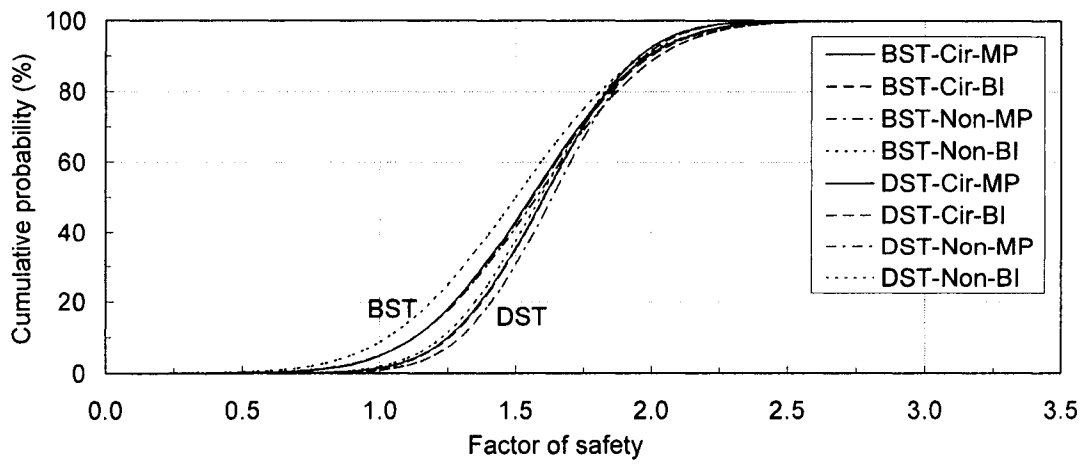
(b)



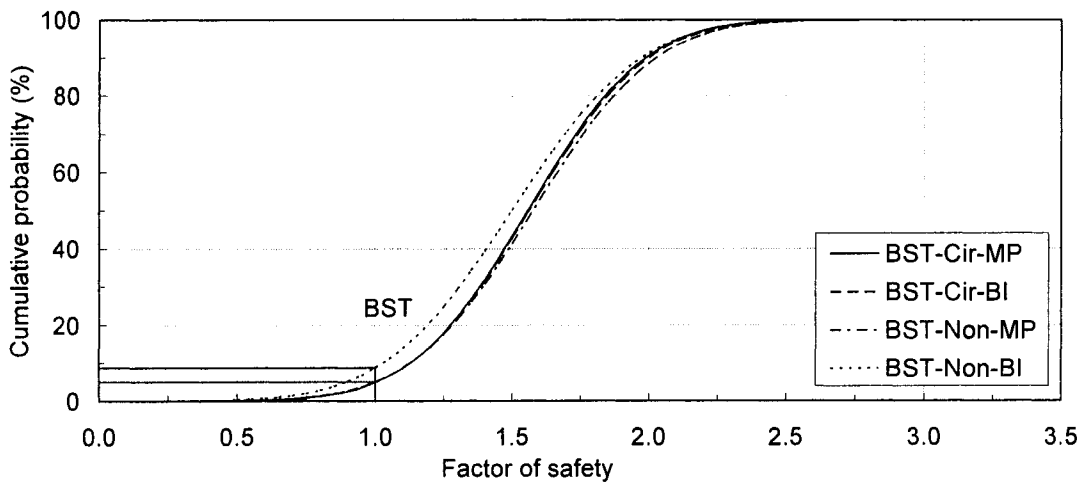
(c)

Figure 5. 8 Probability density functions of factor of safety.

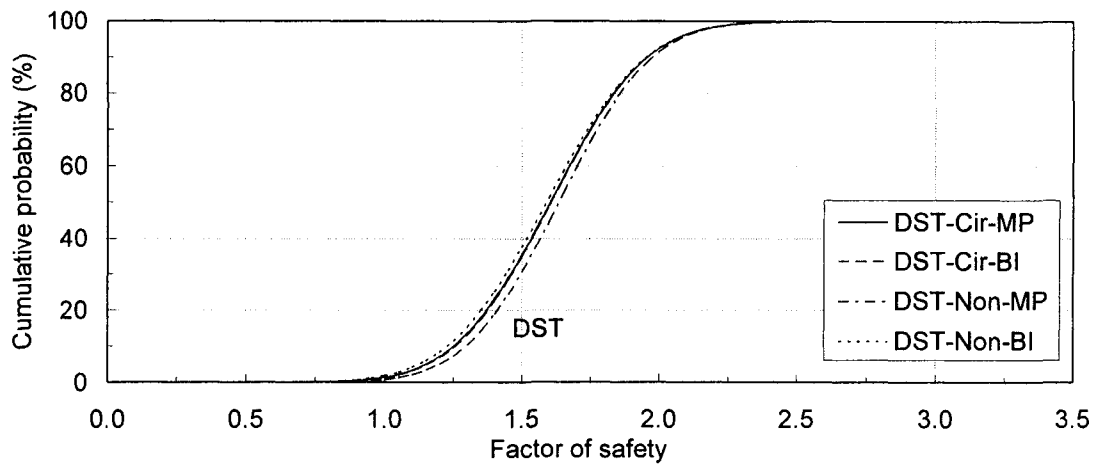
- (a) PDF of FS for all the analyses; (b) PDF of FS for analyses using BST measurements;
 (c) PDF of FS for analyses using DST measurements.



(a)



(b)



(c)

Figure 5. 9 Cumulative distribution functions of factor of safety

Table 5. 1 Statistics of Borehole Shear Test Results

| Soil | Total No. of Tests | 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 |
| H.W.Sh | 10 | 23 | 7 | 12.8 | 4.9 | 66 | 10 | 33.2 | 19.9 |
| M.W.Sh | 5 | 38 | 13 | 21.6 | 9.6 | 334 | 6 | 97 | 134 |
| S.W.Sh | 9 | 41 | 9 | 23.3 | 11.3 | 3970 | 55 | 675 | 1254 |

Notations: H.W.Sh - Highly weathered shale; M.W.Sh - Moderately weathered shale; S.W.Sh - Slightly weathered shale; Ave. - average value (mean value); S.D. - standard deviation.

Table 5. 2 Statistics of Direct Shear Test Results

| Soil | Total No. of Tests | Friction angle, ϕ' (deg.) | | | | Cohesion, c' (kPa) | | | |
|----------|--------------------|--------------------------------|------|------|------|----------------------|------|------|------|
| | | Max. | Min. | Ave. | S.D. | Max. | Min. | Ave. | S.D. |
| Alluvium | 4 | 31 | 23 | 26.5 | 3.8 | 27 | 14 | 18.8 | 5.8 |
| H.W.Sh | 10 | 28 | 12 | 21.4 | 4.8 | 38 | 0 | 20.4 | 10.4 |
| M.W.Sh | 6 | 29 | 14 | 19.4 | 5.7 | 43 | 5 | 23.6 | 15.6 |

Notations: same as for Table 5. 1.

Table 5. 3 Summary of the Results of Slope Stability Analysis

| No. | Analysis | D.FS | Probabilistic Analysis I ^a | | | Probabilistic Analysis II ^b | | |
|-----|------------|-------|---------------------------------------|-------|--------|--|--------|--------|
| | | | M.FS | RI | PF (%) | M.FS | RI | PF (%) |
| 1 | BST-Cir-MP | 1.576 | 1.582 | 1.733 | 3.99 | 1.581 | 1.985 | 2.21 |
| 2 | BST-Cir-BI | 1.587 | 1.588 | 1.725 | 4.18 | 1.587 | 1.979 | 2.41 |
| 3 | BST-Non-MP | 1.540 | 1.601 | 1.714 | 3.68 | 1.576 | 1.704 | 3.62 |
| 4 | BST-Non-BI | 1.521 | 1.524 | 1.422 | 7.78 | 1.517 | 1.469 | 7.04 |
| 5 | DST-Cir-MP | 1.620 | 1.629 | 2.280 | 1.04 | 1.621 | 11.711 | 0.00 |
| 6 | DST-Cir-BI | 1.624 | 1.632 | 2.303 | 1.12 | 1.629 | 11.306 | 0.00 |
| 7 | DST-Non-MP | 1.599 | 1.660 | 2.479 | 0.40 | 1.606 | 12.404 | 0.00 |
| 8 | DST-Non-BI | 1.610 | 1.615 | 2.163 | 1.45 | 1.612 | 11.402 | 0.00 |

Notations - BST: Borehole shear test; DST: direct shear test; Cir: circular slip surface; Non: non-circular slip surface; MP: Morgenstern-Price method; BI: Bishop simplified method; D.FS: deterministic factor of safety; M.FS: mean factor of safety; RI: reliability index; PF: probability of failure.

a. Considering variations of soil properties for all layers and variations of ground water table level.

b. Considering variations of strength parameters for the highly weathered shale only. Parameters for other layers and ground water table level were set to the mean values.

CHAPTER 6. GENERAL CONCLUSIONS AND RECOMMENDATIONS

GENERAL CONCLUSIONS

The most important conclusions and the significance of the study are listed as follows:

1. BSTs are competent to characterize slopes, especially to obtain the *in-situ* soil shear strength parameter values that are essential for slope stability analysis. BSTs have the advantages in that they gave direct, *in-situ* measurement of soil shear strength in a relatively quick manner.
2. The BST measured the peak shear strength and partially softened shear strength, while the ring shear test measured the residual shear strength of the stiff clay shales in the first-time slope failures.
3. A range of mobilized shear strengths at the slope failure was obtained from back calculations due to the unknown ground water conditions at failure. The most probable mobilized shear strength at failure was estimated by considering the partially softened and residual shear strengths in the failure zone.
4. The strength changes, or the “strength path”, due to the slope movement, can be fully established and used to examine the failure mechanisms of the slopes.
5. The evaluated slope failures are attributed to progressive failures, and were likely triggered by high ground water tables.
6. The findings in the first paper represents an improvement compared to the empirical method of using “good engineering judgment or experience” to estimate the mobilized shear strength parameter values for first-time slope failures.
7. The classification of weathering of the shales for the Sugar Creek embankment slope correlates well with the peak shear strength values of the shales, i.e. higher weathering degree consistently corresponds to lower shear strength values; but does not correlate well with residual shear strength values or other soil index properties.
8. The shear strength values obtained from different test methods did not exactly match, but they were comparable and showed reasonable agreement, considering the large variation of the soil.

