

Characterization and stability for Cottonwood riverbank-a reliability study for BSTEM

by

Wenqian Lu

B.S., Kansas State University, 2019

A THESIS

submitted in partial fulfillment of the requirements for the degree

MASTER OF SCIENCE

Department of Civil Engineering
Carl R. Ice College of Engineering

KANSAS STATE UNIVERSITY
Manhattan, Kansas

2022

Approved by:

Major Professor
Stacey E. Kulesza

Copyright

© Wenqian Lu 2022.

Abstract

Riverbank slope failure due to excessive rainfall and flooding can be costly; however, riverbank slopes may also fail due to upstream engineering structures as an unintended consequence. Many methods and models can be used for slope stability analysis to evaluate how upstream engineered structures impact downstream riverbanks. The objective of this study is to determine an efficient implementation of soil properties for the Bank Stability and Toe Erosion Model (BSTEM), which is commonly used for riverbank stability analysis in Kansas. The stability of a bank along the Cottonwood River in Kansas was evaluated using soil properties obtained from various complex test methods. Soil properties were determined from the consolidated undrained and unconsolidated undrained triaxial tests. Correlations based on the soil type, BSTEM default soil data, and empirical correlations based on measured soil properties were also used to obtain the required BSTEM soil properties. The results of this study indicate that the soil properties obtained from an effective stress analysis, as required by BSTEM, using the consolidated undrained triaxial test provide the most accurate factor of safety. Soil properties obtained using total stress analysis and based on soil classification led to more conservative, yet reasonable factors of safety. The use of numerically-correlated soil properties was not appropriate, nor were some built-in soil properties within BSTEM. Thus, while BSTEM calls for effective stress parameters, a slightly more conservative and more simplified analysis can be conducted using either measured total stress parameters, or mean values based on soil classification. Ultimately, researchers must evaluate the appropriate risk when selecting the type of data to use for evaluating riverbank stability using BSTEM. This study recommends that BSTEM include an option for total stress analysis for future model development to represent a more critical failure case and to be more conservative.

Table of Contents

List of Figures	v
List of Tables	vii
Chapter 1 Induction.....	1
Chapter 2 Literature Review	6
2.1 Riverbank Slope Stability.....	6
2.2 Slope Stability Analysis Methods	9
2.3 Slope stability analysis model	16
2.3 Soil strength tests	19
Chapter 3 Methodology	21
3.1 Overview	21
3.2 Site description.....	21
3.3 Soil Sampling.....	22
3.4 Geotechnical analysis.....	23
3.5 Slope stability test methods.....	30
Chapter 4 Results and Discussion.....	34
4.1 Laboratory Results	34
4.2 BSTEM results	41
Chapter 5 Conclusions	49
5.1 Future work	50
References	51
Appendix A.....	55
Appendix B	58
Appendix C	64

List of Figures

Figure 2. 1 Assumed slip surfaces with soil profile (Freeman, 1974)	7
Figure 2. 2 Free surface location (Hossain et al., 2011)	8
Figure 2. 3 Illustration of Ordinary method of slices model (Anderson and Richards, 1987)	13
Figure 2. 4 Illustration of Simplified Bishop Method (Anderson and Richards, 1987)	14
Figure 2. 5 Illustration of Janbu's simplified method (Anderson and Richards, 1987).....	15
Figure 2. 6 Vertical slices method modified from CONCEPTS model (BSTEM V5.5).....	18
Figure 3. 1 Site C102 overview	22
Figure 3. 2 Soil collection in 2019.....	23
Figure 3. 3 Simple model for triaxial test setup.....	25
Figure 3. 4 Correlation between friction angle and plasticity index, developed by Terzaghi et al. (1996).....	28
Figure 3. 5 BSTEM definition of points for bank profile (BSTEM v5.5).....	31
Figure 3. 6 (a) Original bank profile and (b) reselected bank profile	32
Figure 4. 1 Failing bank profile for borehole No.3	47
Figure A. 1 Borehole No.3 data for site C102	55
Figure A. 2 Borehole No. 2 data for site C102	56
Figure A. 3 Borehole No. 2 data for site C102	57
Figure B. 1 BSTEM model required a detailed bank profile input with elevations and stations .	58
Figure B. 2 BSTEM model required an input geometry, bank layer thickness (up to 5 layers)...	58
Figure B. 3 BSTEM model required an input geometry, channel and flow parameters.....	59
Figure B. 4 Bank material could select from default list or insert own data.	59
Figure B. 5 Ground water table depth is needed for pore pressure calculation and bank failure plane will displayed with detailed bank profile	60
Figure C. 1 Mohr's circles to obtain effective friction angle and cohesion for soil sample No. 1, 2, 3.....	64
Figure C. 2 Mohr's circles to obtain effective friction angle and cohesion for soil sample No. 4, 7	64
Figure C. 3 Mohr's circles to obtain effective friction angle and cohesion for soil sample No. 10, 11.....	65

Figure C. 4 Mohr's circles to obtain effective friction angle and cohesion for soil sample No. 12, 14, 15.....	65
---	----

List of Tables

Table 2. 1 Minimum FS from slope stability analysis (Modified from Hossain et al., 2011).....	9
Table 2. 2 Triaxial tests summary.....	20
Table 4. 1 CU test effective stress analysis results for borehole No.1 and No. 3.....	35
Table 4. 2 CU tests total stress analysis results for borehole No. 1 and No. 3	36
Table 4. 3 UU test results.....	37
Table 4. 4 Effective cohesion results using Terzaghi's correlation for each soil sample.....	38
Table 4. 5 Effective cohesion using average effective friction angle for Borehole No. 1	39
Table 4. 6 Effective cohesion using average effective friction angle for Borehole No. 3.....	39
Table 4. 7 ϕ' and c' using in-situ stress obtained with correlation for PI and K_0	40
Table 4. 8 ϕ' and c' using in-situ stress obtained with correlation for friction angle and K_0	41
Table 4. 9 Laboratory CU test results for borehole No. 3.....	42
Table 4. 10 FS for Borehole No. 1 and No. 3 using numerical correlations.....	43
Table 4. 11 FS using BSTEM default data	43
Table 4. 12 FS using USCS clay data	44
Table 4. 13 Discussed results for borehole No. 3	45
Table 4. 14 Factor of safety for original and failing bank profile.....	48
Table B. 1 USGS Soil data	61
Table C. 1 BH-1 Results.....	66
Table C. 2 BH-3 Results	67

Chapter 1 Induction

During excessive rainfall and devastating flood periods, riverbank stability will be challenged by the rapid change in flowrate and water level. Furthermore, riverbanks may become unstable as rivers naturally migrate or as a result of upstream infrastructure which alters the natural flow of the river. Therefore, an accurate understanding of riverbank slope stability is a critical component for any design and development that may alter the river flow. Although programs like GEOS, SLOPE/W, and ABAQUS represent the state of the practice in geotechnical engineering, they are not commonly used by “stream practitioners” who manage and maintain rivers. This study evaluated the slope stability model within the Bank Stability and Toe Erosion Mode (BSTEM), which is commonly used by stream practitioners, including those in the Kansas Water Office, who supported this study. The objective of this research was to provide geotechnical recommendations for stream practitioners using BSTEM to simulate a riverbank stability before, during, and after a simulated flood.

The BSTEM model is considered one of the most advanced and commonly used tools for modeling riverbank retreat (Simon et al., 2000). The original Bank Stability Model (BSM) was developed by the National Sedimentation Laboratory and only included a slope stability analysis (Klavon et al., 2016). Simon et al. (2000) conducted a riverbank stability study using the original BSM model to validate the accuracy of the BSM model's predicted factor of safety. The factor of safety in this case is related to the slope stability. Simon et al. (2000) shows that the BSM was able to provide extremely close bank failure timing compared to observed failing timing. In a later development, BSM was updated with a model to determine the erosion (Klavon et al., 2016). Currently, stream practitioners and researchers in bio-agricultural engineering mainly use the BSTEM model to study erosion, sediment transport, or to evaluate proposed riverbank

stabilization methods on bank stability. For example, Heinley (2010) used BSTEM (V. 5.2) to determine how a reservoir release rates and quantity may affect riverbank erosion. Heinley (2010) identified that the most common “geotechnical” failure mechanism, the BSTEM term for slope stability failure, was a cantilever failure due to toe erosion. Similarly, Konsoer (2016) used the BSTEM (V. 5.4) to determine the rates of retreat for the Wabash River, Ill., again focusing on a failure mechanism due to soil erosion. On the other hand, the geotechnical community tends to be more focused on slope geometry and the soil strength properties to analyze the slope stability instead of a slope failure due to erosion. The most common geotechnical slope stability method is the method of slices as shown in Figure 1. 1, which is also used in the BSTEM. The method of slices is a simple yet effective way to calculate the factor of safety based on the resisting and driving forces along the most critical slip surface in a slope. State of the practice software in geotechnical engineering, such as GEOS, SLOPE/W and ABAQUS, use the method of slices, but the analysis is conducted via the finite element method (FEM). Although the BSTEM model has the ability to simulate riverbank slope failure with various slip surfaces, the model has mainly been used to simulate erosion and cantilever failure or vertical failure due to toe erosion.

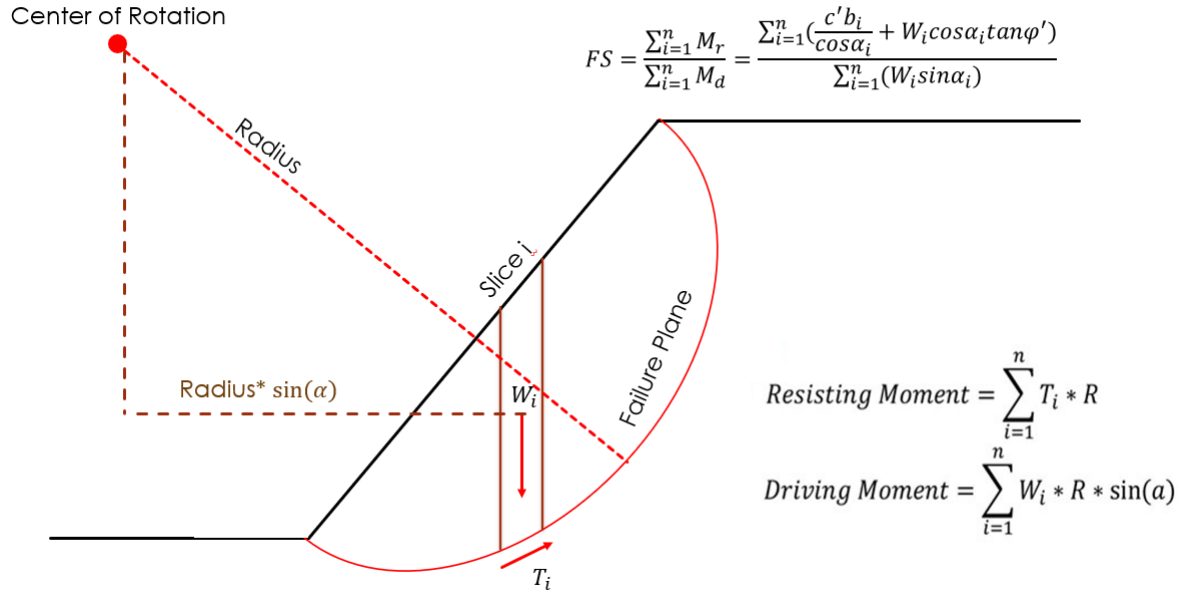


Figure 1. 1 Slope stability analysis model for the method of slices

While the BSTEM model can provide an accurate factor of safety with the appropriate soil strength parameters, a clear understanding of these parameters is critical to simulating the riverbank failure with respect to geometry. BSTEM gives little guidance on how these strength parameters should be obtained and, there are many geotechnical methods available to determine the required parameters. In this research, the soil strength properties were measured using triaxial tests. Triaxial tests are widely used by the geotechnical engineers to assess soil strength. An advantage of the triaxial test is that it can replicate in-situ stresses or a target induced stress by controlling three principal stresses (where the two in the horizontal direction are equal). Then the generalized states of stress can be examined using classic geotechnical principles, such as those by Bishop and Henkel (1962).

The most common triaxial test type is triaxial compression, as opposed to triaxial extension. There are three types of triaxial compression tests: the consolidated drained (CD) test,

the consolidated undrained (CU) test, and the unconsolidated undrained (UU) test. In a CD test, the soil sample is first consolidated, then sheared via axial compression while drainage is permitted throughout the test. Allowing for drainage simulates a long-term, effective stress analysis. In a CU test, the soil is again consolidated first, then sheared via axial compression but drainage is not allowed during the shearing phase. Thus, the CU test is a total stress analysis, however, with additional pore water pressure measurements during shearing, one can also use these data for a long-term effective stress analysis. Finally, in a UU test the soil sample is not consolidated and drainage is not allowed during shearing. The UU test is also known as the zero-friction test because when the soil sample is fully saturated, the friction angle is zero and only total cohesion, or the undrained shear strength, is obtained. The UU test is a fast, simple total stress analysis that provided undrained shear strength.

Among these three types of tests, the CD test will provide the most accurate soil properties to perform long-term drained condition analysis. However, the CD test requires a long time period to perform (2-3 weeks, minimum) and the process is very complicated. Hence the CD test is relatively more expensive than the CU test. Because the CU test with pore water pressure measurements can obtain both total and effective stress analysis, the CU test was selected for this research as the method for obtaining the highest quality data for BSTEM. UU tests were also performed to obtain the undrained shear strength to represent a quick and much more simple method for obtaining soil properties to compare with the CU test results. A total of twelve CU tests and four UU tests were performed. This study also explored other means for obtaining soil properties to represent different levels of confidence in the analysis and to provide recommendations for when triaxial testing is not possible. Assuming the BSTEM modelers have access to soil samples, soil strength properties were obtained from numerical correlations using

the Atterberg Limits as this is a very simple and routine method to classify fine-grained soil samples. Assuming BSTEM modelers knew the general soil stratigraphy, this study used average strength parameters based on the Unified Soil Classification System (USCS) and literature. Finally, the BSTEM default soil properties were also used for the most simple and general case. Standard laboratory tests were conducted on all soil samples as required for soil classification including a sieve analysis, percent passing the US Number 200 sieve, and Atterberg limits as well as water content before and after triaxial testing for quality assurance.

