

Scour Potential of Cohesive Soils

by

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Abstract

Determination of erosion parameters in order to predict scour depth is imperative to designing safe, economic, and efficient bridge foundations. Scour behavior of granular soils is generally understood, and design criteria have been established by the Federal Highway Administration. The same is not true for cohesive soils, and because of their complexity, a universal scour prediction method has not been established by the industry. The Erosion Function Apparatus (EFA) was created to determine the rate of scour of cohesive soils under known shear stresses, which can then be used to predict scour depths under similar conditions.

During this study, nine cohesive soil formations were sampled with the assistance of the Alabama Department of Transportation. Six of these formations were scour tested in an updated EFA featuring an ultrasonic sensor for quantitative erosion measurements. EFA tests were performed to determine erosion functions and whether any formations demonstrated scour resistance. Geotechnical index tests were also performed on these formations to correlate scour to geotechnical properties.

Results of testing verified the performance of the ultrasonic sensor and updated EFA. Three of the tested formations were scour resistant, while three formations showed evidence of scour. Velocity versus scour rate curves were generated for the scourable formations with scour rates upwards of 15 mm per hour. The scour behavior observed was unique among formations limiting the ability to establish correlations between tests.

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List of Abbreviations

FHWA	Federal Highway Administration
HEC	Hydraulic Engineering Circular
SRICOS	Scour Rate in Cohesive Soils
ALDOT	Alabama Department of Transportation
E-SCRICOS	Extended Scour Rate in Cohesive Soils
S-SRICOS	Simple Scour Rate in Cohesive Soils
USGS	United States Geological Survey
RETA	Rotating Erosion Test Apparatus
SERF	Sediment Erosion Rate Flume
LVDT	Linear Variable Differential Transformer
CME	Central Mining Equipment
SPT	Standard Penetration Test
USCS	Unified Soil Classification System
LL	Liquid Limit
PL	Plastic Limit
PI	Plasticity Index

Chapter 1 Introduction

1.1 Background

In 2009, according to the Federal Highway Administration (FHWA), there were approximately 603,000 bridges in the National Bridge Inventory. Of these 603,000 bridges, roughly 83 percent span water (Lagasse et al. 2007). With such a large number of bridges crossing water, scour can be a major concern with accelerated flow conditions such as flooding. Between 1961 and 1976, over 50 percent of the 86 major bridge failures were due to scour (Murillo 1987). More recently, from 1989 to 2000 just over fifteen percent of all bridge failures were due to scour (Wardhana and Hadipriono 2003). From these numbers it is evident that scour is a serious issue in bridge design and maintenance. When scour does occur, remediation measures are extremely costly, due to potential instabilities in the bridge and river bed, as shown in Figure 1-1. Estimates of scour are an important step in the bridge design process, as the estimated depth of scour is a driving force in the foundation system selection and penetration depth.



Figure 1-1. Example of Bridge Scour

Current scour predictions of highway bridges are made using techniques reported in Hydraulic Engineering Circular 18 (Richardson and Davis 2001), abbreviated as HEC 18, and Hydraulic Engineering Circular 20, or HEC 20 (Lagasse et al. 2001). These reports, published by the FHWA, estimate scour depth based on four major variables: channel configuration, stream velocity, soil grain size, and underlying bed material. It is important to note that the methods defined in HEC 18 and HEC 20 were based on predicting scour in cohesionless bed material. Alternatively, it is believed that the variables leading to scour in HEC 18, predominantly grain size, do not translate into accurately predicting scour in cohesive soils. Much work has been completed pertaining to predicting the rate and magnitude of scour of cohesive soils, most notably by Briaud et al. (1999, 2001a, 2001b, 2004). The Erosion Function Apparatus, EFA, was created by Briaud's research group, with the purpose of determining the rate of scour of cohesive soils.

The EFA uses a pump and a flume to create a constant flow, and corresponding shear stress, which is exposed to a one millimeter protrusion of soil. Determining the erosion rates of this one millimeter protrusion at different velocities, or bed stresses, creates an erosion function. This erosion function is then used in accordance with Briaud's Scour Rate in Cohesive Soils (SRICOS) method to predict the maximum depth of scour over flooding events (Briaud 1999). The erosion rates created from the EFA are determined using a viewing window in the flume of the EFA. An observer determines when the volume of the one millimeter protrusion has eroded and records a corresponding time stamp.

Accurate scour predictions are a major contributing factor to the economic foundation design of bridges crossing bodies of water. The depth of scour is a portion of the total depth of foundation needed to provide capacity to carry the bridge loads. If the depth of scour is

overpredicted, the foundation length and construction costs of the bridge are unnecessarily increased. This principle is the driving idea for better predictions of scour depth in cohesive soils. This concept is directly tied to the SRICOS method as it is a relatively accepted method for predicting scour and is approved by the Federal Highway Administration.

1.2 Objectives

The objectives of the study were

- Adapt the EFA to measure scour rates automatically, including the design of an ultrasonic sensor, data acquisition system, and data reduction procedure,
- Conduct EFA tests on cohesive soil samples that were collected from locations below the fall line in the coastal plain of Alabama, and
- Determine if measured scour parameters correlate with common geotechnical parameters such as shear strength, Atterberg Limits, grain size, or Standard Penetration Test N values.

1.3 Scope of Study

The scope of work included the following tasks:

- Update the Auburn University Erosion Function Apparatus with an ultrasonic sensor that can volumetrically measure the mass of eroded material at any point during testing. It is believed that this would add validity to current erosion testing practices,
- Create a testing regimen that incorporates the new updated EFA,
- Obtain samples from cohesive soil formations in the coastal plain of Alabama with the assistance of the Alabama Department of Transportation (ALDOT),
- Perform EFA tests to determine erosion parameters, and
- Perform geotechnical index tests to determine geotechnical parameters.

Chapter 2 Literature Review

2.1 Scour Background Information

Scour is defined as the result of the erosive action of flowing water, which can excavate and carry away material from the bed and banks of streams and from around the piers and abutments of bridges (Richardson and Davis 2001). More specifically scour at bridges can be related to the following factors (Lagasse et al. 2001):

- Channel slope and alignment
- Channel shifting
- Bed sediment size distribution
- Antecedent floods and surging phenomena
- Accumulation of debris, logs, or ice
- Flow contraction, flow alignment, and flow depth
- Pier and abutment geometry and location
- Type of foundation
- Natural or man-induced modification of the stream
- Failure of a nearby structure

Scour is divided into three different classifications: aggradation and degradation, general scour, and local scour. Aggradation and degradation are based on long-term streambed elevation changes (Richardson and Davis 2001). Aggradation is defined by the deposition of upstream material, resulting in the raising of a streambed. Degradation is defined by the lowering of a streambed due to a deficit in the deposits from upstream.

General scour is attributed to the lowering of a streambed across a stream at a bridge or large-scale impingement in flow. General scour often refers to contraction scour, which

occurs when a streambed is narrowed, or contracted, by the addition of a bridge that increases flow velocities (Richardson and Davis 2001). Another type of general scour involves scour around a bend in a river, as velocities tend to vary with respect to distance from the bend. Typically contraction scour occurs across most of the streambed. However general scour and contraction scour are not uniform throughout a given cross section. General scour differs from aggradation and degradation in that general scour is cyclical and often reflects flood activity (Richardson and Davis 2001).

Local scour is defined as the removal of material around objects intercepting flow caused by the acceleration of flow around the objects. With acceleration in flow around a bridge pier comes an increase in shear stress on the stream bed resulting in removal of bed material. HEC 18 recommends calculating total scour at a bridge crossing by adding degradation, general scour, and local scour over the design life of the structure.

Scour occurs when the shear stress exerted on the bed material by the flow of water exceeds the critical shear stress (τ_c) for the bed material. As the shear stress increases beyond the critical shear stress of the bed material, a scour hole develops. Scour can develop around an object, as in local scour, or across a channel in general scour. As the scour increases and more particles are removed, the shear stress on the plane of the bed material decreases. Maximum scour depth is reached once enough material has been removed to reduce the shear stress at the bottom of the scour hole to a level below the critical shear stress (Briaud et al. 1999). This critical shear stress is proportional to the critical velocity (V_c) (in m/s for SI units and ft/s for English units) flowing through a channel. HEC-18 defines the critical shear velocity by equation 2-1 below:

$$V_c = K_u \cdot y^{\frac{1}{16}} \cdot D^{\frac{1}{3}} \quad (2-1)$$

where:

- V_c = Critical velocity above which bed material size D and smaller scours
- y = Average depth of flow upstream of bridge
- D = Particle size correlated to V_c
- K_u = Curve fitting factor (6.19 for SI units and 11.17 for English units)

Equation 2-1 states that the critical shear stress is a function of two different properties, depth of flow and grain size. Grain size is also a variable in calculating the depth of contraction scour. In live-bed contraction scour, grain size is a variable used to determine the mode that bed material is being transported (Richardson and Davis, 2001). Similarly, in clear-water contraction scour the average depth of scour is proportional to the largest nontransportable particle in the bed material.

For calculating the scour depth around a pier, HEC-18 suggests using equation 2-2. This equation was created based on flume experiments in sand using different pier configurations and shapes.

$$\frac{y_s}{a} = 2.0 \cdot K_1 \cdot K_2 \cdot K_3 \cdot K_4 \cdot \left[\frac{y_1}{a} \right]^{0.35} \cdot Fr_1^{0.43} \quad (2-2)$$

where:

- y_s = Scour depth
- y_1 = Flow depth directly upstream of pier
- K_1 = Correction factor for pier nose shape
- K_2 = Correction factor for angle of attack of flow
- K_3 = Correction factor for bed condition
- K_4 = Correction factor for armoring by bed material size
- a = Pier width

L = Length of pier

Fr₁ = Froude Number directly upstream = $\frac{Velocity}{\sqrt{g \cdot y_1}}$

g = Acceleration due to gravity

In equation 2-2 grain size is a variable in the constant K₄, a correction factor for armoring the pier by bed material. This correction factor reduces the amount of pier scour if the pier is considered protected by heavier coarse-grained materials such as gravels. Throughout HEC-18, scour prediction equations are heavily influenced by grain size. This concept is based on the assumption that soils modeled in these scour predictions erode particle by particle. Erosion of coarse-grained material such as sand and gravel occurs typically by rolling, sliding, or plucking of the particle. Close observations and slow motion cameras have determined that scour of these particles is typically a combination of these mechanisms (Briaud et al. 1999). Erosion of sands and gravels is mainly resisted by gravity forces from the weight of the particles. Using this logic, the larger the particle, the more shear stress it can resist. For example, it is understood that the shear stress needed to erode a grain of sand is much less than the shear stress needed to erode a boulder. However, HEC-18 uses these principles to predict the depth of scour of fine-grained cohesive soils. HEC-18 does acknowledge that the rate of scour may be much lower for cohesive soils, but states that the maximum scour depth should still be calculated using the equations above. In this regard, cohesive soils are considered to have a lower critical shear stress than sands using equation 2-1. Also fine-grained cohesive soils do not provide any protection against pier scour using the correction factor K₄ in equation 2-2. In essence, the lack of understanding regarding scour of cohesive soils has resulted in a penalty using current design standards.

2.2 Scour Rate in Cohesive Soils Method

Numerous studies have been performed to create a better understanding of scour of cohesive soils. Most of these studies involved channel or flume tests that simulated a flood event, in which scour was closely monitored. HEC-18 does acknowledge one study in particular as a method for predicting pier scour in cohesive soils. This research, performed at Texas A&M University, resulted in the *Scour Rate in Cohesive Soils* (SRICOS) method, and started by determining the scour mechanisms in sands, clays, and rock (Briaud et al. 1999).

Water velocity ranges from 0.1 to 3.0 m/s in most rivers and streams. This velocity results in average bed shear stresses ranging from 1 to 50 N/m². Since the bed shear stresses resulting in scour are much less than the shear strength parameters typically found in clay, it is presumed that scour is a result of a cyclical failure, which occurs after long-term exposure to relatively low shear stresses (Briaud et al. 1999). Observations of scour in sands showed a rapid, immediate failure compared to the cyclical failure observed in cohesive soils. The different scouring methods of sand and clay suggest that the forces resisting scour are not similar. As previously stated the main force that resists scour in sand is gravity. Gravitational force is relatively small, depending on particle size. The resultant scour rates found in sand and resisted by gravitational forces are represented in meters per hour. Research has shown van der Waals forces, which hold clay particles together, can better resist the constant cyclical loading found in streams, resulting in scour rates in the order of millimeters per hour (Briaud et al. 1999). Similarly, when scour occurs in clay, it does not occur particle by particle but in larger groups of particles or chunks.

The SRICOS method was created to predict the depth of pier scour with respect to time for a known flow velocity in a uniform cohesive soil. The SRICOS method involves

testing a site specific soil sample in an EFA, or Erosion Function Apparatus, and recording scour rates for a range of velocities. The velocities tested in the EFA should encompass the expected shear stresses around the pier under flooding conditions. From EFA tests, an initial scour rate is established as the scour rate corresponding to the maximum shear stress expected during the flood event. A maximum scour depth is calculated based on site specific geometry and flow rates. Using the maximum scour depth, initial scour rate, and flood information, a flood specific scour depth is established.

The SRICOS method allows for an informed scour prediction to be made in cohesive soil. However, the SRICOS method makes certain assumptions that are not realistic in all streams. For instance the SRICOS method produces a depth of scour for one flood event, in a uniform soil layer (Briaud et al. 1999). This is not realistic as bridge piers will resist several floods with varying velocities over a design service period, and commonly are embedded into varying soil layers. To improve the accuracy of the SRICOS method, Briaud et al. (2001a) expanded the SRICOS to encompass the full hydrograph throughout the design period. Also the Briaud et al. (2001a) expanded the SRICOS method to predict the maximum depth of scour in different soil layers. Since the combinations of varying soil layers and flood events can become very complex, a computer program was developed to calculate pier scour using the new Extended SRICOS (E-SRICOS) method. A Simple SRICOS (S-SRICOS) method was also created to accommodate scour predictions similar to the E-SRICOS method without the use of the SRICOS computer program (Briaud et al. 2001a)

The procedure for the E-SRICOS method is similar to the original SRICOS method. The maximum scour depth is still calculated using the mean flow velocity, diameter of the

pier, and viscosity of the water. Samples are collected and tested in the same manner as the SRICOS method, and a velocity versus scour rate curve is created. The flow hydrograph is then created for the bridge, typically using a discharge hydrograph from the United States Geological Survey (USGS) near the bridge location (Briaud et al. 2001a). The flow hydrograph is then transformed into a velocity hydrograph using profile of the stream at the bridge crossing. Using the velocity hydrograph, the velocity versus scour rate curve created in the EFA, soil profiles, and general bridge properties, the SRICOS program calculates the depth of pier scour throughout the entire design life of the bridge (Briaud et al. 2001a). Through several case studies Briaud et al. (2001a) found the maximum scour depth calculated, using HEC 18 standards, was not reached throughout the design life hydrograph

As previously stated the S-SRICOS method was created to produce similar results to the E-SRICOS method without the use of the SRICOS computer program. The maximum scour depth is calculated, usually related to the 100-year flood conditions. Site specific samples are collected and tested in the EFA to obtain erosion functions. It is important that samples are obtained and tested from all soil layers within the maximum scour depth (Briaud et al. 2001a). A single equivalent erosion function is then created by averaging the functions from all soils within the maximum scour depth. Using the flow hydrograph and bridge information, the maximum shear stress is calculated. The calculated maximum shear stress and combined erosion function are used to determine the scour rate corresponding to the maximum shear stress. An equivalent time factor is needed to reduce the number of iterations performed by the SRICOS computer program. The equivalent time is the time required for the maximum velocity in the hydrograph to create the same scour depth as the complete hydrograph (Briaud et al. 2001a). The equivalent time factor is a function of the

design life of the bridge, the maximum velocity of the river, and the initial scour rate corresponding to the maximum shear stress. The total pier scour depth is then calculated using the equivalent time, initial scour rate, and maximum scour depth.

2.3 Erosion Function Apparatus

The quality of the SRICOS prediction is based on the results from the EFA test that is performed on site specific samples. The Erosion Function Apparatus is a closed-channel, flume-like machine equipped with a pump and a stepping motor (Briaud et al. 2001b). The bottom of the flume has a circular opening for testing a Shelby tube sample with a diameter of 76.2 mm. A watertight seal is created between the Shelby tube and the flume by an O-ring. The cross section of the rectangular flume is 101.6 mm x 50.8 mm. The total length of the flume measures 1.22 m. The pump is regulated by a valve on the front of the EFA, and generates velocities ranging from 0.1 to 6 meters per second (m/s). The Shelby tube is held flush at the bottom of the flume during testing. A piston attached to the stepping motor protrudes the sample from the Shelby tube into the flume in 1 mm increments (Briaud et al. 2001b). The EFA can be viewed in Figure 2-1 below. The EFA is instrumented with a thermistor and a paddle flow meter to electronically monitor the temperature and flow rate using a computer.



Figure 2-1. Erosion Function Apparatus

The first step in performing an EFA test proposed by Briaud is to place the sample in the EFA, fill the flume with water and wait an hour to mimic stream conditions. Next the flume velocity is set to 0.3 m/s and the sample was advanced 1 mm into the flume. The EFA is equipped with a viewing glass so that a technician can observe the erosion of the sample. After the sample is advanced 1 mm, as shown in Figure 2-2, the technician measures how much time it takes for the 1 mm sample protrusion to erode. After 1 mm has eroded, or after one hour of testing, the sample is advanced to the 1 mm protrusion location and the velocity increased is to 0.6 m/s. The erosion of the sample is again monitored, and the time for the current 1 mm protrusion to erode is recorded. The sample is sequentially advanced and the erosion timed for velocities of 1 m/s, 1.5 m/s, 2.0 m/s, 3/0 m/s, 4.5 m/s, and 6 m/s (Briaud et al. 2001b). A schematic of a prepared EFA test is shown in Figure 2-2 (Mobley 2009).

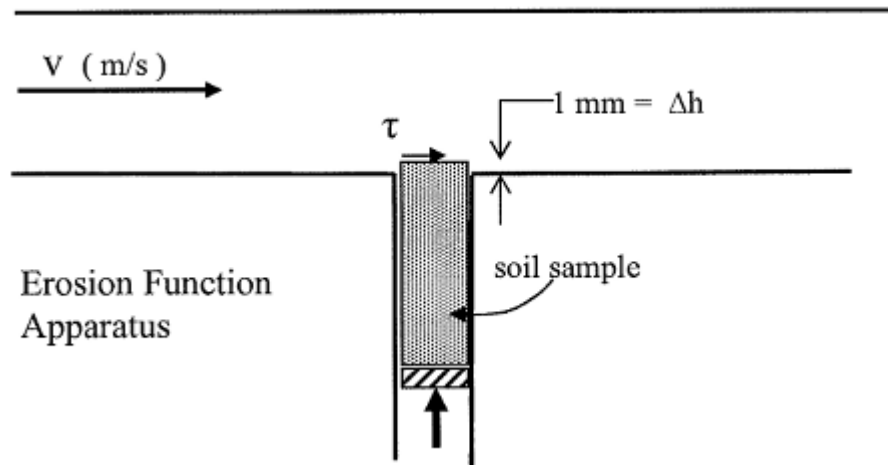


Figure 2-2. Schematic of EFA Test

The results from the EFA are used to create an erosion function, which is used in the scour calculations of the SRICOS method. An example of an erosion function can be viewed in Figure 2-3 (Mobley 2009). The erosion function is a derived from a velocity versus erosion rate relationship that is created from the EFA data. An erosion function shows the relationship between erosion rate (z) and shear stress (τ) for a given soil. Velocity is related to shear stress by using the geometry of the flume, density of water, and friction factor obtained from a Moody Chart. It is imperative the erosion function encompass all shear stresses expected in the stream bed during design flood conditions.

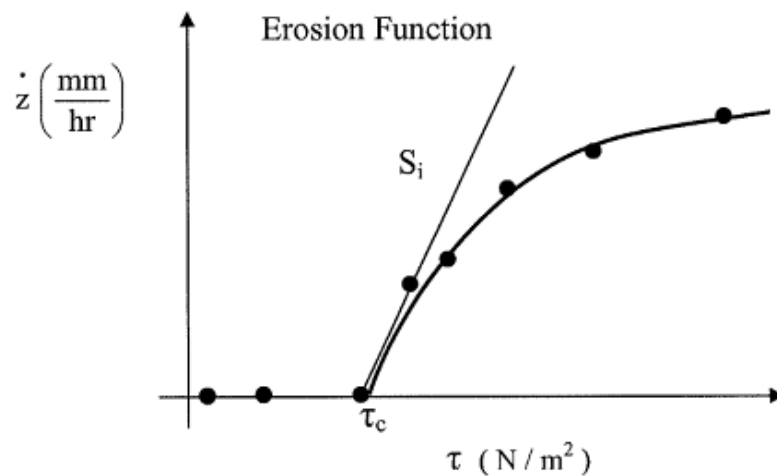


Figure 2-3. Typical Erosion Function

Other than the relationship between shear stress and erosion rate, an erosion function provides two critical values that are used in scour predictions. The critical shear stress (τ_c) is the bed shear stress at which scour first occurs (Mobley 2009). The critical shear stress is found by observation while performing an EFA test. During erosion testing flow is gradually increased until erosion begins. This velocity is converted to a shear stress, recorded, and plotted on the erosion function. The initial erodibility (S_i) measures the rate of scour just after the critical shear stress is reached. The initial erodibility is calculated by drawing a line tangent to the erosion function through the critical shear stress (Crim 2003). Generally, initial erodibility is higher in sands compared to cohesive soils.

Since the results of EFA tests are important to producing quality scour estimates using the SRICOS method, it is integral that scour rates are calculated with accuracy. The determination of scour rates can be a subjective process as erosion is typically not uniform in cohesive soils. Previous research suggested the technician performing an EFA test should estimate when one millimeter of scour has occurred by taking an average across the surface (Crim 2003). However, averaging is subjective and not consistent with different technicians. Several attempts have been made to reduce the subjectivity in erosion testing. The University of Florida created two different apparatuses to objectively measure scour of cohesive soil and soft rock.

2.4 Alternatives to the Erosion Function Apparatus

The Rotating Erosion Test Apparatus (RETA) was designed to determine the volume of erosion of stiff clay, sandstone, and limestone. A schematic of the RETA is shown in Figure 2-4. The RETA performs an erosion test on a 4 in. long cylindrical soil sample with either a 2.4 in. or 4 in. diameter (Sheppard et al. 2005). A ¼ in. diameter hole is drilled

vertically through the sample and a support shaft that attaches to the RETA is inserted through the sample. The opposite end of the shaft is attached to a torque cell and clutch that is fixed to a surface (Sheppard et al. 2005). A larger diameter cylinder is placed over the sample, and water is added to fill the annulus between the sample and the outer cylinder. The outer cylinder is rotated using a motor and pulley system to create a shear stress on the surface of the sample. After a known duration of time the test is stopped and the test sample is measured radially and any eroded material is oven dried to determine the mass of the scoured soil. The shear stress is calculated by knowing the amount of torque applied to the sample along with the initial radius and length of the test specimen. The erosion rate is calculated using the change in radius, duration of the test, mass density of sample, and original sample geometry (Sheppard et al. 2005).

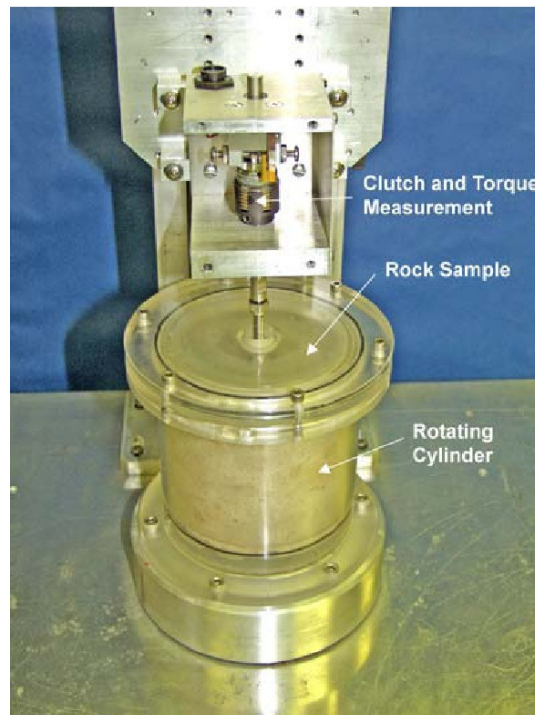


Figure 2-4. Rotating Erosion Test Apparatus (RETA)

The RETA is an acceptable testing apparatus for determining an erosion rate and shear stress relationship. The RETA is not ideal, however, because it does not test the

appropriate failure plane. A soil specimen in a stream bed will be exposed to shear stress and eroded from the top down, where the RETA measures scour on a radial plane. Also, the sample preparation is difficult for the RETA specimen, as it can be difficult to create a consistent curvature around the entire outer diameter of the sample. If the radius of the sample is not consistent, the shear stress will not be equally distributed throughout the sample, causing irregular erosion.

The Sediment Erosion Rate Flume (SERF) is the second apparatus created at the University of Florida for the purpose of measuring relationships between erosion rate and shear stress. The SERF functions similar to the EFA. Like the EFA, the SERF uses a flume with a rectangular cross section, a water pump, a reservoir, and a stepping motor. The SERF does have a larger water capacity than the EFA, as it utilizes two pumps and has an 1100 gal reservoir. The SERF is much more automated than the EFA, using multiple instruments and a Labview program. Images of the SERF are shown in Figure 2-5 and Figure 2-6.



Figure 2-5. Sediment Erosion Rate Flume (SERF)

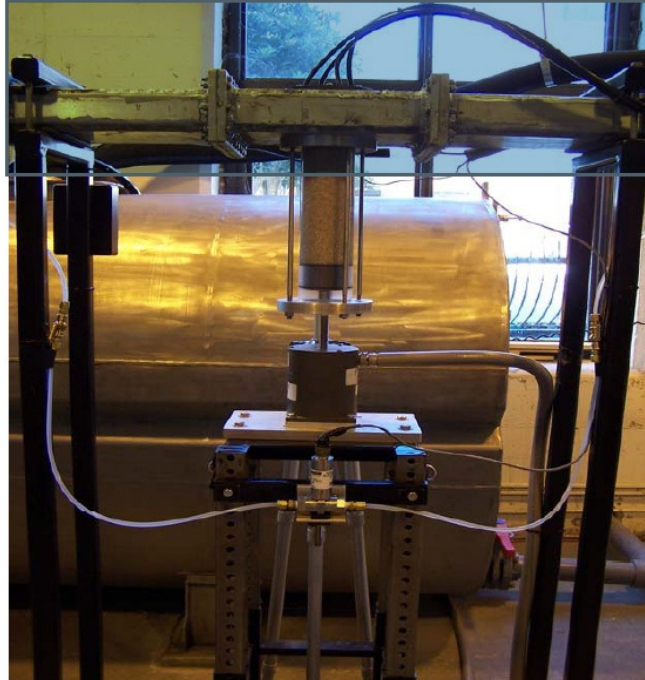


Figure 2-6. SERF Sample Chamber and Stepping Motor

The SERF utilizes a Multiple Transducer Array (MTA) created by SeaTek to scan across a Shelby tube sample (Sheppard et al. 2005). This array consists of twelve ultrasonic transducers that are used to measure the distance from the top of the flume to the surface of the sample. A schematic of the ultrasonic sensor used in the SERF is shown in Figure 2-7 (Sheppard et al 2005). The ultrasonic transducers are spaced so that both a 7.3 cm and 9.5 cm soil/rock sample can be scanned (Sheppard et al. 2005). The data collected by the ultrasonic transducers are used in a Labview program to control the positioning of the sample being tested. Unlike the EFA, the soil sample does not protrude from the bottom of the flume, but is kept level with the base of the flume. Once the sample averages a total scour of 0.5 mm, as detected by the ultrasonic sensor, the sample is automatically advanced by the stepping motor. After the test is completed, the erosion rate is calculated by dividing the length of sample advanced by the stepping motor by the amount of time the test was

performed. A video camera is used to verify the results from the SERF and to determine the method of erosion.