9. The internal friction angles obtained from the BST were generally lower than those obtained from direct shear tests (DST), while the cohesion intercepts obtained from BST were generally larger than those from DST, for both the alluvium and the highly weathered shale. This observation could be mainly attributed to the soil variability, test methods and shear strength anisotropy.
10. The use of the weathering classification and the measured shear strength values for the Sugar Creek project are expected to provide an economical and safe design for the slope and ground improvement measures.
11. The second paper represents a detailed case study for using geotechnical information including *in-situ* BST measurements to characterize weathered shale materials with emphasis on weathering classifications for slope stability analyses.
12. The results of the probabilistic slope stability analyses performed on the Sugar Creek embankment slope show that the location of the critical slip surface using the BST shear strength parameter values is different from that using the DST values.
13. The calculated factors of safety against slope instability are slightly smaller and the probability of failure is higher, when using the BST values compared to the DST values. The difference in results is due to the BST measurements providing a lower mean value but larger variability than the DST measurements.
14. With respect to the assumed slip surfaces, the non-circular critical slip surfaces gave lower factors of safety, but the circular “critical” slip surfaces gave higher probability of failure, indicating inconsistency on the location of the “critical” slip surface resulted from the variability in the input parameters.
15. The Morgenstern-Price method and the Bishop simplified method gave very close results on circular slip surface, but gave considerably different results on non-circular slip surface, especially with respect to the probability of failure of the slope. This may be due to the limitations of the Bishop simplified method on non-circular slip surface.
16. The use of the two independent sources of shear strength parameter values of BST and DST provided a comparison and check for the evaluation of the slope stability and probability of failure.

17. The third paper represents the first detailed analyses and application of in-situ BST results in probabilistic slope stability analysis.

RECOMMENDATIONS

Based on the study, the following recommendations are made for future work:

1. 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.
2. Perform the BST as much as possible in the failure zone of a failed slope or the potential failure zone of a proposed slope as long as the site investigation program is permitted. The failure zone or the potential failure zone can be estimated by trial slope analysis using available information such as slope geometry and empirical shear strength parameter values.
3. Establish long term monitoring of ground water condition and slope deformation for some critical slopes, especially for those newly constructed slopes susceptible to slope instability. This can provide verifications and calibrations for the shear strength parameter values measured by the BST, and information for the possible progressive failures.
4. Accumulate information and 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.
5. Perform quantitative mineralogical analysis for the weathered shales to investigate the possible correlation of the mineralogical compositions with the weathering grades.
6. Perform additional laboratory tests to investigate the anisotropic strength of the weathered shales. The tests can be stress-path triaxial compression tests or direct shear tests with shearing planes of various directions.
7. Investigate the possible effect of the strength anisotropy of the weathered shale on the slope stability from both deterministic and probabilistic perspective.

APPENDIX A - ADDITIONAL DATA FOR ALBIA AND WINTERSET SLOPE

Table A.1 Basic Properties for the Shales at the Albia Slope and the Winterset Slope

| Slope | BH | Depth (m) | Grain Size | | | Atterberg Limit | | | Classification (USCS) | Water content (%) | Total density (kN/m ³) |
|-----------|----|-----------|------------|----------|----------|-----------------|--------|--------|-----------------------|-------------------|------------------------------------|
| | | | Sand (%) | Silt (%) | Clay (%) | LL (%) | PL (%) | PI (%) | | | |
| Albia | 2 | 3.2 | 2 | 46 | 52 | 64 | 24 | 40 | CH | 25.2 | 19.0 |
| | 4 | 0.6 | 2 | 51 | 47 | 59 | 28 | 31 | CH | 30.9 | |
| | 4 | 1.1 | 5 | 47 | 48 | 60 | 25 | 35 | CH | 33.6 | |
| Winterset | 2 | 2.7 | 1 | 64 | 35 | 50 | 23 | 27 | CH | 16.2 | 18.9 |
| | 3 | 2.7 | 3 | 60 | 37 | 59 | 24 | 35 | CH | 29.0 | 19.2 |

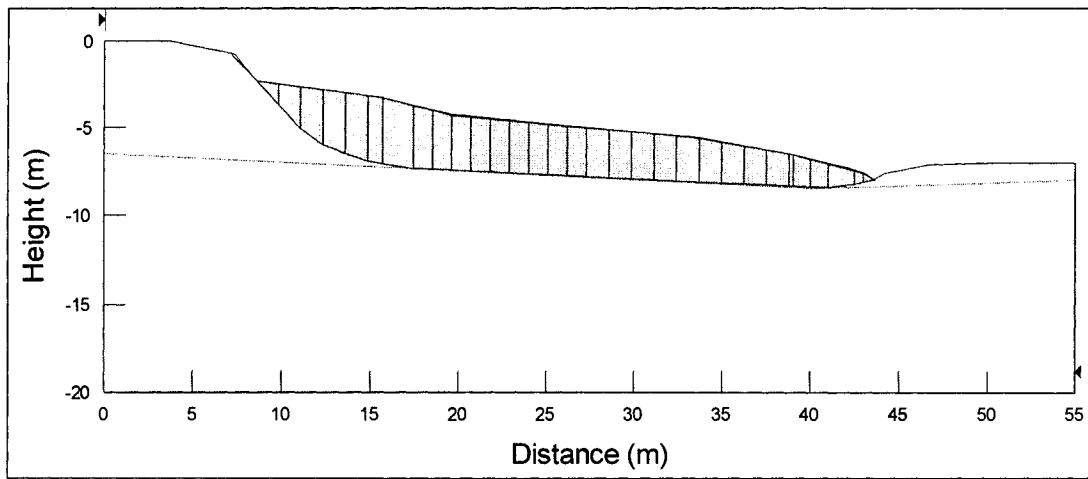


Figure A.1 Slope analysis for the Albia slope

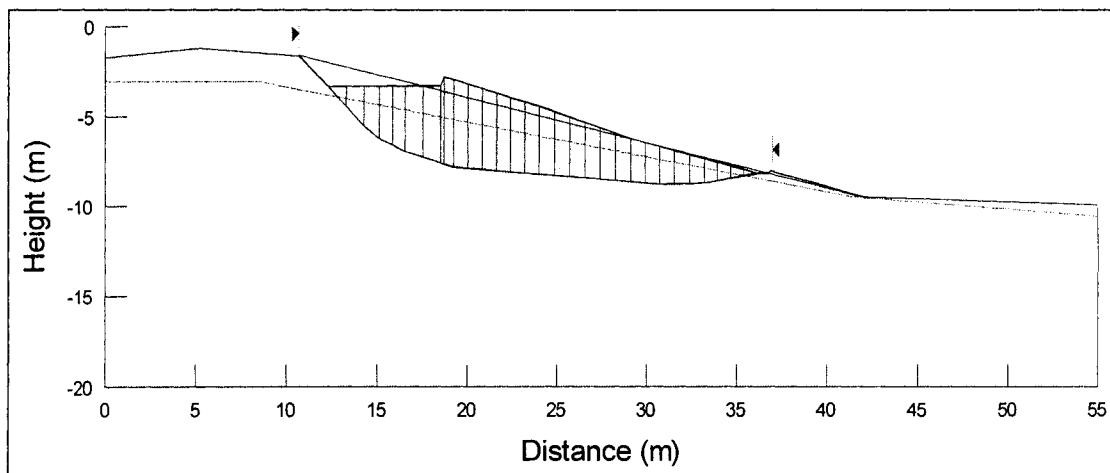


Figure A.2 Slope analysis for the Winterset slope

APPENDIX B - ADDITIONAL DATA FOR SUGAR CREEK SLOPE

Table B.1 Summary of Basic Property Results

| 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 Hill.

Table B.2 Summary of Triaxial and Unconfined Compression Test Results

| BH | Depth (m) | Soil | CU | | CD | | UC s _u (kPa) |
|----|-----------|--------|----------------|-----------|----------------|----------|----------------------------|
| | | | ϕ' (deg.) | c' (kPa) | ϕ' (deg.) | c' (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 Hill.

Table B.3 Summary of BST Results

| 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 B.4 Summary of Direct Shear Tests Results

| 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 Hill.

Table B.5 Summary of Ring Shear Test Results

| BH | Depth (m) | Soil | ϕ_r' (deg.) | c_r' (kPa) | R^2 | Data points | ϕ_r' ($c'=0$) |
|----|-----------|--------|------------------|--------------|--------|-------------|----------------------|
| 2 | 10.5-10.8 | s.w.sh | 6.8 | 0.9 | 0.9997 | 3 | 6.9 |
| 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 |
| 5 | 9.3-9.5 | s.w.sh | 9.1 | 0 | 0.9996 | 3 | 9.1 |
| 6 | 8.85-9.45 | m.w.sh | 9.8 | 0.9 | 1.0000 | 3 | 10.0 |
| 6 | 10.52 | s.w.sh | 9.2 | 0 | 0.9999 | 3 | 9.2 |
| 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 |