The objectives of this research were to make recommendations on how to obtain soil properties in a BSTEM slope stability analysis and to understand how different methods can affect the slope stability in the BSTEM model. Based on a literature review and understanding the state-of-the practice, it was determined that soil properties are often assumed or that incorrect properties are commonly used for the most critical failure case of rapid draw down. The knowledge gained from this research will contribute to future model development, allowing more accurate simulation for different flow scenarios.

There are five chapters in this thesis. The problem statement, background, and the research objectives are described in Chapter 1. In Chapter 2, a detailed literature review on pervious slope stability studies, slope stability analysis methods, and triaxial tests used in this research is described. The methodology of this research, including site description, subsurface sampling, setup for triaxial tests, methods to obtain soil properties, setup for Bank Stability and Toe Erosion Model (BSETM) are provided in Chapter 3. Chapter 4 presents the results for the site and the discussion of the results and recommendations based on the results. Finally, conclusions and future work are provided in Chapter 5.

Chapter 2 Literature Review

2.1 Riverbank Slope Stability

Bank failures can be classified according to the form of the failure surface, specifically planar or rotating. Although rotational failures are more costly in terms of land loss, they are less common along streambanks because they occur at the highest section of streambanks (Simon, 1998). Due to the dynamic nature of river channels over time, planar failures commonly occur earlier when streambanks are lower compared to the rotational failures (Simon, 1998). Planar failures can occur along any critical failure plane, regardless of whether the plane passes through the streambank's toe. Streambank failures are most frequently recorded during periods of recessive stormflow and are frequently analytically as a saturated rapid-drawdown scenario (Thorne, 1990).

There have been several studies on riverbank slope stability that are related to this research. Freeman (1974) conducted a case study to re-evaluate the strength characteristics for Lake Agassiz clays and the mechanism of failure for slopes in these layered clays. A detailed laboratory investigation was carried out on undisturbed soil samples from different sites with tan silt, brown clay, grey clay, and grey plastic clay as defined by the researcher. The riverbank slopes along Red and Assininboine Rivers used in this study were carried out by Mishtak (1964). Freeman (1974) assumed the soil profile and slip surfaces shown in Figure 2. 1.

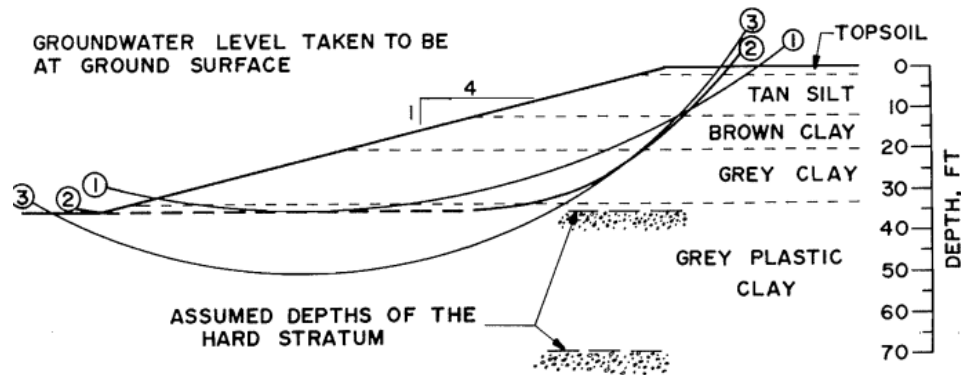


Figure 2. 1 Assumed slip surfaces with soil profile (Freeman, 1974)

With the soil profile and bank slope data, three sets of shear strength assumptions with different slip surfaces were made to perform the slope stability analysis. Freeman (1974) compared residual strength (the load that a damaged the material can still carry without failing) and peak shear strength as:

- (a) Case A. Residual shear strength parameters
- (b) Case B. Peak shear strength parameters
- (c) Case C. Similar to Case A but assume the grey clay extended to hard stratum

Freeman (1973) then carried out slope stability analyses assuming both circular and noncircular slip surfaces. The analyses were performed with software developed by Little and Price (1958) and Morgenstern and Price (1965) for circular slip surfaces and noncircular slip surfaces, respectively. Little and Price (1958) was developed based on Bishop's adaptation of the Swedish method. Morgenstern and Price (1965) was developed based on the method of slices. The Morgenstern and Price (1965) software was able to meet all equilibrium conditions compared to the existing methods. The stability analyses suggested that it is inappropriate to apply residual shear strength estimates along the entire length of the slip surface for slopes in

these clays. The results indicated that residual strength parameters result in an underestimation of the true factor of safety. Following this recommendation, this work also used peak strength over residual strength parameters.

Embankment failure and riverbank erosion are common problems in Bangladesh, every year excessive rainfall and devastating flood are accelerating the failure process led to infrastructures and agriculture damages. Hossain et al. (2011) conducted a slope stability analysis for Manu River embankment to evaluated current embankment design methodology used in Bangladesh. The free phreatic surface was assumed at the saturation line according to design manual of Bangladesh Water Development Board (BWDB) for slope stability analysis as shown in Figure 2.2. A Finite Element Method (FEM) analysis was conducted to determine the free surface via a seepage analysis. After the free surface was determined in both methods as shown in Figure 2.2, limit equilibrium slope stability analysis was conducted to determine the Factor of Safety.

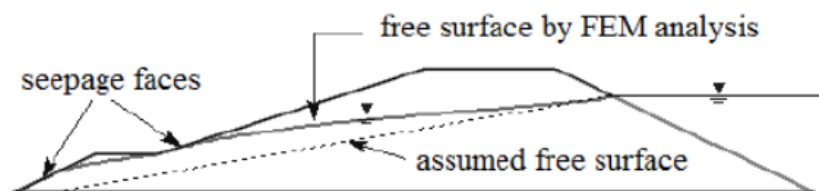


Figure 2. 2 Free surface location (Hossain et al., 2011)

The results from different methods are as shown in Table 2.1. The factor of safety was overestimated by about 22-24% in the assumed free surface.

Table 2. 1 Minimum FS from slope stability analysis (Modified from Hossain et al., 2011)

Methods	Assumed free surface	Predicted free surface	Deviation (%)
Bishop simplified	2.011	1.622	23.98
Janbu simplified	1.880	1.541	21.99
Spencer	2.012	1.622	24.04
Corp of Engrs#1	2.043	1.648	23.96
Morganstern-Price	2.011	1.622	23.98

With the overestimated factor of safety using assumed free surface, the current design method would lead to a less reliable structure especially during the excessive rainfall conditions. This study recommends when designing slope, the use of current design method should be conservative since the seepage analysis provide more reliable factor of safety. Following this recommendation, this work also tested the if current version of BSTEM results reflect the reality.

2.2 Slope Stability Analysis Methods

Slope stability analysis methods vary in their theoretical background and approach, and hence the analysis results vary depending on the used theory. Due to their simplicity and accuracy, limit equilibrium slope stability analyses still are the most popular method among geotechnical engineers. The method of slices, circular methods, and noncircular methods are the three types of two-dimensional slope stability methods that use the limit equilibrium technique. The method of slices is based on slicing the slope into distinct slices and analyzing the failing mass's stability by

considering the static equilibrium of the slices individually as well as the overall forces or moments equilibrium of the failing mass (Coduto, 1998). The circular and noncircular limit equilibrium methods consider the equilibrium of the whole failing mass only, and therefore the internal equilibrium of sliding mass is not considered (Coduto, 1998).

The use of FEM in slope stability analysis has become more widespread. In comparison to limit equilibrium approaches, FEM analysis can represent slopes with complicated geometry, loading sequences, the presence of reinforcement material, variable groundwater tables, and complex soil behavior laws. FEM also allows for a clearer visual representation of soil deformation (Matthews et al., 2014). Currently, many software programs are available for slope stability analysis, such as GEO5 (Fine, 2022), SLOPE/W (Geoslope, 2021), and ABAQUS (Simulia, 2021). While the software analysis relies on accurate soil parameters, it is critical for engineers to build up the knowledge on the principles of slope stability and understand the outputs (Cheng, 2003). Calculations of stresses and displacements in a soil mass subjected to external loads can also be performed using FEM. The stability of a slope, however, cannot be evaluated simply from the finite element analysis. For example, to calculate a factor of safety, only the estimated stresses on a slope can be used. In this regard, FEM is a complicated approach for calculating the factor of safety in slope stability problems. Therefore, simplified programs, such as BSTEM used in this research, are still commonly used by practitioners when a more complex FEM analysis is not required. Although programs such as GEOS, SLOPE/W, and ABAQUS are the state of the practice in geotechnical engineering, this research sought to make recommendations for the simplified and free software, BSTEM, commonly used by Kansas Water Office, who supported this study. The common fundamental methods and assumptions used for slope stability analyses are described below.

Limit equilibrium analysis methods

The soil mass above the slip surface is divided into a number of vertical slices in the methods of slices, and the equilibrium of each of these slices is considered. The number of slices required is determined by the slope geometry and soil profile. To obtain the factor of safety using the method of slices, it is necessary to make assumptions in order to eliminate the extra unknowns, and these assumptions are the key differences between the methods. Circular and noncircular limit equilibrium methods, unlike slice methods, only consider the equilibrium of the entire failing mass, ignoring the internal equilibrium of the sliding mass. If the slope remediation method involves installing structural elements in the sliding mass (such as a geosynthetic reinforcement), such methods may not be appropriate.

Ordinary method of slices. This method is also known as "Fellenius' Method," and it is the most basic method of slicing. The method assumes that any slice's resultant of interslice forces is parallel to its base. As a result, the interslice forces are ignored (Fellenius, 1936). Only the condition of moment equilibrium is met. In this regard, the safety factors calculated using this method are usually conservative (Whitman and Baily, 1967). It is necessary to know the failure surface before this method is applied to compute the factor of safety (Anderson and Richards, 1987).

Mohr-Coulomb failure criteria is expressed as:

$$s = c' + (\sigma - u)\tan\phi' \quad \text{Equation 2. 1}$$

where s is the shear stress (kPa or psf), σ is the total normal stress (kPa or psf), u is pore pressure (kPa or psf), c' is the effective cohesion intercept (kPa or psf), and ϕ' is the effective friction angle (degrees).

As shown in Figure 2. 3, the normal force on the base of the slice can be expressed as:

$$P = W * \cos\alpha \quad \text{Equation 2. 2}$$

where P is the normal force (kN or lb), W (kN or lb) is the weight of the slice, and α is the angle (degrees) between the tangent of the center of the base and the horizontal.

The resisting force on the base of the slice can be expressed as:

$$T = (W * \cos\alpha - u * l) + c' * l \quad \text{Equation 2. 3}$$

where T is resisting force (kN or lb), l is the base length for the slice (m or ft) and rest of the parameters as defined in the previous section.

The factor of safety is derived from the summation of the resisting forces over the summation of the driving force. The summation of the moments can be expressed as:

$$\sum W * R * \sin\alpha = \sum T * R \quad \text{Equation 2. 4}$$

where R is the radius of the assumed failure circle (m or ft). The remaining parameters have been previously defined.

Then the factor of safety with the moments summed from point O can be expressed as:

$$FS = \frac{\sum c' * l + (W * \cos\alpha - u * l) * \tan\phi'}{\sum W * \sin\alpha} \quad \text{Equation 2. 5}$$

where all parameters are previously defined.

To obtain the lowest factor of safety, the procedure repeats with various assumed failure surface until the lowest factor of safety reached.

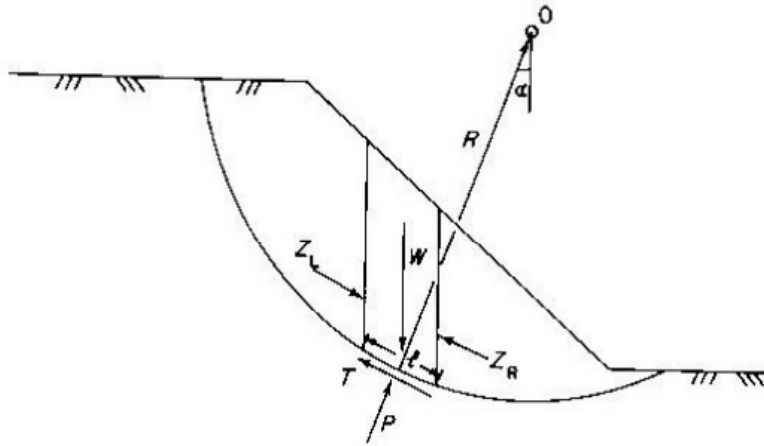


Figure 2. 3 Illustration of Ordinary method of slices model (Anderson and Richards, 1987)

The Ordinary method of slices was the principal method for many other methods such as Simplified Bishop method and Janbu's simplified method discussed below. Also, the ordinary method of slices was used to established BSTEM model used for this study.

Simplified Bishop method. For the Simplified Bishop method as shown in Figure 2. 4, it is assumed that rotation of a circular mass of soil caused the slope failure. The forces between the slices are also assumed to be acting at horizontal direction with no active shear stress between slices. Like the ordinary method of slices, Bishop's method only meets the moment equilibrium.

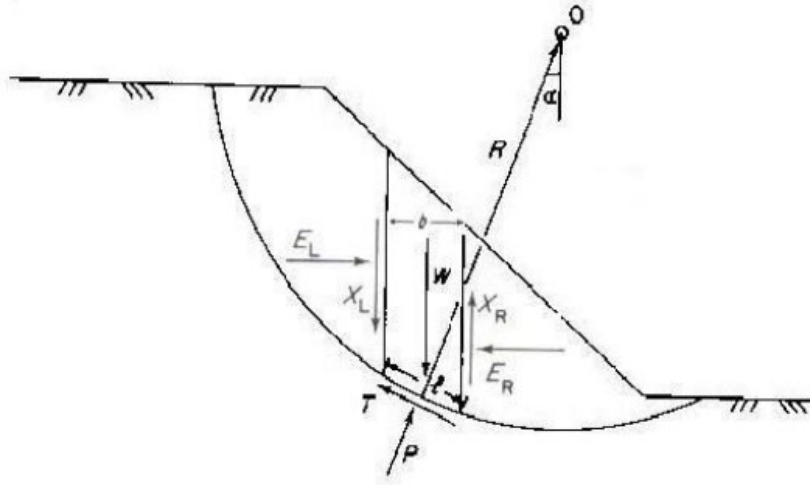


Figure 2. 4 Illustration of Simplified Bishop Method (Anderson and Richards, 1987)

The normal force is assumed to act at the center of the base of each slice, the normal force is given by the expression:

$$P = \frac{\left[W - \frac{1}{F} (c' * l * \sin\alpha - u * l * \tan\phi' \sin\alpha) \right]}{m_a} \quad \text{Equation 2. 6}$$

where,

$$m_a = \cos\alpha + \frac{(\sin\alpha \tan\phi')}{F} \quad \text{Equation 2. 7}$$

The factor of safety then calculated by taking the moments at point O:

$$F = \frac{\sum \left[\frac{c' * l * \cos\alpha + (W - u * l * \cos\alpha) \tan\phi'}{\cos\alpha + (\sin\alpha * \tan\phi')/F} \right]}{\sum W * \sin\alpha} \quad \text{Equation 2. 8}$$

where all parameters are previously defined.