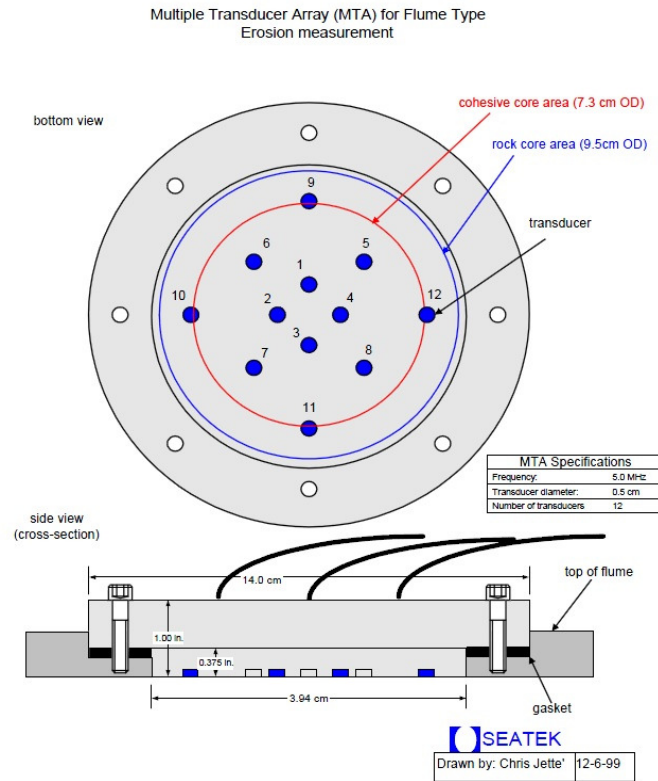


Figure 2-7. Multiple Transducer Array Designed for SERF

Researchers at the Georgia Institute of Technology created and used another alternative to the Erosion Function Apparatus. A rectangular, tilting, recirculating flume was altered and utilized with the same general function of the EFA and SERF machines (Navarro 2004). The flume used was considerably larger than the ones used in the EFA and SERF with a length of 6.1 m, width of 0.38 m, and depth of 0.38 m. Two variable-speed pumps generated flow to the flume. Adhered to the flume bed were gravel particles with a mean diameter of 3.3 mm (Navarro 2004). These gravel particles were used to assure fully developed and fully rough turbulent flow (Navarro 2004). The basin, which fed both pumps, had a holding capacity of approximately two cubic meters.

Shelby tube samples were collected using formations consisting of both cohesive and cohesionless materials. The Shelby tube was placed below a circular opening in the bottom of the flume, and the measurement of erosion was determined using a linear variable differential transformer (LVDT) attached to a piston which advanced the sample into the flume (Navarro 2004). An operator controlled the apparatus that advanced a piston during erosion testing, to ensure the sediment surface was level with the top of the gravel bed in the flume. This procedure is similar to the procedure for operating the SERF. These tests were run continuously at varying velocities throughout the entire length of a Shelby tube sample to mirror typical field stratification (Navarro 2004).

Erosion rates are calculated by converting the vertical displacement measured by the LVDT into eroded mass, using the dry density of the tested sample. The product of the vertical displacement and dry density of the sample is divided by the time interval over which the erosion occurs to determine the erosion rate at a given velocity or shear stress.

2.5 Scour Relationships with Geotechnical Parameters

Generally speaking the critical shear stress of fine-grained soils increases when the soil unit weight increases, plasticity index increases, unconfined compressive strength increases, void ratio decreases, swell decreases, percent fines increases, temperature decreases, and water temperature increases (Briaud et al. 1999). Erosion rates are influenced by the hydraulic shear stress applied, the clay content, the soil and water temperature, the soil and water chemical composition, the soil water content, the plasticity index, the soil unit weight, the soil undrained shear strength, and the mean grain size (Briaud et al. 1999). Briaud et al. (1999) found general qualitative relationships relating critical shear stress and initial erodibility to plasticity index, undrained shear strength, and percent fines (percent

passing #200 sieve). Briaud et al. (1999) also plotted a correlation between critical shear stress and initial erodibility showing that as critical shear strength increases, initial erodibility decreases. Research hints that a correlation between scour parameters and geotechnical parameters is complex and involves a combination of many parameters. Research at the University of Florida developed a correlation between cohesive strength and erosion rates of limestone cores (Sheppard et al. 2006). This function relates erosion rate to applied hydraulic shear stress, unconfined compressive strength, and splitting tensile strength. Further research at the Georgia Institute of Technology suggests a correlation of critical shear stress with the percent fines and median particle diameter (Sturm et al. 2004). This relationship suggests that the smaller the particle size of a given formation, the greater the influence of interparticle forces on the erodibility of the formation. Although these relationships are relatively unproven, it does show that a correlation is possible between cohesive materials and erosion rates.

2.6 Previous Scour Research at Auburn University

Auburn University has owned an Erosion Function Apparatus since 2001. Crim (2003) detailed work to develop a better understanding of the inner workings of the EFA, published a procedure, and created erosion functions as defined in the SRICOS method. With Shelby tube samples provided by the Alabama Department of Transportation (ALDOT), Crim constructed erosion functions of five different Alabama cohesive soils. Crim tested samples from Goose Creek in Wilcox County, culverts on US 84 in Covington county, Linden Bypass in Marengo County, County Road 5 over Cheaha Creek in Talladega County, and Alabama State Road 123 over Choctawatchee River in Dale County. A summary of the results obtained by Crim (2003) are shown in Table 2-1.

Table 2-1: Summary of Crim's Results

Stream Crossing	County	Critical Shear Stresses (N/m ²)	Initial Erodibility (mm/hr/N/m ²)
Goose Creek	Wilcox	0.4 to 4.5	0.2 to 5.6
US 84	Covington	1.1 to 3.1	1.7 to 6.1
Linden Bypass	Marengo	0.9 to 4.7	0.4 to 11.4
County Road 5	Talladega	0.6 to 2.8	0.4 to 410
State Road 123	Dale	1.2 to 2.5	1.0 to 1.2

Overall, Crim witnessed scour rates ranging from 0 mm/hour upwards of 100 mm/hour. Crim plotted the results from the erosion functions and geotechnical tests comparing critical shear stress and plasticity index between samples with varying Standard Penetration Test N Values. Crim also plotted results compatible with Briaud's relationship between initial erodibility and critical shear stress.

Mobley (2009) continued research with the EFA at Auburn. This work also showed erosion testing is highly variable and that few similarities exist in testing different formations. Mobley tested soils from Talladega County, Sumter County, and Dallas County. Mobley witnessed a high variability in erosion rates from the Talladega County soils, encompassing three orders of magnitude. Generally, the critical shear stress of the Talladega County soils ranged from 0.61 N/m² to 4.5 N/m². Mobley tested rock core samples from Sumter County. Mobley was unable to generate an erosion function for the Sumter County samples due to difficulty in sample preparation. One test from Sumter County showed scour resistance with a flume velocity of 0.75 m/s. The Dallas County samples consisted of soil from the Mooreville Chalk formation. Mobley performed three EFA tests on the Dallas County samples at the highest velocity generated by the EFA, approximately 6 m/s. None of these tests exhibited scour over a duration of two hours. Mobley stated the critical shear stress of the Mooreville Chalk was in excess of 45 N/m². Mobley also declared scour

performance was highly variable between formations and was somewhat depended on sample preparation.

Due to limited soil samples, Mobley tested model soils with varying densities and clay content. Mobley determined that model soils with higher density and higher clay content resulted in higher critical stresses and lower erosion rates.

2.7 Previous Scour Research by Others

Sheppard et al. (2006) determined the critical shear stresses, using the SERF, for uniform sands ranging from 0.08 N/m^2 to 0.75 N/m^2 . The critical shear stresses measured were increasing with mean grain size. Erosion rates for these tests exceeded 300 mm/s .

Sheppard et al. (2006) also created erosion rates for natural limestone core samples. Eight of these cores were tested in the RETA with once core tested in the SERF. The SERF, limited by shorter test duration, observed no scour during testing. Erosion functions for the limestone cores were created using the RETA. These tests yielded a minimum critical shear stress of 20 N/m^2 ranging upwards of 35 N/m^2 . The erosion rates calculated on the limestone were much lower than those observed in the uniform sands with a maximum erosion rate of 0.6 centimeters per year at a shear stress of 70 N/m^2 .

In addition to the tests described above Sheppard et al. (2006) performed erosion tests on cemented sands in an effort to mimic the scour behavior of sands with cohesive materials. The results of these tests varied due to the difficulty of sample preparation. The critical shear stress of these samples ranged from 5 N/m^2 to 10 N/m^2 . An estimate of erosion rate was created for these samples using the erosion function, splitting tensile strength, and unconfined compression strength.

Navarro (2004) performed erosion tests in a large scale flume, as described above, to produce critical shear stresses of soil samples obtained in Georgia. The results of these tests are summarized in Table 2-2 below. The critical shear stresses observed by Navarro (2004) are compatible with critical shear values ranging from 0.5 N/m² to 5.0 N/m² reported by Briaud et al. (1999).

Table 2-2: Summary of Navarro's Results

County	Critical Shear Stresses (N/m ²)	Classification
Murray	3 to 21	Silty Sand
Towns	6.8 to 21	Sandy Silt
Habersham	2.5 to 21	Sandy Silt
Haralson	3 to 12	Silty Sand
Bibb	3.3 to 9.7	Sand to Lean Clay
Wilkinson	0.4 to 21	Sand to Fat Clay
Effingham	3.24 to 21	Sand to Clayey Sand
Decatur	2.5 to 7.9	Clayey Sand
Berrien	> 21	Clayey Sand and Fat Clay
McIntosh	17.17	Clayey Sand

Chapter 3 Testing System and Methods

3.1 EFA Testing Equipment

As stated earlier, the Erosion Function Apparatus is the device Briaud (1999) created at Texas A&M University in order to collect the scour rates used to construct erosion functions. These erosion functions were used to estimate the depth of scour over the design life of a bridge. Figure 2-1 shows an image of an EFA. Figure 3-1 shows a schematic of an EFA featuring all of the integral components used for erosion testing (Crim 2003). The EFA contains a reservoir, a pump, and a flume. Water is pumped from the reservoir, through the flume, and across the sample during testing before being recirculated back into the reservoir. The valve on the front of the EFA is used to regulate the flow velocity during testing. A flow straightener is used to develop a full flow condition before the water flows over the soil sample.

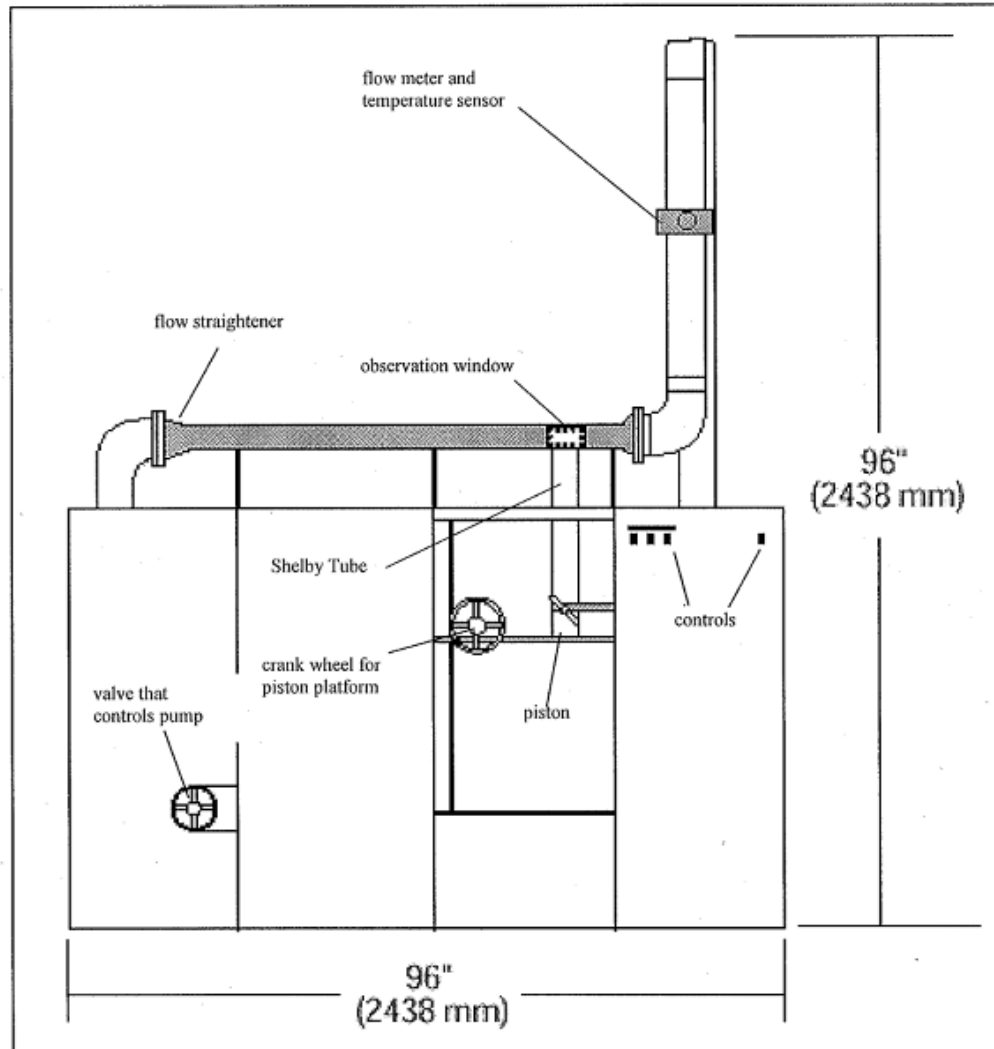


Figure 3-1. Schematic Showing Important Components of the EFA

The EFA is instrumented with a flow meter and temperature sensor in order to monitor flow velocity and water temperature during testing. The sample tube is positioned in the flume using the crank wheel attached to the piston platform. The piston platform is then locked into place so that the sample tube positioning does not change during testing. A motor is attached to the piston and the sample can be brought to the proper elevation in the flume by using a control switch. During testing, scour is observed through the observation window. Other controls shown in Figure 3-1 include a power switch to the EFA, a power switch to the pump, and a power switch to a sump pump, which is located in the reservoir.

EFA tests are instrumented to monitor the temperature and flow rate of the water used during a test. It is important for the flow velocity to be monitored and kept consistent during an EFA test as it directly reflects the shear stress imposed on a sample. Temperature is monitored to ensure the conditions in the EFA during testing are consistent with those in a stream or river. The EFA software consists of a Labview readout in which the temperature and flow velocity can be monitored and recorded. Figure 3-2 shows an image of the digital readout provided by the EFA software.

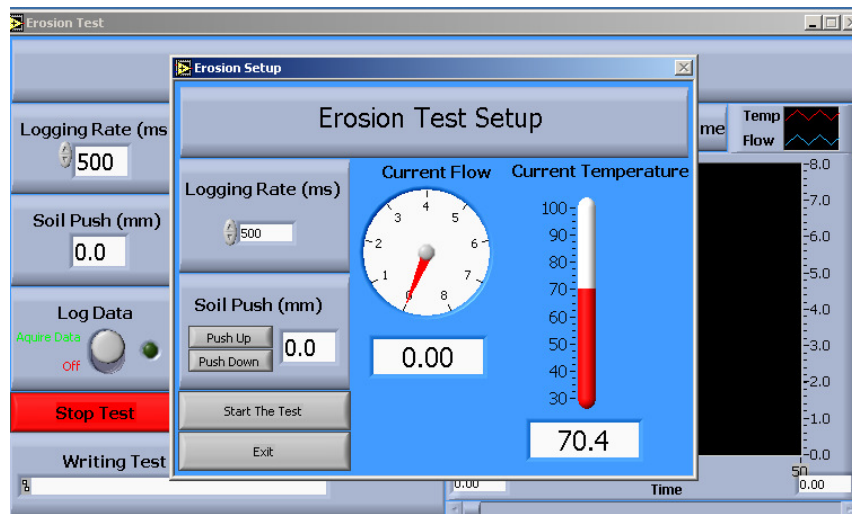


Figure 3-2. Digital Readout from EFA

The top window in Figure 3-2 shows the readout of the current flow velocity and current temperature. The readout reports the flow velocity by dividing the flow rate calculated from the flow meter by the cross sectional area of the flume. The sample can also be advanced in 0.5 mm increments by clicking the “Push Up” button. A test can be started by setting the logging rate and clicking “start the test”. During a test the attached data acquisition system records the temperature, flow velocity, and soil push location at the specified logging interval. This data logging method was used to calculate erosion rates in previous research at Auburn University by Crim (2003) and Mobley (2009). The screen in

the background of Figure 3-2 shows a graph that logs the velocity and temperature with respect to time during an EFA test.

3.2 Ultrasonic Sensor

Similar to the work performed by Sheppard et al. (2005) at the University of Florida, an ultrasonic sensor was designed and installed on Briaud's Erosion Function Apparatus at Auburn University. The SERF created by Sheppard and Bloomquist was designed to not only assist in calculating erosion rates, but to obtain a pattern of erosion. The ultrasonic sensor on the SERF consists of twelve transducers. Eight of the transducers are used in measuring scour of a 7.3 cm sample, while all twelve transducers are used to measure scour of a 9.5 cm sample. The design criteria for the EFA ultrasonic sensor was to create a similar array of transducers that will measure scour in cohesive soils, instead of rock, and consist of a tight grouping of transducers creating a clearly mapped soil surface. This ultrasonic sensor was designed and created with the help of Seatek and consists of sixteen transducers. The sensor is connected to a stainless steel housing, and mounted to the top of the flume of the EFA above the location of the test sample. A schematic of the ultrasonic sensor designed by Seatek is shown below in image 3-3.

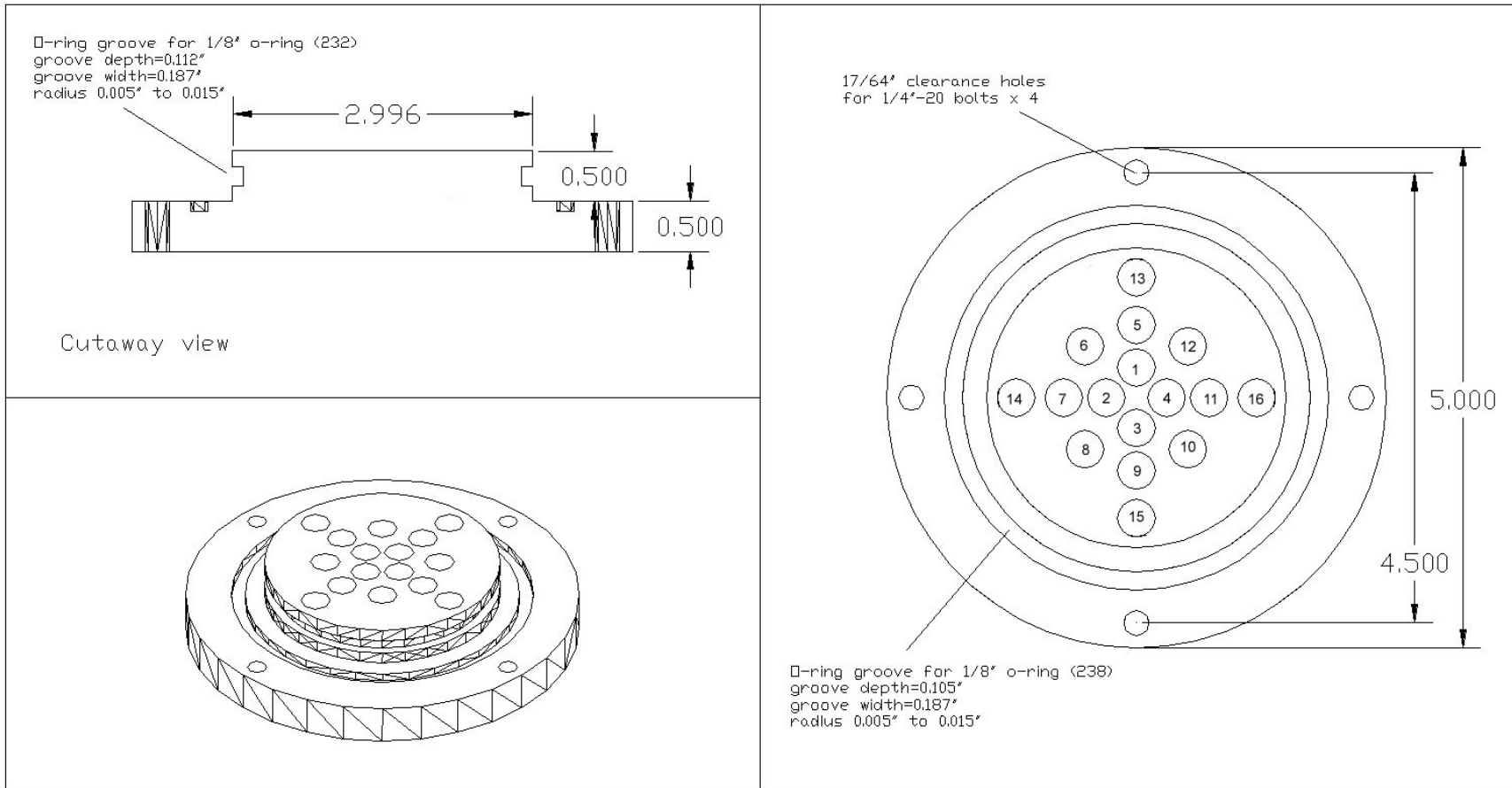


Figure 3-3. Auburn Ultrasonic Sensor Schematic

The transducers function at 5 MHz, with a physical diameter of 0.5 cm and an acoustic footprint of 0.8 cm at a distance of 5 cm (Jette 2010). The ultrasonic sensor was set into an aluminum cover, which replaced the original acrylic cover of the EFA. The ultrasonic sensor was sealed to the aluminum cover by using two O-rings. Careful considerations were made to ensure that the ultrasonic sensor was flush with the bottom on the EFA aluminum cover. The ultrasonic sensor could not protrude from the EFA cap, as this would result in irregular flow conditions. The ultrasonic sensor also could not be raised above the EFA cap, as the transducers must be submerged to function properly. The ultrasonic sensor is able to collect data from 16 different points across the area of a Shelby tube soil sample (71.12 mm). Also 12 data points can be obtained from standard rock coring soil samples, which is convenient if a soil formation is too stiff to be sampled by pushing Shelby Tubes.

The data acquisition system used with the ultrasonic sensor is capable of collecting data from all 16 transducers, along with the ability to sample up to 4 external analog channels. If any external analog channels are used, the output voltage must range between 0 V and 4 V. The electronics package is able to communicate via an RS232 (serial) connection. The software used to communicate with the data acquisition system was CrossTalk. CrossTalk is used to set the datalogging parameters for the system and produces an output ASCII text file from the data received that can easily be imported into a spreadsheet. The CrossTalk software and data acquisition system are set up on an updated secondary computer that is separate from the computer that acquires the temperature and flow velocity from the EFA.

When opening the CrossTalk program, a new session should be started, as seen in Figure 3-4. Next, the Connection Settings window should be open to ensure that the proper

parameters are input so CrossTalk can record the signal from the data acquisition system. The connection should be set to “Local COM Port (Direct Connect)”. The correct COM port should be chosen along with a Baud rate of 19200. Also, the Flow Control should be set to “XONXOFF”. The Connection Settings window should match the settings displayed in Figure 3-5. When setting up the CrossTalk software for an EFA test, the correct script should be chosen. This script is entitled “SEATEK.AU SCRIPT.XWC” and can be chosen as shown in Figure 3-6. The CrossTalk Script Dialog box will appear after the Seatek script has been selected, and “Begin data collection” should be selected. The Script Dialog box completed with typical test setup settings can be viewed in Figure 3-7.