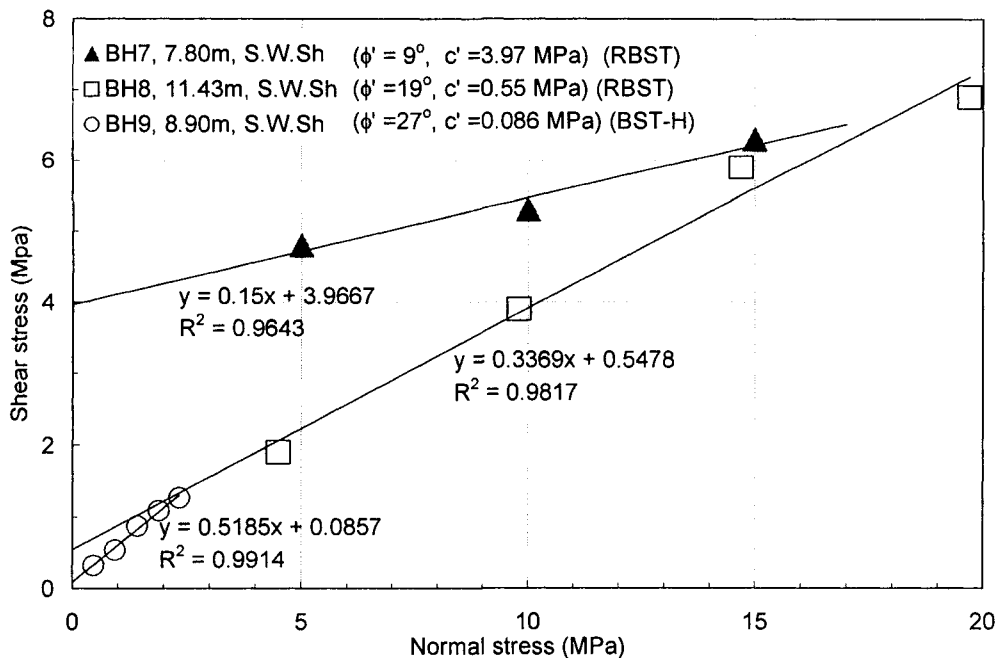


Figure B.1 Rock Borehole Shear Test results

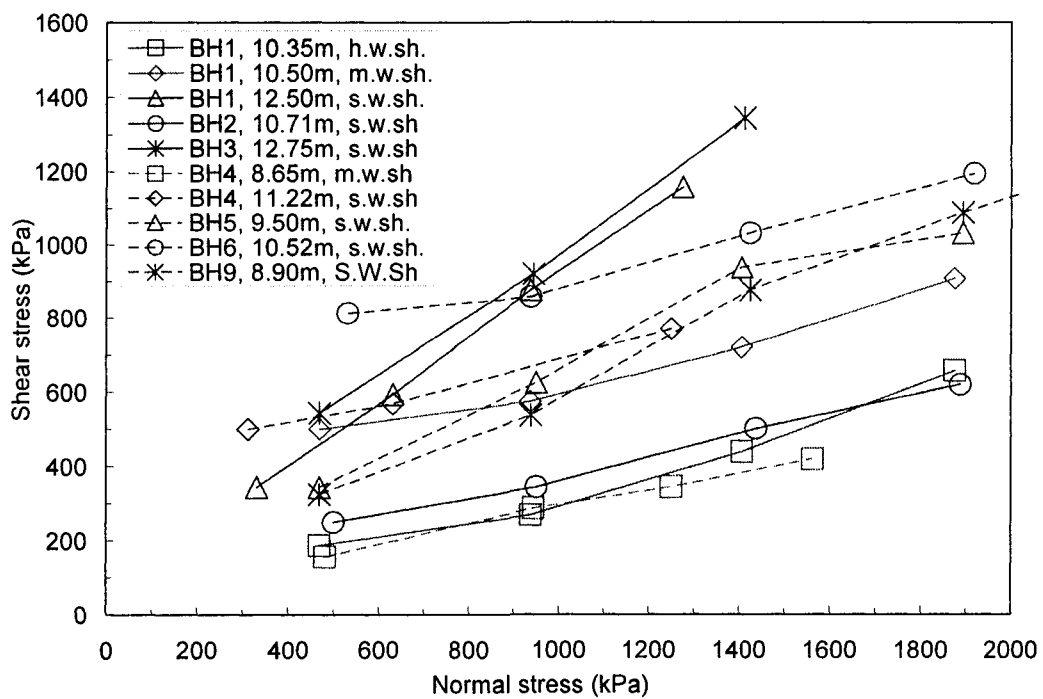


Figure B.2 Borehole Shear Test results (1 of 4)

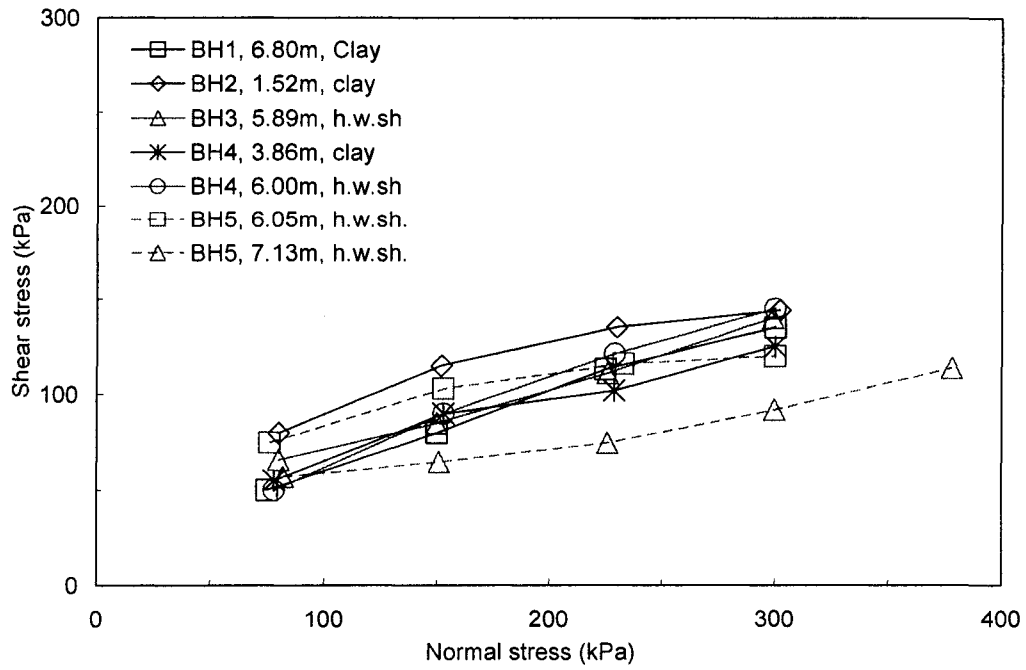


Figure B.3 Borehole Shear Test results (2 of 4)

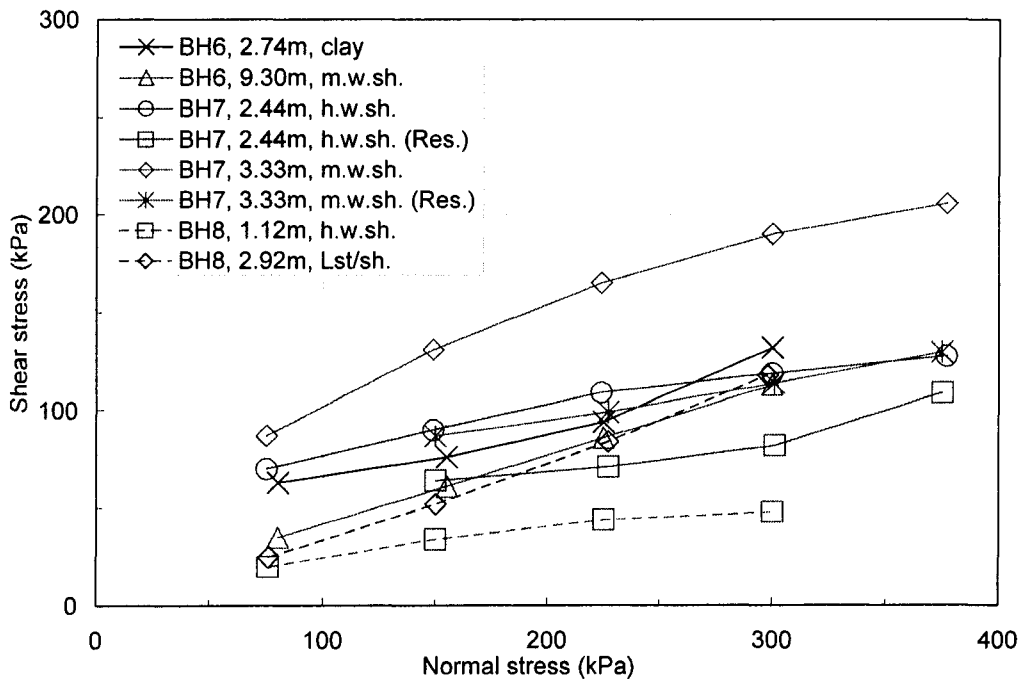


Figure B.4 Borehole Shear Test results (3 of 4)

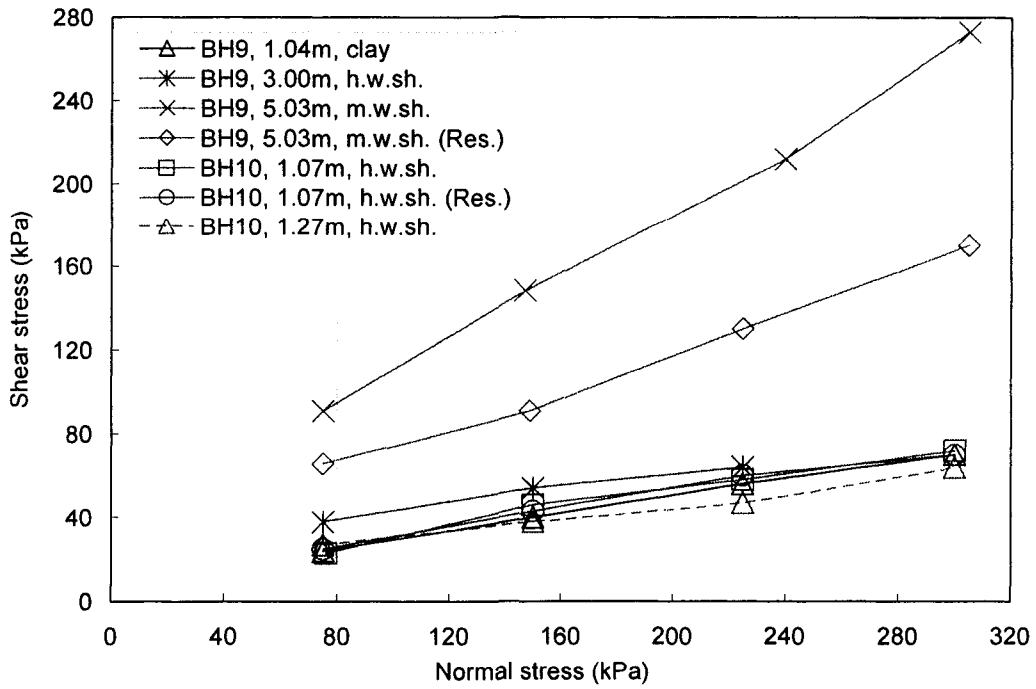


Figure B.5 Borehole Shear Test results (4 of 4)