With the equation has factor of safety on both sides, the factor of safety in simplified Bishop method can only be solved iteratively.

Even though the simplified Bishop method does not achieve complete static equilibrium, the procedure provides relatively accurate factor of safety values (Bishop, 1955). The Simplified Bishop method is more accurate than the Ordinary Method of Slices, particularly when effective stress analysis with high porewater pressure is required (Bishop, 1955).

Janbu's simplified method. Similar to Bishop's simplified method, Janbu assumed a horizontal interslice force and zero shear force, but he did so use force equilibrium instead of moment equilibrium (Janbu, 1954). Janbu's simplified method is applicable to non-circular slip surface as shown in Figure 2. 5. Therefore, the expression obtained for the total normal force on the base of each slice is the same as Bishop's simplified method:

$$P = \frac{\left[W - \frac{1}{F} (c' * l * \sin\alpha - u * l * \tan\phi' * \sin\alpha) \right]}{m_a} \quad \text{Equation 2. 9}$$

Then the factor of safety is obtained with overall horizontal force equilibrium:

$$FS = \frac{\sum (c' * l + (P - u * l) \tan\phi') * \sec\alpha}{\sum W * \tan\alpha} \quad \text{Equation 2. 10}$$

where all parameters are previously defined.

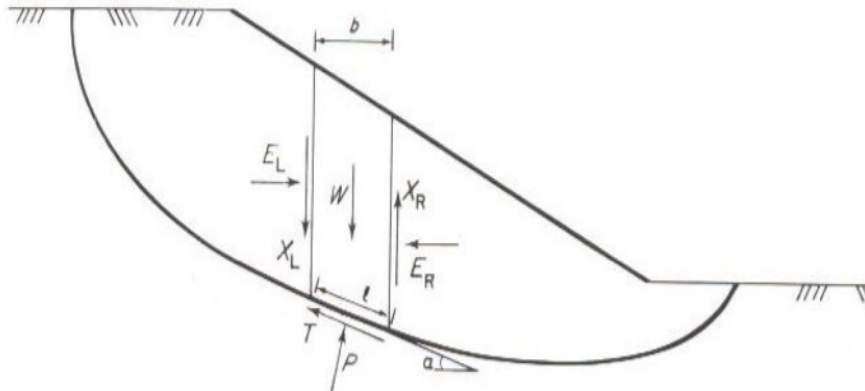


Figure 2. 5 Illustration of Janbu's simplified method (Anderson and Richards, 1987)

Interslice normal and shear forces have been shown to have a significant impact on FS in stability analyses of hillslopes and embankments (Langendoen and Simon, 2008). Janbu showed that his method may underestimate factor of safety by approximately 5% (Janbu, 1973)

Swedish circle method. This method is the simplest circular analysis used to determine short-term stability for both homogeneous and inhomogeneous slope (Abramson et al., 2001). The failure surface for this model is assumed as a rigid, cylindrical block that will fail by rotation about its center and the friction angle is zero. Therefore, the shear strength will only be due to cohesion. The Swedish circle method is simple model to determine the factor of safety for soil with zero cohesion for both total and effective stress scenarios.

2.3 Slope stability analysis model

Bank Stability and Toe Erosion Model (BSTEM).

The USDA-ARS National Sedimentation Laboratory's Bank-Stability and Toe-Erosion Model (BSTEM) has been used to quantify streambank-loading rates, lateral retreat, and potential reduction in sediment loadings under various bank-mitigation measures in a variety of geomorphic settings across the country (Simon et al., 2010). The force-equilibrium factor of safety of either planar- or cantilever-shear failure in a layered streambank is calculated using BSTEM. This research used the least BSTEM version 5.5 to conduct slope stability analysis.

The methods BSTEM simulated are horizontal layers (Simon et al., 2000), vertical slices with tension crack (Langendoen and Simon, 2008) and cantilever shear failures (Thorne and Tovey, 1981).

Horizontal layers method. The horizontal layer method is a further development of the wedge failure developed by Simon and Curini in 1998. This method using Mohr-Coulomb

failure criterion for the saturated portion of the wedge, the expression as show in Eq. 2.11. For the unsaturated portion, this method used Fredlund failure criterion (Fredlund et al., 1978) as:

$$s = c' + (\sigma - u_w)\tan\phi' + (u_a - u_w)\tan\phi'' \quad \text{Equation 2. 11}$$

where u_w is pore water pressure (kPa), u_a is pore air pressure (kPa), and ϕ'' is friction angle (degrees) with respect to changes in $(u_a - u_w)$ when $(\sigma - u_w)$ is held constant.

Then the model divided the bank profile up to five user defined layers then the factor of safety can be expressed as:

$$FS = \frac{\sum_{i=1}^I (c'_i * L_i + (u_a - u_w)_i L_i \tan\phi_i^b + [W_i \cos\beta - u_{ai} * L_i + P_i \cos(\alpha - \beta)] \tan\phi_i')}{\sum_{i=1}^I (W_i \sin\beta - P_i \sin[\alpha - \beta])} \quad \text{Equation 2. 12}$$

where c'_i is effective cohesion (kPa) of i^{th} layer, L_i is the length (meters) of the failure plane incorporated within the i^{th} layer, W_i is the weight (kg) of i^{th} layer, P_i is the hydrostatic-confining force (kN) due to external water level acting on i^{th} layer, β is the failure-plane angle (degrees) from horizontal, α is the local bank angle (degrees) from horizontal, and I is the number layers. The rest parameters are previously defined.

Vertical slices with tension crack. The vertical slice method is adapted from the method employed in CONCEPTS model (Langendoen and Simon, 2008). The vertical slice method evaluates normal and shear forces active in segments of the failure block. Figure 2. 6 shows an assumed failure block and its subdivision into slices. The bank will be divided into an equal number of slices and layers, and each slice into three subslices.

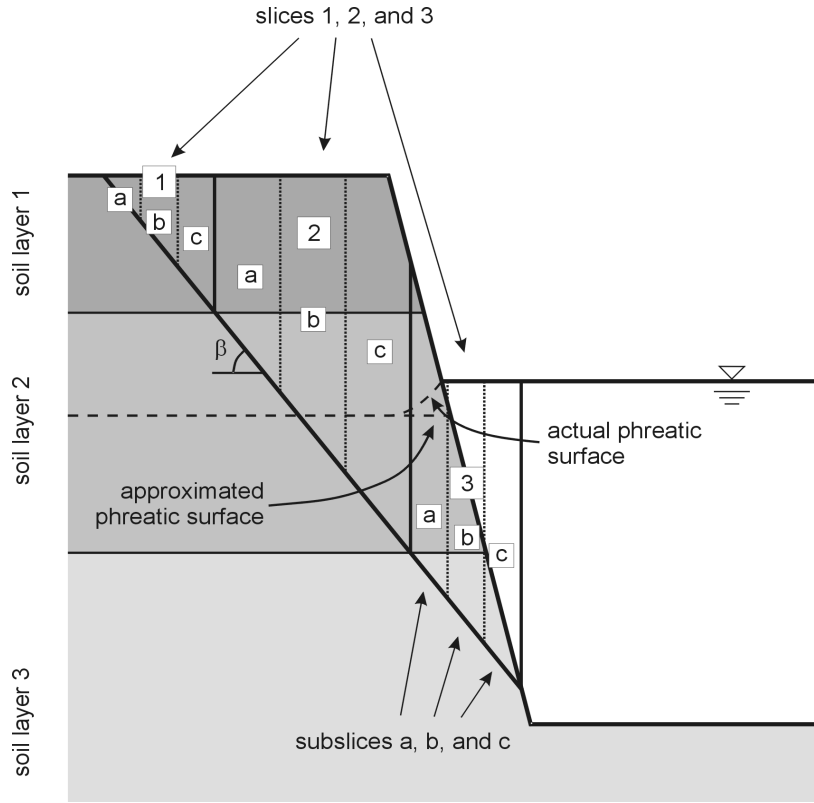


Figure 2. 6 Vertical slices method modified from CONCEPTS model (BSTEM V5.5)

The factor of safety for this model can be expressed as:

$$FS = \frac{\cos\beta \sum_{i=1}^I (c'_i * L_i + (u_a - u_w)_i L_i * \tan\phi_i^b + [N_i - u_{ai} * L_i] \tan\phi_i')}{\sin\beta \sum_{i=1}^I (N_i) - P_i} \quad \text{Equation 2. 13}$$

where all the parameters are previously defined.

Cantilever shear failures. This method is modified the horizontal layer method equation by using β as 90° . The factor of safety can be expressed as:

$$FS = \frac{\sum_{i=1}^I (c'_i * L_i + (u_a - u_w)_i L_i * \tan\phi_i^b + [P_i * \sin\alpha - u_{ai} * L_i] \tan\phi_i')}{\sum_{i=1}^I (W_i - P_i \cos\alpha)} \quad \text{Equation 2. 14}$$

where all parameters are previously defined.

The effects of vegetation on bank stability are also considered by BSTEM. Soil is generally strong in compression but weak in tension. Tree and herbaceous species' fibrous roots are strong in tension but weak in compression. As a result, root-permeated soil forms a composite material with increased strength (Thorne, 1990).

Unless the user's own pore water pressure data were used, the BSTEM Version 5.5 assumed hydrostatic conditions below the water table and a linear interpolation if matric suction was above the water table.

In the BSTEM model, a bank is considered "stable" if its factor of safety is greater than 1.3. If the FS number is between 1.0 and 1.3, the bank is "conditionally stable." If the FS value is less than 1.0, the bank is considered "unstable."

2.3 Soil strength tests

The shear strength of a soil has an impact on slope stability issues in geotechnical engineering. Soil shear strength is determined by the resistance to mass deformation created by particle movement, particle interlocking, physical connections, and chemical bonding across particles (Terzaghi, 1996). With all the slope stability methods discussed in previous section, cohesion and friction are two soil characteristics that are commonly used to express soil shear strength. To obtain these two parameters, different tests and methods could be used. For this research, triaxial tests were performed to obtain the data needed to compute friction angle and cohesion.

Most of the soil samples collected from the site were clay and based on boring logs. Undrained testing would be appropriate with these cohesive soil samples (Bishop and Henkel, 1962). Pore pressure must be monitored during the test to obtain an efficient friction angle and cohesion. The pore pressure could be determined using the consolidated drained (CD) test, but it

would take longer and have a higher probability of error than the consolidated undrained (CU) test. The Unconsolidated Undrained (UU) test is the quickest of the three triaxial tests, but it cannot determine pore pressure to perform meaningful effective stress analysis. The CU test was used for this study after considering all the variables. Table 2. 2 shows the differences between three types of triaxial tests.

Table 2. 2 Triaxial tests summary

Type of test	Duration	Type of analysis	Drainage Condition
Unconsolidated Undrained	Under 30 mins	Total Stress	Short-term
Consolidated Undrained	Days (sand) to weeks (clay)	Total and Effective Stress	Short-term and Long- term
Consolidated Drained	Weeks to months	Effective Stress	Long-term

Chapter 3 Methodology

3.1 Overview

This section includes the research methodology for this project, including field and laboratory work. In order to determine the effective friction angle and cohesion of the soil samples, triaxial tests were conducted for undisturbed samples identified as clay by boring logs. Unused soil samples from the triaxial tests were used for the liquid limit test, plastic limit test, and soil classification.

3.2 Site description

The study segment was selected from Cottonwood River located in Lyon County, Kan., south of Highway 50 as shown in Figure 3.1. The Kansas Water Office and the Kansas Department of Agriculture Division of Conservation (KDA-DOC) identified this location as a streambank erosion hotspot and highlighted it as a prospective streambank stabilization site to prevent sedimentation downstream of John Redmond Reservoir. The study team chose this location from a list of numerous suggested bank stabilization sites to investigate the effects of bank stabilization upstream and downstream of the project since the upstream and downstream meander bends were not designated erosion hot zones.



Figure 3. 1 Site C102 overview

3.3 Soil Sampling

Undisturbed soil samples were collected using 7.2-cm diameter Shelby tube. Soil samples were collected every 91 cm until the depth of termination in the sand layer at approximately 10.97 to 11.58 m. Figure 3.2 shows soil sample collection in 2019. Visual soil identifications were performed and recorded on boring logs which are included in Appendix A. After sample collection, all soil samples were sealed at both ends using plastic endcaps. Duct tape was used to seal the endcaps to the soil sample tube. This was done to preserve the in-situ water content of each soil sample during storage. All samples were labeled with the site number, borehole number, and sample number. All samples were stored in a 100% humidity-controlled room until

performing laboratory tests. A field log of soil samples collected at each borehole was created and recorded the following: site number, borehole number, borehole location, borehole depth, soil sample description, and groundwater level. For site C102, the main site in this study, total of 17 Shelby tube samples were collected and stored.



Figure 3. 2 Soil collection in 2019

3.4 Geotechnical analysis

All soil samples were characterized for basic geotechnical parameters including: water content (ASTM D 4959-16), sieve analysis (ASTM D 422-63), Atterberg limits (ASTM D 4318-17), and then classified according to the Unified Soil Classification System. The dry unit weight of each sample was determined via measurements in the Shelby tube and using phase relationships.

These data were required to fully characterize the samples and to ensure the triaxial strength tests were set up appropriately. Two types of triaxial strength tests were conducted at C102 to evaluate different levels of data that could be obtained for the B-STEM analysis, consolidated undrained (ASTM D 4767-11) and unconsolidated undrained (ASTM D 2850-15) tests.