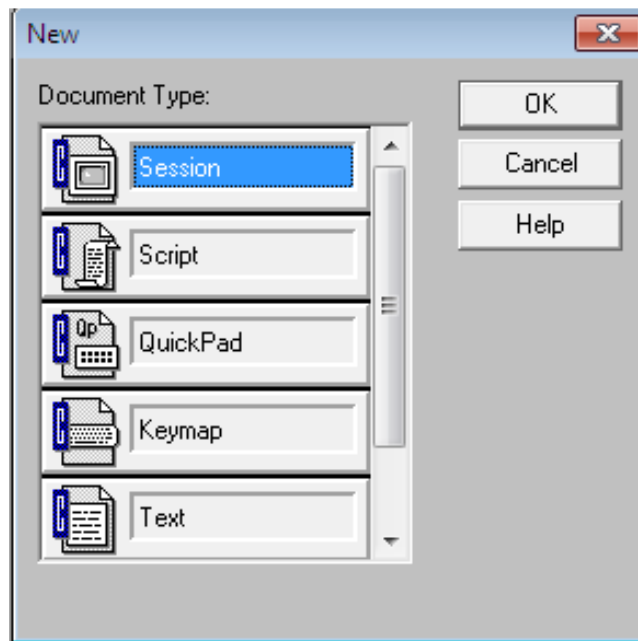


Figure 3-4. CrossTalk Startup Dialog Box

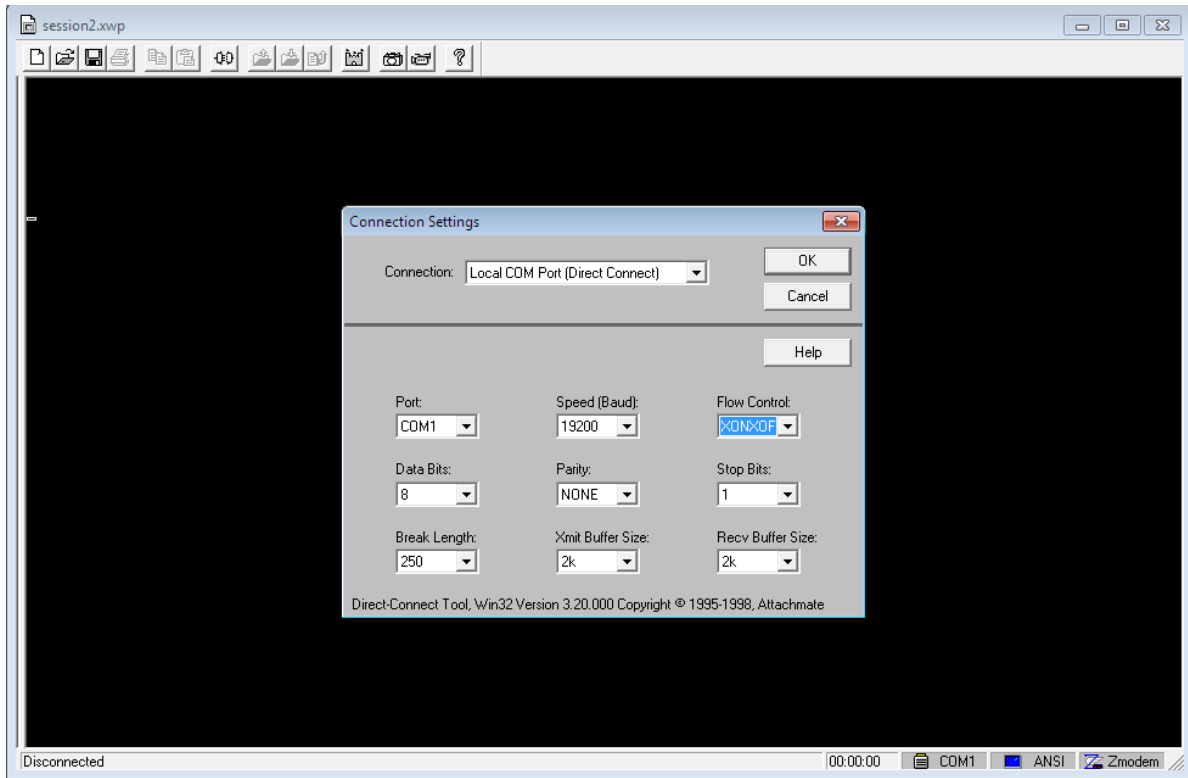


Figure 3-5. CrossTalk Connection Settings

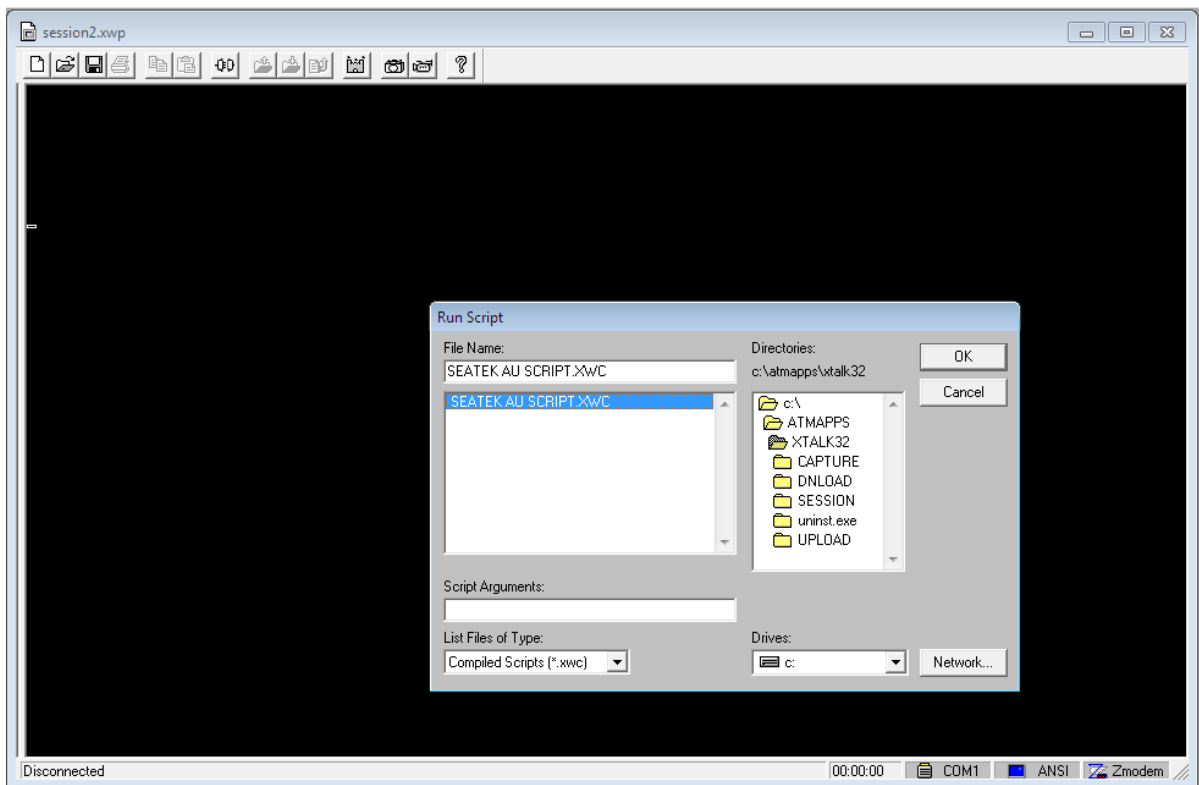


Figure 3-6. CrossTalk Script Dialog Box

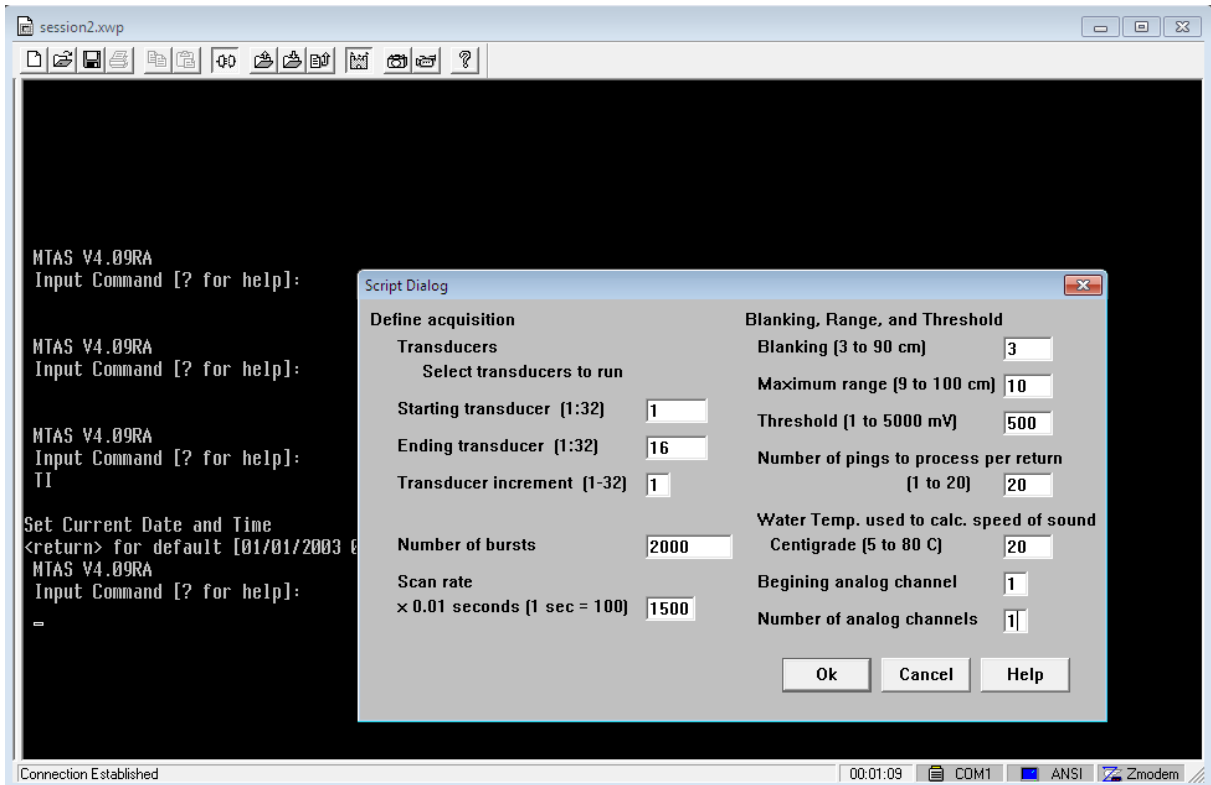


Figure 3-7. CrossTalk Script Dialog Box with Typical Settings

The script dialog box sets the parameters on each individual scan along with setting the duration of the entire test. For the ease of data reduction, transducer 1 was set to the starting transducer and transducer 16 was the ending transducer. The transducers were recorded in an increment of 1. The scan rate was typically set to 1500 corresponding to a scan every 15 seconds. It is important that acceptable distance parameters were established in order to qualify the scan readings from the ultrasonic sensor. The minimum or blanking distance for the ultrasonic sensor was 3 cm. The maximum distance was typically determined by a value that is not expected to be exceeded during an individual run. For most tests, the minimum distance was 3 cm and the maximum distance was 10 cm. As stated above these limits qualify the readings reported by the data acquisition system. If an errant reading was recorded that was outside of the acceptable limits, the reading was recorded as 0, disregarded, and not calculated in the data reduction.

The data acquisition system was able to sample up to 20 individual range readings, or pings, for every recorded reading. This capability greatly reduced the variability between recorded values by eliminating errant readings. If 20 pings were used and returned acceptable ranges, the three shortest and the three longest ranges were excluded from the data reading. The remaining 14 readings were then averaged, and the average was reported for each transducer. The ultrasonic sensor records a range measurement when the processed acoustic return exceeds a threshold voltage (Jette 2010). This threshold voltage was set through trial and error by adjusting the threshold voltage until distance fluctuations were a minimum. Since the ultrasonic sensor was used to measure scour in cohesive soils and cohesive soils are often smooth in nature in comparison to granular soils, the threshold voltage was typically set to a relatively low value of 500 mV.

The ultrasonic sensor designed by Seatek for Auburn University was created with the idea of improving scour measurements taken using Briaud's EFA. However, the 16 transducer ultrasonic sensor did have a few constraints that were addressed before a cohesive-soil testing regimen was adopted. As previously stated the bottom of the ultrasonic sensor, where the transducers contact the flow of water, could neither be elevated above the water or protruding into the flow of water. The ultrasonic sensor was originally designed to be mounted on top of the current acrylic cover of the EFA. This preliminary design left approximately a 13 mm hole above the flow of water allowing large air pockets to build up around the transducers. It was determined that the acoustic pulsing of the ultrasonic sensor was disturbed by these air pockets resulting in many blank readings. As a result a counterbore was machined into the acrylic cover of the EFA to recess the ultrasonic sensor flush with the bottom of the EFA cover. This design worked in theory, but the acrylic cover

did not allow for a waterproof seal to be formed around the ultrasonic sensor. In an attempt to tighten the seal between the EFA cover and the ultrasonic sensor the acrylic in the ultrasonic sensor was cracked allowing more water to leak through the seal. A third EFA cover was designed by counterboring the ultrasonic sensor into an aluminum stock plate. The aluminum plate was identical to the original acrylic cover, and strong enough to form a waterproof seal with the ultrasonic sensor.

Along with assuring the ultrasonic sensor was mounted flush in the EFA cover creating a waterproof seal, additional care was taken to ensure the transducers were not damaged. It was important the cables were not crimped, pulled, or forced into a small radius bend at any point during testing. When the cables are coiled, the signal-to-noise ratio may decrease resulting in blank or disturbed readings (Jette 2010). The transducers should also be supported at all times, and the cable alone should never support the transducers. To ensure that the cables were always supported, a wooden frame was built onto some of the flume supports of the EFA and the cables were draped over the frame.

The installed sensor measures distance readings by using ultra-sonic transducers; therefore, temperature effects must be considered. The water temperature is an input in the CrossTalk “Script Dialog” box as seen in Figure 3-7. However, the data acquisition system does not allow for any changes in water temperature throughout the duration of a test. This presents an issue as the data acquisition system assumes a certain water temperature in calculating the wave speed dictates the scanned distance. A six-hour continuous test showed that the water temperature in the EFA rose from 26 °C to 54 °C. This temperature change is related to the EFA’s pump energy and flume friction. The EFA contains a water tank that is approximately 1.36 cubic meters. To reduce the magnitude of temperature change, water

was continuously circulated through the EFA tank. Water was circulated into the EFA tank using a water hose from a spigot, while the water was pumped out by using the EFA's drain and sump pump. During a three hour test with water continuously circulated through the EFA the water temperature ranged from 19.2 °C to 21.3 °C. It is important to keep the water temperature in a reasonable range to mimic stream conditions as closely as possible. Due to the effects of temperature on wave speed, a temperature correction was necessary to overcome the variance in temperature. During a typical test the temperature was set at 20 °C, and a temperature correction was performed during data reduction.

As previously stated, the EFA contained a separate data acquisition system and software package intended to detect and record flow rate and water temperature. It was determined the thermistor installed on the EFA fit the voltage parameters of the analog channels on the ultrasonic sensor data acquisition package. The EFA thermistor was then installed and calibrated as an analog input for the ultra-sonic sensor. During the setup for a typical test, it was necessary to include one analog channel on each scan in the CrossTalk "Script Dialog" box shown in Figure 3-7.

The thermistor was calibrated using a voltmeter and the digital readout from the EFA. This calibration was then plotted and equation relating temperature to voltage was derived. The temperature calibration can be viewed in equation 3-1, and an image of the temperature calibration can be viewed in Figure 3-8. The Auburn EFA flow meter was calibrated by Mobley (2008). This calibration was performed manually by measuring the amount of water coming out of the spout of the EFA and comparing it to the velocity shown in the EFA software. Mobley concluded there is up to 10 percent error in velocity readings less than 1.0

meter per second. At higher velocities there is not any appreciable error between the true flume velocity and the measured velocity by the EFA's flow meter.

$$^{\circ}F = 0.1232 \cdot mV - 133.83 \quad (3-1)$$

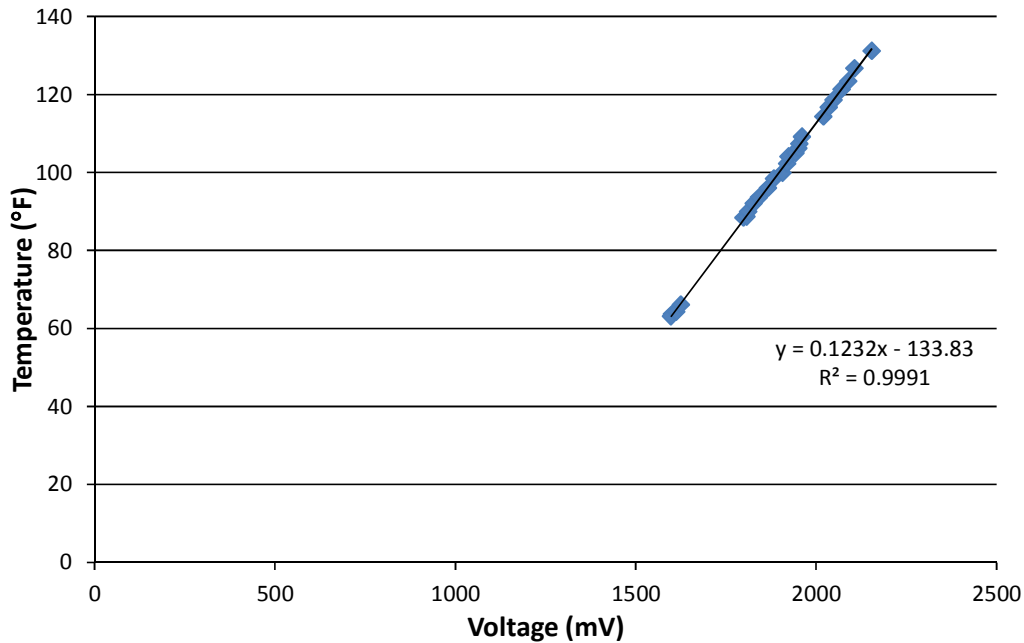


Figure 3-8. Temperature Calibration for EFA Thermistor

3.3 Data Reduction

For each test and scan performed, data reduction was necessary to determine a change in mass. Each scan from the Seatek sensor contained a time stamp, a distance measurement from all 16 transducers, and a voltage reading from the thermistor. Each time stamp was converted into an elapsed time, so that erosion rates may be calculated. If a distance measurement returns a blank value (reading of 0.00), an average distance of neighboring transducers was taken and recorded over the blank value. The voltage recorded from the thermistor was also converted to a temperature, using equation 3-1, so that temperature changes could be monitored throughout testing.

Since the Seatek sensor utilized ultrasonic wave propagation, any changes in water temperature affected distance measurements. Therefore, a temperature correction was

required with every scan. The speed of sound was calculated by using Equation 3-2 below (Seatek 2010), where speed of sound is represented in meters per second, and temperature (T) is given in degrees Celsius.

$$SOS = 0.0029T^3 - 0.055T^2 + 4.95T + 1402.3 \quad (3-2)$$

An initial speed of sound (SOS1) was calculated, in meters, based on the input temperature given in the data acquisition setup as shown in Figure 3-7. The final speed of sound (SOS2) was calculated based on the temperature at the time of each scan. The final speed of sound represents the true water temperature in the EFA at the time a reading is taken. Using the initial speed of sound, the elapsed time, in seconds, of each scan was calculated by using Equation 3-3.

$$Elapsed\ Time = \left[\frac{SOS1 * 100}{Initial\ Measurement} \right]^{-1} \quad (3-3)$$

The elapsed time represents the time for the ultrasonic wave to travel from the sensor to the surface of the sample and back to the sensor. This time is used by the data acquisition system to calculate a measured distance according to the input parameters in Figure 3-7. The elapsed scan time was used to calculate the corrected distance measurement, as shown in Equation 3-4.

$$Corrected\ Distance = SOS2 * Elapsed\ Time * 100 \quad (3-4)$$

The transducers reported an average distance of 20 pings per scan, and scans were performed every 15 seconds to reduce the level of variability between readings. Four consecutive corrected distance measurements were averaged to represent the measured distance per elapsed minute.

The final step of data reduction process was to convert the measured distances at any given time, to a volume of erosion. Each transducer was assigned a tributary area based in

scan area such that a weighted average could be taken. Since the ultrasonic sensor commonly measured two different sample sizes, two tributary areas were assigned to each transducer. Figure 3-9 maps the tributary areas for both the 5.72 cm and 6.35 cm samples. Tributary areas were only calculated for transducers 1 through 12, as transducers 13 through 16 were outside the sample area.

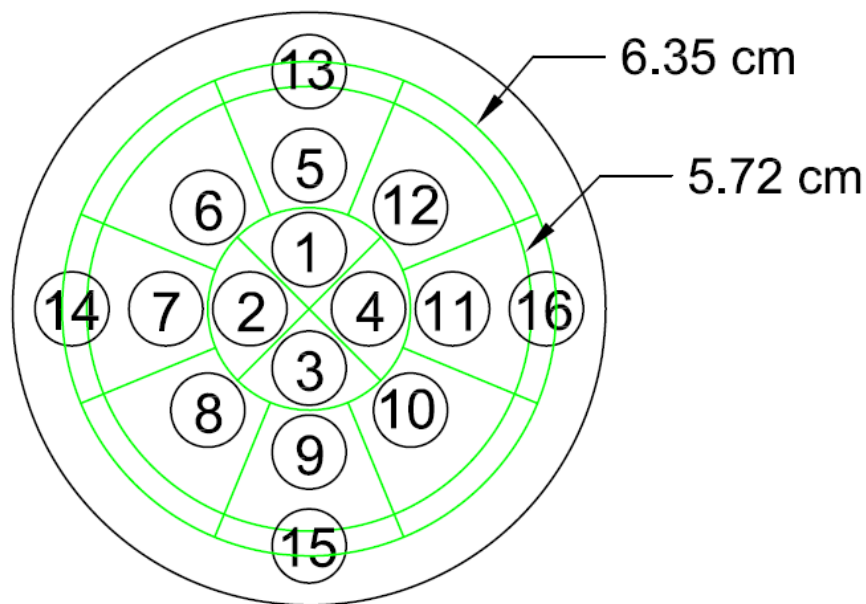


Figure 3-9. Sensor Tributary Areas

During any given scan, an initial distance measurement, or datum, was established over the first minute of scans, for each transducer. The corrected average distance at any given time was subtracted from the initial datum, and multiplied by the tributary area. The sums of these products were summed over transducers 1 through 12. The volume of erosion at any time interval, in cubic centimeters, can be calculated by using Equation 3-5.

$$\sum_{n=1}^{12} (D_i - D_f) \cdot A_t \quad (3-5)$$

The volume of erosion was converted to cubic millimeters, and an average height of erosion was calculated, by dividing the volume of erosion by the area of the sample. The average height of erosion represents a height change assuming all calculated scour was uniform. For EFA testing, once the overall height change reflected a decrease of one millimeter, the sample was advanced. This average height of erosion is precisely calculated adding an advantage of traditional visual observations presented by Briaud. The elapsed time for one millimeter of erosion can then be converted to an erosion rate.

3.4 Verification of Sensor Operation

Verification of the ultra-sonic sensor was performed through several trial scans, using a non-erodible sample. The non-erodible sample was created by applying a sand surface to an aluminum rod with a diameter of 6.35 cm. The goal of each trial scan was to successfully show sample movement by advancing the sample at a known time and distance. The trial scans mirrored the results and procedure of a typical EFA test. Since the EFA advances the sample from the bottom, the non-erodible sample trial scans displayed movement as the target became closer to the sensor. In a typical EFA test, the sample will be advanced and then progressively move farther away, as scour occurs, from the sensor located at the top of the flume. For the sake of verifying the effectiveness of the ultrasonic sensor it was determined that as long as movement can be accurately measured, the relative direction of movement did not matter.

Figure 3-10 below shows the results of a trial scan. The non-erodible aluminum sample was placed in the EFA and scanned for approximately 150 minutes. During this scan two separate one millimeter pushes were induced, at 42 and 128 minutes respectively. Figure 3-10 proves that the ultrasonic sensor, data acquisition package, and data reduction methods

produced the expected results, as one millimeter jumps are observed at 42 and 128 minutes. It is important to notice that variation in these readings were small, with a maximum of 0.08 mm from the target distance. These small variations could be attributed to variations in errant scans or temperature corrections.

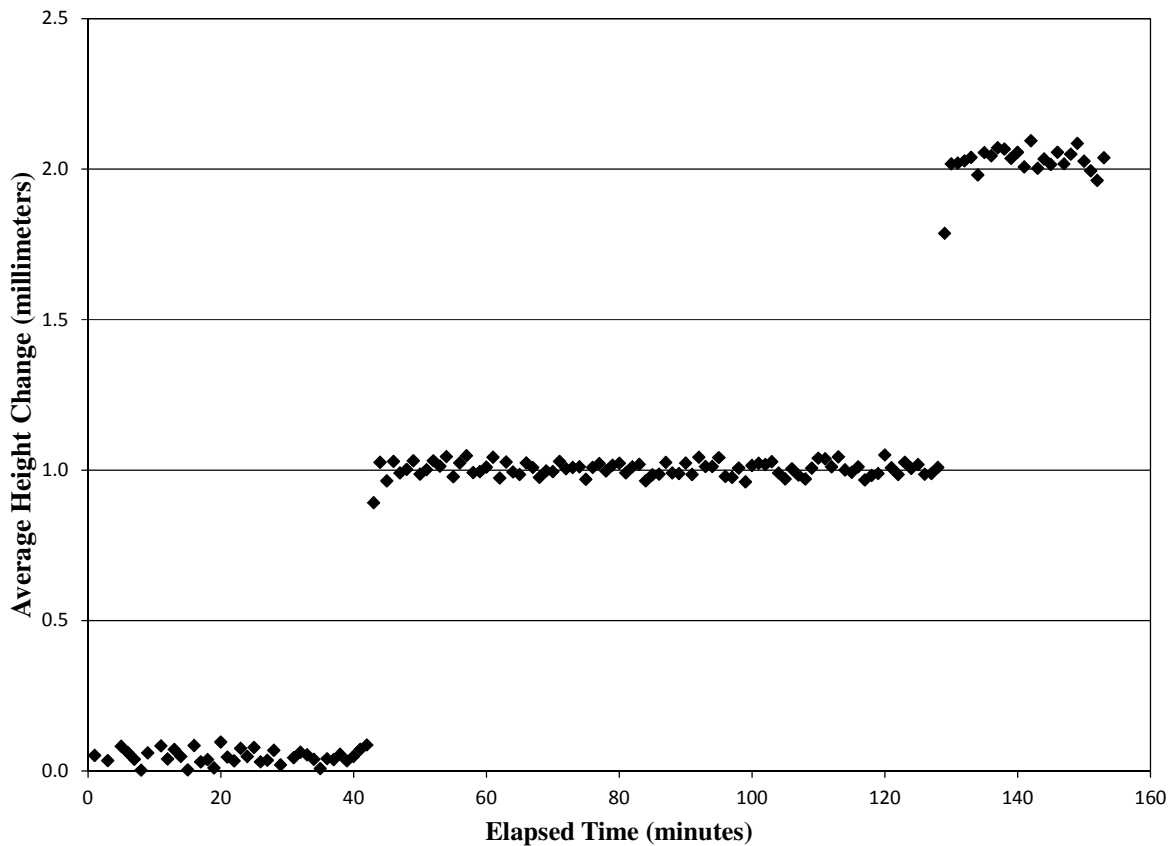


Figure 3-10. Sensor Verification Test

3.5 Testing Procedure

3.5.1 Sample Procurement

Drilling operations were coordinated with the Alabama Department of Transportation (ALDOT) to acquire cohesive soil samples for testing. ALDOT stated that these hard cohesive soils and chinks would not be conducive to sampling via Shelby Tube. Since the materials to be sampled were so stiff, a typical all terrain vehicle or truck drill rig would not be able to advance a Shelby tube sample without damaging the tube. Two sampling

alternatives were derived to ensure that samples could both be acquired with relative ease and tested using the ultrasonic sensor at Auburn. Option one involved using either 44.5 mm or 47.6 mm rock core samples, while option two involved using 57.2 mm continuous samples, a new sampling method acquired by ALDOT. Preliminary tests were performed with the EFA using rock core samples, and it was determined that the rock core samples were not ideal and should only be used as an alternative. This was decided as it was necessary for rock core samples to be completely vertical and plumb or else the samples would not fit into the previously created EFA testing tube. Also, it was determined that short segments could not be used with rock core samples as the flume created a “suction” like force pulling the loose sample towards the ultrasonic sensor. This sample movement upward could not be tolerated as any movement other than scour and planned sample advancements recorded would conflict with the results presented by the ultrasonic sensor.

Therefore, it was decided to sample with the new continuous sampling technique that ALDOT recently acquired. This sampler easily fits onto an ATV- or truck-mounted drill rig and can be used to acquire undisturbed soil samples in difficult to sample soil formations. The sampling technique involves first drilling down to the sample level with a hollow-stem auger. The 1.52 m sampler is attached to either AW or NW threaded drill rod and lowered to the bottom of the hollow stem auger. Once connected the sampler is pneumatically advanced as the hollow stem auger cuts around the obtained soil sample. A sealed bearing assembly conveys the thrust from the drill rig to the tube and sampler shoe while isolating the sample tube from rotation of the auger (CME 2012). An image of the Central Mining Equipment (CME) continuous sampling setup used is shown in Figure 3-11. A sampling shoe is used to contain the sample in the tubes for a maximum recovery. The sampler itself is a split spoon

sampler that includes two acrylic 57.2 mm diameter tubes that are each 762 mm long. Once samples are gathered lids are placed on top and bottom of each sample allowing tubes to be tested and stored separately.