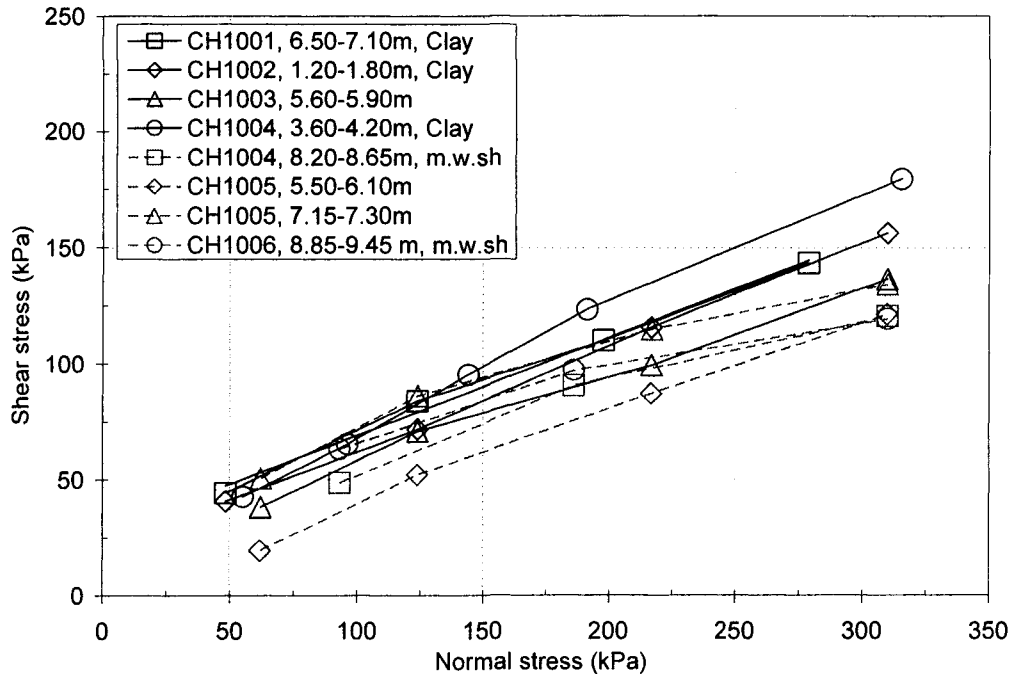


Figure B.6 Direct shear test results (1 of 2)

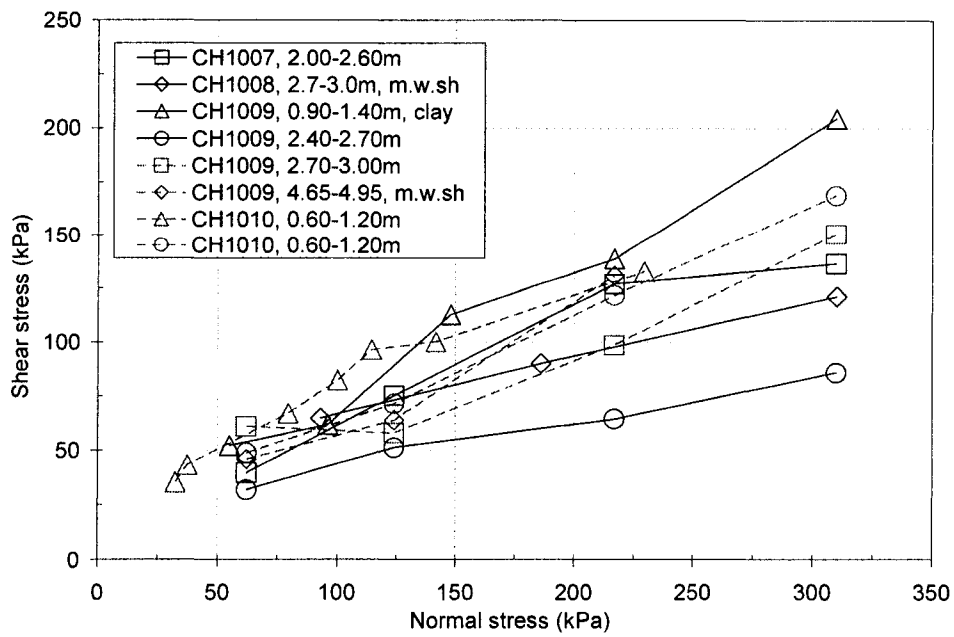


Figure B.7 Direct shear test results (2 of 2)

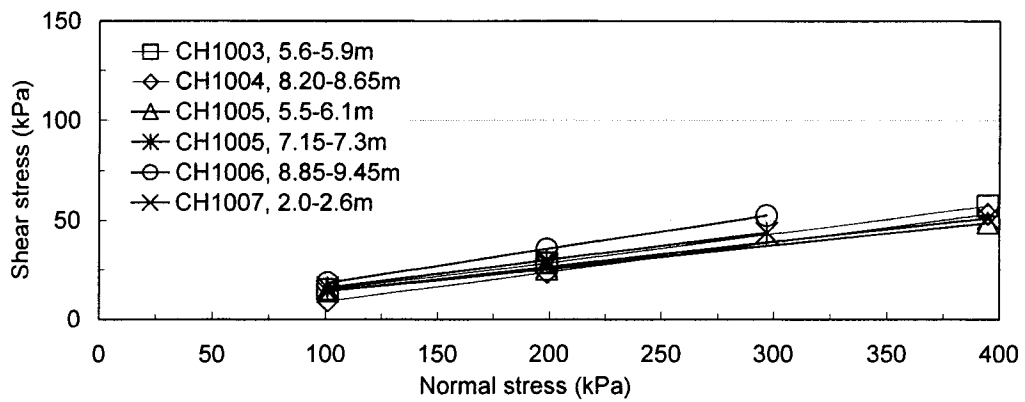


Figure B.8 Results of ring shear tests for H.W.Sh (1 of 2)

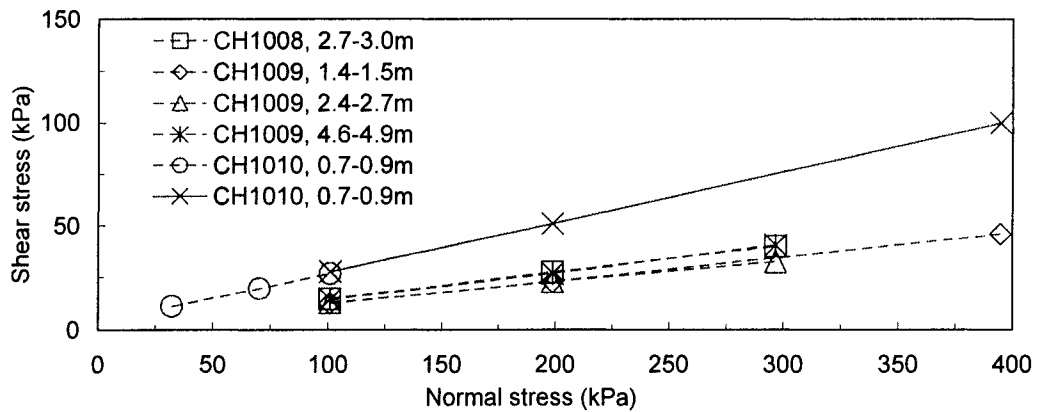
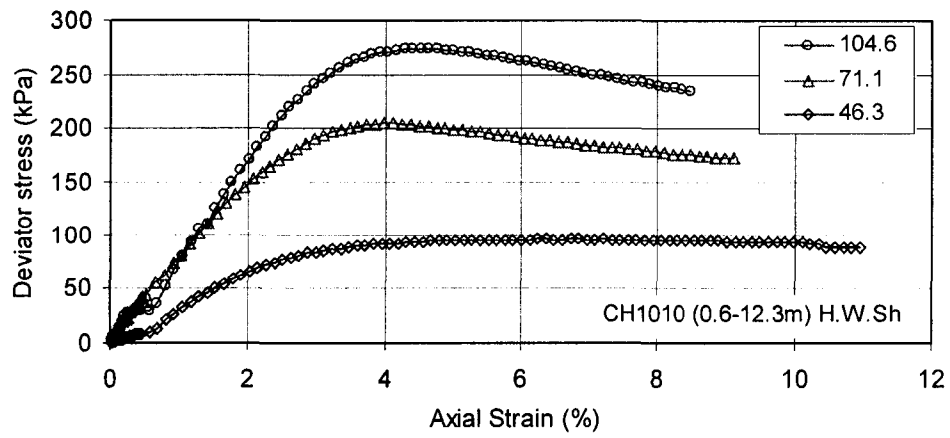
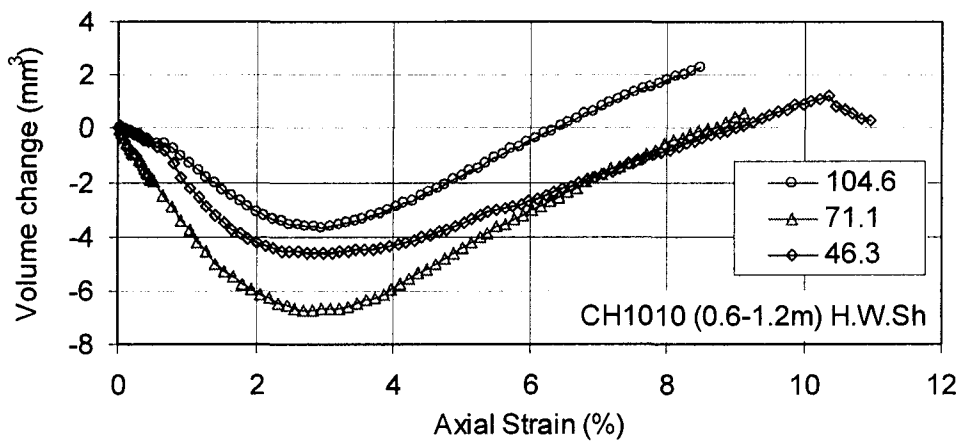