Triaxial testing specimen preparation. The goal of conducting a consolidated undrained test was to determine the friction angle and cohesion for the soil samples from site C102. It is advantageous to run CU tests with porewater pressure measurements because it is relatively faster than a consolidated drained test and it allows for determining both the total and effective friction and cohesion parameters based on a Mohr-Coulomb analysis. In order to perform the test, the soil samples stored in the humidity room needed to be cut into two or three 20 cm length segments depending on the actual length of recovery in each tube. After the soil sample was cut, one sample was used to perform the CU test as data point one and the unused segments were resealed and stored in humidity room for future CU tests. This ensured that three tests were conducted at increasing confining stresses on the same soil to fully define the Mohr-Coulomb failure envelope. Unfortunately, some samples were not long enough or were damaged during extraction, thus some analyses only included two CU tests.

Each specimen in the testing segment was extruded by soil extruder (initially via a motor-controlled extruder and later via a hand extruder) into a 70-mm diameter, two-way split mold. The soil sample was then carefully removed from the mold without damaging the sample, especially the sides. Hand calipers were used to measure the diameter at the top, bottom, and middle of the sample to determine the average sample diameter. Similarly, the length was measured at three different locations for the average. After the measurement, the soil sample was placed into a rubber membrane using a 70-mm diameter suction membrane device. The thickness

of the membrane was measured with a hand caliper by folding it into multiple layers. Porous discs were placed on both ends of the soil sample along with filter paper. Both porous discs and filter paper were saturated prior to testing. The triaxial cell was then assembled with the trimmed soil sample and testing according to ASTM D 4767-11 commenced. A simplified schematic of the triaxial test is shown in Figure 3.3.

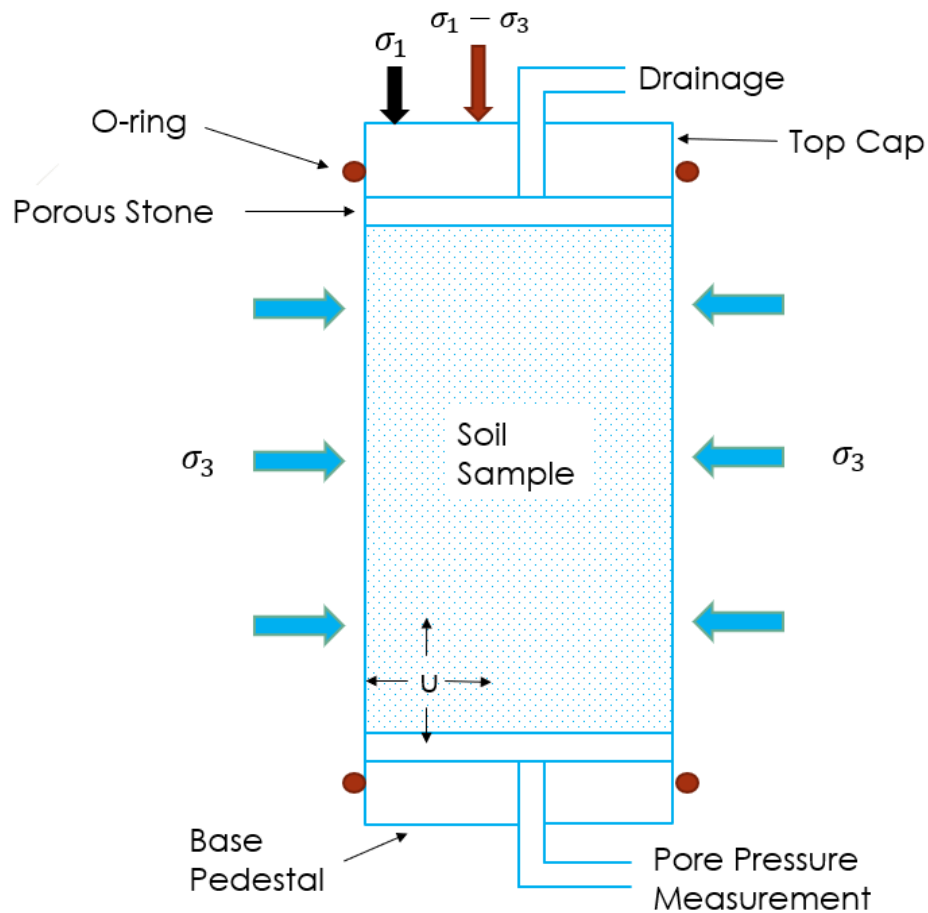


Figure 3. 3 Simple model for triaxial test setup

The first CU test was run at a confining stress near the in-situ **stress** based on the unit weight of the soil and average sample depth. The depth to groundwater was below all samples, as shown in the bore logs in Appendix A. The remaining two tests were run at higher confining

stresses to determine the Mohr-Coulomb failure envelope and required cohesion and friction angles. A porewater pressure sensor was used during all testing so that the effective stress parameters could also be determined. Some samples did not achieve sufficient material in retrieval for more than one CU test. Therefore, two samples were also tested for undrained shear strength using the UU method at the in situ confining stress. The UU test is very fast and common, therefore it was assumed this may be the only strength information available for a B-STEM analysis. Thus, it was included in this study. The sample preparation and in situ stresses for UU testing followed the same procedure as CU testing previously described.

Data processing. CU data analysis determines the major and minor effective stress for each soil sample. All cell pressure, porewater pressure, force, and displacement data were collected during testing by DS7.3 (ELE, 2018). Due to inherent fluctuations during testing, the minor effective stress was taken at the maximum deviator stress and determined by the measured cell pressure and pore pressure at 4% strain (ASTM D 4767-11) because most soil samples had peak stress around. The minor principal effective stress was calculated as:

$$\sigma'_3 = \sigma_c - u \quad \text{Equation 3. 1}$$

where σ'_3 is minor principal effective stress, σ_c is cell pressure, u is porewater pressure

The major principal effective stress was determined by minor effective stress and deviator stress at 4% strain applied to the specimen. The major effective stress was calculated as:

$$\sigma'_1 = \sigma'_3 + \sigma_d \quad \text{Equation 3. 2}$$

where σ'_1 is major principal effective stress, σ_d is deviator stress.

The UU test data were used to determine the undrained shear strength. All pressure and displacement data were collected during testing by DS7.3 software. With measured soil sample area and strain, the corrected area was calculated as:

$$A_c = \frac{A}{1 - \varepsilon} \quad \text{Equation 3.3}$$

where A_c is corrected area, A is original sample area, ε is measured strain.

Then deviator stress was calculated as:

$$\sigma_d = \frac{F}{A_c} \quad \text{Equation 3.4}$$

where F is measured force.

For UU test, the confining pressure or cell pressure is minor principal pressure, and major principal pressure calculated as:

$$\sigma_1 = \sigma_3 + \sigma_d \quad \text{Equation 3.5}$$

where σ_1 is major principal total stress, σ_3 is minor principal total stress.

For CU test data, two or more sets of major and minor effective stresses were needed to draw the Mohr's circles to determine the failure plane which provided the effective friction angle and cohesion. For UU test data, two sets of major and minor total stresses were needed to draw the Mohr's circles to determine the failure plane which provided the undrained shear strength.

Terzaghi's numerical correlation. For normally consolidated clay-like soil, Terzaghi et al. (1996) proposed a relationship between effective friction angle and plasticity index as shown in Figure 3.4. This correlation is applicable for soil with a plasticity index less than 60.

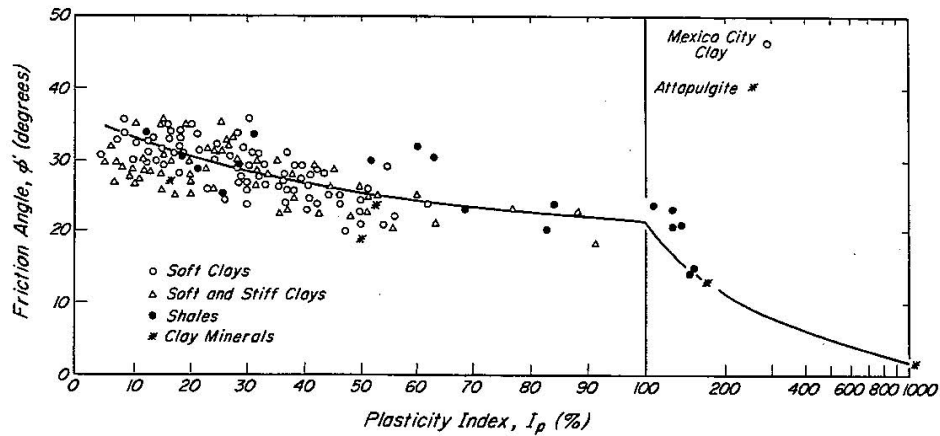


Figure 3. 4 Correlation between friction angle and plasticity index, developed by Terzaghi et al. (1996)

The correlation also can be expressed as the following equation:

$$\phi' = 35.7 - 0.28PI + 0.00145PI^2 \quad \text{Equation 3.6}$$

where PI is plasticity index.

After the effective friction angle was obtained using the Terzaghi et al. (1996) approximation, Mohr's circle was drawn with the laboratory measured major and minor effective stresses. The failure plane with the calculated angle was drawn tangent to Mohr's circles and projected backwards to obtain the cohesion.

Sorensen's numerical correlations. Alternative to Terzaghi's correlation, Sorensen and Okkels (2013) developed four different correlations to calculate effective friction angle based on plasticity index and different soil conditions. These correlations are more suitable for clay-like soil with clay friction (CF) less than 90% due to the available soil data when developing these equations as shown in Figure 3.3 (Sorensen and Okkels, 2013). In this research, these

correlations were used because plasticity index (PI) has a greater influence than CF does (Sorensen and Okkels, 2013).

For normally consolidated samples, the effective friction angle was calculated as:

$$\phi' = 39 - 11\log(PI) \quad \text{Equation 3.7}$$

$$\phi' = 43 - 10\log(PI) \quad \text{Equation 3.8}$$

where Eq. 3.7 is the lower bound of the friction angle (ϕ'), Eq. 3.8 is the mean value for friction angle. For over consolidated samples, the effective friction angle was calculated as:

$$\phi' = 39 - 6\log(PI) \quad \text{Equation 3.9}$$

$$\phi' = 45 - 14\log(PI) \quad \text{Equation 3.10}$$

where Eq. 3.6 is the lower bound of the friction angle, Eq. 3.7 is the mean value for friction angle. The cohesion for these relationships were obtained like Terzaghi's correlation approach as described above.

In-situ soil property. Effective friction angle and cohesion also were obtained from in-situ stresses. In order to draw the Mohr's circle, the coefficient of earth pressure at rest, K_0 , was calculated using PI and friction angle. The correlation between PI and K_0 was established for normally consolidated clay soils by Massarsch (1977) using Brooker and Ireland (1965) results. The correlation can be expressed as:

$$K_0 = 0.44 + 0.42 \left(\frac{PI}{100} \right) \quad \text{Equation 3.11}$$

Brooker and Ireland (1965) also provided the correlation between effective friction angle and PI as:

$$K_0 = (1 - \sin\phi') \quad \text{Equation 3.12}$$

With the coefficient of earth pressure, the lateral earth pressure at rest (i.e., the horizontal stress) could be obtained as:

$$\sigma_h = K_0 \sigma_v \quad \text{Equation 3. 13}$$

where σ_h is the horizontal stress, and σ_v is the vertical stress. Friction angle and cohesion were then obtained followed the same procedure as Terzaghi's correlation previously described.

General soil property. Soil properties for inorganic clays (CL) of low to medium plasticity were used according to Ghavami (2017). The data used in BSTEM were saturated cohesion and friction angle ranged from 11 to 15 kPa and 26 to 30 degrees. The mean, lower boundary, upper boundary values were used during analysis. This table include many soil types, and the full table can be found in Appendix B.

BSTEM soil property. BSTM has a set of default soil property data pre-loaded. These data were used to simulate the situation when no laboratory results are available or when a preliminary analysis is needed to provide a general idea about the slope stability.

3.5 Slope stability test methods

There are many different methods to calculate the slope stability as described in Chapter 2, for the purpose of the research, Bank Stability and Toe Erosion Model (BSTEM) (USDA, 2016) was used to evaluated to bank slope stability. As mentioned earlier, BSTEM is a simple and free model commonly used by agriculture engineers.

Bank stability and toe erosion model (BSTEM). BSTEM is a simple model that combines a horizontal layer method, vertical slices method, and cantilever failure method to evaluate the stability for riverbank. For site C102, the bank station and elevation were scanned at three boring locations in 2018. BSTEM's definition of points used in bank profile as shown in

Figure 3.5. The model requires the input bank profile data match the defined points. All the required input parameters for BSTEM can be found in Appendix B.