During a typical sampling trip ALDOT geologists located possible drilling sites containing the target formations. ALDOT geologists also determined the depth where the target formation was reached. Once the target formation was confirmed from drill cuttings and split spoon samples, a Standard Penetration Test (SPT) was performed to obtain the SPT N value for each formation. Typically two to three standard penetration tests were performed in each formation depending on the approximate depth of the formation provided by the ALDOT geologist. Once the SPT was completed, the AW rod was removed from the boring and a hollow stem auger was used to the depth of the previously drilled hole. Typically a Shelby Tube was pushed inside the hollow stem auger to ensure that any loose cuttings would not be sampled. Sampling was then performed as described above with the CME continuous sampling system. Typically 2-3 continuous samples were gathered in each formation resulting in 4-6 sample tubes. It was believed that obtaining 1.5-2.3 m of each formation would be sufficient for both EFA testing and geotechnical testing. After collection, sample tubes were capped and taped to maintain field moisture conditions. Since EFA and geotechnical testing could take place months after sampling, samples were stored in a curing room to preserve field moisture conditions. Sample tubes were also marked according to depth and formation prior to storage.

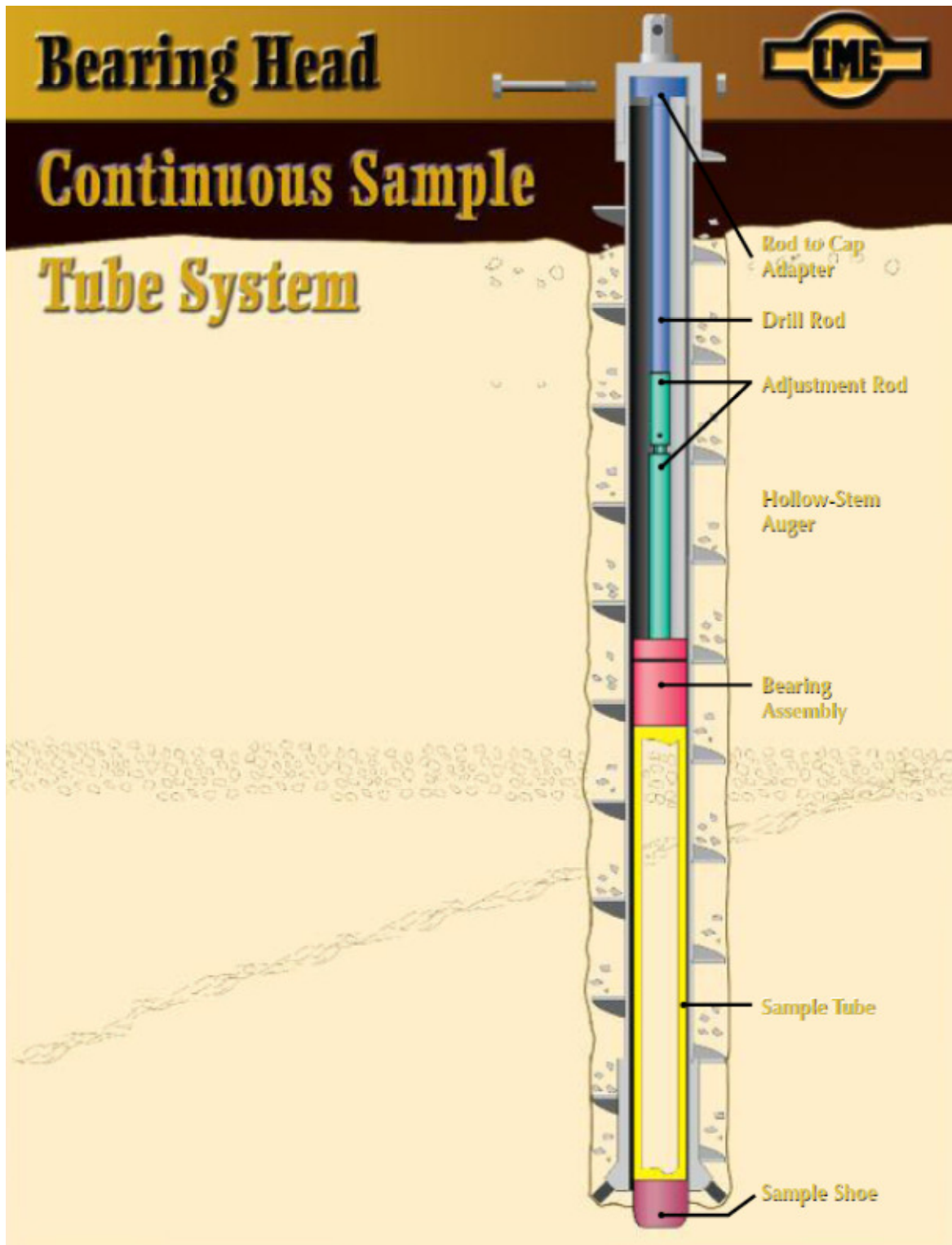


Figure 3-11. CME Continuous Sampling System

3.5.2 Sample Preparation

A testable EFA sample was not cracked radially or longitudinally with a length of at least 101 mm. It was important that testable sections were not cracked as cracks would have induced shear planes that would alter the results of testing. A typical sample tube had an outside diameter of 63.5 mm, an inside or sample diameter of 57.2 mm, and a length of 762 mm. The pushing arm of the EFA could accommodate a sample tube length up to 457 mm. An automatically advancing hacksaw was used to cut the sample tubes to accommodate the dimensions of the EFA. A testable section for the EFA was located and marked on the sample tube. The section was then loaded into the saw and clamped to ensure that a square cut was made. It was important the clamp was not so tight to distort the acrylic tube. Figure 3-12 below shows a sample section being cut with the automatically advancing hacksaw.



Figure 3-12. Automatically Advancing Hacksaw Cutting Sample Tube

Once the sample section was cut, the sample was extruded approximately 5 mm in order to prepare the surface for testing. Using a spatula the surface of the sample was cleaned of any loose debris. The sample was also cut level with the sample tube, and the surface was smoothed with the spatula. At times a wire saw was used along with a spatula to prepare the surface for testing. Figure 3-13 shows the sample being extruded, and Figure 3-14 shows the sample surface being prepared for testing.



Figure 3-13. Extruding Sample



Figure 3-14. Cutting and Cleaning Surface of Sample

3.5.3 EFA Testing Procedure

Once the EFA test sample was cut, extruded, and cleaned the below procedure was followed for performing a typical EFA test.

1. The EFA tank was rinsed of any suspended material or debris to clean out the tank.
2. The EFA tank was filled with clean water, approximately 2/3 of the tank height.
3. The drainage valve at the bottom of the EFA was opened to allow water to drain from the EFA tank.
4. The sump pump on the EFA was turned on to assist the drainage valve in draining water from the tank.
5. Once the tank was full, drainage valve open, and sump pump on, clean water was added to the tank. This completed the continuous circulation of clean water in the EFA to control temperature levels.
6. The water level was monitored throughout the testing process to ensure that the water level did not drop below the pump intake or rise within 300 mm of the top of the tank.
7. The blank sample was loaded into EFA, and the pump was turned on. The flow velocity was adjusted to the target testing velocity. This step, suggested by Crim (2003), allowed for velocity to be adjusted between tests without exposing the soil sample to varying shear stresses.
8. Once the target velocity was reached the pump was turned off and the blank sample was removed from the EFA.
9. The prepared soil sample was then loaded into the EFA, assuring that the sample tube was flush with the lip of the base plate of the flume.

10. Using the stepping motor, the sample was advanced through the base plate until the sample is level with the base of the flume. Figure 3-15 shows the soil plunger being pushed against the base of the soil sample, and Figure 3-16 shows the sample pushed level with the base of the flume.
11. The CrossTalk program was started, connection settings entered, and the script dialog box was filled out according the settings shown in Figure 3-7. It is important to note that this step occurred before the flume was started. Also it is important the CrossTalk script was simply set-up and not started, as blank scans at the beginning of a test would interfere with data reduction.
12. The EFA pump was started and the water velocity was allowed to accelerate until the target velocity was reached. The velocity was monitored using the software shown in Figure 3-2.
13. The CrossTalk script was quickly started by pressing “OK” on the Script Dialog Box shown in Figure 3-7 and naming the data file according to the sample being tested. It is important that this step be started immediately after the flume fills with water, as any soil mass lost before the CrossTalk script was started would not be recorded.
14. Depending on the scour characteristics of the soil being tested, the Crosstalk script was usually allowed to run approximately three minutes with the soil sample level with the base of the flume. This step was essential is establishing a clear baseline before the sample was advanced and allowed to scour. However, when testing at high velocities that resulted in higher scour rates this amount of time was reduced to 1.5 minutes to ensure the soil mass did not scour prior to being advanced.

15. The sample was advanced 1mm into the flume using the EFA software. Figure 3-17 shows a sample advanced into the flume.
16. Throughout the entire test, scour behavior was visually monitored and recorded to verify the results provided by the ultrasonic sensor. Scour was also visually monitored to determine any trends in the scour mechanism of each formation tested.
17. Once the data acquisition system and EFA test were running smoothly, the real time data reduction Excel spreadsheet template was started. It is important to note that this step is not necessary as data reduction can be performed with the created data file after the test is completed. However, one of the benefits of CrossTalk software and data acquisition system is that data can be reduced in real time and scour results can be presented complimentary to visual confirmation.
18. With the template Microsoft Excel data reduction file open, the “Data” tab was selected. The “From Text” icon was selected and the CrossTalk text file was imported. It was determined that the text should be delimited by tab and space so that each data column would be imported separately. The “Properties” tab was opened when prompted to select the cell in which the data will be placed. In the “External Data Range Properties” window the “Save query definition”, “Adjust Column Width”, and “Preserve Cell Formatting” boxes were selected. Under the “Refresh Control” heading, the box was selected to refresh the data prompt every 2 minutes. The Excel sheet was set to refresh the data from the text file every 2 minutes in order to monitor scour every in real time. Screen shots from this step are shown in Figure 3-18 and Figure 3-19.

19. Scour was monitored in the “Erosion Height Change” tab of the Excel template sheet. Typically the 1 mm push and subsequent scour was clearly visible. Once the “Erosion Height Change” tab showed erosion totaling 1 mm the test was ended. Again it was important to visually confirm that 1 mm of scour occurred before stopping the CrossTalk software or the EFA pump.
20. Once erosion occurred and the test was completed, the CrossTalk software was stopped by pressing “Control-C”. The EFA pump and the water supply were stopped once the test ended.
21. The water was allowed to drain from the flume before the sample was removed from the base plate of the EFA. The sample surface was cleaned similarly to the method described in the “Sample Preparation” section. This action was necessary to ensure that water did not percolate through the sample over time compromising future tests from being performed at field conditions.
22. Once the sample surface was cleaned and level with the sample tube the sample was capped, sealed with tape, and placed back into the curing room until the next EFA test was performed.
23. The EFA was drained and rinsed of any debris that resulted from erosion testing. It was important that the amount of suspended soil particles in the EFA were minimized as erosion could result by the collision of particles in future tests.
24. The erosion rate for each test was determined by dividing the height of erosion (1 mm) by the amount of time it took to achieve the height of erosion. This could be clearly mapped using the Excel data reduction spreadsheet.

25. The CrossTalk text file and the Excel data reduction spreadsheet were saved with the title of each test so that any additional post processing could be performed if necessary.

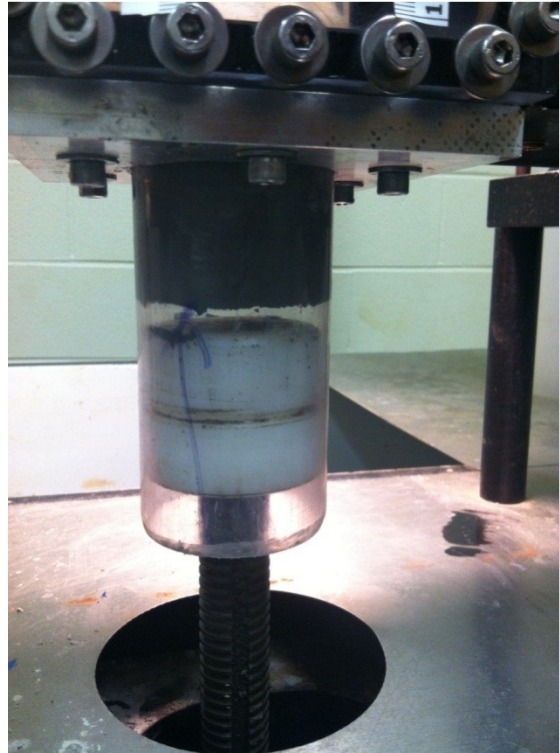


Figure 3-15. Soil Plunger Advancing Sample



Figure 3-16. Soil Sample in EFA Level with Base of Flume

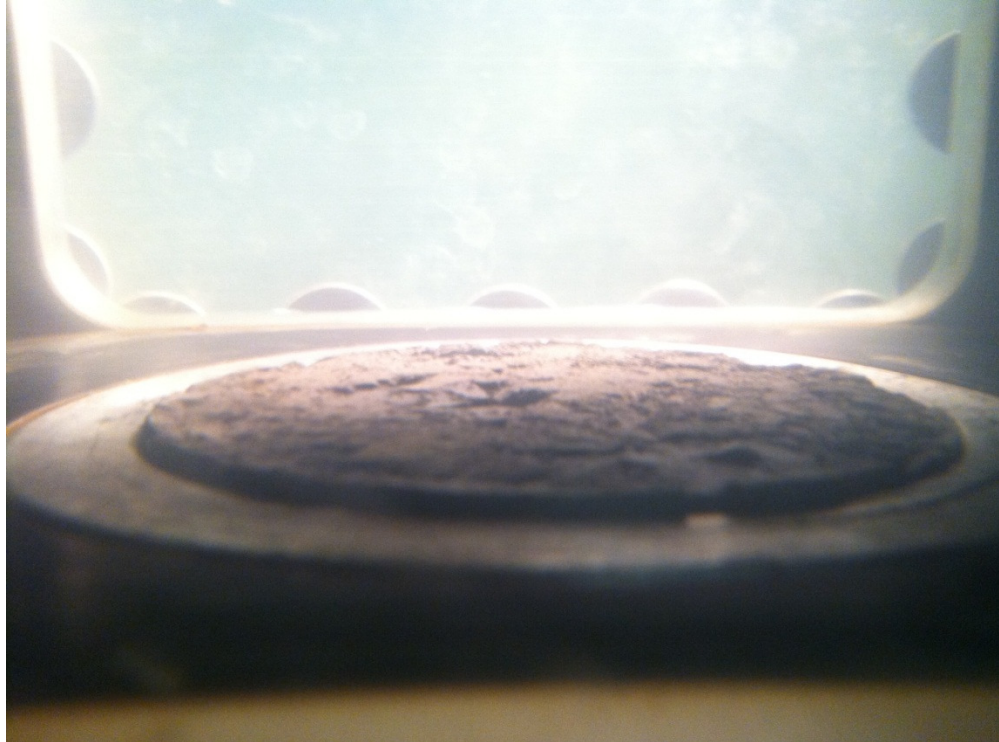


Figure 3-17. Soil Sample in EFA Advanced 1mm

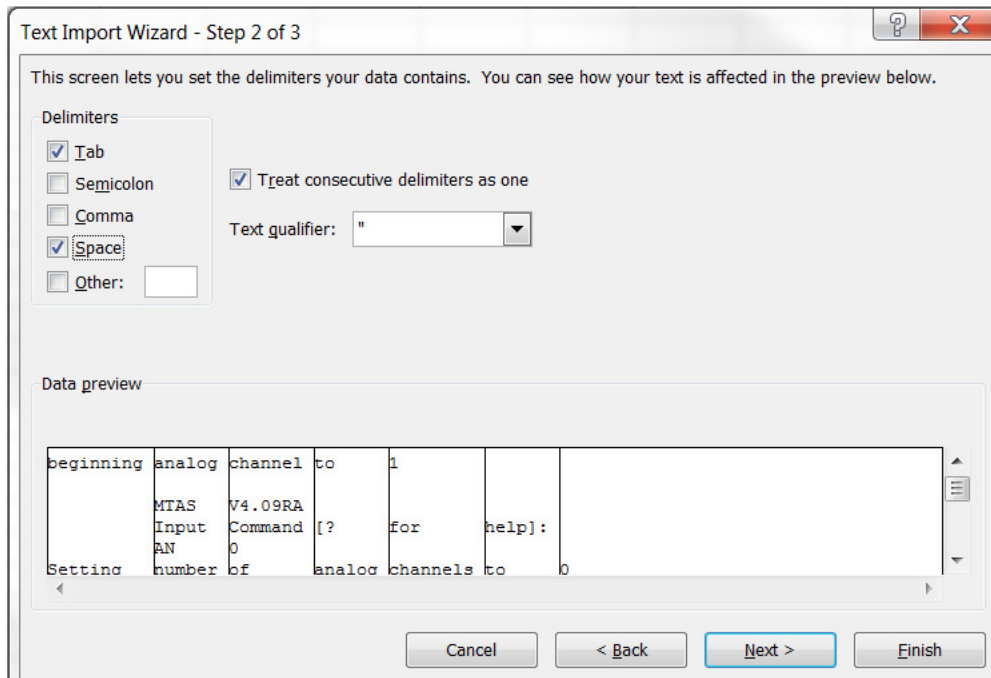


Figure 3-18. Text Import Wizard, Delimited Settings for CrossTalk File

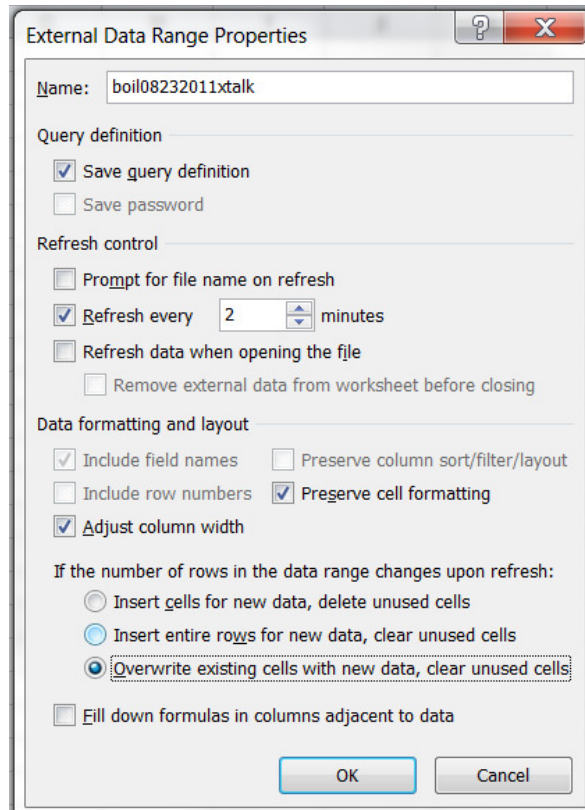


Figure 3-19. Import Properties for Data Reduction Spreadsheet

3.6 Testing Regimen

Based on the original EFA reports by Briaud et al. (1999), previous work at Auburn University (Crim 2003; Mobley 2009), and the maximum stream velocity expected in Alabama rivers, a testing system was created to include six different testing velocities. These EFA testing velocities include 0.3 m/s, 0.6 m/s, 1.0 m/s, 1.5 m/s, 2.0 m/s, and 3.0 m/s. Typically a formation was first tested at 0.3 m/s, and the velocity was gradually increased until scour occurred. A formation was considered to resist scour at a certain velocity if scour did not occur with one hour of testing. After it was determined that a given formation was scour resistant at a certain velocity, the velocity was increased to the next highest velocity. Once scour occurred at a given velocity, numerous tests were performed at each of the

remaining testing velocities so that averages could be established. Typically a minimum of three EFA tests were performed at each velocity step greater than the threshold velocity related to the critical shear stress. The amount of tests performed at each velocity was dependent upon the amount of testable soil recovered during sampling.

After testing was completed across all of the testing velocities, a threshold velocity test was performed to determine the velocity that correlates to the critical shear stress. This test was started by exposing the soil sample to the highest velocity that did not show any sign of erosion. The flume velocity was then steadily and carefully increased until scour started. Once scour started the velocity was recorded and used as the threshold velocity in the creation of erosion functions.

If a formation was determined to be resistant to scour at all of the listed EFA testing velocities, a multiple events test was performed. This test attempted to model the performance of a formation against changing shear stress cycles. For the purposes of this research, the multiple events test included running an EFA test on a sample for one hour at 3.0 m/s, reducing the velocity to 1.0 m/s for 30 minutes, and increasing the flume velocity to 3.0 m/s for another hour. The multiple events test was not performed on any formation with a threshold velocity less than 3.0 m/s. It was determined that adequate scour data could not be collected on a sample that had already scoured 1 mm and was simply advanced another millimeter. In previous work at Auburn University (Mobley 2009) it was noted that scour in cohesive soils does not occur in a uniform fashion. Therefore, advancing a sample 1 mm after it has already eroded would not expose an even shear stress across the plane of the sample as some of the test specimen would be higher than the necessary 1 mm and parts of the specimen would be lower than the necessary 1 mm.

3.7 Geotechnical Testing

As previously stated, a major goal of this research was to correlate scour parameters and behavior to common geotechnical parameters. In an attempt to determine any trends between scour behavior and geotechnical parameters, several geotechnical laboratory and field tests were performed. During sampling Standard Penetration Tests were performed on each formation. Natural moisture contents were taken from sampling tubes prior to EFA tests. Excess soil from sampling was used to perform a sieve analysis (ASTM D422), hydrometer, liquid limit and plastic limit test (ASTM D318) for each formation. It was intended that unconfined compression tests be performed on each formation in an attempt to correlate scour parameters to shear strength. However, after studying the sample tubes, there was not a sufficient uncracked sample length of 11.43 cm to perform an unconfined compression test among any of the sampled formations.

Chapter 4 Test Results

4.1 General Sampling

As previously stated, the sampling program for this research used the CME continuous sampling system. Sampling was coordinated with the assistance of ALDOT geologists and drill crews. Nine total cohesive soil and chalk formations were sampled with the intent to determine scour and geotechnical parameters. These drilling locations along with the seven previous drill sites tested at Auburn are shown below in Figure 4-1.

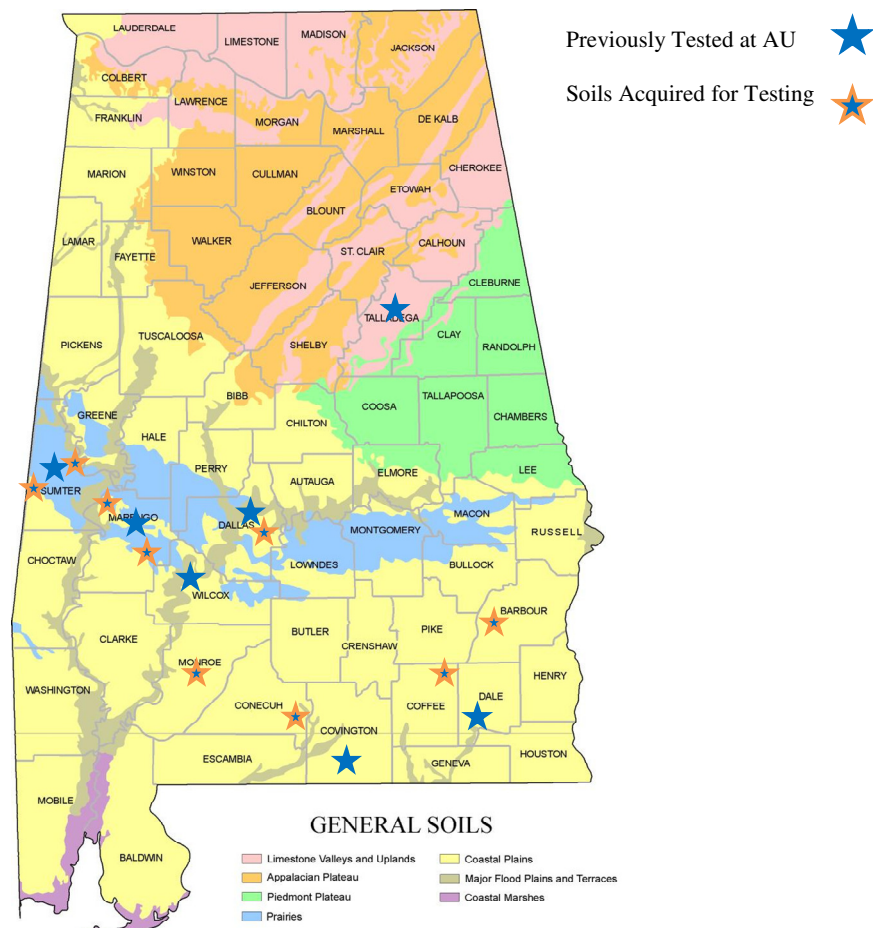


Figure 4-1. Locations of Sampled and Tested Formations

The current study's drill sites were located in the southern and western portions of Alabama, specifically in the coastal plains and prairies. These locations were drilled between April and June of 2012.

Typical tests were annotated using a combination of the formation, depth, and test number of each sample. For example, a test name was titled "Bucatumna27.0_1". The first word in the title represents the formation of the sample, the Bucatumna Clay formation. The first set of numbers in the title represents the approximate depth (in feet) the sample was procured, or 27 feet in the example title above. Finally, the number after the underscore represents the test number performed on any given sample.

4.2 Bucatumna Clay

The first formation tested using the updated EFA featuring the ultrasonic sensor was the Bucatumna Clay formation located in southern Alabama. The Bucatumna Clay formation was sampled using the CME continuous sample system. EFA tests along with geotechnical tests were also performed on the Bucatumna Clay formation.

4.2.1 Sampling

The Bucatumna Clay formation was sampled on April 5, 2012 in Monroe County, Alabama. An ALDOT geologist classified the Bucatumna Clay formation as a dark gray to brown clay. The formation was located with the assistance of an ALDOT geologist at a depth of 3.35 m below the ground surface. A Standard Penetration Test was performed between 3.5 and 3.96 m below the ground surface, yielding a SPT N value of 6. Three different continuous samples were taken from depths 4.11 to 5.64 m, 5.64 to 7.16 m, and 7.16 to 8.69 m. Since this was the first sample procured using the CME continuous sample system and because the Bucatumna Clay was relatively soft, a Shelby Tube was advanced from 8.69 to 9.30 m. The

Shelby Tube sample was acquired as a back-up in case EFA testing could not be performed with the continuous samples. Lastly, another Standard Penetration Test was performed between 9.30 and 9.75 m with a SPT N value of 9.

The continuous sample from 4.11 m to 5.64 m was determined to be unusable for EFA testing. The top of the sample was not usable as it had a plug missing from the Standard Penetration test taken above. The sample gathered from 4.11 to 5.64 m was also smaller radially than the sample tube at the base of the sample as shown in Figure 4-2.



Figure 4-2. Bucatunna Clay Sample Smaller Radially than Sample Tube

The second sample of the Bucatunna Clay formation ranged from 5.64 to 7.16 m and was mostly unusable. The entire top half of the sample was severely fractured in all directions and could not be used for EFA testing. However, the bottom third of the bottom half of the sample was mostly uncracked and sections could be used for EFA testing. The third and final sample from 7.16 to 8.69 m was wrinkled at the top with the bottom 20 to 30 mm un-cracked. It appeared in the second and third samples that the soil was failing in shear during sampling along

the tube walls. This would be possible if the rotating head of the continuous sampler did not fully separate the sample from the rotation of the hollow stem auger and the sample was shoved and twisted into the tubes. This phenomenon was observed and monitored throughout the sampling of all formations and these disturbed and cracked regions were not tested in the EFA. These regions were however used in geotechnical testing where soil was processed prior to testing.