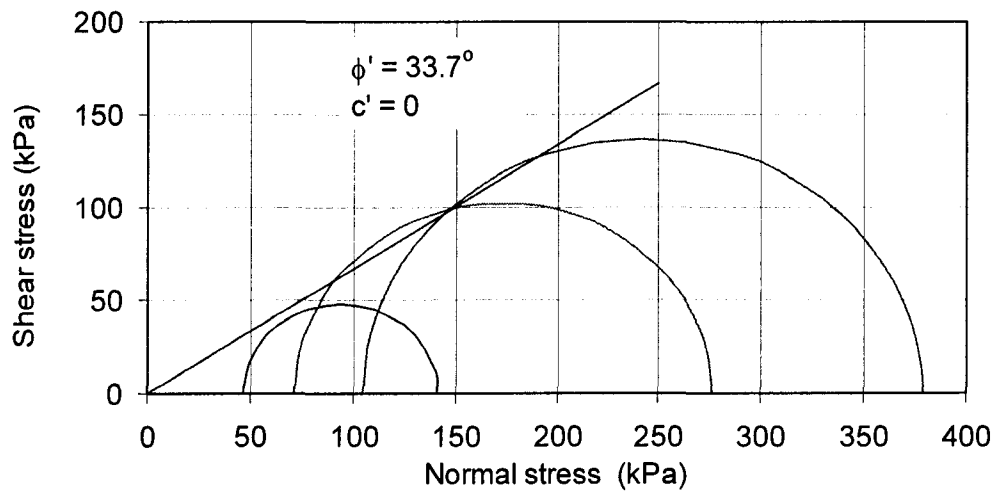
Figure B.9 Results of ring shear tests for H.W.Sh (2 of 2)



(a)

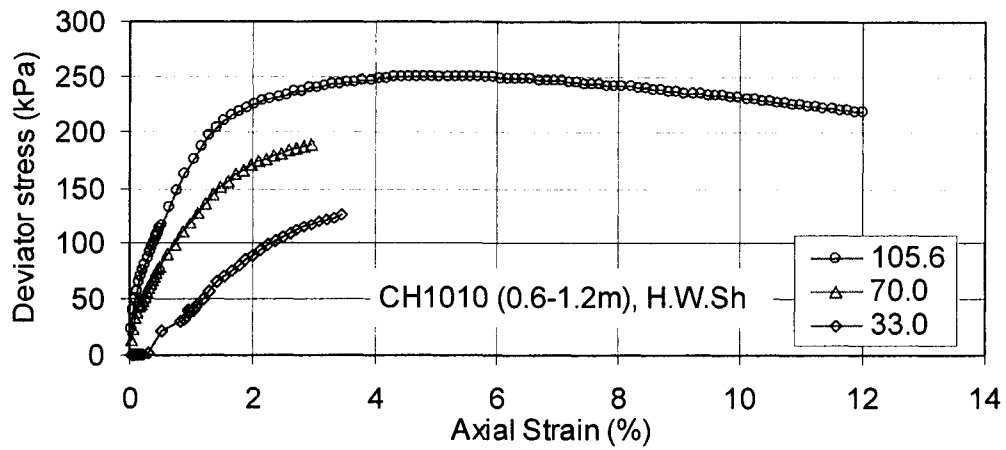


(b)

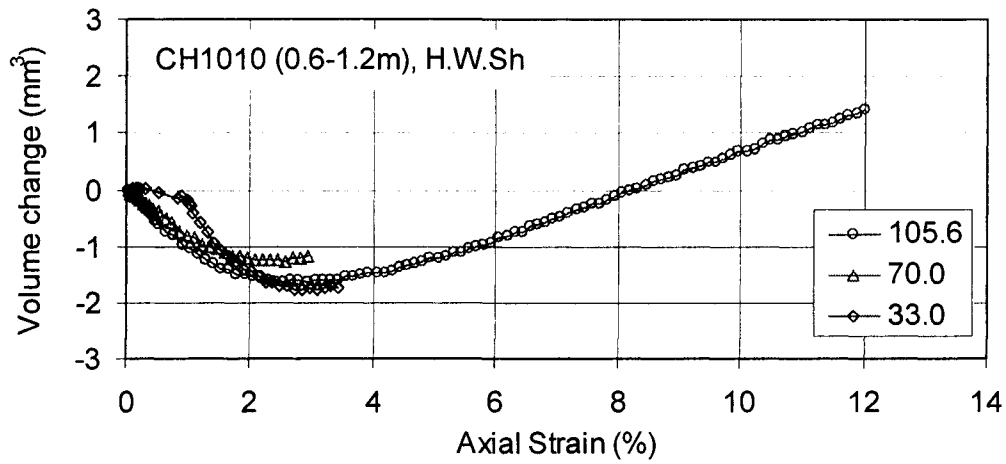


(c)

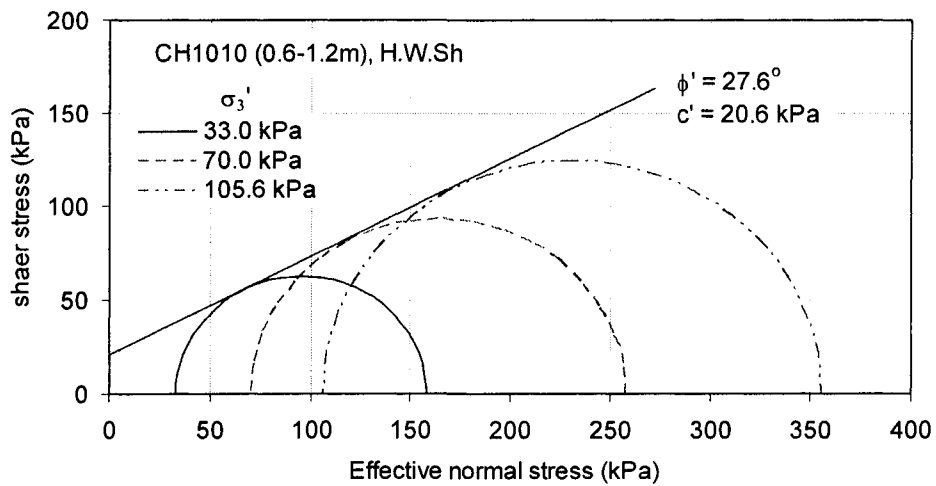
Figure B.10 CU triaxial compression test results for H.W.Sh at 0.6-1.2m in BH10. (a) Deviator stress versus strain; (b) Volume change versus strain; (c) Mohr circles.



(a)



(b)



(c)

Figure B.11 CD triaxial compression test results for H.W.Sh at 0.6-1.2m in BH10.
 (a) Deviator stress versus strain; (b) Volume change versus strain; (c) Mohr circles.

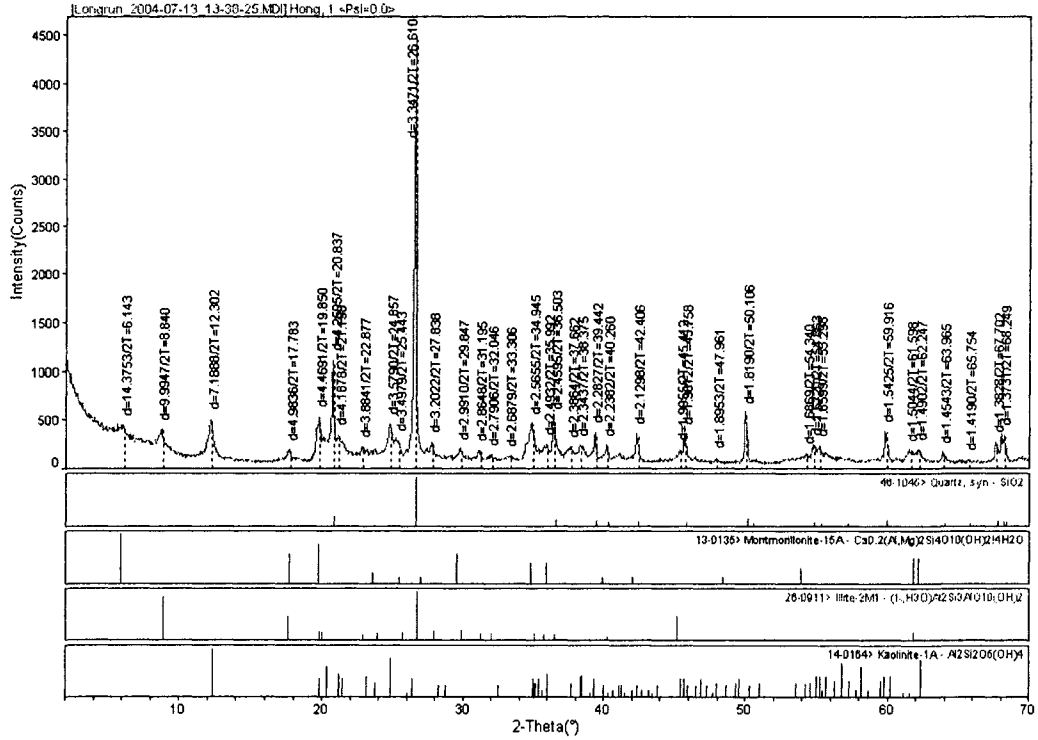


Figure B.12 XRD result (1 of 10) (CH1003, 5.6-5.9m, highly weathered shale)

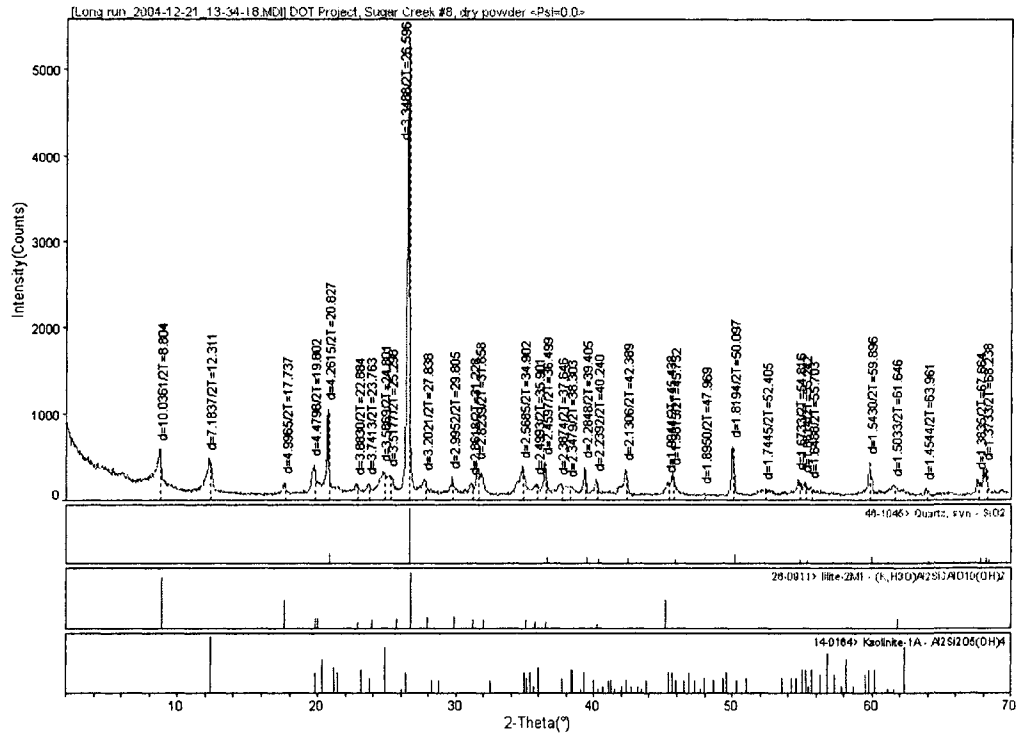


Figure B.13 XRD result (2 of 10) (CH1003, 12.75m, slightly weathered shale)