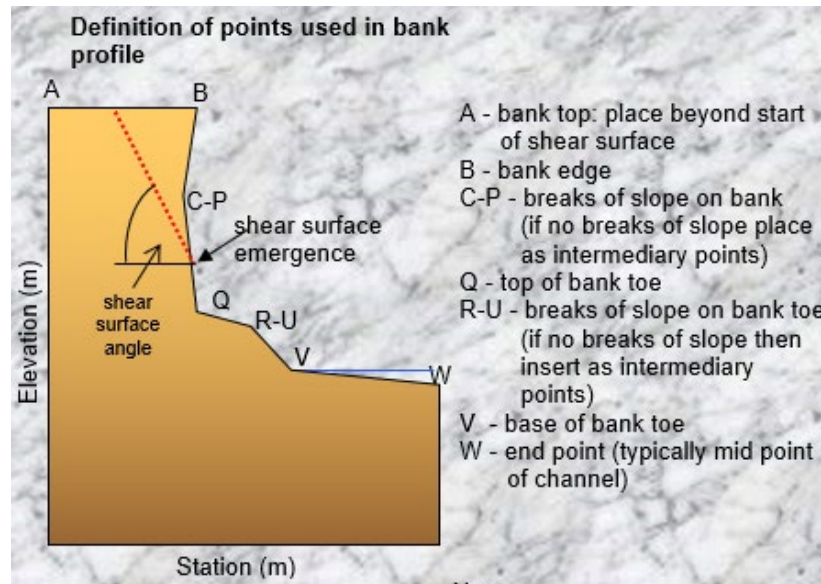
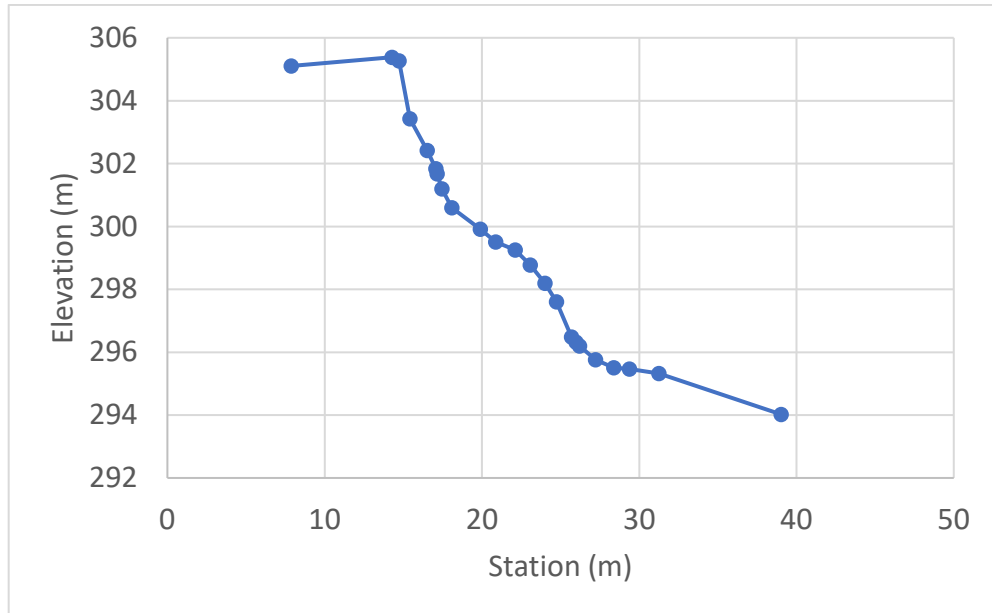
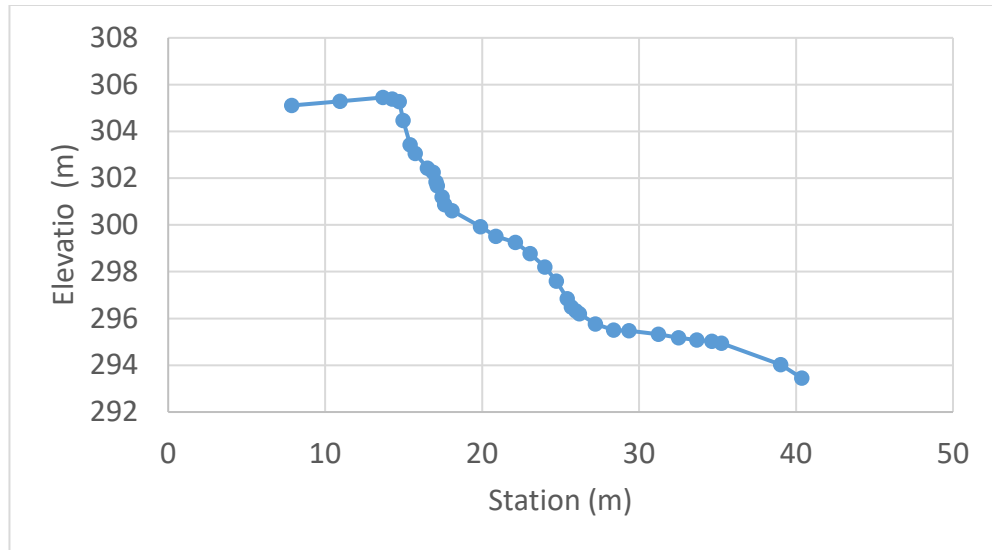


Figure 3. 5 BSTEM definition of points for bank profile (BSTEM v5.5)

For this research, the scanned station and elevation data points exceeded the required numbers. All input bank profile data for site C102 were reelected to match the BSTEM-defined points. The original bank profile and reselected bank profile for borehole No. 1 as shown in Figure 3. 6.



(b)

Figure 3. 6 (a) Original bank profile and (b) reselected bank profile

BSTEM processes bank profile data from left to right as shown in Figure 3.5, therefore relative elevations that are measured from right to left (i.e., a mirror image of Figure 3.5) must be transformed to follow BSTEM protocol. The program cannot solve for the most critical failure

surface without this transformation and will run for several hours without a solution compared to several minutes as designed.

For the reach length and water flow rate, the data were obtained from the USGS water gate (USGS, 2021). The BSTEM model calculates the Factor of Safety for bank slope and bank toe, however, only bank slope analysis was used for this research. BSTEM also is able to take vegetation protection into account, for this research, no vegetation protection was added during analysis due to the nature of the site. The water tables for both bore hole were set as field observation. With the soil properties obtained with the CU tests, Terzaghi's correlation, Sorenson's correlation, ASSHTO soil classification, in-situ stresses, and BSTEM default soil properties, the BSTEM was able to calculate the factor of safety for the bank slope. The water table level was set with the data from the subsurface investigations as shown in boring logs in Appendix A. The analysis was performed for both before and during flood water level in 2019.

Chapter 4 Results and Discussion

4.1 Laboratory Results

The soil parameters used in BSTEM were obtained from various sources. Table 4.1 shows the CU test results for borehole No.1 and borehole No.3 with shearing rates ranging from 0.017 to 0.1mm/min. Borehole No.2 results were not included because most of soil samples were damaged during preliminary testing during training to learn the CU test procedure as well as several unpredictable equipment/laboratory outages. Specify there were at least two times that the compressed air in the building was failed and damaged the samples. Thus, recuperating these data were not practical and did not add value to the study. The shearing rate for borehole No.1 was chosen based on the visual soil description on the boring logs in Appendix A. For borehole No.3, soil samples 1, 2, and 3 were tested by a previous researcher. Sample 4 was tested at the same shearing rate as sample 7 due to the same visual soil description so they could be combined for analyses. The effective friction angles and effective cohesions are shown in Table 4.1. All effective friction angles are in the range of natural clay soil according to Ouyang and Mayne (2017).

Table 4. 1 CU test effective stress analysis results for borehole No.1 and No. 3

Borehole No.	Sample No.	Shearing Rate (mm/min)	Confining Pressures (kPa)	Minor Effective Stress (kPa)	Major Effective Stress (kPa)	ϕ' (°)	c' (kPa)
1	14 B	0.1	105	46	157	26.0	12
1	15T	0.1	150	80	229	26.0	12
1	15B	0.1	200	74	210	26.0	12
3	1	0.017	40	37	177	30.6	18
3	2	0.017	60	35	181	30.6	18
3	3	0.025	110	29	166	30.6	18
3	4	0.017	100	51	239	35.6	12

The total stress analysis was performed using based on CU tests results. The total friction angle and cohesion for borehole No. 1 and No. 3 are as shown in Table 4. 2.

Table 4. 2 CU tests total stress analysis results for borehole No. 1 and No. 3

Borehole No.	Sample No.	Shearing Rate (mm/min)	Confining Pressures (kPa)	Minor Total Stress (kPa)	Major Total Stress (kPa)	ϕ (°)	c (kPa)
1	14 B	0.1	105	250	361	8	24
1	15T	0.1	150	292	441	8	24
1	15B	0.1	200	239	375	8	24
3	1	0.017	40	231	371	1	62
3	2	0.017	60	258	404	1	62
3	3	0.025	110	360	497	1	62
3	4	0.017	100	296	484	11	7

Table 4. 3 shows the UU test results for borehole No. 1 and No. 3. The UU tests were performed with shearing rate chosen based on visual soil description. All soil sample shown in the table were described as sandy clay. The confining pressures were selected based on in-situ stress for sample 5T and sample 6.

Table 4. 3 UU test results

Borehole No.	Sample No.	Shearing Rate (mm/min)	Confining Pressures (kPa)	Major Total Stress (kPa)	s_u (kPa)
3	5T	0.0167	160	242	28
3	5B	0.0167	190	309	28
3	6	0.2	160	189	11
2	12B	0.1	201	252	11

In Table 4. 4, the effective friction angles were calculated using the Terzaghi et al. (1996) correlation with PI. The calculated friction angles are all in the range of natural clay soil. The effective cohesion was first obtained with only one Mohr's circle using measured minor and major effective stress for each soil sample. The angle of failure plane was defined as:

$$\alpha_f = 45^\circ + \frac{\phi'}{2} \quad \text{Equation 4. 1}$$

where α_f is the failure plane, and other variables have previously been defined. This failure plane defines the point of tangency on Mohr's circle where the Mohr-Coulomb failure surface touches the circle at the calculated angle, ϕ' , allowing for back calculation via projection of the failure envelope to determine c' . For sample 15T and 15B, when using only one Mohr circle, the cohesion would be negative if using the defined failure plane as Eq. 4. 1. Therefore, it was assumed these were normally consolidated samples with zero cohesion.

Table 4. 4 Effective cohesion results using Terzaghi's correlation for each soil sample

Borehole No.	Sample No.	Plasticity Index	Minor Effective Stress (kPa)	Major Effective Stress (kPa)	ϕ' (°)	c' (kPa)
1	14 B	20	46	157	30.7	4.9
1	15T	20	80	229	30.7	0
1	15B	19	74	210	30.9	0
3	1	42	37	177	26.5	25.5
3	2	38	35	181	27.2	27.0
3	3	44	29	166	26.2	28.9
3	4	37	51	239	27.3	31.4

Table 4. 5 and Table 4. 6 show the average effective friction angles for the entire borehole calculated using Terzaghi et al. (1996) and Sorensen and Okkels (2013) correlations. In these two tables, to be consistent with the average effective friction angle, the average effective major and minor principal stress for the entire borehole was used. In Table 4. 5, for Terzaghi and Sorensen NC mean correlations, the cohesion would be negative when using Eq. 4. 1 defined failure plane. Like Sample 15T and 15B, zero cohesion was used for Terzaghi and Sorensen NC mean results.

Table 4. 5 Effective cohesion using average effective friction angle for Borehole No. 1

	Terzaghi	Sorensen NC Lower	Sorensen NC Mean	Sorensen OC Mean	Sorensen OC Lower
Avg ϕ' (°)	30.8	24.8	30.1	26.9	22.2
Avg Major Stress (kPa)	198.7	198.7	198.7	198.7	198.7
Avg Minor Stress (kPa)	66.7	66.7	66.7	66.7	66.7
C' (kPa)	0	11.8	0	7.8	17.7

Table 4. 6 Effective cohesion using average effective friction angle for Borehole No. 3

	Terzaghi	Sorensen NC Lower	Sorensen NC Mean	Sorensen OC Mean	Sorensen OC Lower
Avg ϕ' (°)	26.6	21.2	26.8	22.4	20.3
Avg Major Stress (kPa)	174.7	174.7	174.7	174.7	174.7
Avg Minor Stress (kPa)	33.7	33.7	33.7	33.7	33.7
C' (kPa)	27.5	35.3	26.5	33.3	37.3

The friction angle and cohesion shown in Table 4. 7 and Table 4. 8 were obtained with in-situ stress. The horizontal stresses were obtained using the coefficient of earth pressure at rest, K_0 . For Table 4. 7, K_0 was calculated with the plasticity index. For Table 4. 8, K_0 was calculated with Terzaghi's friction angle. The friction angles obtained from these two methods are in the range of natural clay soil according to Ouyang and Mayne (2017). However, the cohesions

obtained are very small compared to the CU test results in Table 4. 1 and compared to the values known for general clay.

Table 4. 7 ϕ' and c' using in-situ stress obtained with correlation for PI and K_0

Borehole No.	Sample No.	Plasticity Index	K_0	Sample Depth (m)	In-situ Vertical Stress (kPa)	In-situ Horizontal Stress (kPa)	ϕ' (°)	c' (kPa)
1	14 B	20	0.524	5.5	100.1	52.5	18.2	0.3
1	15T	20	0.524	7.7	143.3	75.1	18.2	0.3
1	15B	19	0.519	9.5	176.3	91.6	18.2	0.3
3	1	42	0.616	1.5	30.2	18.6	13.7	0.1
3	2	38	0.599	4.0	80.1	48.1	13.7	0.1
3	3	44	0.624	6.0	119.3	74.6	13.7	0.1
3	4	37	0.595	8.0	158.6	94.4	13.7	0.1

Table 4. 8 ϕ' and c' using in-situ stress obtained with correlation for friction angle and K_0

Borehole No.	Sample No.	Terzaghi Friction angle	K_0	Sample Depth (m)	In-situ Vertical Stress (kPa)	In-situ Horizontal Stress (kPa)	ϕ' (°)	c' (kPa)
1	14 B	30.7	0.489	5.5	100.1	49.0	19.4	0.1
1	15T	30.7	0.489	7.7	143.3	70.2	19.4	0.1
1	15B	30.9	0.486	9.5	176.3	85.7	19.4	0.1
3	1	26.5	0.553	1.5	30.2	16.8	17.3	0.4
3	2	27.1	0.543	4.0	80.1	43.6	17.3	0.4
3	3	26.2	0.558	6.0	119.3	66.7	17.3	0.4
3	4	27.3	0.541	8.0	158.5	85.8	17.3	0.4

4.2 BSTEM results

The following section is the factor of safety results obtained from the BSTEM model. The model was run with before, during, and after the flood stream elevation for borehole No. 1 and No. 3. From the output, the before and after flood results are identical, so only before and during flood results are discussed in this section. Since each borehole has outliners, those results are discussed as well. The complete results for both boreholes can be found in Appendix B. The factors of safety based on the laboratory test results for borehole No. 3 are shown in Table 4. 9. Borehole No. 1 results are not included in this table since only the soil samples from borehole No. 3 were used to perform UU tests. If the factor of safety (FS) from the model shows “Stable”, it means the FS is greater than 999999. From the results, this slope will not fail even during the flood and both total stress analyses show more conservative results than the effective stress analysis.