4.2.2 EFA Testing

The Bucatunna Clay was the first formation tested at Auburn University using the ultrasonic sensor and data acquisition system. In total 32 EFA tests were performed on the Bucatunna Clay formation with 27 tests providing results for determining a scour rate versus velocity relationship. The first sample used for EFA testing was located at approximately 8.22 m below the ground surface. The first test titled “Bucatunna27.0_1” was performed to determine a starting velocity for testing. The data collected from “Bucatunna27.0_1” is shown below in Figure 4-3.

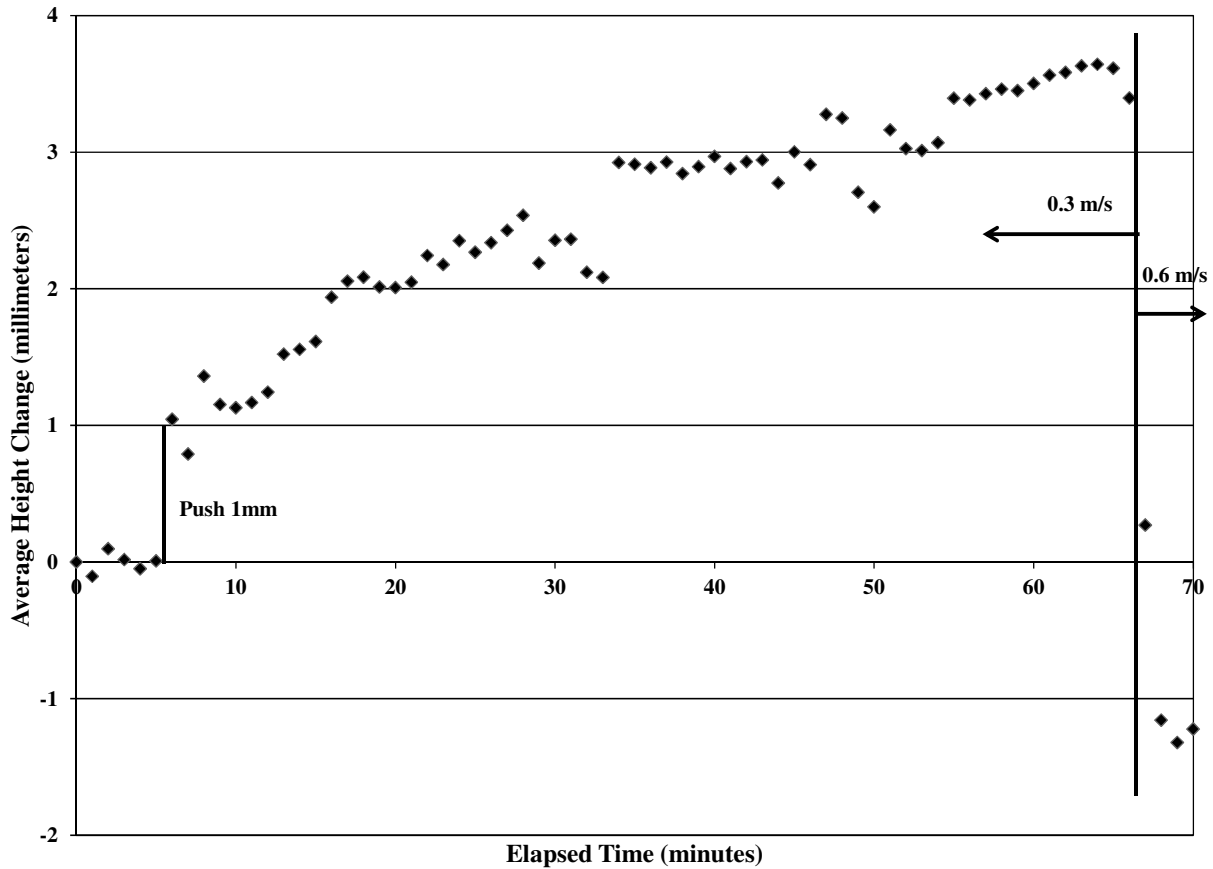


Figure 4-3. “Bucatanna 27.0_1” Results

The “Bucatanna 27.0_1” test specimen began at a starting velocity of 0.3 m/s, and was advanced 1mm into the flume after approximately 5 minutes. This can be viewed in the bump at the five minute mark in Figure 4-3, and further proves the validity of the ultrasonic sensor. The “Bucatanna 27.0_1” sample actually showed a net gain in height over the next hour while the velocity was held at 0.3 m/s. This seemed odd that the sample was actually rising into the flume as the shear stress was applied to the sample, but this movement was visually confirmed. After one hour, no scour was observed and the sample had raised an average of 2.5 mm. It was believed that this swelling was due to the Bucatanna Clay being a swelling clay formation, and this issue would be confirmed during geotechnical testing. Once the flume velocity was increased to 0.6 m/s the “Bucatanna27.0_1” sample scoured drastically, as shown in Figure 4-3.

From the results of “Bucatanna 27.0_1” it was determined that the Bucatanna Clay formation did not scour at 0.3 m/s, the threshold velocity was between 0.3 and 0.6m/s, and that further scour testing should start at a flume velocity of 0.6 m/s.

A total of five EFA tests were performed on the Bucatanna Clay formation at a flume velocity of 0.6 m/s. Table 4-1 below shows the test results obtained by visual observation along with the ultrasonic sensor.

Table 4-1: Bucatanna Clay Results at 0.6 m/s

Sample:	Elapsed Time Ultrasonic (min)	Elapsed Time Visual (min)	Scour Rate Ultrasonic (mm/hr)	Scour Rate Visual (mm/hr)
Bucatanna 27.0_3	14	13	4.29	4.62
Bucatanna 27.0_4	17	15	3.53	4.00
Bucatanna 27.0_5	12	13	5.00	4.62
Bucatanna 27.0_6	22	22	2.73	2.73
Bucatanna 27.0_7	15	15	4.00	4.00

As seen in Table 4-1, the elapsed time for erosion as determined by the EFA were similar to the elapsed time observed. The largest difference in the observed and measured elapsed time was two minutes as seen in test “Bucatanna 27.0_4”. The scour rates for the five tests were similar with “Bucatanna 27.0_5” having a slightly higher scour rate of 5.0 mm/hour and “Bucatanna 27.0_6” having a slightly lower scour rate. The average scour rate at 0.6 m/s as calculated by the ultrasonic sensor was 3.91 mm/hr.

A total of 7 EFA tests were performed at a flume velocity of 1.0 meters per second. Below, Table 4-2 shows the results obtained for the tests performed at 1.0 m/s. The elapsed time and subsequent scour rates observed at 1.0 m/s were more variable than those observed at 0.6 meters per second. Seven tests were performed as two tests “Bucatanna 27.5_3” and “Bucatanna 27.5_4” were uncharacteristic of the rest of the data set with elapsed times measured at 7 and 38 minutes respectively. As seen in Table 4-2, the “Bucatanna 27.5_3” test was unable to be

visually observed, but visual observations were once again relatively close to ultrasonic sensor calculations. It was also observed that the “Bucatanna 27.5_3” test appeared to be softer than insitu conditions prior to testing, explaining the high scour rate. The average erosion rate for the 7 EFA tests performed on the Bucatanna Clay formation at a flume velocity of 1.0 m/s was 4.07 mm/hr.

Table 4-2: Bucatanna Clay Results at 1.0 m/s

Sample:	Elapsed Time Ultrasonic (min)	Elapsed Time Visual (min)	Scour Rate Ultrasonic (mm/hr)	Scour Rate Visual (mm/hr)
Bucatanna 27.5_2	13	14	4.62	4.29
Bucatanna27.5_3	7	N/A	8.57	N/A
Bucatanna27.5_4	38	24	1.58	2.50
Bucatanna27.5_5	15	16	4.00	3.75
Bucatanna27.5_6	17	17	3.53	3.53
Bucatanna27.5_7	18	18	3.33	3.33
Bucatanna27.5_8	21	17	2.86	3.53

Five tests were performed at a flume velocity of 1.5 m/s, and the results can be viewed in Table 4-3. As expected the recorded scour rates increased compared to those recorded at lower velocities, with an average scour rate of 5.59 mm/hr. The data set acquired from this velocity had a low variability with the exception on the “Bucatanna 26.5_3” test that had a higher scour rate. It was encouraging that this data set was relatively close with respect to scour rates as it was the first data set tested using two different sample depths. It was noted during testing that a sand seam was located approximately 50 mm above the “Bucatanna 26.5_3” test sample. However, subsequent tests using this sample showed the same scour characteristics as previous test samples.

Table 4-3: Bucatunna Clay Results at 1.5 m/s

Sample:	Elapsed Time Ultrasonic (min)	Elapsed Time Visual (min)	Scour Rate Ultrasonic (mm/hr)	Scour Rate Visual (mm/hr)
Bucatanunna 27.5_9	14	11	4.29	5.45
Bucatanunna 26.5_2	12	11	5.00	5.45
Bucatanunna26.5_3	7	8	8.57	7.50
Bucatanunna26.5_4	13	14	4.62	4.29
Bucatanunna26.5_5	11	11	5.45	5.45

Five EFA tests were performed on the Bucatunna Clay formation at a flume velocity of 2.0 m/s. The results from these tests can be viewed below in Table 4-4. The test titled “Bucatanunna 26.5_7” was the lone outlier in this data set with an elapsed time of 22 minutes. However, there were not any oddities or special notes recorded with regards to this test, it just seemed as though this test was more scour resistant than other tests at this velocity. Overall the average scour rate at a flume velocity of 2.0 m/s was 6.67 mm/hr.

Table 4-4: Bucatunna Clay Results at 2.0 m/s

Sample:	Elapsed Time Ultrasonic (min)	Elapsed Time Visual (min)	Scour Rate Ultrasonic (mm/hr)	Scour Rate Visual (mm/hr)
Bucatanunna 26.5_6	10	10	6.00	6.00
Bucatanunna26.5_7	22	20	2.73	3.00
Bucatanunna26.5_8	10	10	6.00	6.00
Bucatanunna26.5_9	7	8	8.57	7.50
Bucatanunna26.5_10	6	8	10.00	7.50

The final velocity used in EFA testing of the Bucatunna Clay formation was at 3.0 m/s. Five EFA tests were performed at this velocity and two separate samples were used, similar to the tests performed at 1.5 m/s. The results from the tests performed at 3.0 m/s can be viewed below in Table 4-5. At this high velocity, erosion occurred rather quickly averaging less than 6 minutes per test. Since the time for erosion was so brief, instead of advancing the sample after five minutes, the sample was advanced into the flume after only being exposed to flow for two minutes. This minimized the possibility of erosion prior to advancing the sample. The average

scour rate at 3.0 m/s was 11.01 mm/hr, a jump of 5.0 mm/hr above the average scour rate at a flume velocity of 2.0 m/s.

Table 4-5: Bucatunna Clay Results at 3.0 m/s

Sample:	Elapsed Time Ultrasonic (min)	Elapsed Time Visual (min)	Scour Rate Ultrasonic (mm/hr)	Scour Rate Visual (mm/hr)
Bucatanunna 26.5_11	8	10	7.50	6.00
Bucatanunna26.5_12	7	8	8.57	7.50
Bucatanunna 23.0_1	5	8	12.00	7.50
Bucatanunna 23.0_2	5	5	12.00	12.00
Bucatanunna 23.0_3	4	5	15.00	12.00

The final test performed on the Bucatunna Clay formation was aimed at determining the critical shear velocity of the formation. Since previous tests on the Bucatunna Clay formation was scour resistant, and actually swelled, at 0.3 meters per second and scoured at 0.6 m/s, the critical shear velocity was located between the two velocities. The test titled “Bucatanunna 23.0_4” was started with a flume velocity of 0.3 m/s and gradually increased until scour was observed. The flume velocity was held between 0.35 and 0.4 m/s for approximately 15 minutes, and again swelling was observed instead of scour. Slowly the flume velocity was increased between 0.45 and 0.5 m/s. At this velocity a large chunk eroded from the sample. Figure 4-4 below shows the sample immediately after erosion occurred. This image agrees with the ultrasonic sensor, while portions of the sample are located above the base of the flume, the majority of the sample has eroded in deep pockets below the flume. From this test it was determined that the threshold velocity was 0.45 m/s.



Figure 4-4. “Bucatumna 23.0_4” After Erosion Occurred

As previously stated a total of 27 EFA tests were performed on the Bucatumna Clay formation at five different velocities. Figure 4-5 below shows the all of the results for the EFA test results with respect to scour rate and flume velocity. For the most part the data sets are tightly bunched with similar scour rates at each velocity interval. Figure 4-6 below shows the drawn-in velocity versus scour rate curve for the Bucatumna Clay formation as derived from the mean scour rate at each velocity.

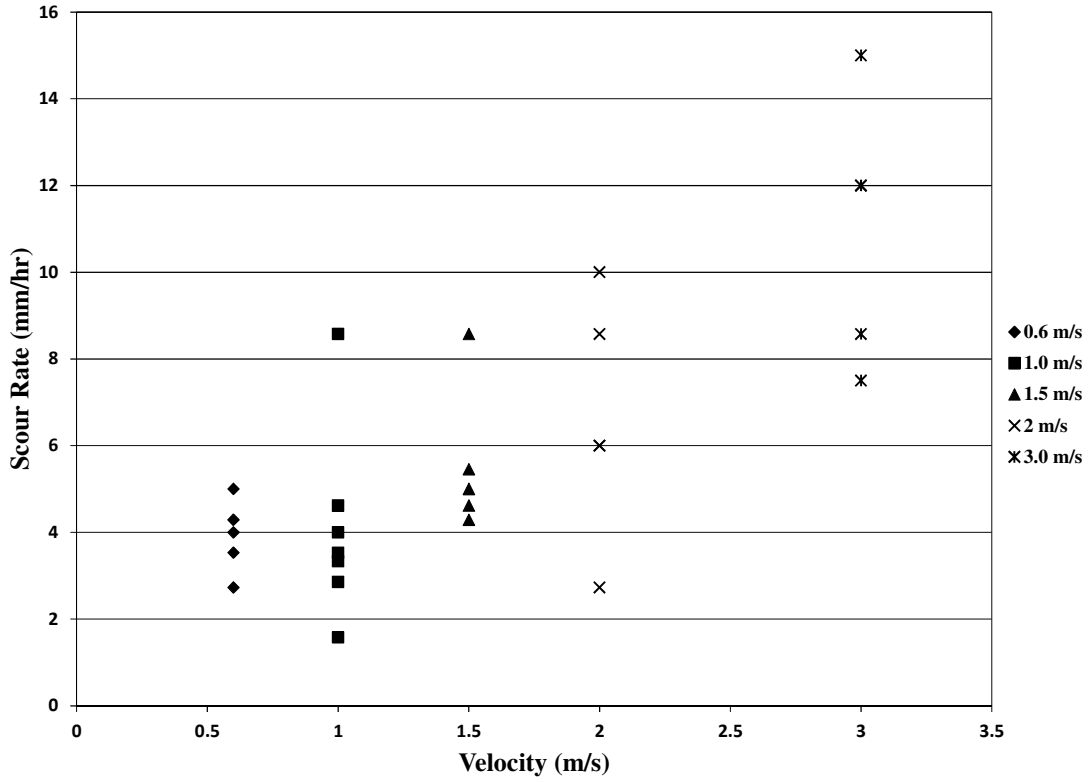


Figure 4-5. EFA Tests Results, Bucatunna Clay

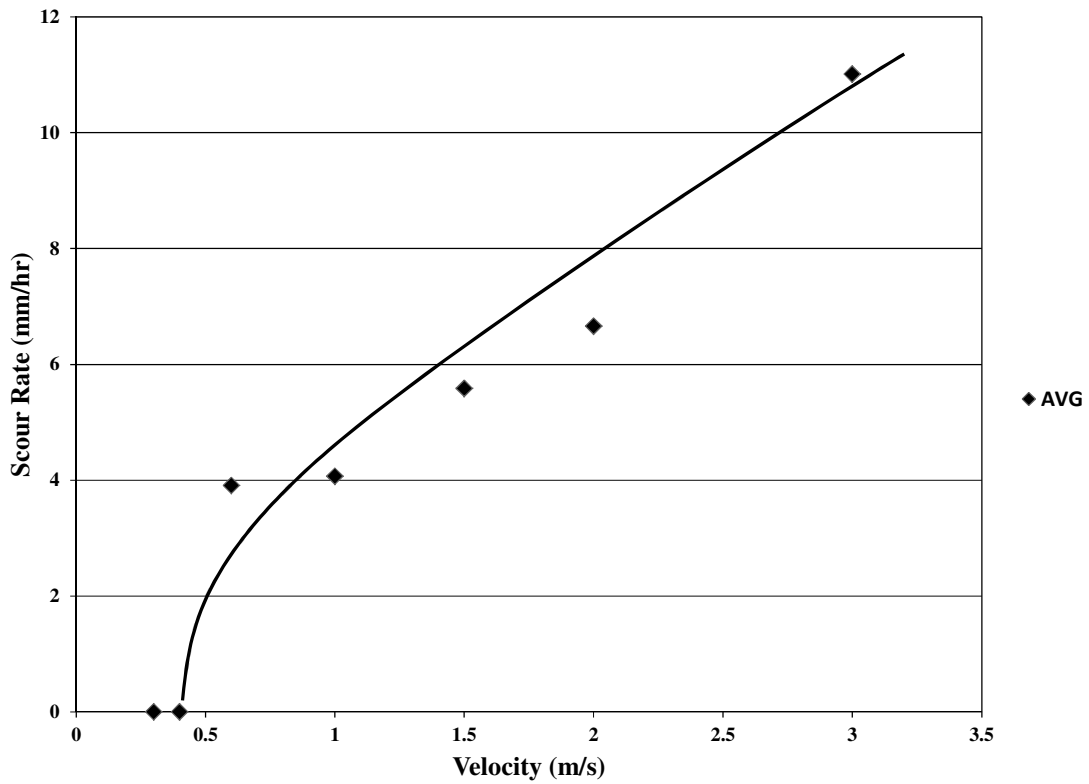


Figure 4-6. Bucatunna Clay Velocity versus Scour Rate

4.2.3 Geotechnical Testing

As part of the scope of this research several common geotechnical index tests were performed on the Bucatunna Clay formation. Two initial moisture contents were taken to obtain the insitu moisture content of the formation prior to EFA testing. These tests yielded moisture contents of 49.5 and 46.2 percent, with an average of 47.9 percent. Using ASTM D421, several kilograms of the soil obtained during sampling was processed and oven dried. Using this processed soil, a full grain size distribution was performed on the formation following ASTM D422. The results of the grain size analysis are shown in Figure 4-7. Since this research revolved around fine-grained materials, a coarse sieve analysis was not performed as the maximum particle size diameter for this formation was approximately 0.43 mm. From the grain size analysis the mean particle diameter (D_{50}) was calculated to be 0.033 mm. Approximately 65 percent of the sample passed the number 200 sieve, classifying the formation in the silt and clay family.

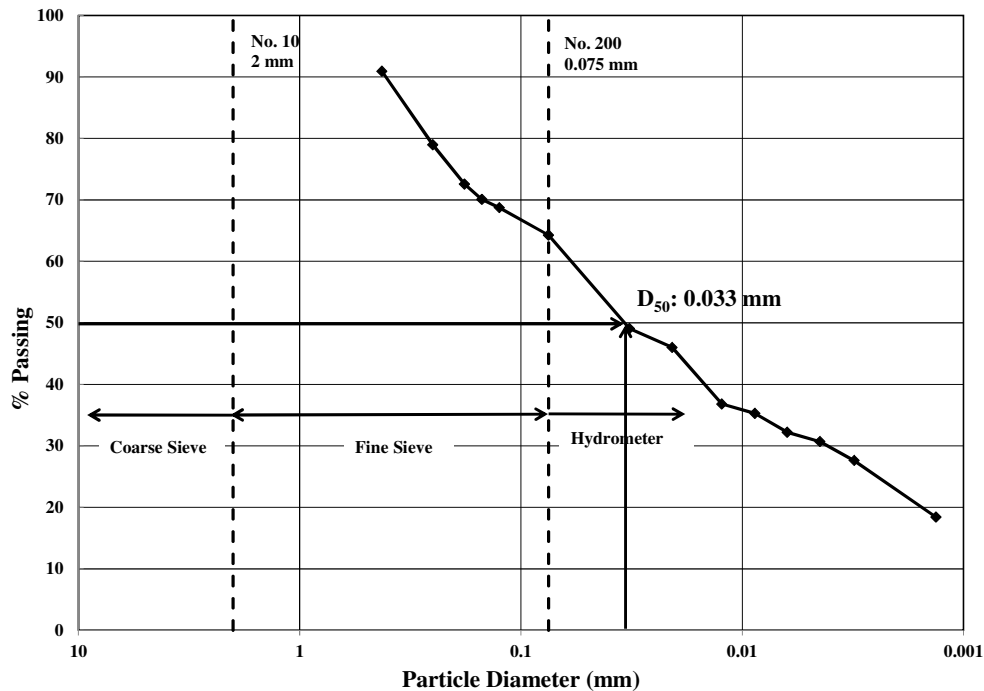


Figure 4-7. Bucatunna Clay Grain Size Distribution

Atterberg limits were performed to complete the soil classification and geotechnical testing of the Bucatunna Clay formation. The procedure presented by ASTM D318 was followed when performing these Atterberg limits. The liquid limit (LL) was determined to be approximately 68, while the plastic limit (PL) was found to be 39. Using the Atterberg limits, the plasticity index was 29. Using these values along with the grain size distribution in Figure 4-7, the Bucatunna Clay formation was classified as a sandy elastic silt (USCS Classification MH). The high plasticity index of 29 is greater than a plasticity index of 10, which is commonly used as the threshold to classify a clay with swelling potential. This explains the swelling phenomenon observed in a few tests at velocities below the critical shear velocity.

As previously stated, the recovered sample was not conducive to being used in an unconfined compression test as the sample was cracked at intervals tighter than the minimum test length of 115 mm. Individual EFA test and geotechnical test results can be viewed in Appendix A.

4.3 Yazoo Clay

The next formation tested using the updated EFA featuring the ultrasonic sensor was the Yazoo Clay formation located in southern Alabama. The Yazoo Clay formation was sampled using the CME continuous sample system. EFA tests along with geotechnical index tests were also performed on the Yazoo Clay formation.

4.3.1 Sampling

The Yazoo Clay formation was sampled on April 6, 2012 in Conecuh County, Alabama. An ALDOT geologist classified the Yazoo Clay formation as a light colored stiff gray clay. The Yazoo Clay formation was sampled above a box culvert crossing a stream with a visible outcrop of the formation viewed in the streambed. An ALDOT geologist identified the formation in drill

cuttings and split spoon samples at approximately 4.11 m below the ground surface. A Standard Penetration Test was performed at the top of the formation yielding an N value of 15 blows. Three total CME continuous sample runs were performed on the Yazoo Clay formation with varying results.

The first run was taken from 4.27 to 5.79 m below ground surface. The top half was disturbed and unusable for EFA testing due to the Standard Penetration Test which was terminated at 4.6 m. The bottom half of the first run provided several sections that were testable in the EFA. The second continuous sample run was performed between 5.79 and 7.32 m below the ground surface. The top tube of the run had many cracked sections that were not able to be used in the EFA, but could be used for classification and index testing. The bottom tube of the sample run from 6.55 to 7.32 m had several EFA testable sections. The third and final sample run was performed between 7.32 and 8.84 m below ground surface. After this continuous sample was obtained it was a sandy material was noticed at a depth of 8.2 m. The Yazoo clay material obtained in this third run was completely unusable as a transition into the sandy material below was apparent. Due to the change in material, another Standard Penetration Test was not performed after sampling.

4.3.2 EFA Testing

A total of twelve EFA tests were performed on the Yazoo Clay formation. After the samples were brought back to Auburn University and stored for testing, it was determined that much less of the formation was able to be tested in the EFA than originally thought. The best section for EFA testing was approximately one third of a meter long between 5.48 and 5.80 m below the ground surface. It was noted during sample preparation that the Yazoo Clay formation

was much tougher to cut and create a smooth sample surface. It was apparent that the sand content in the Yazoo Clay formation was much higher than the Bucatunna Clay formation.

The first test performed on the Yazoo Clay formation was used to help determine the starting test velocity and was titled “Yazoo Clay 18.5_1”. The “Yazoo Clay 18.5_1” test started with a flume velocity of 0.3 m/s, and scour was not observed. After one hour the flume velocity was increased to 0.45 m/s and immediately a wedge of the sample scoured away. This wedge developed shortly after the test was started and grew in size throughout the test, and image of this wedge can be viewed in Figure 4-8. However, since this method of failure had not yet been observed a second test titled “Yazoo Clay 18.5_2” was performed starting at a flume velocity of 0.45 m/s.



Figure 4-8. “Yazoo Clay 18.5_1” Test Failure Wedge

The “Yazoo Clay 18.5_2” test did not perform similarly to the previous test but exhibited a swelling characteristic similar to that observed in the Bucatunna Clay formation. The test was started at a flume velocity of 0.45 m/s and did not scour, after one hour. The flume velocity was then increased to 0.6 m/s for an hour in which scour was not observed. The flume velocity was then increased temporarily to 0.8 m/s for fifteen minutes before being increased once again to 1.0

m/s. During this time the test sample rose and average of 1.75 mm, and developed a large crack in the center of the sample as shown in Figure 4-9. After the test sample was exposed for 25 minutes at a flume velocity of 1.0 m/s, the sample scoured well below the bottom of flume as shown in Figure 4-10. The test results showing the gradual swell in the “Yazoo Clay 18.5_2” test can be viewed in Figure 4-11. The test was analyzed and it was determined that the testing regiment should be started at a flume velocity of 1.0 m/s.