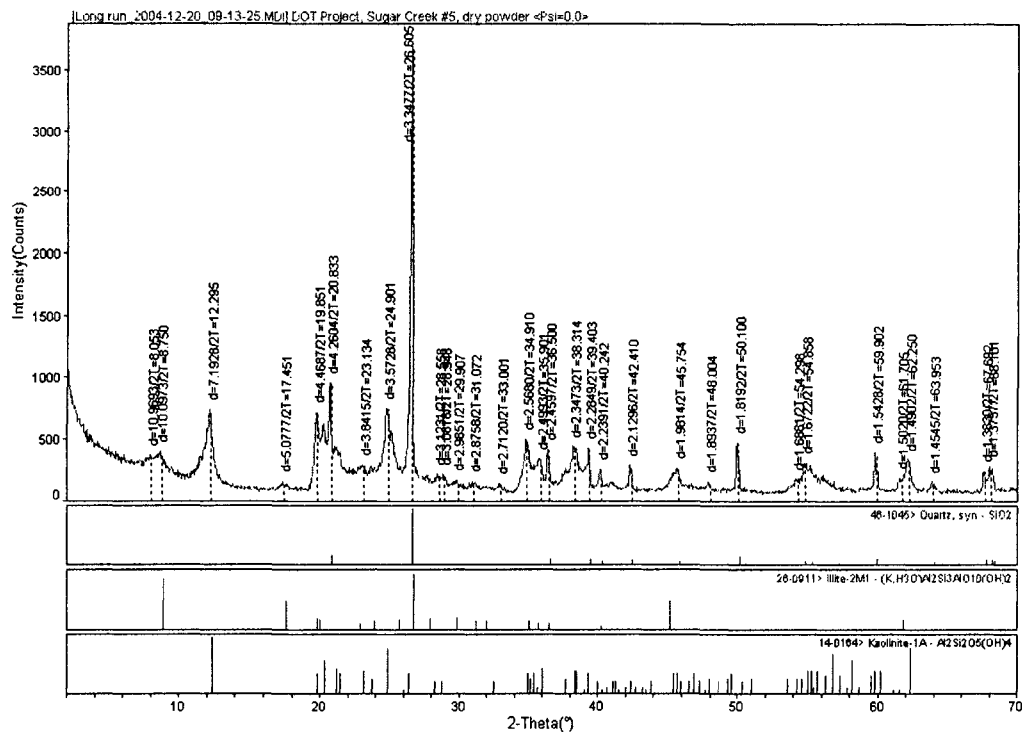


Figure B.14 XRD result (3 of 10) (CH1004, 8.2-8.65m, moderately weathered shale)

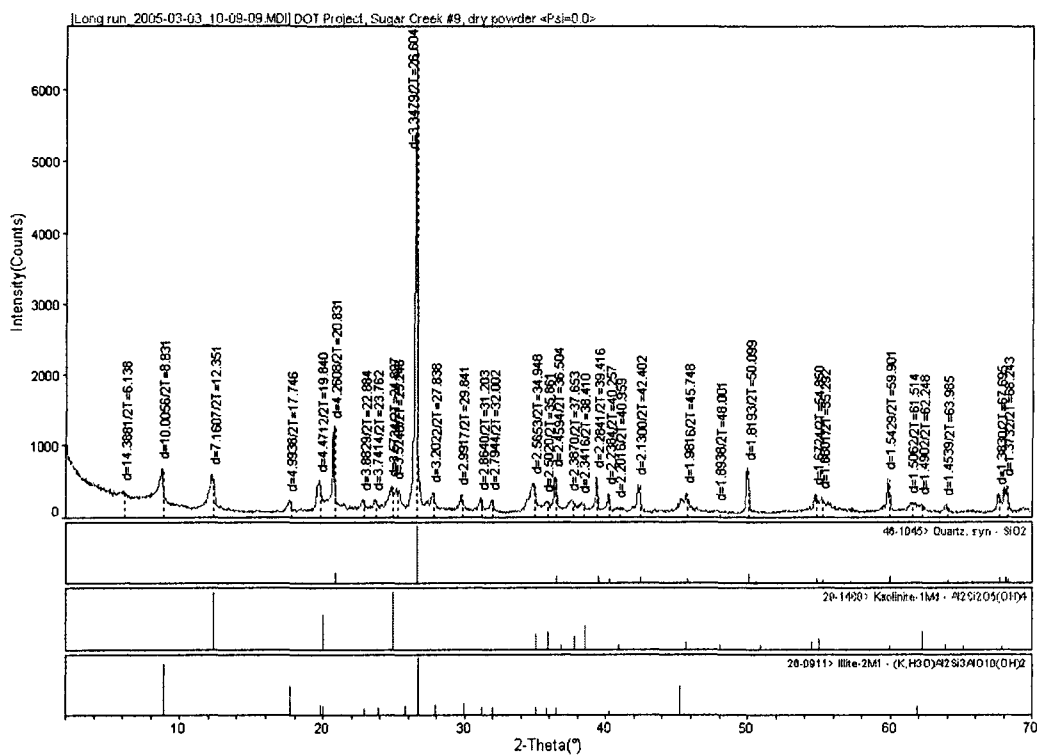


Figure B.15 XRD result (4 of 10) (CH1004, 11.22m, slightly weathered shale)

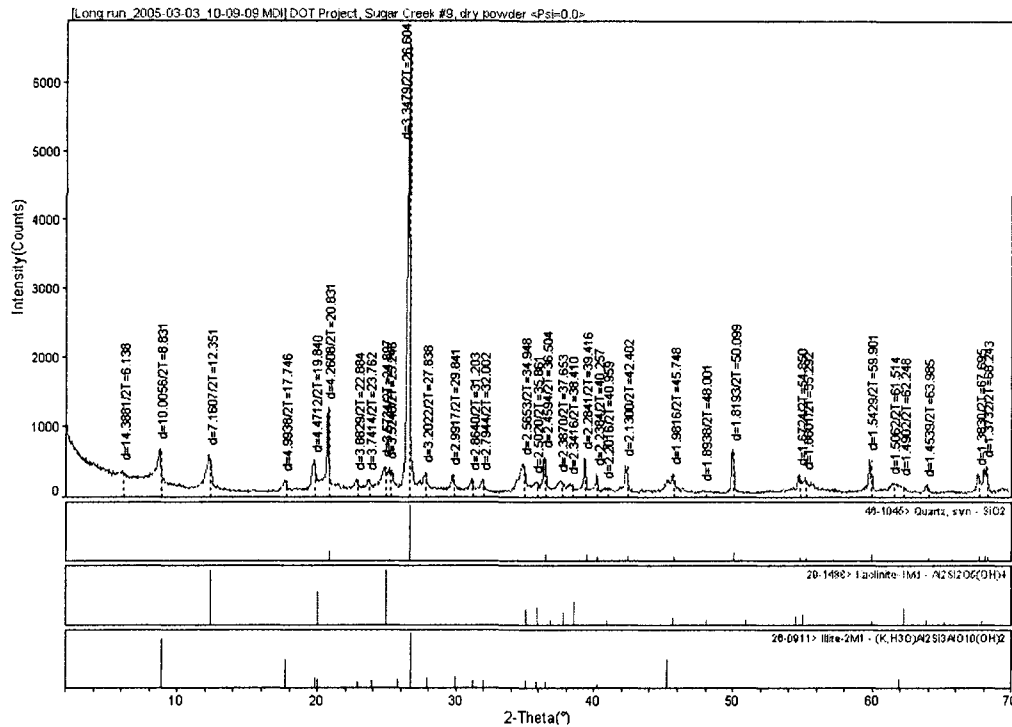


Figure B.16 XRD result (5 of 10) (CH1005, 5.5-6.1m, highly weathered shale)

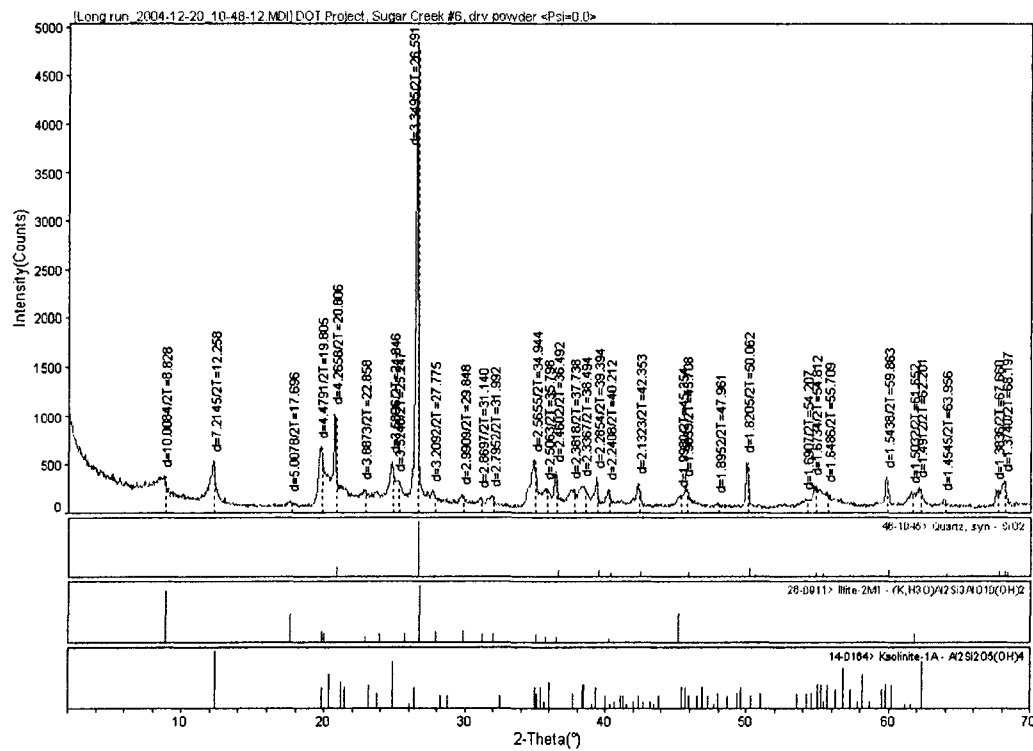


Figure B.17 XRD result (6 of 10) (CH1005, 7.15-7.3m, highly weathered shale)