Table 4. 9 Laboratory CU test results for borehole No. 3

Method	Factor of Safety	
	Before Flood	During Flood
CU Effective stress analysis	6.25	Stable
CU Total stress analysis	4.21	14.19
UU Test Results	3.71	16.34

All results from numerical correlations are shown in Table 4. 10. For borehole No. 3, the results from correlations are close to each other for before flood and comparable to the CU tests results. The results are highly variable for the “during flood” condition compared to CU tests results. For borehole No. 1, the FS using Terzaghi’s correlation with average principal stress is smaller than effective stress analysis result and make this bank unstable. Note that the existing banks are stable, therefore this result is invalid. This was due to the use of zero cohesion, assumed when the correlation provided a negative cohesion. The same issue was observed with the Sorensen NC mean value correlation. For both boreholes, the in-situ stress factors of safety are in the unstable range, which again, is not the case. With the variable outputs, these correlations are considered as not reliable to use in BSTEM.

Table 4. 10 FS for Borehole No. 1 and No. 3 using numerical correlations

	Factor of Safety			
	Borehole No. 1		Borehole No. 3	
Method	Before Flood	During Flood	Before Flood	During Flood
Φ' w/ Terzaghi, avg stress	0.37	13.73	8.85	Stable
Φ' w/ Terzaghi by layers	1.61	20.51	8.24	Stable
Φ' w/ Sorensen NC low, avg stress	2.34	16.47	8.55	120.03
Φ' w/ Sorensen NC mean, avg stress	0.36	13.21	8.58	Stable
Φ' w/ Sorensen OC low, avg stress	3.02	15.48	7.97	222.89
Φ' w/ Sorensen OC mean, avg stress	1.89	16.84	9.74	93.55
In-situ stresses $K_0(\text{PI})$ by layers	0.34	7.14	0.24	6.16
In-situ stresses $K_0(\Phi')$ by layers	0.27	7.73	0.45	8.72

Table 4. 11 shows the FS obtained with BSTEM default soil data. The stiff clay data results are comparable to lab results. The soft clay data were not close to the actual results mainly because most of the soil samples used in this study are stiff clay.

Table 4. 11 FS using BSTEM default data

	Factor of Safety			
	Borehole No. 1		Borehole No. 3	
Method	Before Flood	During Flood	Before Flood	During Flood
BSTEM Default Soft Clay Data	10.53	30.95	12.85	Stable
BSTEM Default Stiff Clay Data	3.87	21.11	8.22	70.4

In Table 4. 12, the USCS clay data were used to obtain a FS. Because the USCS data were undrained soil parameters, the results were more comparable to the total stress analysis.

Table 4. 12 FS using USCS clay data

	Factor of Safety			
	Borehole No. 1		Borehole No. 3	
Method	Before Flood	During Flood	Before Flood	During Flood
USCS Clay Data Low	4.03	Stable	2.33	19.59
USCS Clay Data Mean	4.69	Stable	2.67	25.05
USCS Clay Data High	5.35	Stable	3.02	28.84

Based on the results shown in the previous sections, the effective friction angle and cohesion obtained from numerical correlations likely lead to unreliable factor of safety. Although both slopes were not failing, half of the numerical correlation results showed that borehole No. 1 were unstable before the flood. The BSETM default data define the parameters differently with soft clay and stiff clay. The factor of safety obtained from the stiff clay was more comparable to the CU tests results. The USCS soil properties gave more conservative results compared to CU and UU tests results, and did not show the slope as unstable, which matched the slope profile. Compared to the CU test results, most total stress analysis results as shown in Table 4. 13 are comparable without large deviation.

Table 4. 13 Discussed results for borehole No. 3

Method	Factor of Safety	
	Before Flood	During Flood
CU Effective stress analysis	6.25	Stable
CU Total stress analysis	4.21	14.19
UU Test Results	3.71	16.34
BSTEM Default Stiff Clay	8.22	70.4
USCS Clay Data Low	2.33	19.59
USCS Clay Data Mean	2.67	25.05
USCS Clay Data High	3.02	28.84

The BSTEM model requires an effective stress analysis (based on the program inputs) to gain the soil parameters. Note that an effective stress analysis is considered as drained, or long-term condition, whereas total stress analysis is considered as the undrained, short-term condition. As previously noted in the literature review and directly by Thorne (1990), the risk of bank failure is highest from the saturated rapid drawdown scenario which is the undrained condition. As discussed above, when using the undrained condition results, the factor of safety stays in the range without large deviations. The data from the previous discussion shows that, the effective stress analysis provides most accurate results since it follows the requirements in BSTEM. The total stress analysis and general soil properties from USCS and BSTEM provided more conservative results. While the CU test could provide measurement for pore water pressure to perform effective stress analysis, the CU test procedure is complex and takes a long time (days to weeks) to finish compared to the UU test (hours). The length of the experiment and many

stages required in the process make the CU test prone to mistakes. In fact, several samples were damaged in this study due to errors during testing or due to the equipment failing from a loss of compressed air in the building over weekends resulting in ruined samples. Also, the CU test is more expensive than the UU test. With these observations, the following recommendations of how to use data in BSTEM were made:

1. With the existing version of BSTEM, if time and budget allow, conduct CU tests with pore water pressure to obtain the most accurate factor of safety.
2. With the existing version of BSTEM, if time and budget are limited but samples are available, conduct UU tests to obtain the factor of safety.
3. With the existing version of BSTEM, if one only needs to gain a general picture of the slope stability, analysis based on USCS ranges may be used, but a clear understanding of the soil is a must to avoid inaccurate results.

For BSTEM itself, while the current version requires one to conduct CU tests with pore water pressure measurements to obtain effective stress analysis results, it may not be the appropriate analysis method. Again, since the most critical failure case in riverbank slope stability is more commonly related to short-term rapid drawdown, the undrained condition with total stress analysis is more appropriate. It is recommended that BSTEM consider total stress analysis parameters in future versions to allow the user simulate multiple failure cases and be closer to real-life scenarios.

To validate these recommendations, a force to fail test was conducted for Borehole No. 3. The test was conducted for the same methods shown in Table 4. 1 but the slope profile was changed to find the failure point for the slope using the CU test effective stress analysis. This new failing slope profile (shown in Figure 4. 1) was then used with the soil properties for all

other methods previously described, and BSTEM analysis was run for borehole No. 3 with before flood flow conditions.

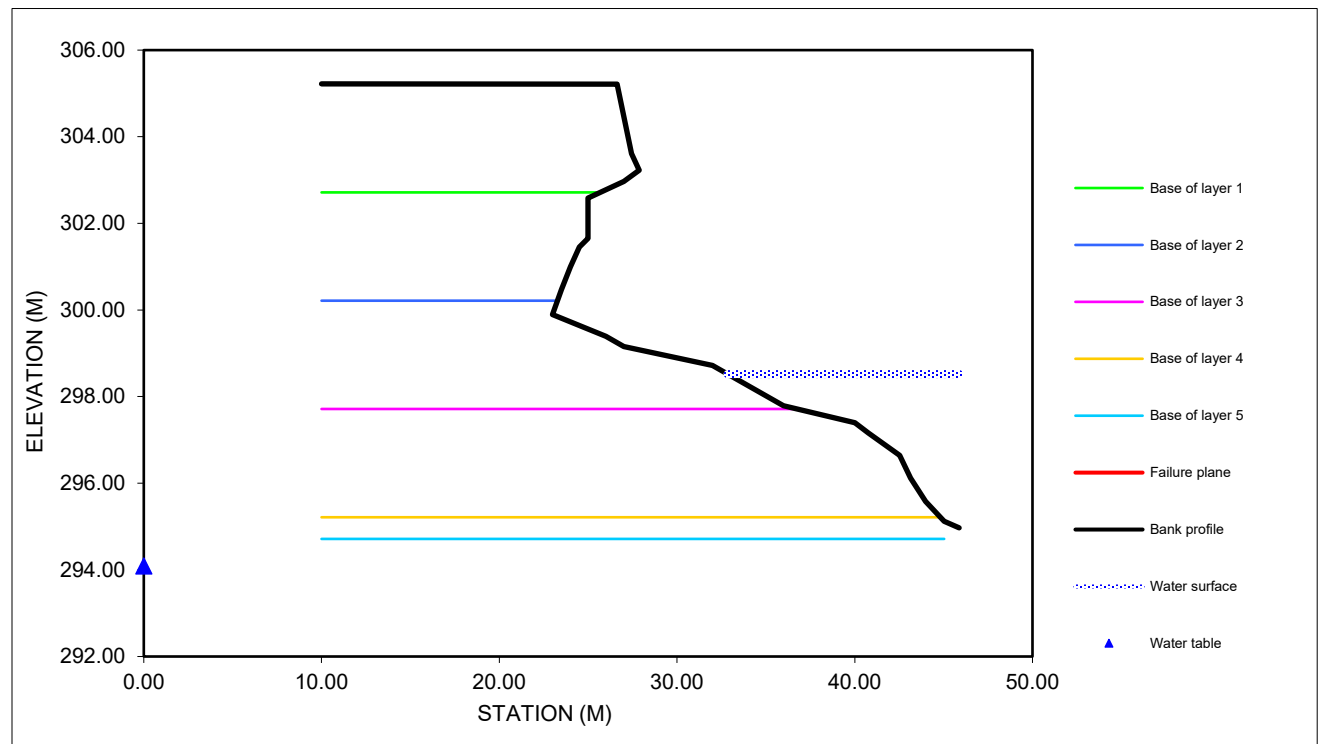


Figure 4. 1 Failing bank profile for borehole No.3

The CU total stress test results give a factor of safety of one, as shown in Table 4. 14, which is defined as “conditionally stable” in BSTEM but would be considered unstable in geotechnical engineering. Table 4.14 shows the total stress analysis and USCS soil properties lead to a conservative factor of safety compared to that from the effective stress analysis resulting in a matching the trend observed before with the stable bank profile. The BSTEM model is a well-validated and commonly used model to evaluate riverbank slope stability, primarily for erosion studies. After exploring the slope model, it was determined that the model can produce an accurate factor of safety with adequate soil strength properties. However, there is

a need for BSTEM modelers to have a comprehensive understanding of the model inputs to simulate the most critical failure case.

Table 4. 14 Factor of safety for original and failing bank profile

Methods	Factor of Safety	
	Original Profile	Selected Failing Profile
CU Effective stress analysis	6.25	1.35
CU Total stress analysis	4.21	1.00
UU Test Results	3.71	0.98
USCS Clay Data Low	2.33	0.87
USCS Clay Data Mean	2.67	0.91
USCS Clay Data High	3.02	0.95

Chapter 5 Conclusions

Riverbank stability is a critical case when evaluating the impact of flooding, high flow rates, or remediation methods in stream management. Although programs like GEOS, SLOPE/W, and ABAQUS are the state-of-the practice in geotechnical engineering, they are not generally used to study riverbank stability, particularly where toe erosion is expected to be significant. This study used BSTEM, which is often used by agriculture engineers and riverbank practitioners. The objectives of this research were to make recommendations for how to obtain soil properties in a BSTEM slope stability analysis and to understand how different methods can affect the slope stability in the BSTEM model. A detailed site investigation was conducted along the Cottonwood River in Kansas. Undisturbed soil samples collected in thin walled Shelby tubes from two boreholes were used in the laboratory tests. In addition to standard methods for classifying all soil samples, 12 Consolidated Undrained (CU) tests and four Unconsolidated Undrained (UU) tests were performed. Numerical correlations using plasticity index were performed for all soil samples. The results from this research show that the effective stress analysis required by BSTEM provides the most accurate factor of safety based. The total stress analysis results in this research show a more conservative factor of safety compared to the effective stress analysis. In this research, the data also show that soil properties obtained from simple numerical correlations are not suitable for this model.

The recommendations from this research are that if one is using the current version of BSTEM (v5.5) and if the time and budget allows, CU tests with porewater pressure measurements should be performed to obtain the most accurate factor of safety based on BSTEM requirements. If the time and/or budget are limited, UU tests are an appropriate alternative. If only needs to gain a general understand of the slope stability, average values from USCS tables

or default BSTEM soil properties can be used. This study also recommends including a total stress analysis option for future model development to represent a more conservative case that is closer to the real-world scenarios of rapid-draw down. Finally, this study suggests that a better understanding of soil strength parameters used in BSTEM would assist the researchers in interpretation of their findings.

5.1 Future work

A continuation of this study should involve increased number of UU tests. This would allow researchers more total stress analysis data from a simple method to verify the reliability of the conclusion that total stress analysis gives conservative results. Furthermore, this study was limited to one site where soil samples are lean clay. Sandy clay, silt, and sand should also be included in a future study. Finally, a risk assessment that could determine the risk of failure should be conducted in a future work to provide information about the accuracy range for the tests the researchers should run.

References

- Abramson, Lee W., et al. *Slope stability and stabilization methods*. John Wiley & Sons, 2001.
- Anderson, M. G. and K. S. Richards. *Slope stability : geotechnical engineering and geomorphology*. Chichester, New York : John Wiley & Sons, 1987.
<<https://onlinelibrary.wiley.com/doi/abs/10.1002/esp.3290140308>>.
- ASTM. *D2850-15: Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils*. West Conshohocken, PA: ASTM International, 2016.
- ASTM. *D422-63: Standard Test Method for Particle-Size Analysis of Soils*. West Conshohocken, PA: ASTM International, 2007.
- ASTM. *D4318-17: Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils*. West Conshohocken, PA: ASTM International, 2017.
- ASTM. *D4767-11: Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils*. West Conshohocken, PA: ASTM International, 2020.
- ASTM. *D4959-16: Standard Test Method for Determination of Water Content of Soil By Direct Heating*. West Conshohocken, PA: ASTM International, 2016.
- Bishop, Alan W. "The use of the slip circle in the stability analysis of slopes." *Geotechnique* 5 (1955): 7–17.
- Bishop, Alan Wilfred and David John Henkel. "The measurement of soil properties in the triaxial test." (1962).
- Brooker, Elmer W. and Herbert O. Ireland. "Earth pressures at rest related to stress history." *Canadian geotechnical journal* 2 (1965): 1–15.