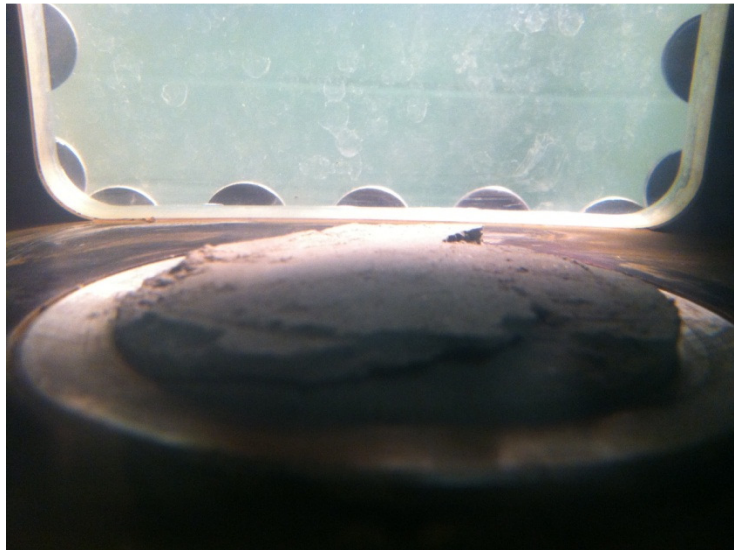


Figure 4-9. “Yazoo Clay 18.5_2” Test Swelling and Cracking



Figure 4-10. “Yazoo Clay 18.5_2” Test After Failure

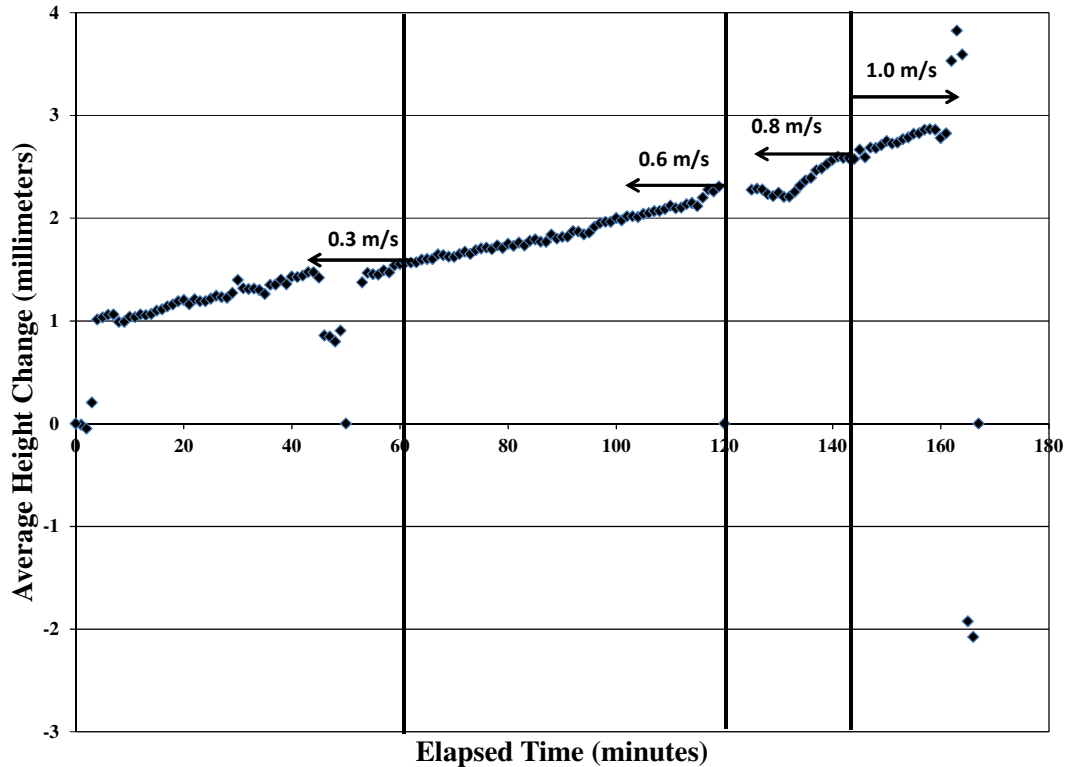


Figure 4-11. “Yazoo Clay 18.5_2” Test Results

Five EFA tests were performed on the Yazoo Clay formation at a flume velocity of 1.0 m/s. The average scour rate among these five tests was 7.33 mm/hr, but included very high variability. Four of the five tests at this velocity resulted in scour times ranging from five to eight minutes. The fifth test, titled “Yazoo Clay 18.5_7”, at this velocity did not scour under a flume velocity of 1.0 m/s after one hour. After an hour the flume velocity was increased to 1.2 m/s. At this velocity thin flakes began to scour immediately after the velocity was increased, but stopped, and the flume velocity was increased to 1.5 m/s. After fifteen minutes at a flume velocity of 1.5 m/s, no major scour was observed and the flume velocity was increased once again to 2.0 m/s. Five minutes later at a flume velocity of 2.0 m/s a large chunk of material scoured from the surface of the test. Once the “Yazoo Clay 18.5_7” test was complete and results analyzed, it was determined that the testing regiment should be continued at a flume

velocity of 1.5 m/s. A summary table of the tests performed at a flume velocity of 1.0 m/s is shown below in Table 4-6.

Table 4-6: Yazoo Clay Results at 1.0 m/s

Sample:	Elapsed Time Ultrasonic (min)	Elapsed Time Visual (min)	Scour Rate Ultrasonic (mm/hr)	Scour Rate Visual (mm/hr)
Yazoo Clay 18.5_3	5	5	12.00	12.00
Yazoo Clay 18.5_4	8	8	7.50	7.50
Yazoo Clay 18.5_5	7	7	8.57	8.57
Yazoo Clay 18.5_6	7	7	8.57	8.57
Yazoo Clay 18.5_7*	N/A	N/A	0.00	0.00
* Did Not Scour				

Two EFA tests were performed at a flume velocity of 1.5 m/s, and the results mirrored the “Yazoo Clay 18.5_7” test results. The first test performed at a flume velocity of 1.5 m/s showed minimum scour for the first 40 minutes of the test; however a huge chunk immediately scoured away after 43 minutes resulting in a scour rate of 1.40 mm/hr. The second EFA test performed at a flume velocity of 1.5 meters per second had varying results mostly due to connection issues with the ultrasonic sensor. Once the second test, “Yazoo Clay 18.5_9”, was started approximately one third of the sample scoured on the back half of the sample. This scour was not noticed much ultrasonically, due to high peaks in the scour surface being located directly under the transducers. Transducers 9 and 10 which were located directly above the location the scour occurred continued to record readings on par with the rest of the transducers. Transducers 11 and 12 showed very erratic readings during a fifteen minute stretch of the test. These erratic readings were most likely due to connection issues with the ultrasonic sensor and were rarely noted during testing. After one hour the “Yazoo Clay 18.5_7” test showed very little signs of scour other than the portion that eroded immediately. Images of this scoured section can be viewed below in Figure 4-12 and Figure 4-13. It is very plausible that the high readings

observed in transducers 9 and 10 are directly associated with high peaks and steep valleys in the scoured area.

Since the two samples that were tested at a flume velocity of 1.5 m/s showed smaller amounts of scour than the tests at 1.0 m/s, the testing schedule was continued at a flume velocity of 2.0 m/s. Overall the average scour rate of the two tests performed at a flume velocity of 1.5 m/s was 0.70 mm/hr. A summary of these two tests is shown in Table 4-7.



Figure 4-12. “Yazoo Clay 18.5_9” Test Scoured Area from Front

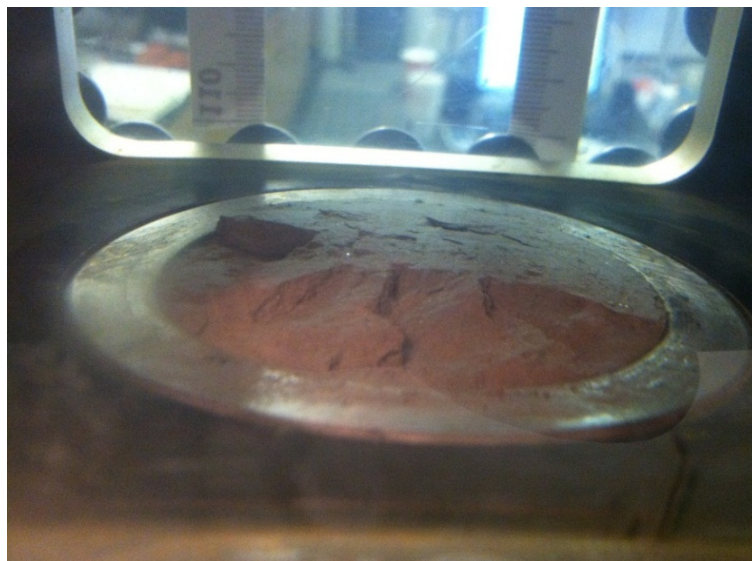


Figure 4-13. “Yazoo Clay 18.5_9” Test Scoured Area from Behind

Table 4-7: Yazoo Clay Results at 1.5 m/s

Sample:	Elapsed Time Ultrasonic (min)	Elapsed Time Visual (min)	Scour Rate Ultrasonic (mm/hr)	Scour Rate Visual (mm/hr)
Yazoo Clay 18.5_8	43	43	1.40	1.40
Yazoo Clay 18.5_9*	N/A	N/A	0.00	0.00
* Did Not Scour				

A total of three tests were performed on the Yazoo Clay formation at a flume velocity of 2.0 m/s. The first test, titled “Yazoo Clay 18.5_10” scoured had a resulting scour rate of 4.29 mm/hr. However, the test did not scour at a constant rate or even in small chunks as observed in previous Yazoo Clay EFA tests. The test showed minimum surface scouring for the first thirteen minutes of the test until a very large chunk scoured away. Using the ultrasonic sensor this chunk encompassed the entire sample and was approximately ten millimeters deep into the flume. Two additional tests were performed at a flume velocity of 2.0 m/s and both showed similar magnitudes of scour as the “Yazoo clay 18.5_10” test immediately after the pump was started. Table 4-8 below shows the three test results at a flume velocity of 2.0 m/s.

Table 4-8: Yazoo Clay Results at 2.0 m/s

Sample:	Elapsed Time Ultrasonic (min)	Elapsed Time Visual (min)	Scour Rate Ultrasonic (mm/hr)	Scour Rate Visual (mm/hr)
Yazoo Clay 18.5_10	14	11	4.29	5.45
Yazoo Clay 18.0_1**	0	0	N/A	N/A
Yazoo Clay 18.0_2**	0	0	N/A	N/A
**Complete Failure Prior to Push				

The sporadic scour behavior of the Yazoo Clay formation lead to the pausing of the EFA testing schedule as most of the tests yielded contradictory scour results. The only consistent results obtained from this formation were at a flume velocity of 1.0 m/s, which yielded an average scour rate of 7.33 mm/hr. The test performed at both 1.5 and 2.0 m/s were highly

variable and inconsistent yielding no consistent results. Individual test results of the Yazoo Clay formation can be viewed in Appendix B.

4.3.3 Geotechnical Testing

Two initial moisture contents were taken to obtain the insitu moisture content of the formation prior to EFA testing. These tests yielded moisture contents of 62.2 and 57.6 percent, with an average of 59.9 percent. Using soil processed by following ASTM D421, a full grain size distribution was performed on the formation following ASTM D422. The results of the grain size analysis are shown below in Figure 4-14. Similar to the Bucatunna Clay formation one hundred percent of the formation passed through the number 10 sieve, thus negating the need for a coarse sieve analysis. From the grain size analysis the mean particle diameter was calculated as 0.088 mm. Approximately 44 percent of the sample tested passed the number 200 sieve classifying the sample as a sand.

Atterberg limits were performed to complete the soil classification and geotechnical testing of the Yazoo Clay formation. The liquid limit was determined to be approximately 57, and the formation was determined to be non-plastic. Using these values along with the information gathered from the grain size distribution, the sample was classified as a non-plastic silty sand (USCS Classification SM).

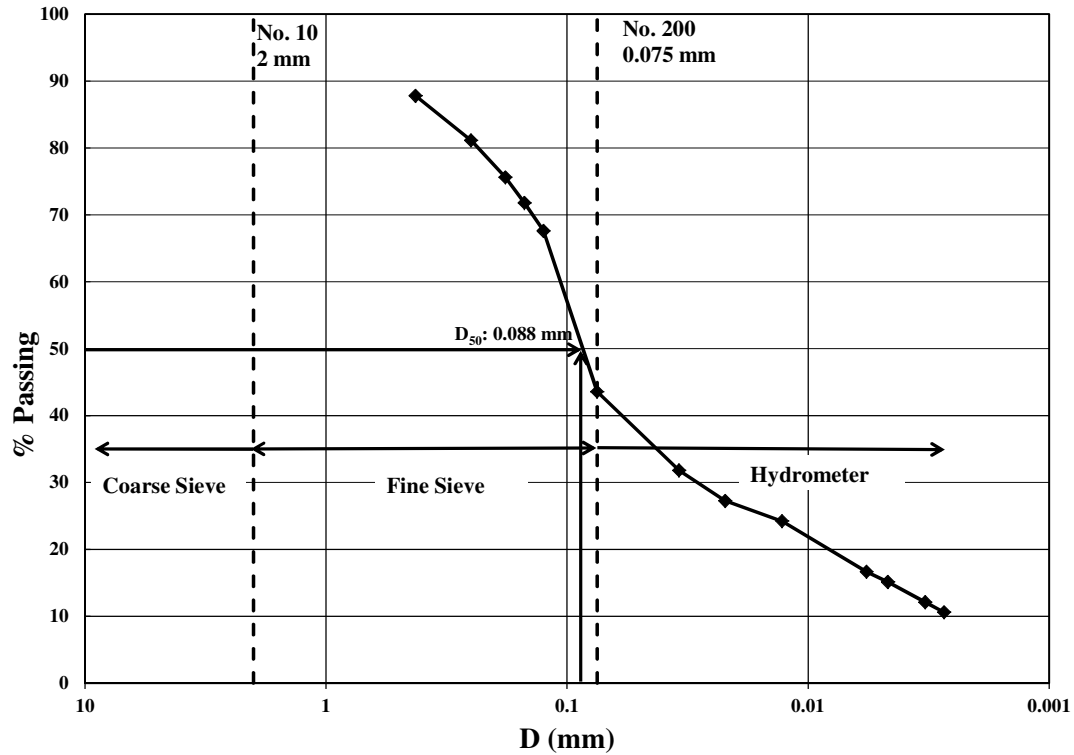


Figure 4-14. Yazoo Clay Formation Grain Size Distribution

Due to the results of the geotechnical index tests along with the sporadic EFA test data, it was apparent that the sample acquired to represent the Yazoo Clay formation, was not purely Yazoo Clay, but had a high sand content. This sand content was not evenly distributed throughout the sample as several EFA test results show potential for scour resistance. Likewise, a high percentage of the sample was retained on the number 200 sieve and appeared to be a very fine silty sand. Since this layer of clay was sampled between two sandy layers it appears the sample tested was not a pure sample of the Yazoo Clay formation.

4.4 Demopolis Chalk

4.4.1 Sampling

The Demopolis Chalk formation was sampled on May 5, 2012 in Sumter County, Alabama. An ALDOT geologist classified the Demopolis Chalk formation as a very stiff light gray chalk. The Demopolis Chalk formation was sampled beside a bridge crossing the

Tombigbee River. An outcrop of the formation could be viewed in the bluffs comprising the river bed. These bluffs were approximately 15 m above the river, entirely comprised of the Demopolis Chalk formation. These bluffs can be viewed below in Figure 4-15.



Figure 4-15. Demopolis Clay Formation Sampling Location

An ALDOT geologist identified the formation in drill cuttings and split spoon samples at approximately 4.27 m below the ground surface. A Standard Penetration Test was performed at the top of the formation yielding a high N value of 92 blows. The ALDOT drill crew performed two CME continuous sample runs on the Demopolis Chalk formation.

The first run was taken from 4.57 to 6.10 m below ground surface. The top half was disturbed and unusable for EFA testing due to the Standard Penetration Test which was terminated at approximately 4.65 m. An EFA testable section was between 5.8 and 6.1 m below ground surface. The middle portion of this top run was cracked in both the vertical and radial directions.

The second continuous sample run was performed from 6.1 to 7.62 m below the ground surface. The top and middle portions of this sample were severely cracked in both the vertical and radial direction. The bottom third of the sample was uncracked; however, during sampling the sample tube was bowed and stretched. It was noted during sampling that the stiffness of the Demopolis Chalk formation made advancing the sample tubes increasingly difficult with depth. The torque necessary to advance the continuous sampler in the formation was great enough to distort the continuous sample tube and increase the outside diameter from 64 mm to approximately 74 mm. This increase in diameter of the sample tubes made this section of the Demopolis Chalk formation unable to be tested and advanced in the EFA. Due to the difficulty in sampling the stiff Demopolis Chalk formation only two continuous sample runs were performed, resulting in approximately 30 cm of EFA testable sample.

4.4.2 EFA Testing

A total of five Erosion Function Apparatus tests were performed on the Demopolis Chalk formation. The Demopolis Chalk Formation was unable to be automatically advanced into the EFA during testing due to the stiffness of the sample. Therefore, prior to each EFA test the sample was manually advanced approximately one millimeter using a hydraulic extruder, and then placed into the EFA at the testing height. The height of the test specimen protruding from the flume could be calculated by subtracting the known flume height from the corrected distances measured by the ultrasonic sensor. This changed the shape of the erosion rate versus elapsed time figures generated by the ultrasonic sensor and data acquisition system. Instead of a figure showing a starting location at 0, a push advancing the sample approximately one millimeter, and subsequent scour movement, the figure showed a starting location at 0 and subsequent scour movement from that datum.

The first test performed on the Demopolis Chalk formation was titled “DemopolisChalk19.0_1”, and was started at a flume velocity of 1.2 m/s. Using scales attached to the outside of the EFA viewing window it appeared that the sample protruded from the flume approximately one millimeter. After 40 minutes of not witnessing any visually or by ultrasonic readings, the flume velocity was increased to 2.0 m/s. Another 20 minutes passed without any evidence of scour, and the velocity was increased to the maximum test velocity of 3.0 m/s. The test was allowed to run for one hour at the maximum velocity and no scour was witnessed. It was determined that three more tests should be performed at 3.0 m/s to determine if the Demopolis Chalk formation was scour resistant according to the parameters of this study.

The three tests mentioned above were titled “DemopolisChalk19.0_2”, “DemopolisChalk19.0_3”, and “DemopolisChalk19.0_4”. These three tests averaged a protruded distance of 1.13 mm above the flume with values of 0.3, 1.5, and 1.6 mm respectively. All three of these tests were performed for at least one hour, and all three did not show any signs of major scour. The only scour witnessed was a very minor rounding of the sharp edges of the test samples. It is believed that this is due to stress concentrations that are not observed in the field with a continuous stream bed. This slight rounding of edges was visually observed and was not significant enough to be recorded by the ultrasonic sensor. The “DemopolisChalk19.0_3” and “DemopolisChalk19.0_4” tests both showed a slight raise in height in the ultrasonic output, averaging 0.2 mm over the duration of the tests, hinting that some minor swelling behavior is possible in the formation.

The final test performed on the Demopolis Chalk formation was titled “DemopolisChalk19.0_5”. This test was intended to mirror recurring storm events on a small scale. The first hour of the test the flume velocity was 3.0 meters per second, dropping to 1.0

m/s for the next 30 minutes, and then increased to 3.0 m/s for another hour. It was calculated that the test specimen started at an average height of 1.7 mm above the base of the flume. No major scour was observed during the 2.5 hour test. Again only a slight rounding off of the front edge of the sample was noticed as viewed below in Figure 4-16. In summary a total of five tests were performed on the Demopolis Chalk formation and all five did not show any signs of measurable scour.

The results of all five tests may be viewed in Appendix C. This was the first sample tested in this study that did not exhibit any scour behavior with the set testing regiment. In case future testing of the Demopolis Chalk formation is necessary, the remaining testable section of the formation was sealed and placed into a curing chamber to preserve field moisture conditions.



Figure 4-16. “DemopolisChalk19.0_5” After Testing

4.4.3 Geotechnical Testing

Three initial moisture contents were taken to obtain the insitu moisture content of the Demopolis Chalk formation prior to EFA testing. These tests yielded moisture contents of 22.5, 21.5, and 21.4 percent, with an average of 21.8 percent. Using soil processed by following ASTM D421, a full grain size distribution was performed on the formation following ASTM D422. The results of the grain size analysis can be viewed below in Figure 4-17. The grain size

analysis on the Demopolis Chalk showed that the formation consisted of very fine-grained particles with approximately 97 percent of the sample passing the number 200 sieve. From the grain size analysis the mean particle diameter was calculated to be 0.021 mm.

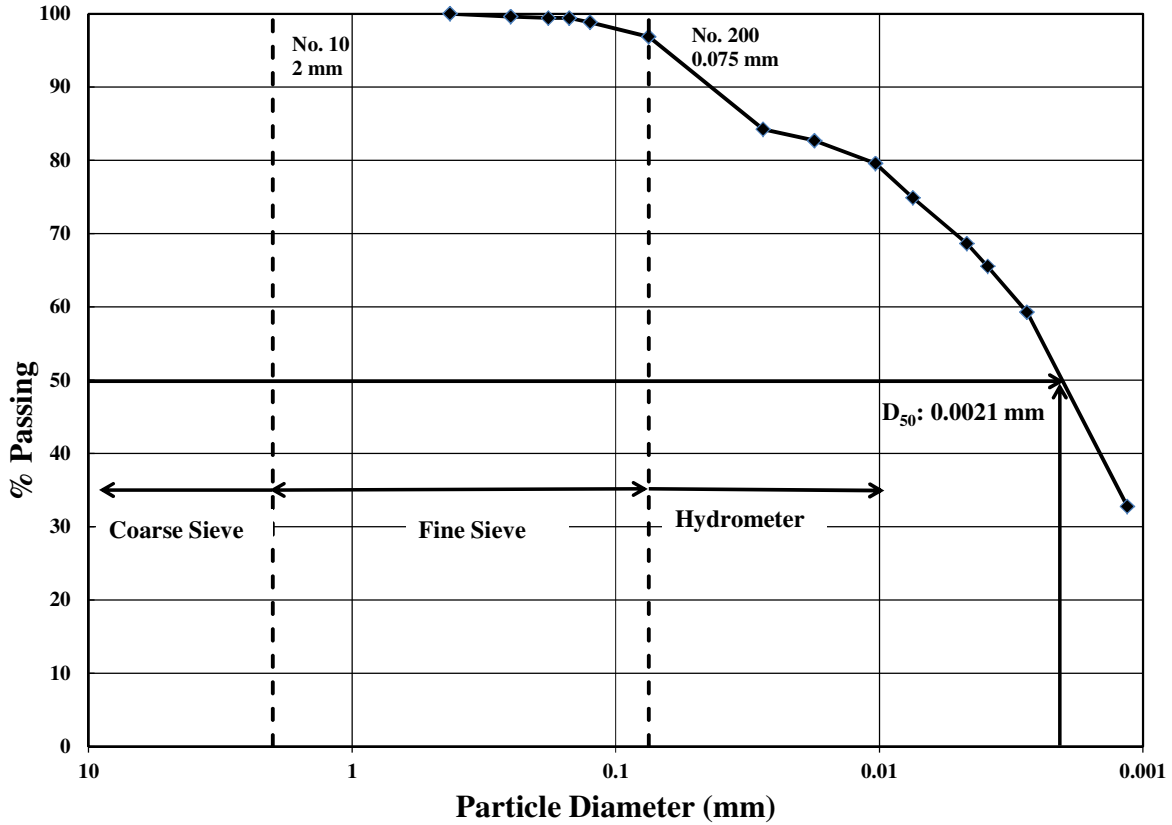


Figure 4-17. Demopolis Chalk Formation Grain Size Distribution

Atterberg limits were performed to complete the soil classification and geotechnical testing of the Demopolis Chalk formation. The liquid limit was determined to be approximately 37, and the plastic limit was determined to be approximately 27, resulting in a plasticity index of 10. The calculated plasticity index validated the earlier observation during EFA testing, that this formation has minor swelling potential. Using these values along with the information gathered from the grain size distribution, the sample was classified as a lean clay (USCS Classification CL).

4.5 Mooreville Chalk

4.5.1 Sampling

The Mooreville Chalk formation was sampled on April 30, 2012 in Dallas County just west of Selma, Alabama. An ALDOT geologist classified the Mooreville Chalk formation as a stiff gray chalk. The ALDOT geologist identified the formation in drill cuttings and split spoon samples at approximately 5 m below the ground surface. A Standard Penetration Test was performed at the top of the formation yielding an N value of 60 blows. The ALDOT drill crew performed three CME continuous sample runs on the Mooreville Chalk formation.

The sampling of the Mooreville Chalk formation occurred in two drill holes, the Standard Penetration Test and first sample run were performed in the first drill hole. The Standard Penetration Test was taken from 6.1 to 6.55 m. The CME continuous sample was taken from 6.85 to 8.38 m below the ground surface. During sampling it was determined that a plug had developed in the drill hole and the continuous sampler had to be retracted. After the sample tubes were recovered, the sample tube from 7.62 to 8.38 m showed approximately 30 cm of testable section. However, the sample tubes were extremely bowed approaching an outside diameter of 75 mm.

The second drill hole was located approximately 3 meters from the first, and the formation was encountered at a similar depth. Two continuous sample runs were performed in this hole. The first run was taken from 5.33 to 6.85 m below the ground surface. A small testable section, approximately 15 cm long, was located in the top half of the sample. Another testable section with a similar length was located at the bottom of the sample run. The rest of the run was severely cracked in both the vertical and radial directions. The second CME continuous sample run was performed from 6.85 to 8.38 m below the ground surface. The top half of the

sample run was severely cracked and unusable for EFA testing. A testable section with a length of approximately 30 cm was located at the base of the sample run. However, 20 cm of this section was bowed out similar to the sample tube in the first sample hole. The Mooreville Chalk formation similar to the Demopolis Chalk formation proved difficult to sample with the CME continuous sampling system, but sufficient testable sections were recovered.

4.5.2 EFA Testing

Ten Erosion Function Apparatus tests were performed on the Mooreville Chalk formation resulting in varying results. The section used to test the Mooreville Chalk was from the second drill hole mentioned above at a depth of 6.7 m. This section was the testable section with the least bowed sample tube. The sample tube was cut according the methods provided in Chapter 3, but also had to be sanded in order to fit into the flume of the EFA. Once the sample tube was altered to fit into the base of the flume it was determined that the automatic advancing motor on the EFA could not advance the sample. The sample was manually advanced in a similar manner to that of the Demopolis Chalk formation.

The first test on the Mooreville Chalk formation was performed to determine the base velocity for testing the formation. The test titled “MoorevilleChalk22.0_2” was started at 1.0 m/s and quickly showed measurable scour. It was determined that this test was not indicative of the scour parameters of the formation as it was calculated that the sample protruded from the base of the flume two millimeters which was double the distance specified by Briaud. The next test performed on the Mooreville Chalk formation was titled “MoorevilleChalk22.0_3”, was started at a flume velocity of 0.3 m/s, and was able to be automatically advanced into the EFA flume. This test showed no signs of scour at 0.3 m/s. Some large masses of material scoured in chunks at 0.6 m/s, and scour continued once the flume velocity was increased to 1.0 m/s. From

this test it was determined that testing should start at 0.6 m/s. It was noted that the sample section used to obtain the results from these two tests could have been exposed to varying stress states, not primarily associated to EFA testing. It appeared that the test sample could have been exposed to differing shear and torsional stress states, by being manually advanced with an extruder through a bowed sample tube.

Two EFA tests were performed at a flume velocity of 0.6 m/s. The first titled “MoorevilleChalk22.0_4” resulted in a scour rate of 1.4 mm/hr. However, all the scour recorded was measured from two large mass scours in which a large chunk of material accounted for the measured scour. The second test performed at a flume rate of 0.6 m/s did not show any signs of scour visually or ultrasonically. These two tests did not demonstrate the behavior as the previous two with regards to varying stress states, as both were easily automatically advanced. An additional test was performed at a flume velocity of 1.0 m/s on the Mooreville Chalk formation and no measurable scour was recorded.

Two EFA tests were performed at a flume rate of 1.5 m/s on the Mooreville Chalk formation. The first of these two tests was titled “MoorevilleChalk22.0_7” and resulted in one third of the volume of the protrusion being scoured away by one large chunk in the upstream portion of the sample. The final result of this test can be viewed below in Figure 4-18. The second test performed at a flume velocity of 1.5 m/s resulted in no measurable amount of scour. The only scour was that occurred during this test was witnessed visually and consisted of the rounding off of the front edges of the sample similar to that witnessed in testing the Demopolis Chalk formation.