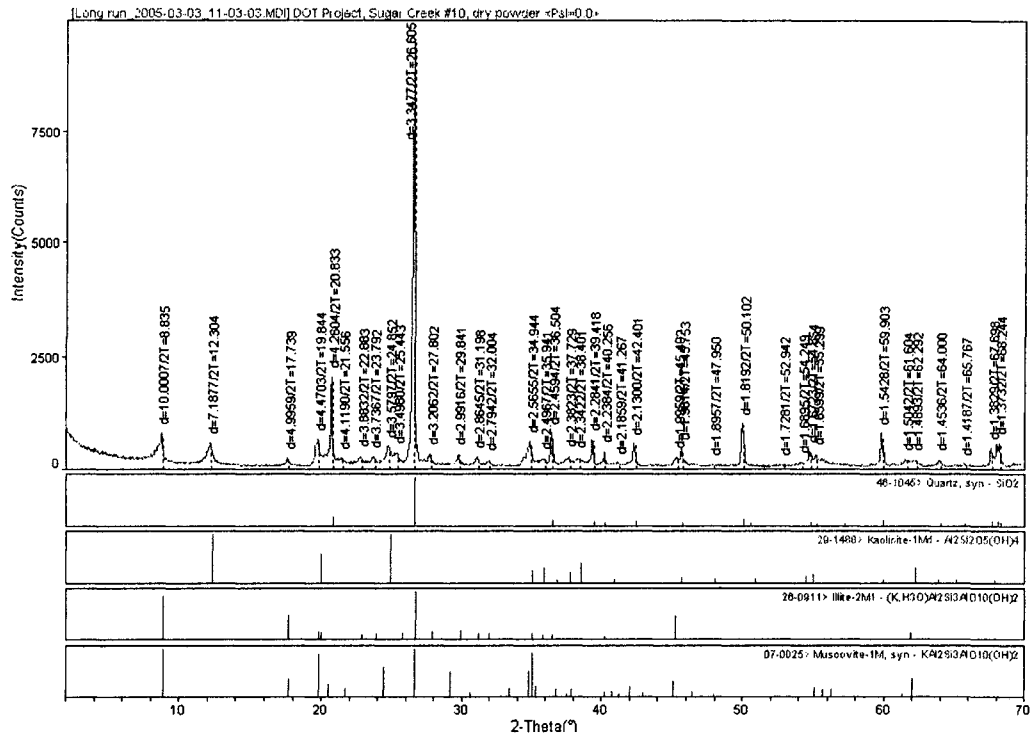


Figure B.18 XRD result (7 of 10) (CH1005, 9.2-10.7m, slightly weathered shale)

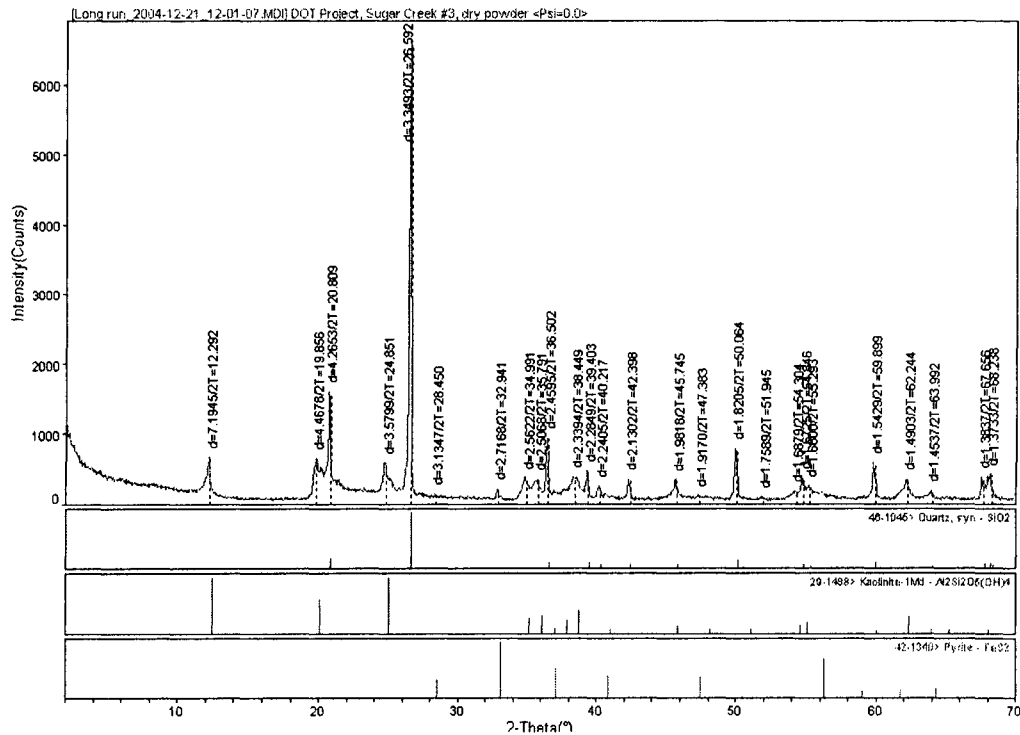


Figure B.19 XRD result (8 of 10) (CH1007, 2.0-2.6m, highly weathered shale)

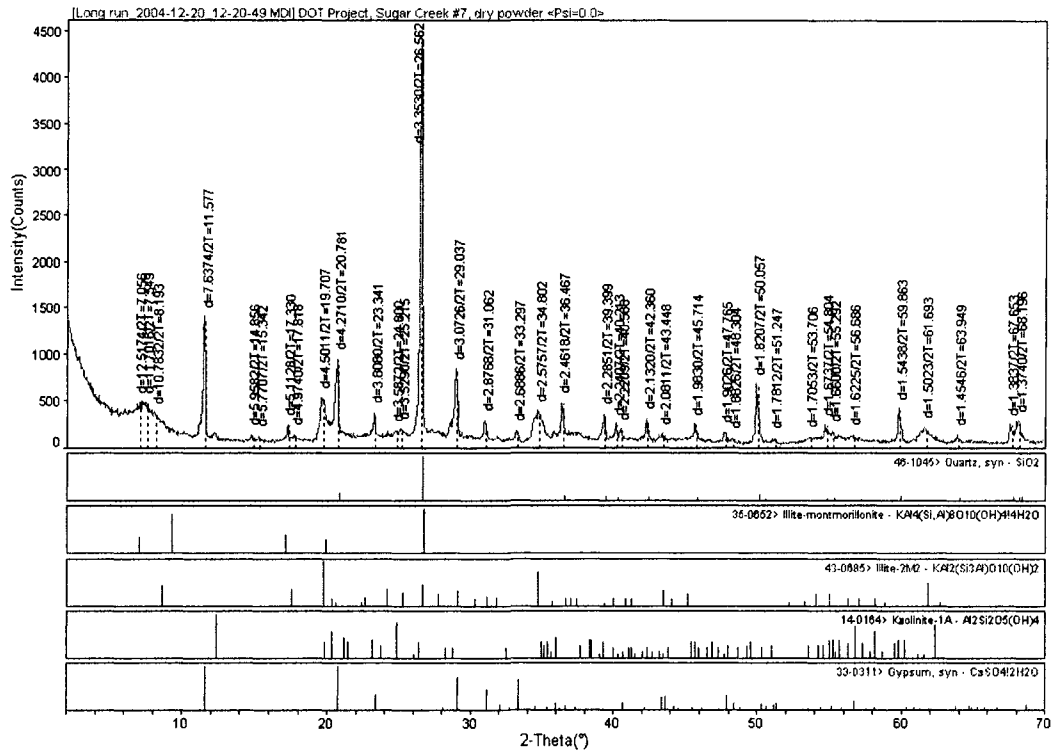


Figure B.20 XRD result (9 of 10) (CH1009, 2.4-2.7m, highly weathered shale)

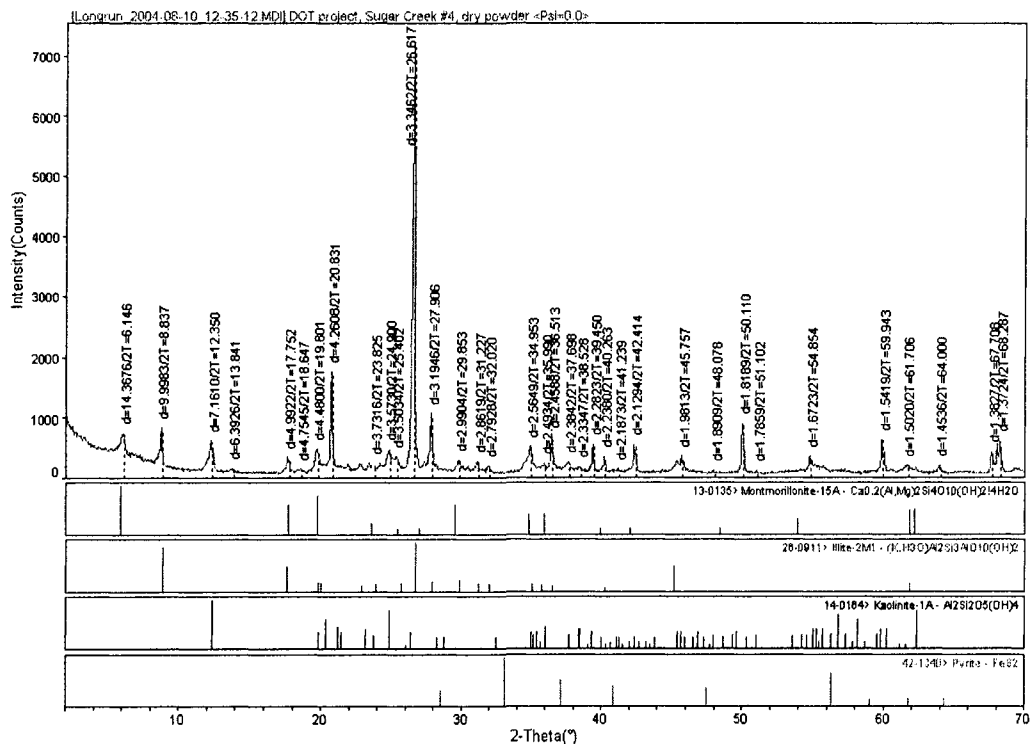


Figure B.21 XRD result (10 of 10) (CH1010, 0.6-1.2m, highly weathered shale)