- Cheng, Yung Ming. "Location of critical failure surface and some further studies on slope stability analysis." *Computers and geotechnics* 30 (2003): 255–267.
- Coduto, Donald P. *Geotechnical engineering: principles and practices*. 1999.
- ELE. "DS 7.3." *DS 7.3*. ELE, 2018.
- Fellenius, Wolmar. "Calculation of stability of earth dam." 1936.
- Fine. "GEO 5." *GEO 5*. Fine, 2022.
- Fredlund, D. G., N. R. Morgenstern and R. A. Widger. "The shear strength of unsaturated soils." *Canadian geotechnical journal* 15 (1978): 313–321.
- Freeman, W. S. and Hugh B. Sutherland. "Slope stability analysis in anisotropic Winnipeg clays." *Canadian Geotechnical Journal* 11 (1974): 59–71.
- Geoslope. "SLOPE/W." *SLOPE/W*. Geoslope, 2021.
- Ghavami, Sadegh. "Can anyone provide reference about estimation of c' and ϕ' for both clay and sand please?" *Can anyone provide reference about estimation of c' and ϕ' for both clay and sand please?* July 2017.
- Heinley, Kathryn Nicole. "Stability of streambanks subjected to highly variable streamflows: the Osage River downstream of Bagnell Dam." (2010).
- Hossain, Md Bellal, Toshinori Sakai and Md Zakaria Hossain. "River embankment and bank failure: a study on geotechnical characteristics and stability analysis." *American Journal of Environmental Sciences* 7 (2011): 102.

- Janbu, N. "Application of composite slip surface for stability analysis." *Proceedings of European Conference on Stability of Earth Slopes, Sweden, 1954*. 1954. 43–49.
- Janbu, N. "Slope stability computations–Embankment–Dam Engineering, Casagrande Volume." *John Wiley and Sons, New York* (1973): 47–86.
- Khanal, A., et al. "Comparison of linear and nonlinear models for cohesive sediment detachment: Rill erosion, hole erosion test, and streambank erosion studies." *Journal of Hydraulic Engineering* 142 (2016): 04016026.
- Langendoen, Eddy J. and Andrew Simon. "Modeling the evolution of incised streams. II: Streambank erosion." *Journal of hydraulic engineering* 134 (2008): 905–915.
- Little, A. L. and V. E. Price. "The use of an electronic computer for slope stability analysis." *Geotechnique* 8 (1958): 113–116.
- Massarsch, K. R. "Fracturing of soil caused by pile driving in clay." *Proc. of 9th Intel. Conf. of SMFE*. 1977. 197–200.
- Matthews, Carol, Zeena Farook and Peter Helm. "Slope stability analysis–limit equilibrium or the finite element method." *Ground Engineering* 48 (2014): 22–28.
- Mishtak, J. "Soil mechanics aspects of the Red River floodway." *Canadian Geotechnical Journal* 1 (1964): 133–146.
- Morgenstern, N.R. u and V. Eo Price. "The analysis of the stability of general slip surfaces." *Geotechnique* 15 (1965): 79–93.
- Ouyang, Zhongkun and Paul W. Mayne. "Effective friction angle of clays and silts from piezocone penetration tests." *Canadian Geotechnical Journal* 55 (2018): 1230–1247.

- Simon, A., et al. "Iterative bank-stability and toe-erosion modeling for predicting streambank loading rates and potential load reductions." *USDA-ARS, National Sedimentation Laboratory* (2010).
- Simon, Andrew and Andrea Curini. "Pore pressure and bank stability: The influence of matric suction." *Hydraulic engineering* 98 (1998).
- Simon, Andrew, et al. "Bank and near-bank processes in an incised channel." *Geomorphology* 35 (2000): 193–217.
- Simulia. "ABAQUS." *ABAQUS*. Simulia, 2021.
- Sorensen, K. K. and N. Okkels. "Correlation between drained shear strength and plasticity index of undisturbed overconsolidated clays." *Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris*. 2013. 423–428.
- Terzaghi, Karl, Ralph B. Peck and Gholamreza Mesri. *Soil mechanics in engineering practice*. John Wiley & Sons, 1996.
- Thorne, Colin R. and N. Keith Tovey. "Stability of composite river banks." *Earth Surface Processes and Landforms* 6 (1981): 469–484.
- Thorne, Colin R. "Effects of vegetation on riverbank erosion and stability." *Vegetation and erosion* (1990).
- Whitman, Robert V. and William A. Bailey. "Use of computers for slope stability analysis." *Journal of the Soil Mechanics and Foundations Division* 93 (1967): 475–498.

Appendix A.

Boring No.: B-3 **Project No.:** 02195032 **Project Name:**

Latitude: 38.39454 **Longitude:** 096.34743 **Surface Elevation:**

Operator: D. Bowles **Logger:** B. Bowles **Project Mgr.:** Kevin, F.

Rig No.: 884 **Rig Type:** 850X **Remarks:** C102 - Downstream Borehole

Boring Advancement

Start (date/time): 4/26/19

Finish (date/time):

Depth: 0-36 Method: HS

No.	Sample		Sample Collection Time	Penetration Record			Recovery (Inches)	Depth From To	Sample Description and Lithology
	From	To		Blows	6"	6"			
1	0	3.5							Br s. clay fat clay
2	3.5	5.5	5T				14"	20	Br Fat clay
3	5.5	10	HS				16"	30	SHA
4	10	12	5T				18"	25	Br Fat clay
5	12	16.5	HS				18"	25	SHA
6	16.5	18.5	5T				21	25	SHA moist
7	18.5	23	HS				23	25	Gr Fine sand
8	23	25	5T						
9	25	29.5	HS						
10	29.5	31.5	5T						
11	31.5	36	HS						
12	36	38	5T						

Surface Cover & Thickness

Vegetation:

Topsail: 6" TOPSOIL

Asphalt:

Concrete:

Aggregate:

Water Level Observations

First Encountered: 36.5

At Completion: 36

feet, on (date)

feet, on (date)

feet, on (date)

Water Loss From: To:

Water Loss (%):

Cave In: ☐ Depth:

Artesian: ☐ Height:

Boring Abandonment

Cuttings: ☐

Grout: (write mixture below) ☐

Well Constructed: (see back) ☐

Bentonite Chips: ☐

Sand: ☐

Sheet: 1 of 1

Additional Remarks: Tubes → Double check cutting < 3k

Figure A. 1 Borehole No.3 data for site C102

Boring No.: B-1		Project No.: 02195032		Project Name:	
Latitude: 38.40021		Longitude: 096.34609		Surface Elevation:	
Operator: D. Basles		Logger: B. Basles		Project Mgr.: Kevin F	
Rig No.: 884		Rig Type: 850X		Remarks: C102 - Upstream Borehole	

Sample		Sample Collection Time	Penetration Record			Recovery (Inches)	Depth From To	Sample Description and Lithology
No.	From To		Blows	6"	6"			
14	0 3	HS						6" BP soil BRS:17Y clay
15	3 8	1						BRS:17Y clay
16	8 13	1						BRS:17Y clay
17	13 15	ST						BRS:17Y clay soft moist
18	15 23	HS						6" Fat clay
19	23 25	ST						Pushed Tube NO Recovery
20	25 28	HS						BRS:17Y clay w/ sand
21	28 30	ST						
22	30 34.5	HS						
23	34.5 36	ST						

Boring Advancement	
Start (date/time):	4/26/19
Finish (date/time):	
Depth	0-34.5 HS
Method	

Surface Cover & Thickness	
Vegetation:	
Topsoil:	6" BP
Asphalt:	
Concrete:	
Aggregate:	

Water Level Observations	
First Encountered:	29
At Completion:	28
feet, on (date)	
feet, on (date)	
feet, on (date)	
Water Loss From:	To:
Water Loss (%):	
Cave In: ☐	Depth: ☐
Artesian: ☐	Height: ☐

Boring Abandonment	
Cuttings:	☐
Grout: (write mixture below)	☐
Well Constructed: (see back)	☐
Bentonite Chips:	☐
Sand:	☐

Sheet: _____ of _____

Additional Remarks:

Terracon

Created by M. Maher (Exploration Services) on 3/5/2018.

Figure A. 3 Borehole No. 2 data for site C102

Appendix B

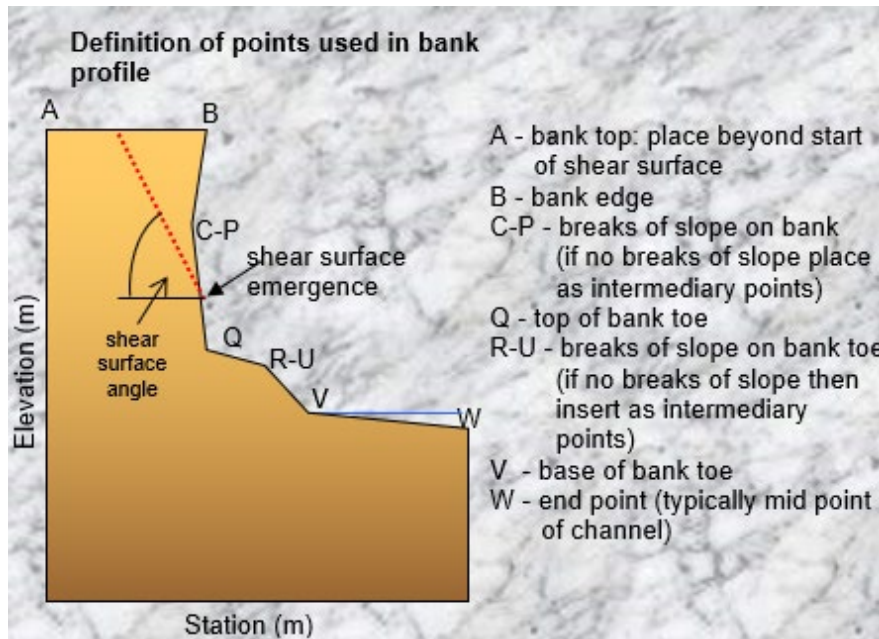


Figure B. 1 BSTEM model required a detailed bank profile input with elevations and stations

Bank layer thickness (m)

Elevation of layer base (m)

Top Layer

Layer 1 #VALUE!

Layer 2 #VALUE!

Layer 3 #VALUE!

Layer 4 #VALUE!

Layer 5 #VALUE!

Bottom Layer

Parallel layers, starting from point B

Figure B. 2 BSTEM model required an input geometry, bank layer thickness (up to 5 layers)

Channel and flow parameters

Input reach length (m)

Input reach slope (m/m)

Input elevation of flow (m)

Input duration of flow (hrs)

Figure B. 3 BSTEM model required an input geometry, channel and flow parameters

Select material types (or select "own data" and add values below)

Bank Material

Layer 1
Layer 2
Layer 3
Layer 4
Layer 5

Erodible soft clay ▼

Own data ▼

Own data ▼

Own data ▼

Own data ▼

Bank and bank-toe material data tables.

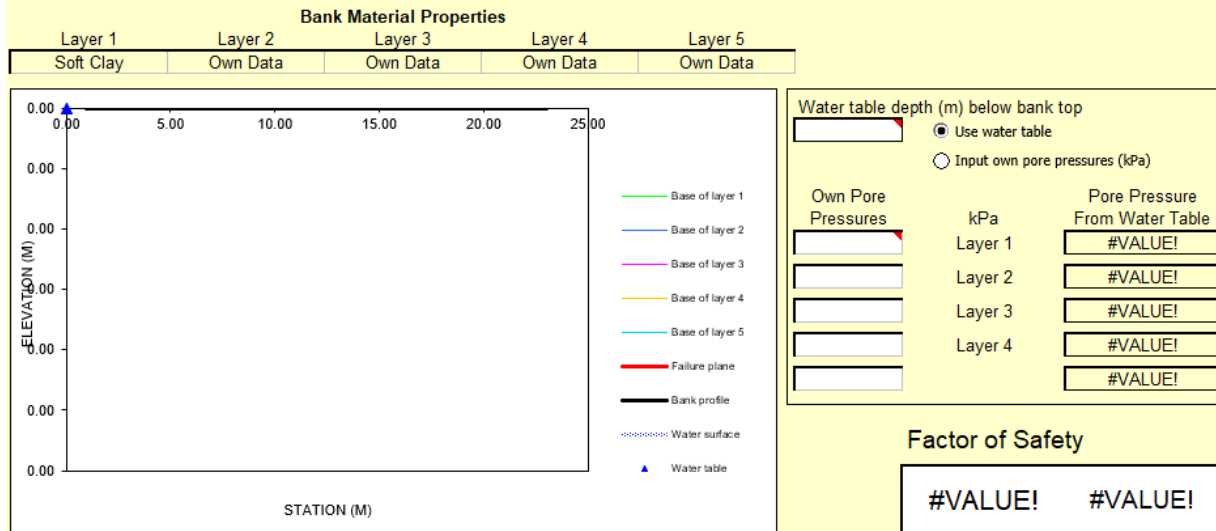
These are the default parameters used in the model. Changing the values or descriptions will change the values used when selecting soil types from the list boxes above. Add your own data using the white boxes.

Material Descriptors			Bank Model Input Data			
Bank material type	Description	Mean grain size, D_{50} (m)	Friction angle ϕ' (degrees)	Cohesion c' (kPa)	Saturated unit weight (kN/m ³)	ϕ^b (degrees)
1	Boulders	0.512	42.0	0.0	20.0	15.0
2	Cobbles	0.128	42.0	0.0	20.0	15.0
3	Gravel	0.0113	36.0	0.0	20.0	15.0
4a and 4b	Angular sand	0.00035	32.3	0.4	18.5	15.0
5a and 5b	Rounded sand	0.00035	28.3	0.4	18.5	15.0
6a, 6b and 6c	Silt	-	26.6	4.3	18.0	15.0
7a, 7b and 7c	Soft clay	-	26.4	8.2	17.7	15.0
8a, 8b and 8c	Stiff clay	-	21.1	12.6	17.7	15.0
9	Own data layer 1					
	Own data layer 2					
	Own data layer 3					
	Own data layer 4					
	Own data layer 5					
	Own data Bank Toe					

Figure B. 4 Bank material could select from default list or insert own data.

Bank model output

Verify the bank material and bank and bank-toe protection information entered in the "Bank Material" and "Bank Vegetation and Protection" worksheets. Once you are satisfied that you have completed all necessary inputs, hit the "Run Bank-Stability Model" button.