Figure 4-18. “MoorevilleChalk22.0_7” After Testing

Since there was a limited amount of testable sample obtained from the Mooreville Chalk formation it was imperative that testing continued regardless of slightly varying results. The next EFA test performed on the Mooreville Chalk formation was performed at a flume velocity of 2.0 m/s and no scour was recorded. The flume velocity was increased to 3.0 m/s for the next EFA test and no scour was recorded. The final test performed on the Mooreville Chalk formation was titled “MoorevilleChalk22.0_11” and was a repeat even test similar to that performed on the Demopolis Chalk formation. This test was intended to be performed with a flume velocity of 3.0 m/s for one hour, 1.0 m/s for thirty minutes, and 3.0 meters per second for another hour. This test showed no signs of scour through the first hour and a half of the test. However, soon after the velocity was increased from 1.0 to 3.0 m/s two masses of material scoured from the sample. The smaller of the two masses came from the far side portion of the sample, while the larger mass scoured from the near side of the sample. After these two masses scoured away no other scour was noticed or measured throughout the duration of the test. The

volume of the scour measured from these two masses was approximately one third of the total protrusion.

In summary, the behavior of the Mooreville Chalk formation varied, but it was recognized that the formation could be scour resistant. Testing was limited to the amount of testable sections recovered during sampling. Once the sample was able to be automatically advanced in the EFA, the formation showed minimal scour rates with no tests recording scour rates greater than 1.0 mm/hr. Finally, it was determined that the Mooreville Chalk formation did not scour uniformly in a particle by particle or even flake by flake fashion as observed in sands and earlier tested clays. The scour observed in the Mooreville Chalk formation consisted of large mass chunks of the material scouring at once. All EFA test data for the Mooreville Chalk formation can be viewed in Appendix D.

4.5.3 Geotechnical Testing

Two initial moisture contents were taken to obtain the insitu moisture content of the Mooreville Chalk formation. These tests yielded moisture contents of 21.9 and 24.9 percent, with an average of 23.4 percent. Using soil processed by following ASTM D421, a full grain size distribution was performed on the formation following ASTM D422. The results of the grain size analysis can be viewed below in Figure 4-19. The grain size analysis on the Mooreville Chalk showed that the formation consisted of very fine-grained particles with approximately 92 percent of the sample passing the number 200 sieve. From the grain size analysis the mean particle diameter (D_{50}) was calculated to be 0.024 mm. The mean particle diameter and percent of the Mooreville Chalk sample passing the number 200 sieve mirrored that of the Demopolis Chalk formation. Also after 24 hours in the Hydrometer both formations had at least thirty percent of the sample still in solution.

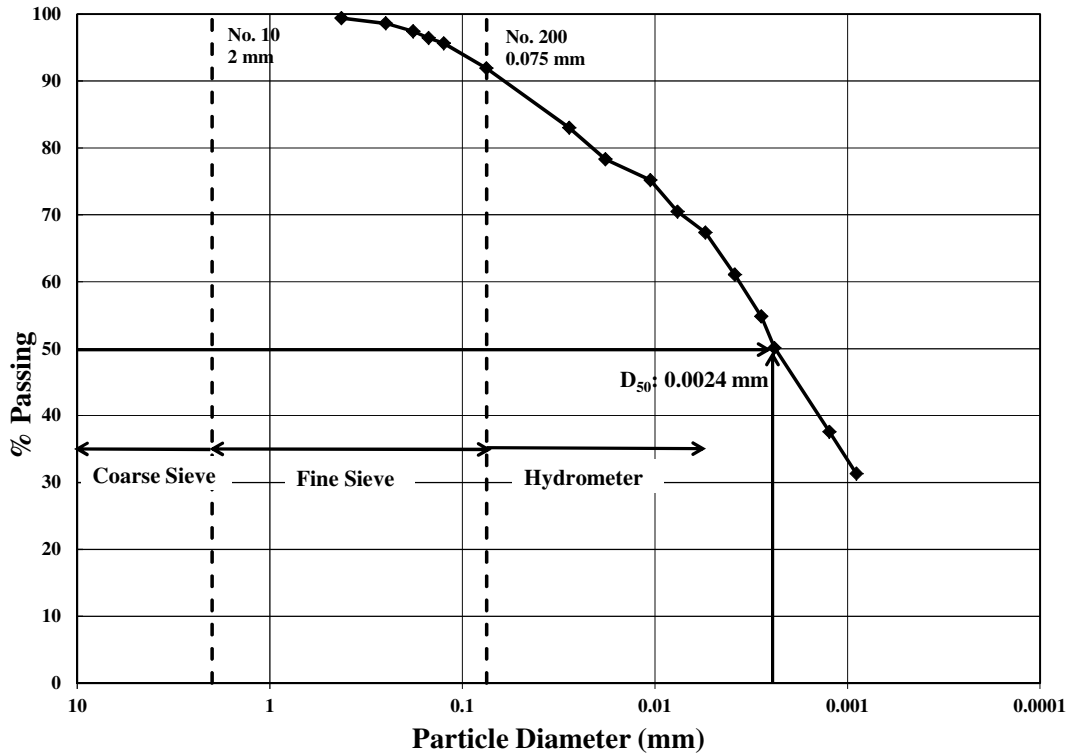


Figure 4-19. Mooreville Chalk Formation Grain Size Distribution

Atterberg limits were performed to complete the soil classification and geotechnical testing of the Mooreville Chalk formation. The liquid limit was determined to be approximately 52, and the plastic limit was determined to be approximately 25, resulting in a plasticity index of 27. The calculated plasticity index is similar to that observed in the Bucatunna Clay formation. Using these values along with the information gathered from the grain size distribution, the sample was classified as a fat clay (USCS Classification CH).

4.6 Prairie Bluff Chalk

4.6.1 Sampling

The Prairie Bluff Chalk formation was sampled on May 1, 2012 in Marengo County west of Demopolis, Alabama. An ALDOT geologist classified the Prairie Bluff Chalk formation as a stiff gray chalk. Outcrops of the formation were viewed in a stream bed beside the drilling location. The ALDOT geologist identified the formation in drill cuttings and split spoon samples

at approximately 3.5 m below the ground surface. A Standard Penetration Test was performed at the top of the formation yielding an N value of 86 blows. The ALDOT drill crew performed three CME continuous sample runs on the Prairie Bluff Chalk formation.

The first sample run was taken from 4.57 to 6.10 m below the ground surface. The top half of the sample consisted of a transitioning layer from the Standard Penetration Test. The bottom half of the sample was uncracked and acceptable for EFA testing. The second continuous sample run was taken from 6.10 to 7.62 m below the ground surface. Similar to the previously sample chalk formations the Prairie Bluff Chalk was very stiff and difficult to sample. The second sample run was to be very difficult to advance, and upon recovery it was discovered that the acrylic sample tubes melted during sampling. The sections of the tubes that did not melt were severely cracked and could not be used for EFA testing. The third CME continuous sample run was taken from 7.62 to 9.14 m below the ground surface. This run was also very difficult to advance as the sampler bowed out during drilling and became wedged the hollow stem auger. Once the sample was recovered, it was discovered that most of the sample tube was melted due to a plug that developed at the bottom of the drill hole. Upon recommendation from the ALDOT drill crew, sampling of the Prairie Bluff Chalk was abandoned due to difficult sampling conditions. A total of approximately 30 cm of the Prairie Bluff Chalk formation was collected for EFA testing.

4.6.2 EFA Testing

Seven Erosion Function Apparatus tests were performed on the Prairie Bluff Chalk formation to determine erosion rates. Once the samples were brought back to Auburn University it was discovered that the amount of testable sample was very limited due to bowed sample tubes. A 15 cm section of the sample was cut and prepared for testing. Similar to the Mooreville

Chalk formation, the exterior of the sample tube had to be sanded down to fit into the flume of the EFA. Once the sample was prepared, it was determined that the automatic motor on the EFA could not advance the sample into the flume. Therefore, the sample was manually advanced using an extruder approximately 1 mm and placed into the EFA at the testing height. The height of each protrusion could be calculated by the values provided from the ultrasonic sensor.

The first test on the Prairie Bluff Chalk formation started at a flume velocity of 1.0 m/s. The protrusion into the flume was calculated to be 1.51 mm or fifty percent higher than the height specified by Briaud. After fifty minutes a large mass chunk scoured away, the volume of the scoured section was approximately equal to five times the volume of the protrusion. It was noted during testing that the sample seemed dry and loose in some areas, and that the large scoured volume started from the loose areas of the sample. Another EFA test was performed at a flume velocity of 1.0 m/s, with a total protrusion of 1.85 mm. No scour was observed during this test, suggesting that the flume velocity should be increased for future tests. The idea that the test samples that had altered tubes and been manually advanced were exposed to differing stress states than initially intended during EFA testing was confirmed during testing of the Prairie Bluff Chalk formation. It was thought that these samples were exposed to shear or torsional forces during extruding. This idea was reinforced by cracking on the edges of the advanced sample as viewed in Figure 4-20 and Figure 4-21. These shear and torsional forces could have been developed through an eccentricity between the bowed sample tube and the extruder advancing the sample. It is unknown if these forces negatively influenced EFA testing. It was noted that these cracks that developed in the EFA sample created shear planes for scouring to occur that would not naturally exist in a stream bed.



Figure 4-20. Prairie Bluff Chalk Formation Cracking Example 1

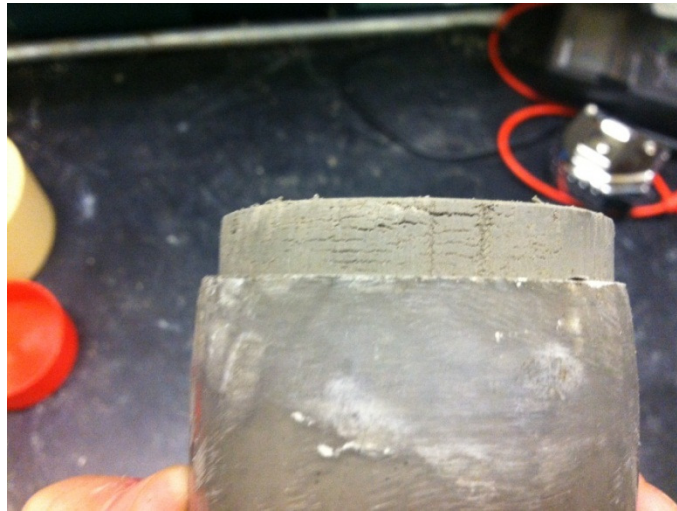


Figure 4-21. Prairie Bluff Chalk Formation Cracking Example 2

An EFA test was performed at a flume velocity of 1.5 meters per second, and a protrusion into the flume of 0.8 mm. This test showed one millimeter of erosion after thirty-four minutes resulting in a scour rate of 1.76 mm/hr. The scour started from dry loose areas noticed in the middle of the sample during sample preparation. The next EFA test performed on the Prairie Bluff formation was performed at a flume velocity of 2.0 m/s. The total protrusion into the flume from this test was calculated to be 1.33 mm. After eighty minutes no scour was observed visually or ultrasonically.

Three tests were performed at a flume velocity of 3.0 m/s, and all three tests showed no measurable scour. The protrusions from all three of these tests were greater than 1.0 mm and in one case the protrusion was double the specified height. It was noted that these samples did not seem to be as dry and loose as samples noted above. Also the cracks due to manually extruding these samples were smaller in size than those noticed during earlier tests that exhibited scour. Due to the lack of testable sample material, a repeat event test was not performed on the Prairie Bluff Chalk formation. It was also recognized that the samples tested that did not seem to be disturbed during sampling or extruding exhibited a resistance to scour at flume velocities up to 3.0 m/s. All EFA tests performed on the Prairie Bluff Chalk formation can be viewed in Appendix E.

4.6.3 Geotechnical Testing

Three initial moisture contents were taken to obtain the insitu moisture content of the Prairie Bluff Chalk formation. These tests provided moisture contents of 16, 17.1, and 20.1 percent with an average of 17.7 percent. As noted during EFA testing, these moisture contents were approximately five percent less than those observed from the other two chalk formations. Using soil processed by following ASTM D421, a full grain size distribution was performed on the formation following ASTM D422. The results of the grain size analysis can be viewed below in Figure 4-22. The grain size analysis on the Prairie Bluff Chalk showed that the formation consisted of fine-grained particles with approximately 82 percent of the sample passing the number 200 sieve. From the grain size analysis the mean particle diameter (D_{50}) was calculated to be 0.028 mm. The mean particle diameter and percent of the Prairie Bluff Chalk sample passing the number 200 sieve were similar to the other two chalk formations tested. The

the top of the formation yielding an N value of 30 blows. The ALDOT drill crew performed three CME continuous sample runs on the Porter's Creek Clay formation.

The first continuous sample run was performed between 3.05 and 4.57 m below the ground surface. Unlike the previous chalk samples, the Porter's Creek Clay did not have any trouble advancing the sampler. Most of the sample run was unusable with cracks in the vertical and radial directions. A testable section was located in the bottom of this run and was approximately 15 cm in length. The second continuous sample run was performed between 4.57 and 6.1 m below the ground surface. A layer of sand was located in the top half of this run and was not used for EFA testing. The bottom half of the run consisted of a 30 cm EFA testable section. It was noticed during sampling that some cracks due to weathering separated the testable sections. The third continuous sample run was performed between 6.1 and 7.62 m below the ground surface. The top half of the sample was determined to be unusable for EFA testing due to heavy cracking and the presence of a wet seam. The bottom half of the sample was uncracked and could be used for EFA testing. The ALDOT geologist also noticed that the testable sections were divided by cracks representing weathered areas.

4.7.2 EFA Testing

A total of 20 Erosion Function Apparatus tests were performed on the Porter's Creek Clay formation. Three different test samples were cut and prepared for EFA testing ranging from 5.7 to 6.1 m below ground surface. The first test performed was started at a flume velocity of 0.3 m/s and did not scour after one hour, and swelling similar to that of the Bucatunna Clay was observed. The flume velocity was increased to 0.6 m/s for the next EFA test and the sample exhibited one millimeter of scour after 15 minutes, resulting in a scour rate of 4.0 mm/hr. Two more EFA tests were performed at a flume velocity of resulting in scour rates of 3.33 and 4.62

mm/hr. The average scour rate for the three EFA tests performed at a flume velocity of 0.6 m/s was 3.98 mm/hr. The results of the EFA tests performed at a flume velocity of 0.6 meters per second are shown in Table 4-9 below.

Table 4-9: Porter’s Creek Clay Results at 0.6 m/s

Sample:	Elapsed Time Ultrasonic (min)	Elapsed Time Visual (min)	Scour Rate Ultrasonic (mm/hr)	Scour Rate Visual (mm/hr)
Porter's Creek Clay 19.5_2	15	15	4.00	4.00
Porter's Creek Clay 19.5_3	18	18	3.33	3.33
Porter's Creek Clay 19.5_4	13	13	4.62	4.62

Three EFA tests were performed on the Porter’s Creek Clay formation at velocity of 1.0 m/s. During these tests, the advancement of the sample occurred one minute after the flume and ultrasonic sensor was started. This time period was typically 3 to 5 minutes, allowing for a clear baseline to be created during testing. However, significant surface roughening had occurred during prior tests before the sample was advanced. The ultrasonic sensor and data reduction software only required one minute to set an adequate base line for readings. The three tests performed at a flume velocity of 1.0 m/s resulted in an average scour rate of 9.17 mm/hr. Table 4-10 below shows the results of these three tests.

Table 4-10: Porter’s Creek Clay Results at 1.0 m/s

Sample:	Elapsed Time Ultrasonic (min)	Elapsed Time Visual (min)	Scour Rate Ultrasonic (mm/hr)	Scour Rate Visual (mm/hr)
Porter's Creek Clay 19.5_5	6	6	10.00	10.00
Porter's Creek Clay 19.5_7	6	6	10.00	10.00
Porter's Creek Clay 19.0_1	8	8	7.50	7.50

The Porter’s Creek Clay formation was tested four times with a flume velocity of 1.5 m/s. However, one of these tests scoured immediately due to a weathered surface. The results of these tests are shown in Table 4-11.

Table 4-11: Porter’s Creek Clay Results at 1.5 m/s

Sample:	Elapsed Time Ultrasonic (min)	Elapsed Time Visual (min)	Scour Rate Ultrasonic (mm/hr)	Scour Rate Visual (mm/hr)
Porter's Creek Clay 19.0_2	5	5	12.00	12.00
Porter's Creek Clay 19.0_3	7	7	8.57	8.57
Porter's Creek Clay 19.0_5	6	6	10.00	10.00
*Porter's Creek Clay 19.0_4 Instant Scour (Weathered Sample)				

The average scour rate of these tests was calculated to be 10.19 mm/hr. The scour observed during these tests mirrored that of the Bucatunna Clay formation and not that of the three chalk formations previously tested. The scour did not typically involve large mass chunks, but small flakes that consistently scoured away the protrusion into the flume over time. This scour did occur rather quickly as elapsed times for scour at a flume velocity of 1.5 m/s was 5, 7, and 6 minutes respectively.

Three EFA tests were also performed at a velocity of 2.0 m/s. The mechanism for scour was similar to those tests performed at a velocity of 1.5 m/s. The average scour rate for these three tests was 10.86 mm/hr. The results of these tests are shown below in Figure 4-12. The average scour rates for the EFA tests performed for flume velocities of 1.0, 1.5, and 2.0 m/s were similar with a difference of only 1.69 mm/hr.

Table 4-12: Porter’s Creek Clay Results at 2.0 m/s

Sample:	Elapsed Time Ultrasonic (min)	Elapsed Time Visual (min)	Scour Rate Ultrasonic (mm/hr)	Scour Rate Visual (mm/hr)
Porter's Creek Clay 19.0_6	5	5	12.00	12.00
Porter's Creek Clay 19.0_7	7	7	8.57	8.57
Porter's Creek Clay 19.0_8	5	5	12.00	12.00

The final set of EFA tests, used to determine scour rates, performed on the Porter’s Creek Clay formation were executed at a flume velocity of 3.0 m/s. The average scour rate on these three tests jumped to 15.67 mm/hr. The results of these tests are shown below in Table 4-13.

Table 4-13: Porter’s Creek Clay Results at 3.0 m/s

Sample:	Elapsed Time Ultrasonic (min)	Elapsed Time Visual (min)	Scour Rate Ultrasonic (mm/hr)	Scour Rate Visual (mm/hr)
Porter's Creek Clay 19.0_9	5	5	12.00	12.00
Porter's Creek Clay 19.0_12	4	4	15.00	15.00
Porter's Creek Clay 19.0_14	3	3	20.00	20.00

During testing of the Porter’s Creek Clay formation it was noted that weathered planes existed within the tested samples. It was also noted that scour typically started on or around these weathered planes, which appeared to be planes of weakness with respect to scour resistance. As a result of these weathered lines and planes in the prepared sample four EFA tests were abandoned immediately following advancement into the flume. All four of these tests had large mass scouring occur at the location of the weathered planes immediately following the push. The volume of scour was not measured by the ultrasonic sensor as scour occurred very quickly before a scan from the sensor could be performed. Visually the volume of scour in these four instances was greater than the one millimeter protrusion into the flume. An image of these weathered lines and planes can be viewed below in Figure 4-23. When the weathered lines were noticed in split spoon samples during sampling the ALDOT geologist confirmed that this weathering pattern was quite common in samples from the Porter’s Creek Clay formation.



Figure 4-23. Porter’s Creek Clay Formation Scoured along Weathered Planes

The last test performed on the Porter’s Creek Clay formation was intended to determine the threshold velocity, which scour first occurs. From previous tests it was determined that the threshold velocity, or critical shear velocity, was between 0.3 and 0.6 m/s. The velocity during the first test performed on the Porter’s Creek Clay formation typically ranged from 0.3 to 0.35 m/s. The velocity for the final test on the formation, titled “Porter’s Creek Clay 18.75_1”, was started at 0.4 meters per second. At first surface scouring began to occur, at a reduced rate of that observed in the samples tested at 0.6 m/s. After about ten minutes small flakes began to scour away from the specimen, while the flume velocity ranged from 0.4 to 0.42 m/s. This scouring was substantial enough for the threshold velocity to be established at 0.4 m/s. It should be noted that the threshold velocity obtained is not an exact number. Previous work at Auburn University involved the calibration of the flow meter located in the EFA. This work showed at velocities less than 0.5 m/s the flow meter could have a margin of error of approximately ten percent (Mobley, 2009). The total results for the testing of the Porter’s Creek Clay formation can be viewed in Figure 4-24, with the averaged results in Figure 4-25.

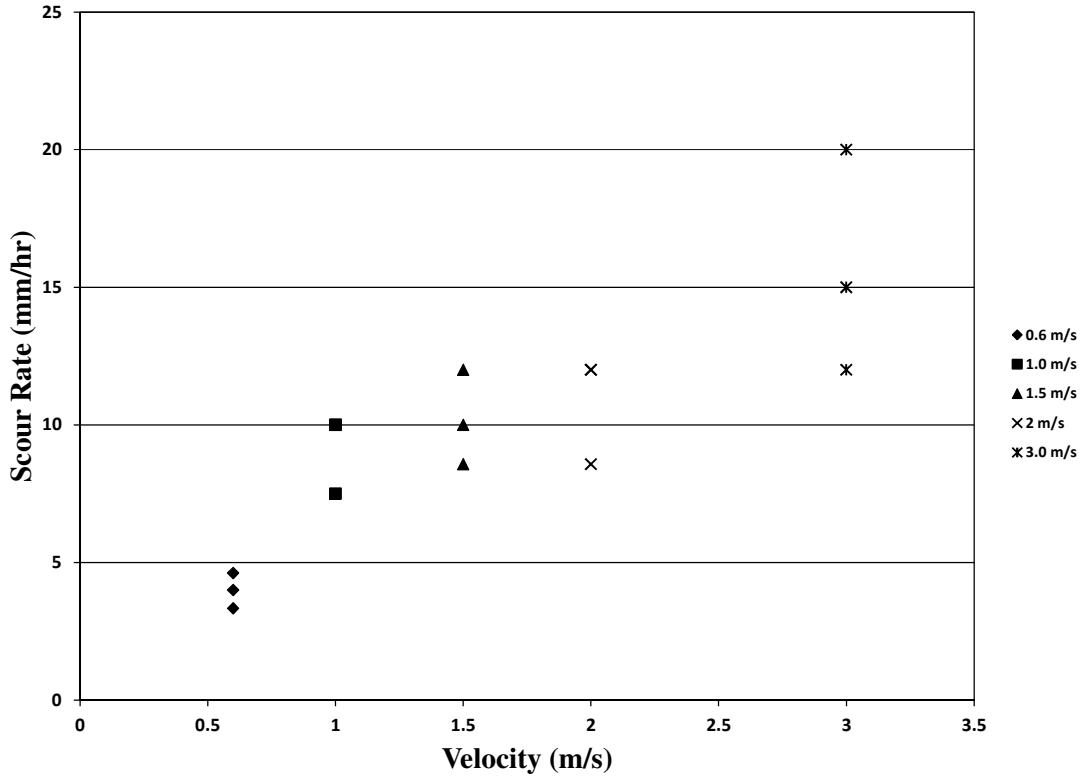


Figure 4-24. EFA Test Results, Porter's Creek Clay Formation

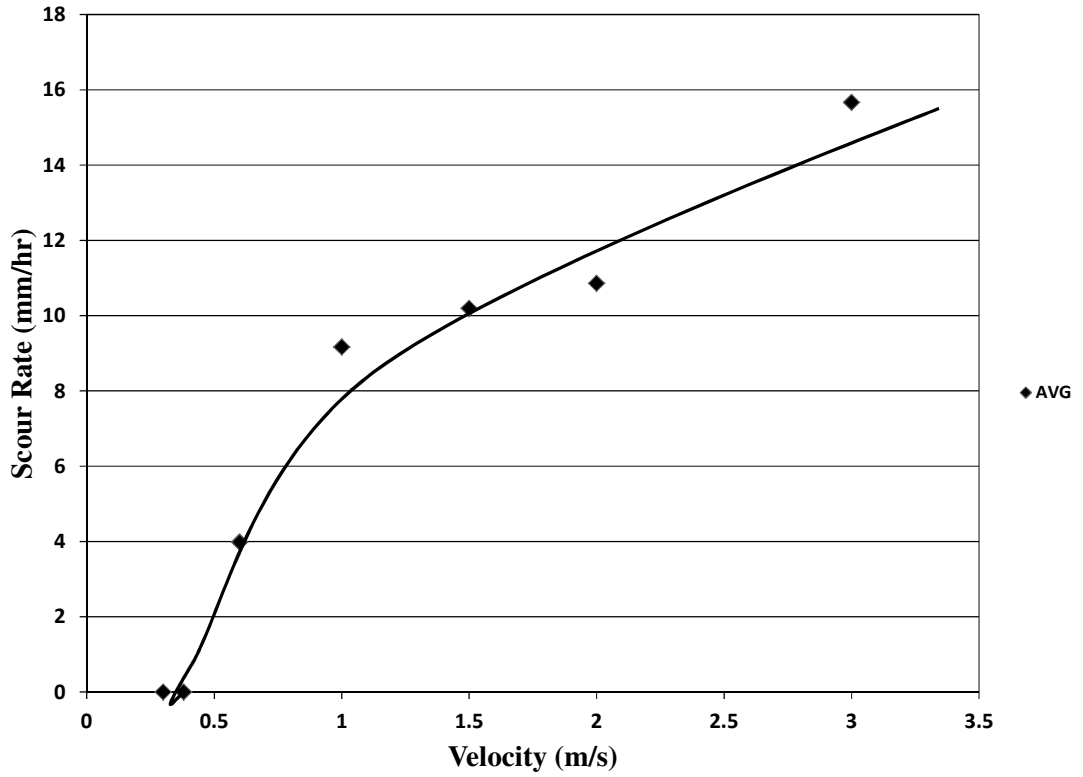


Figure 4-25. Porter's Creek Clay Velocity versus Scour Rate

In summary, the Porter's Creek Clay formation behaved similarly to the Bucatunna Clay formation during EFA testing. The major concern in testing the Porter's Creek Clay formation was the scour behavior of the formation with respect to the weathered lines and planes within the formation. As stated above these weathered lines created planes of weakness during EFA testing increasing the observed scour rates. All EFA test results for the Porter's Creek Clay formation can be viewed in Appendix F.

4.7.3 Geotechnical Testing

Three initial moisture contents were taken to obtain the insitu moisture content of the Porter's Creek Clay formation. These tests provided moisture contents of 36.7, 33.6, and 36.9 percent with an average of 35.7 percent. Using soil processed by following ASTM D421, a full grain size distribution was performed on the formation following ASTM D422. The results of the grain size analysis can be viewed below in Figure 4-25. The grain size analysis on the Porter's Creek Clay showed that the formation consisted of fine-grained particles with approximately 90 percent of the sample passing the number 200 sieve. From the grain size analysis the mean particle diameter (D_{50}) was calculated to be 0.082 mm. The percent of the Porter's Creek Clay sample passing the number 200 sieve was similar to the three chalk formations that were tested.

Atterberg limits were performed to complete the soil classification and geotechnical testing of the Porter's Creek Clay formation. The liquid limit was determined to be approximately 62, and the plastic limit was determined to be approximately 53, resulting in a plasticity index of 9. Using these values along with the information gathered from the grain size distribution, the sample was classified as an elastic silt (USCS Classification MH).