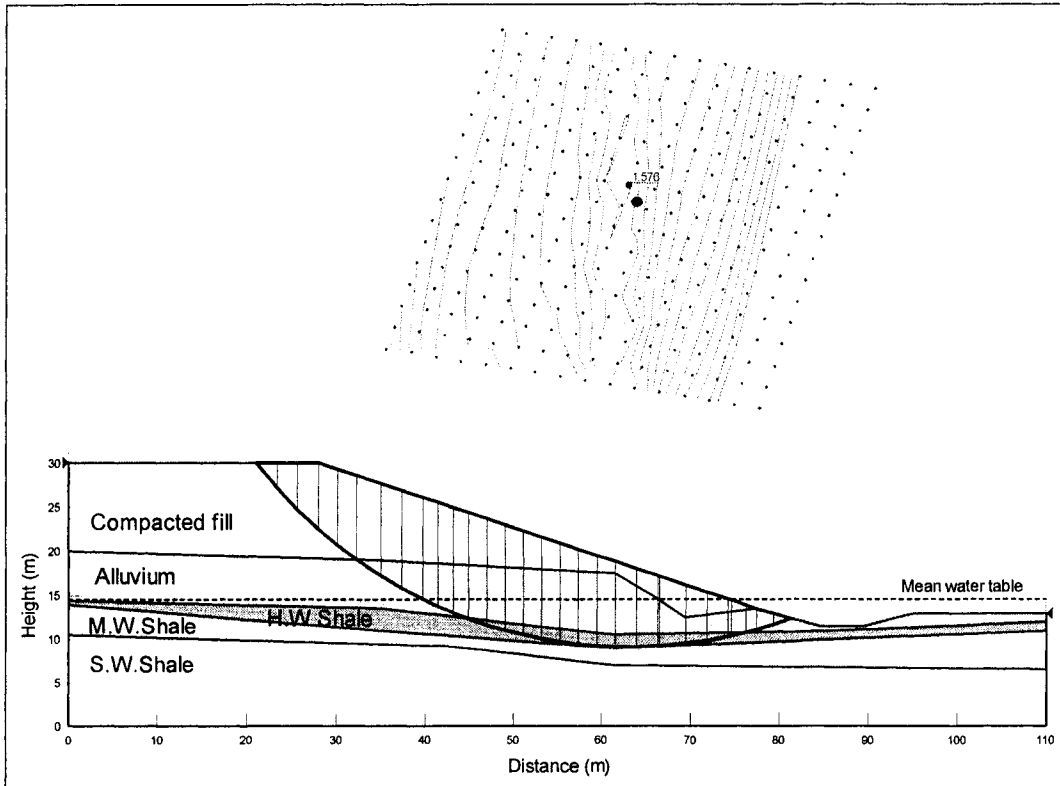


Figure B.22 Slope stability analysis using BST measurements for circular slip surface

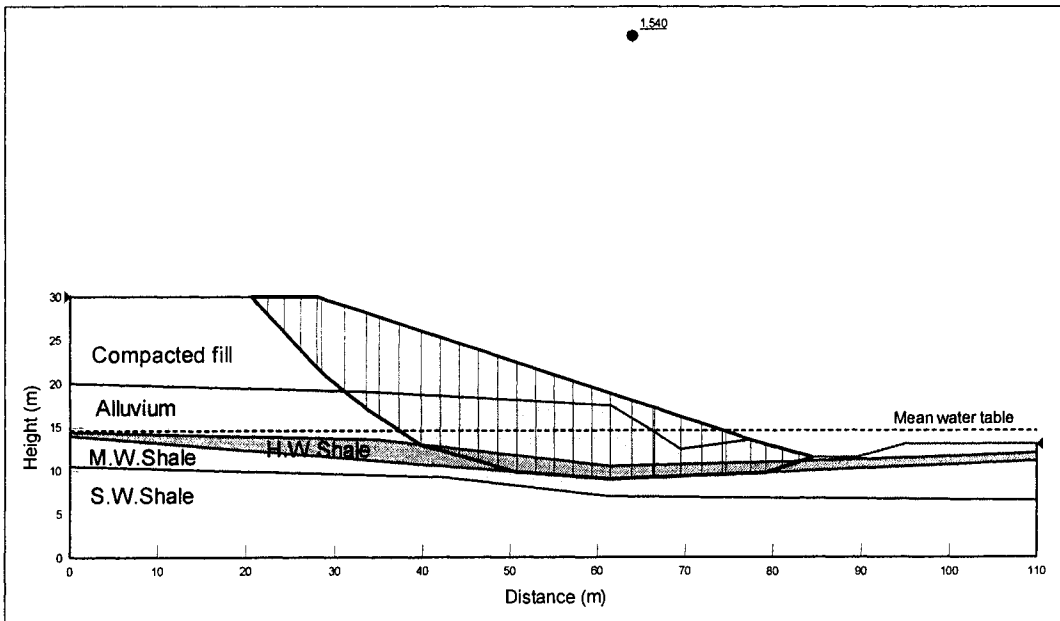


Figure B.23 Slope stability analysis using BST measurements for non-circular slip surface

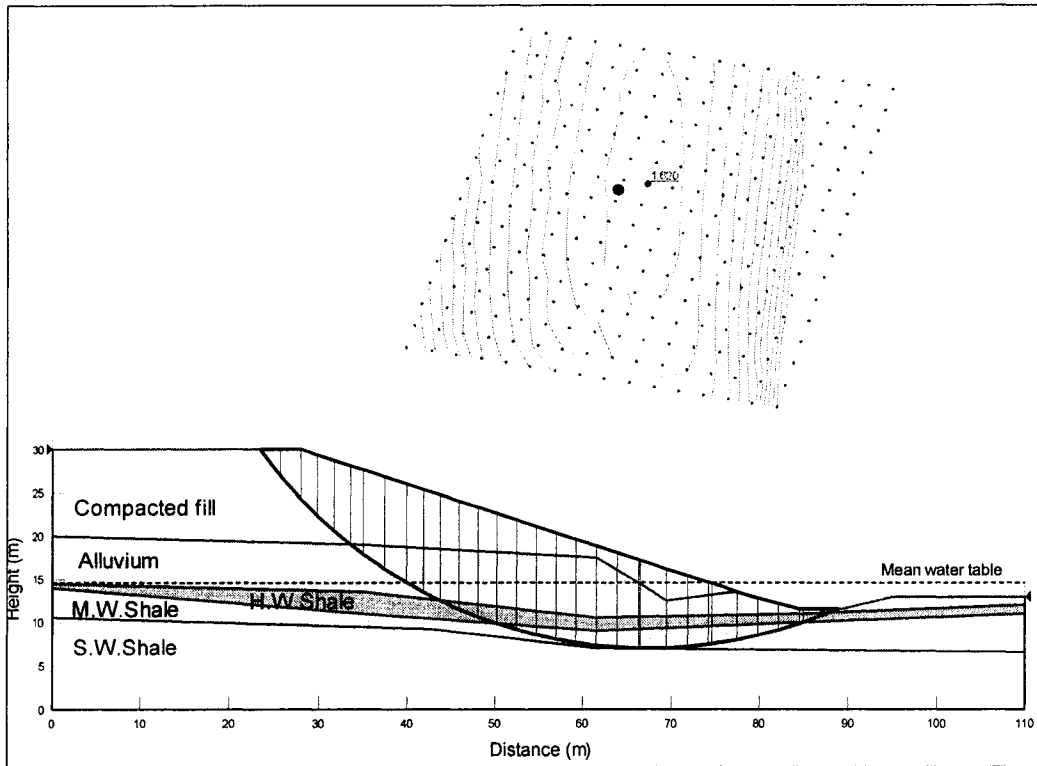


Figure B.24 Slope stability analysis using DST measurements for circular slip surface

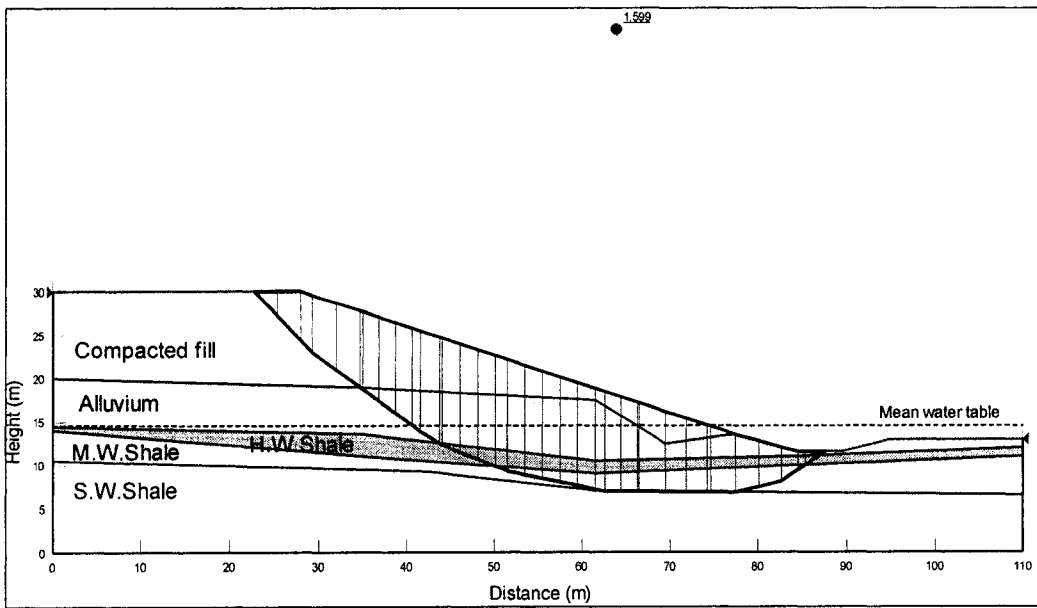


Figure B.25 Slope stability analysis using DST measurements for non-circular slip surface

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