Run Bank-Stability Model

Figure B. 5 Ground water table depth is needed for pore pressure calculation and bank failure plane will displayed with detailed bank profile

Table B. 1 USGS Soil data

Unified Classification	Soil Type	Standard Proctor Compaction		Compacted Cohesion, c_0 (kPa)	Saturated Cohesion, c_{sat} (kPa)	Friction Angle, ϕ (deg)
		Maximum Dry Density (kN/m^3)	Optimum Moisture Content (%)			
GW	well graded clean gravels, gravel-sand mixture	>18.7	<13.3	*	*	>38
GP	Poorly graded clean gravels, gravel sand mixture	>17.3	<12.4	*	*	>37
GM	silt gravels, poorly graded gravel-sand- silt	>17.9	<14.4	*	*	>34
GC	Clayey gravels, poorly graded gravel- sand-clay	>18.1	<14.7	*	*	>31
SW	well graded clean sands, gravelly sands	18.7 ± 0.8	13.3 ± 2.5	39 ± 4	*	38 ± 1

Table B. 1 Continued

Unified Classification	Soil Type	Standard Proctor Compaction		Compacted Cohesion, c_0 (kPa)	Saturated Cohesion, c_{sat} (kPa)	Friction Angle, ϕ (deg)
		Maximum Dry Density (kN/m^3)	Optimum Moisture Content (%)			
SP	poorly graded clean sands, sand-gravel mixture	17.3 ± 0.3	12.4 ± 1.0	23 ± 6	*	37 ± 1
SM	silty sands, poorly graded sand-silt mixture	17.9 ± 0.2	14.5 ± 0.4	51 ± 6	20 ± 7	34 ± 1
SM-SC	Sand-silt-clay with slightly plastic fines	18.7 ± 0.2	12.8 ± 0.5	50 ± 21	14 ± 6	33 ± 4
SC	Clayey sands, poorly graded sand-clay mixture	18.1 ± 0.2	14.7 ± 0.4	75 ± 15	11 ± 6	31 ± 4
ML	Inorganic silts and clayey silts	16.2 ± 0.2	19.2 ± 0.7	67 ± 10	$9 \pm *$	32 ± 2
ML-CL	Mixtures of inorganic silts and clays	17.1 ± 0.3	16.8 ± 0.7	63 ± 17	$22 \pm *$	32 ± 3

Table B.1 Continued

Unified Classification	Soil Type	Standard Proctor Compaction		Compacted Cohesion, c_0 (kPa)	Saturated Cohesion, c_{sat} (kPa)	Friction Angle, ϕ (deg)
		Maximum Dry Density (kN/m^3)	Optimum Moisture Content (%)			
CL	Inorganic clays of low to medium plasticity	17.0 ± 0.2	17.3 ± 0.3	87 ± 10	13 ± 2	28 ± 2
OL	Organic silts and silty clays of low plasticity	*	*	*	*	*
MH	Inorganic clayey silts, elastic silts	12.9 ± 0.6	36.3 ± 3.2	72 ± 30	20 ± 9	25 ± 3
CH	Inorganic clays of high plasticity	14.8 ± 0.3	25.5 ± 1.2	103 ± 34	11 ± 6	19 ± 5
OH	Organic clays and silty clays	*	*	*	*	*

* denotes insufficient data

Appendix C

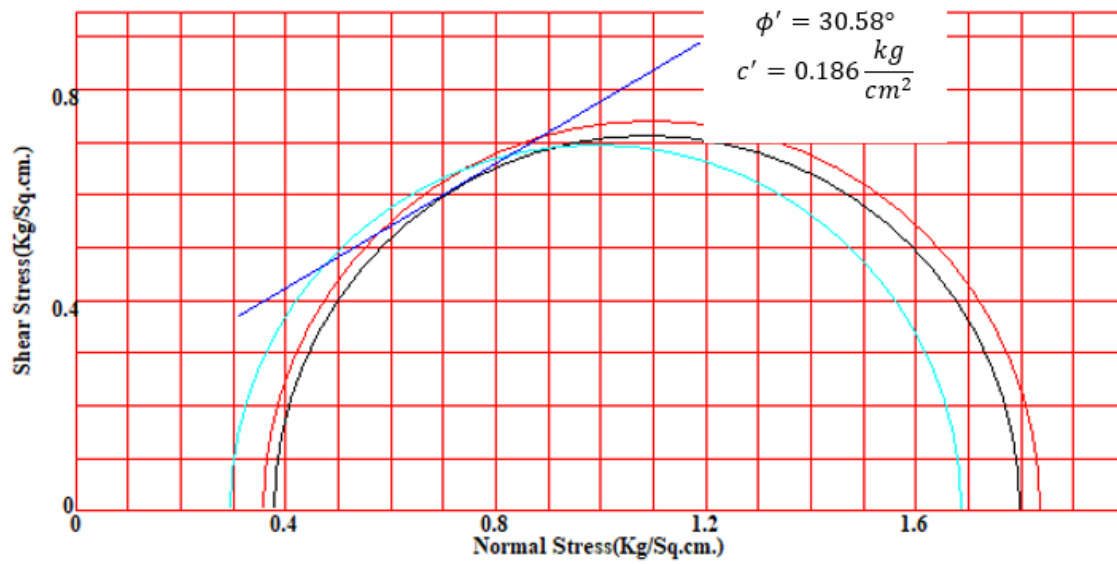


Figure C. 1 Mohr's circles to obtain effective friction angle and cohesion for soil sample

No. 1, 2, 3

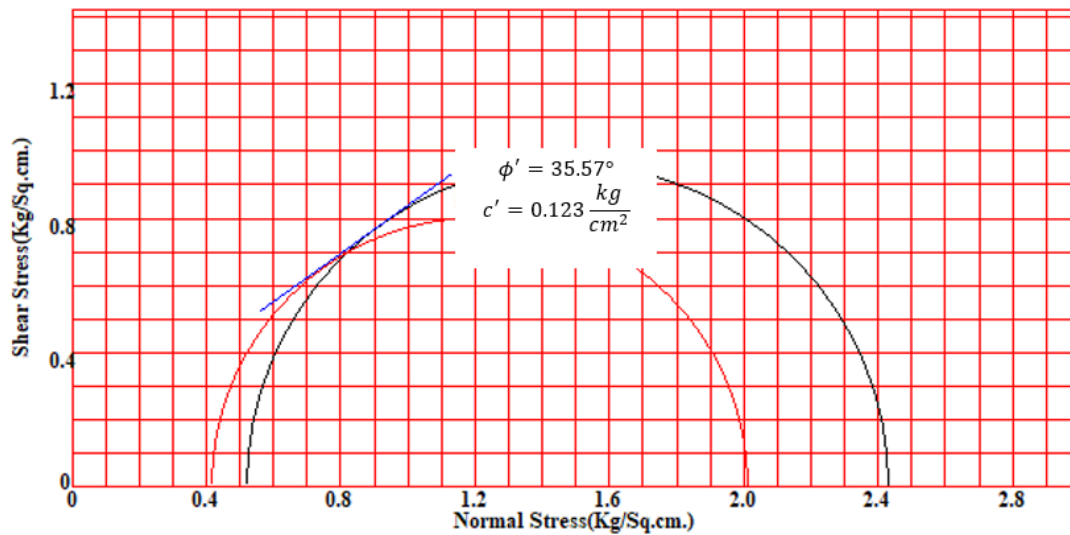


Figure C. 2 Mohr's circles to obtain effective friction angle and cohesion for soil sample No.

4, 7

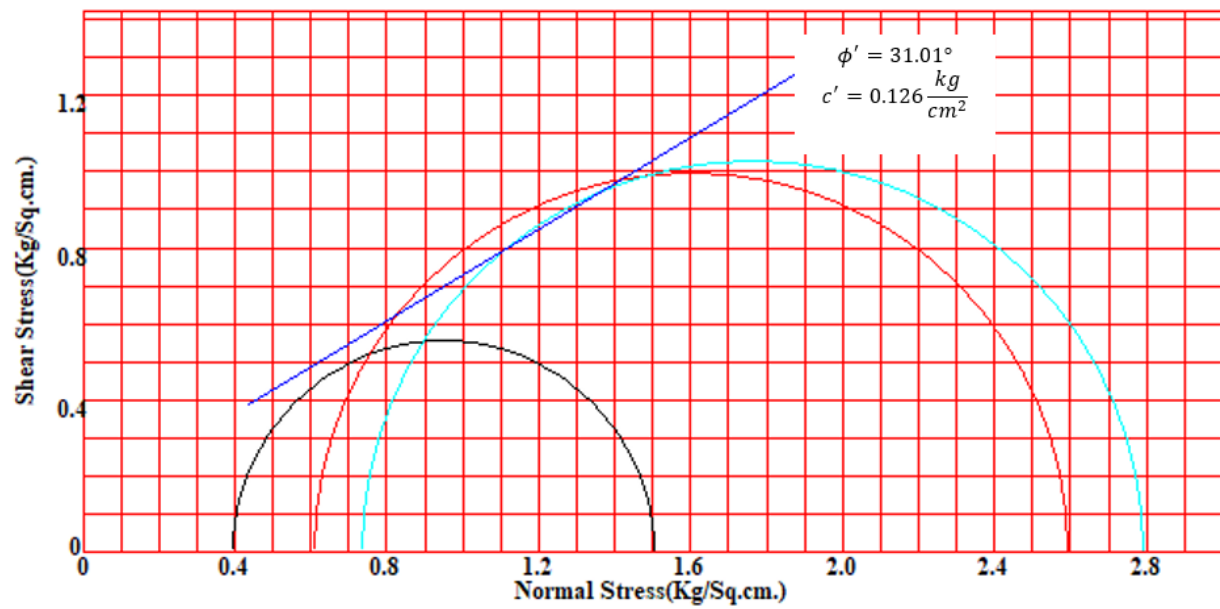


Figure C. 3 Mohr's circles to obtain effective friction angle and cohesion for soil sample No.
10, 11

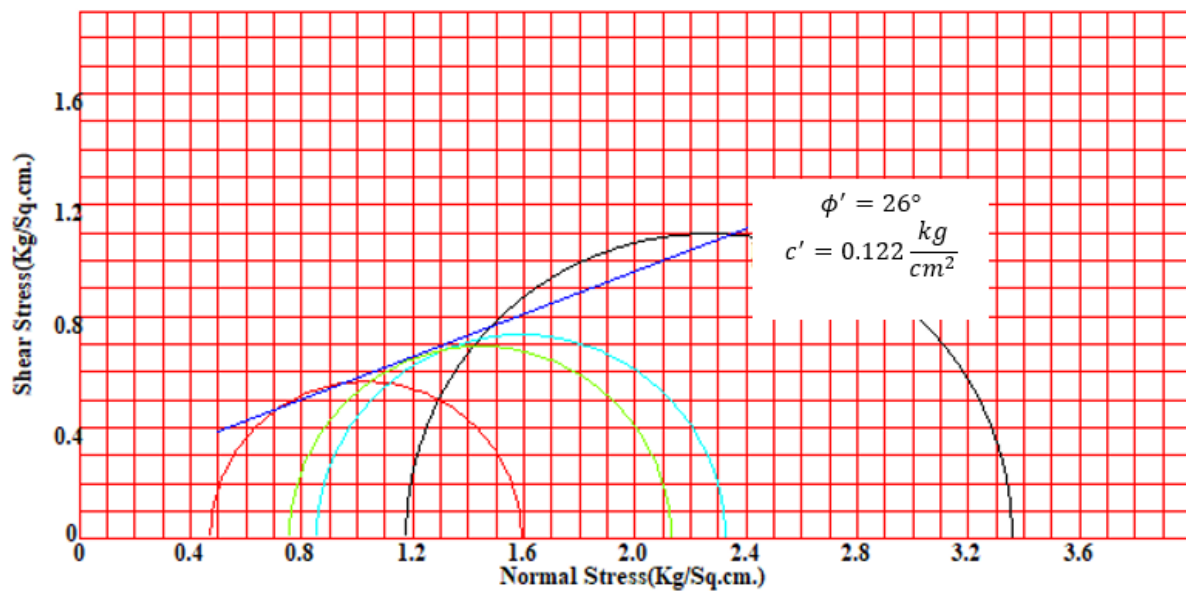


Figure C. 4 Mohr's circles to obtain effective friction angle and cohesion for soil sample No.
12, 14, 15

Table C. 1 BH-1 Results

	Before Flood	During Flood	After Flood
CU Effective stress analysis	2.45	19.28	2.45
CU Total stress analysis	1.89	10.41	1.89
Φ' w/ Terzaghi, one circle	0.37	13.73	0.37
Φ' w/ Sorensen NC low, one circle	2.34	16.47	2.34
Φ' w/ Sorensen NC mean, one circle	0.36	13.21	0.36
Φ' w/ Sorensen OC low, one circle	3.02	15.48	3.02
Φ' w/ Sorensen OC mean, one circle	1.89	16.84	1.89
Φ' w/ Terzaghi for each sample, one circle	1.61	20.51	1.61
In-situ stresses $K_0(P_i)$ by layers, multi circles	0.34	7.14	0.34
In-situ stresses $K_0(\Phi')$ by layers, multi circles	0.27	7.73	0.27
BSTEM Default Soft Clay Data	10.53	30.95	10.53
BSTEM Default Stiff Clay Data	3.87	21.11	3.82
USCS Clay Data Low	2.33	19.59	2.33
USCS Clay Data Mean	2.67	25.05	2.67
USCS Clay Data High	3.02	28.84	3.02

Table C. 2 BH-3 Results

	Before Flood	During Flood	After Flood
CU Effective stress analysis	6.25	Stable	6.25
CU Total stress analysis	4.21	14.19	4.21
UU Test Results	3.71	16.34	3.71
Φ' w/ Terzaghi, one circle	8.85	Stable	8.85
Φ' w/ Sorensen NC low, one circle	8.55	120.03	8.55
Φ' w/ Sorensen NC mean, one circle	8.58	Stable	8.58
Φ' w/ Sorensen OC low, one circle	7.97	222.89	7.97
Φ' w/ Sorensen OC mean, one circle	9.74	93.55	9.74
Φ' w/ Terzaghi for each sample, one circle	8.24	Stable	8.24
In-situ stresses $K_0(\text{PI})$ by layers, multi circles	0.24	6.16	0.24
In-situ stresses $K_0(\Phi')$ by layers, multi circles	0.45	8.72	0.45
BSTEM Default Soft Clay Data	12.85	Stable	12.85
BSTEM Default Stiff Clay Data	8.22	70.4	8.07
USCS Clay Data Low	4.03	Stable	4.03
USCS Clay Data Mean	4.69	Stable	4.69
USCS Clay Data High	5.35	Stable	5.35