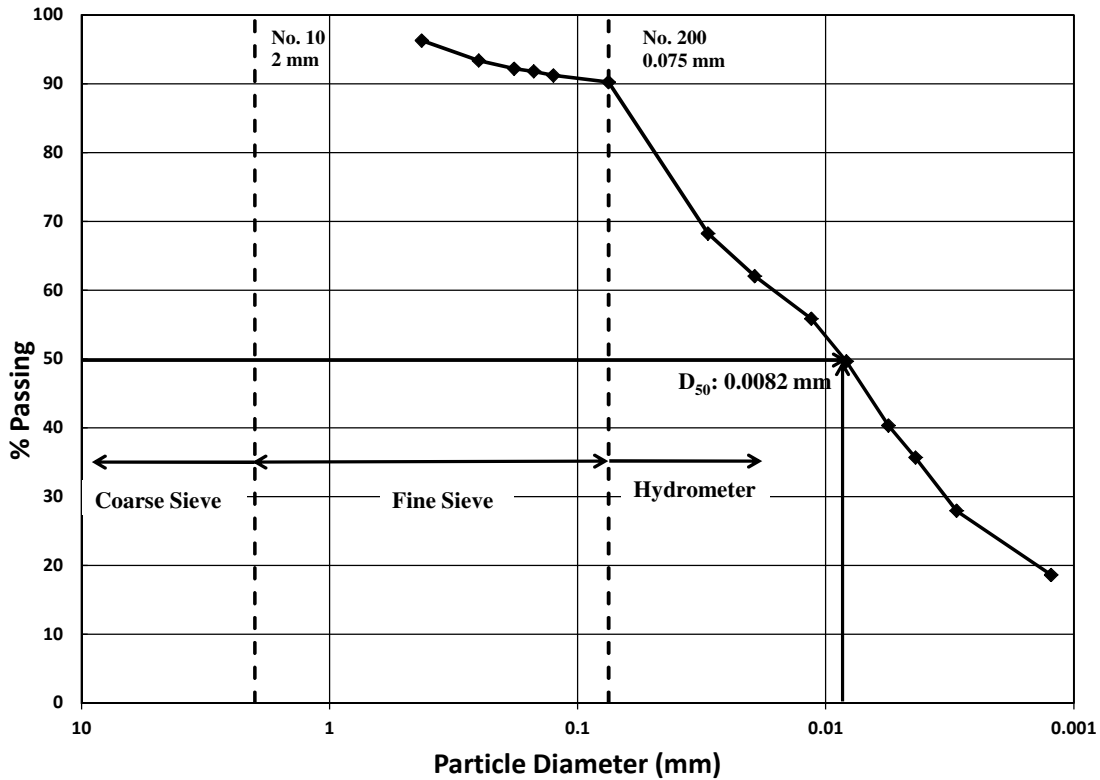


Figure 4-26. Porter's Creek Clay Formation Grain Size Distribution

4.8 Clayton Formation

4.8.1 Sampling

Three other clay formations were sampled to complete this study, however geotechnical and EFA testing was not performed on these formations. The testing of these formations will continue in the future. The Clayton Clay formation was sampled on June 21, 2012 in Barbour County, Alabama. An ALDOT geologist classified the Clayton formation as a light brown clay. The Clayton formation was difficult to locate as the formation is traditionally thin in parts of southern Alabama. Sampling the Clayton formation was made additionally difficult as the formation often underlies varying layers of sand. The ALDOT geologist identified the formation in drill cuttings and split spoon samples at approximately 8.7 m below the ground surface. A Standard Penetration Test was performed at the top of the formation yielding an N value of 23

blows. After the Standard Penetration Test was performed on the Clayton formation, the drill hole caved in due to sand in the upper part of the hole. The drill crew started another drill hole approximately 5 m away from the first and the formation was located approximately 7.3 m below the ground surface. The ALDOT drill crew performed three CME continuous sample runs on the Clayton formation.

The first of the continuous sample runs was performed between 7.32 and 8.84 meters below the ground surface. The top acrylic sample tube in this run was completely empty with no recovery. The bottom half of the sample had a small testable section about 30 cm in length. The second continuous sample run was performed between 8.84 and 10.36 m below the ground surface. This sample run included many small testable sections between cracks in the formation, but none of the sections were greater than 15 cm in length. The third continuous sample run was performed between 10.36 and 11.89 m below the ground surface. The sample tube hit the top of bedrock during sample resulting in very little recovered sample. Among this recovered sample there were no EFA testable sections.

4.9 Nanafalia Clay Formation

4.9.1 Sampling

The Nanafalia Clay formation was sampled on June 6, 2012 in Coffee County, Alabama. An ALDOT geologist classified the Nanafalia Clay formation as a plastic brown clay. The ALDOT geologist identified the formation in drill cuttings and split spoon samples at approximately 2.5 m below the ground surface. A Standard Penetration Test was performed at the top of the formation yielding an N value of 13 blows. The ALDOT drill crew performed three CME continuous sample runs on the Nanafalia Clay formation.

The first CME sampler run was performed from 2.74 to 4.26 m below the ground surface. The top half of the sample was not usable for EFA testing and included some transition from the Standard Penetration Test. A testable section was located in the top 15 centimeters of the bottom acrylic sampling tube. The second continuous sample run was performed from 4.26 to 5.79 m below the ground surface. Most of the top half of the sample was not recovered, and some small usable sections were located in the bottom half of the sample. The third and final continuous sample run on the Nanafalia Clay formation was performed from 5.79 to 7.31 m below the ground surface. The top half of this sample run was mostly unusable for EFA testing, but could be used for future geotechnical testing. The bottom acrylic sampling tube was almost entirely uncracked, and could be used for EFA testing. The bottom half of this continuous sample was the first sample obtained that could have an unconfined compression test performed without the presence of multiple cracks.

4.10 Naheola Clay Formation

4.10.1 Sampling

The Naheola Clay formation was sampled on June 7, 2012 in Marengo County, Alabama. An ALDOT geologist classified the Naheola Clay formation as a gray clay. The ALDOT geologist identified the formation in drill cuttings and split spoon samples at approximately 3.9 m below the ground surface. A Standard Penetration Test was performed at the top of the formation yielding an N value of 16 blows. The ALDOT drill crew performed three CME continuous sample runs on the Naheola Clay formation.

The first continuous sample run on the Naheola Clay formation as performed between 3.96 and 5.49 m below the ground surface. The top half of the sample was disturbed by the Standard Penetration test. The bottom half of the sample was mostly usable for EFA testing.

The color of the formation changed from gray to dark yellow in the bottom half of the sample. The ALDOT geologist observed the sample and determined that the colored change was not unordinary for the Naheola Clay formation. The second CME sample run was performed between 5.49 and 7.01 m below the ground surface. The color of the formation continued to change intermittingly between gray and yellow in the second sample. The top half of the sample run was severely cracked and not able to be used for EFA testing. The bottom half of the sample contained approximately 40 cm of EFA testable sample. The third continuous sample run was performed from 7.01 to 8.53 m below the ground surface. The top half of the sample did not recover any of the Naheola Clay formation. The bottom half of sample contained several small rock segments. A 25 centimeter testable section was located at the bottom of the sample. With approximately 80 cm of uncracked Naheola Clay formation collected, unconfined compression tests can be performed on the formation in the future.

Chapter 5 Results Discussion

5.1 Sampling Observations

As stated in earlier chapters, nine clay and chalk formations from the coastal plains in southern Alabama were sampled using the Central Mining Equipment continuous sample tube system. The quality and consistency of the acquired samples varied between formations. Ideally the samples gathered would serve three purposes and be used for EFA testing, shear strength testing, and geotechnical index testing.

All samples were successfully used for EFA testing. However, the clay formations tested in the EFA were easier to test than the chalk formations. It appeared the very stiff chalk formations created a high amount of skin friction between the sample and the acrylic tube. At times this skin friction was too large for the sample to be automatically advanced using the stepping motor on the Erosion Function Apparatus. In these cases EFA testing was not ideal because the protrusion into the flume not consistent with the calibrated one millimeter protrusion set forth in Briaud's (1999) procedure. The clay samples acquired, the Bucatunna Clay, Yazoo Clay, and Porter's Creek Clay formations, were able to be EFA tested without any difficulties.

The samples acquired, using the CME continuous sampler, were successfully used for geotechnical index testing. Samples were extruded from the sample tubes and processed for testing. However, the samples acquired were not conducive for shear strength testing. Given the cohesive material used for this study, the unconfined compression test is ideal for shear strength testing. With the diameter of the continuous sample being 5.7 cm, an acceptable

unconfined compression sample would need to be at least 11.4 cm. Unfortunately the sampling method did not yield uncracked samples with a minimum length of 114 mm. Therefore, it was not possible for an unconfined compression test to be performed on any of the acquired samples.

During sampling it was noted that most, if not all, of the uncracked sections acceptable for use in EFA testing were located in the bottom half of the acrylic sampling tubes. It is thought that this was due to friction being created between the cohesive surface of the sample and the acrylic sample tube as the sampler was advanced during sampling. It appeared that after a certain length the sampled material began to fail in friction and crack vertically within the sampling tube. This would explain the bottom of the samples being uncracked, as this segment of sample would be exposed to small or negligible frictional forces along the edges of the sample. This cracking was nominally improved by spraying a lubricant inside the sample tubes, although the effectiveness of the lubricant varied depending on the stiffness of the formation being sampled. Another issue associated with the cracking of sampled material is that the bearing assembly was not always successful in keeping the sample tube isolated from the rotation of the drill rig. This could expose the sample to torsional and shear forces during sample, which would be evident by the cracks in the recovered sample. This also could compromise the undisturbed state of the sample specimen. It is important to note this was not always the case and was only observed when sampling very stiff formations.

Another issue observed during sampling of the chalk formations was the bowing of the sample tube assembly. In these cases it appeared as if the acrylic sampling tubes were heated, due to the friction of material, allowing the tube to yield and expand radially. This

too could question the undisturbed state of the obtained samples. Also the bowing of the sample tube made it difficult to test these samples in the EFA, as the opening in the flume of the EFA is equal to the diameter of the sample tube.

5.2 EFA Testing Observations

Six of the sampled formations were tested in the EFA to determine scour parameters. Of the six formations tested, three were clay formations and three were chalk formations. The trends in EFA testing appear to illustrate a clear difference in the scour behavior of clay and chalk formations. The three clay formations tested were the Bucatunna Clay formation, the Yazoo Clay formation, and the Porter's Creek Clay formation.

The Bucatunna Clay formation scoured with scour rates ranging from 4 to 11 mm/hr after scour initially occurred at a flume velocity of 0.45 m/s. The Bucatunna Clay formation was easy to prepare for EFA tests and was easily advanced into the flume. Scour typically occurred in small flakes resulting in nearly linear erosion, until the entire protrusion was eroded. At velocities close to or lower than the threshold velocity, the formation appeared to swell during testing. At times the magnitude of this swelling was approximately five times the initial protrusion. Swelling did not occur at velocities above the threshold velocity because the formation was not able to absorb moisture and swell while scouring.

The Yazoo Clay formation also scoured with highly variable scour rates which caused doubt in establishing a threshold velocity. The Yazoo Clay proved difficult to prepare as larger particle sizes made it difficult to create a level surface. EFA testing was eventually suspended due to high variability. Scour rates observed in the Yazoo Clay formation were higher at a flume velocity of 1.0 m/s than at a flume velocity of 1.5 m/s. Likewise, two consecutive tests completely failed before the ultrasonic sensor was even allowed to start. It

appeared that the sample was inconsistent with respect to depth and contained a high sand content. The sand content was excessive in this sample, upwards of fifty-five percent, which is uncharacteristic of the formation.

The Porter's Creek Clay formation was also scoured at the tested velocities. The scour rates for the Porter's Creek Clay formation ranged from 4 to 15 mm/hr. The Porter's Creek Clay formation behaved similarly to the Bucatunna Clay during EFA testing. The Porter's Creek Clay formation was also similar to the Bucatunna Clay formation in regards to ease of sample preparation. The sample generally scoured in a flake by flake pattern with some larger chunks eroding at times. The Porter's Creek Clay formation sample contained many weathered lines and planes. Scour was observed along these planes, and these planes became shear planes during testing.

The observations presented above are consistent with the results presented by Crim (2003) and Mobley (2009). Both Crim and Mobley tested samples classified as silts and clays. Crim (2003) and Mobley (2009) observed scour rates ranging from 0 mm/hour upwards of 100 mm/hour. Typical erosion rates observed by Crim and Mobley were approximately 20 mm/hr. These results are similar to those observed in the Bucatunna and Porter's Creek formations.

The three chalk formations tested were the Demopolis Chalk, the Mooreville Chalk, and the Prairie Bluff Chalk. All three of these formations were scour resistant according to the testing regiment established in this study. Mobley (2009) also tested the Mooreville Chalk formation in the EFA. Mobley observed that the formation was resistant to scour in velocities upwards of 6 m/s.

However, the Demopolis Chalk, Mooreville Chalk, and Prairie Bluff Chalk samples were difficult to test as all three samples had to be manually extruded during EFA testing. This manual extrusion resulted in a loss in calibration of the protrusion height according to the procedure set forth by Briaud. Typically the manual protrusion height was greater than one millimeter meaning that the shear stress acting on the sample was actually greater than the intended shear stress. Although these samples were exposed to higher shear stresses scour was not observed. After the samples were manually extruded small cracks were noticed on the surface of the samples. These cracks were attributed to possible shear and torsional forces exerted on the sample during sampling and extruding. Although these cracks would eventually affect the scour performance of the formations, scour was not observed in any of the three chalk formations. These formations were also difficult to level during sample preparation due to the stiffness of the chalk formations.

5.3 Geotechnical Testing Observations

The geotechnical testing component of this study helped reinforce the observations made during EFA testing. Similarly the geotechnical testing program assigned additional properties to the formations tested in the EFA, to find trends between geotechnical and scour parameters. One example showing the validity of the geotechnical testing program in providing explanation for observations during EFA testing is the Bucatunna Clay formation. As previously stated, the Bucatunna Clay formation swelled during EFA testing. This swelling behavior was noted and quantified during EFA testing. During geotechnical testing this observation was validated as the formation had a plasticity index of 29 which is well above the limit for a swelling clay.

Another example of the results in geotechnical testing providing validity to observations made in EFA testing was the Yazoo Clay formation. The Yazoo Clay formation was highly variable with large particles noticed during some EFA tests. The geotechnical testing it was determined the sample acquired did not classify as a clay, but as a silty sand. This evidence along with the variability and observations obtained during EFA testing hint that the obtained sample was not a pure sample of the Yazoo Clay formation, and the results obtained were not indicative of the entire formation.

Similarly the geotechnical testing provided some clarity in the testing of the three chalk formations. The Demopolis Chalk, Mooreville Chalk, and Prairie Bluff Chalk formations were all determined to be scour resistant during EFA testing. These formations also had similar geotechnical index properties, which explained some of the similarity in EFA testing. All three formations had similar mean particle diameters within a range of 0.007 millimeters. Also the percent passing the number 200 sieve was within 15 percent for the three formations with a minimum of 82 percent and a maximum of 97 percent.

5.4 Correlations in Testing

Several general scour trends were noticed while correlating the scour results to the geotechnical index properties obtained for the six formations tested. Generally speaking, scour-resistant chalk formations had higher Standard Penetration Test N values, lower insitu moisture contents, higher percentage of percent passing the number 200 sieve, and a smaller mean particle diameter. These trends correlate to previous research that states the critical shear stress increases when plasticity index increases, when undrained shear strength increases, and with the percent passing the number 200 sieve (Briaud 2001b). The trend stating that plasticity index increases as the critical shear strength increases was not affirmed

in this study. There were no correlations made between scour parameters and plasticity index, as the plasticity index for all six formations was highly variable. The Standard Penetration Test N value can roughly represent the undrained shear strength. Briaud (2001b) stated that the critical shear stress increases with the shear strength of the soil. This theory was confirmed in this study as the formations with higher SPT N values were scour resistant. Although the samples tested hinted that the insitu moisture content influences the critical shear stress and scour rate of a formation, this value is highly variable both between different formations and within one formation.

Both the evidence of this study and prior research tend to show that the percentage of particles passing the number 200 sieve influences the critical shear stress of the formation. The three chalk formations that were proven to be scour resistant all had a high percentage of particles passing the number 200 sieve. The Bucatunna Clay and Yazoo Clay formations, which both showed scour at the tested velocities, had lower percentages of particles passing the number 200 sieve than the three scour resistant formations. The outlier by this measure was the Porter's Creek Clay, which showed scour at the tested velocities, but had 90 percent passing the number 200 sieve. An explanation for the Porter's Creek Clay scouring with a high percentage passing the number 200 sieve could be the weathered planes that were observed during EFA testing. It was noted that scour routinely occurred along these planes, which could explain why the formation does not abide by this correlation. Previous research stated that the rate of scour was increased when the percentage of clay particles decreased (Briaud, 2001b). This trend is indirectly similar to the trend stated above that the critical shear stress increases with the percentage passing the number 200 sieve. Again all the formations tested except the Porter's Creek Clay formation follow this qualitative trend.

An observation was made with the scour test results and the geotechnical index parameters using the moisture content and SPT N value. In Figure 5-1 below, a horizontal and vertical line are drawn, and approximately intersect at a moisture content of 23 percent and an SPT N value of 60. All of the formations falling down and to the right of this intersection, the Bucatunna Clay formation, the Yazoo Clay formation, and the Porter's Creek Clay formation all scoured during testing. The formations up and to the left of the intersection, the Demopolis Chalk formation, the Mooreville Chalk formation, and the Prairie Bluff Chalk formation were scour resistant. This observation combined all three aspects of this study in sampling, EFA testing, and geotechnical testing. The SPT N value was obtained during sampling, the scour data was obtained during EFA testing, and the moisture content was collected during geotechnical testing. Additionally, one data point from Mobley (2009) was added to this data set and also follows this observation.

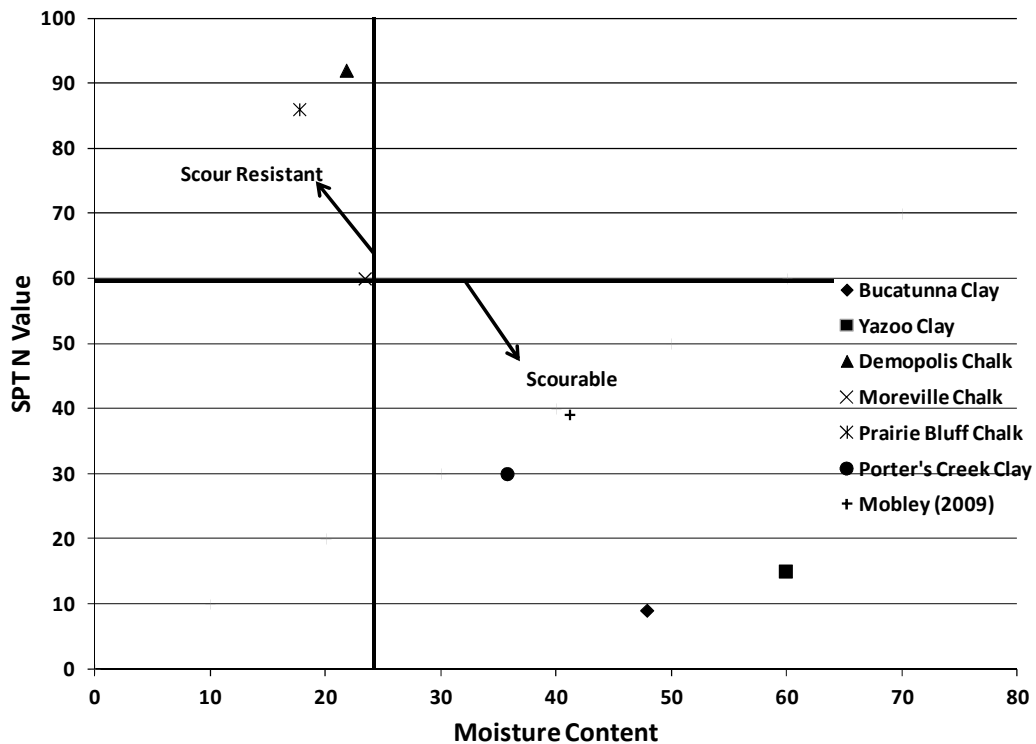


Figure 5-1. Scour Observation using Moisture Content and SPT N Value

Similarly, an observation was also made between SPT N Value and the percent of particles passing the number 200 sieve. Figure 5-2 shows this observation, as all of the formations falling below the line scoured during testing and all the formations plotted above the line were scour resistant. Similar to Figure 5-1, data from Mobley (2009) was added to this plot. Mobley (2009) observed scour in this sample with a SPT N value of 39 and 98 percent of the sample passing the #200 sieve, which follows the observation in Figure 5-2. This observation between scour parameters, SPT N value, and grain size is further described by Figure 5-3.

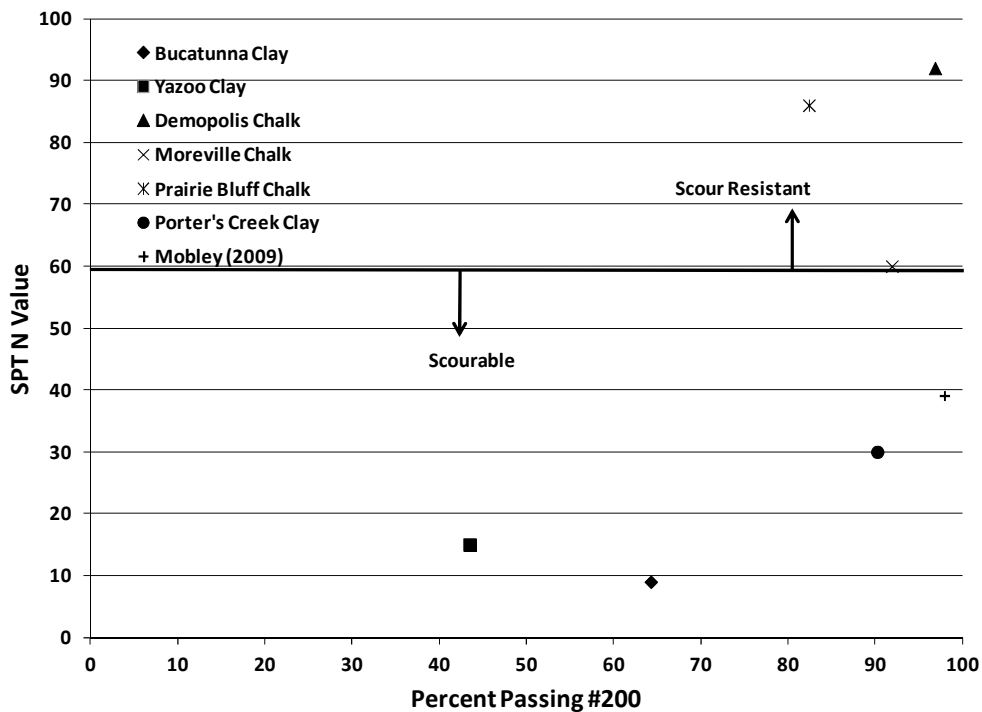


Figure 5-2. Scour Observation using Percent Passing #200 and SPT N Value

Figure 5-3 shows the mean particle size (D_{50}) on a logarithmic scale plotted with the SPT N value. In Figure 5-3, a horizontal and vertical line are placed approximately intersecting at a mean grain size of 0.0082 mm and an SPT N value of 60. All the formations plotted above and to the left of the intersection line were scour resistant and the formations

plotted below and to the right of the intersection scoured during EFA testing. Six additional samples tested by Crim (2003) are plotted on Figure 5-3 and follow the same observation, as all six samples fall below and to the right of the intersection and exhibited scour behavior during testing.

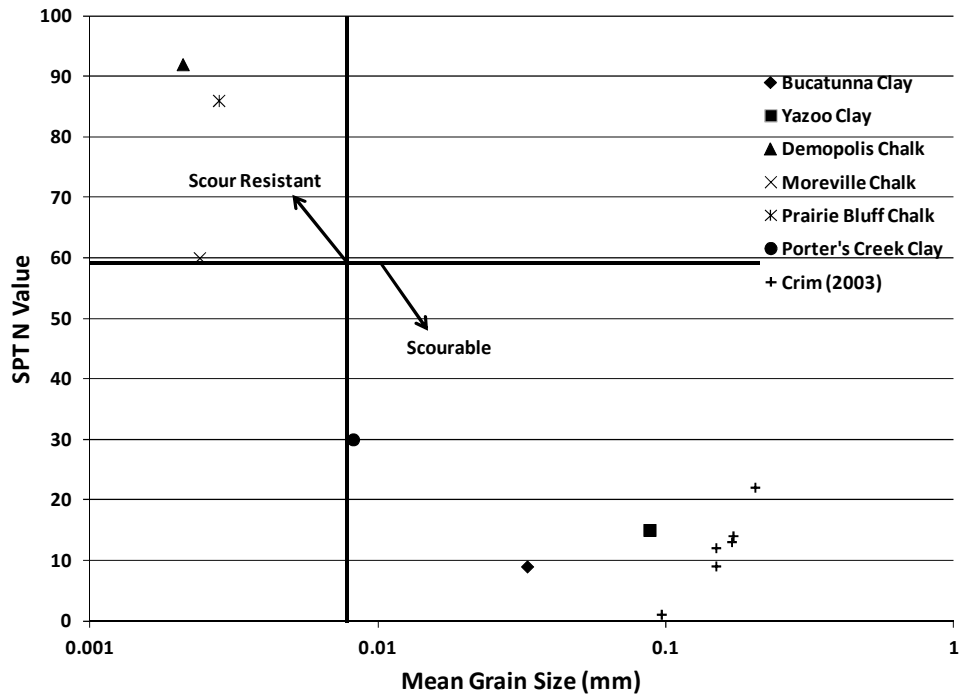


Figure 5-3. Scour Observation using Mean Grain Size and SPT N Value

In summary, all six of the formations tested proved to be unique. Even the chalk formations that generally performed the same in geotechnical and EFA testing, were each unique. This uniqueness made creating and proving quantitative correlations between scour behavior and geotechnical behavior more difficult.

Chapter 6 Summary, Conclusions, and Recommendations

6.1 Summary

The ability to determine scour parameters in order to accurately predict scour depth is imperative to designing safe, economic, and efficient bridge foundations. As previously stated, scour behavior is well understood for cohesionless soils; however, much research has been performed in an effort to better understand scour behavior of cohesive soils. In this study the research objectives were to adapt the EFA to measure scour rates automatically with the aid of an ultrasonic sensor, create an EFA testing procedure and regimen that incorporates the updated sensor, and conduct EFA tests on cohesive soil samples that were collected from the coastal plain of Alabama. Additionally, common geotechnical parameters such as Atterberg limits and grain size distribution were measured to determine if any correlations exists with scour parameters.

The Erosion Function Apparatus was updated with a 16 transducer ultrasonic sensor. Using the ultrasonic sensor and the updated EFA, a new testing regimen was created for determining scour parameters of cohesive soils. To obtain these samples, cooperation with the Alabama Department of Transportation was necessary. With the advice of ALDOT drilling coordinators, the sampling method used for this research was the Central Mining Equipment continuous sample tube system. ALDOT collected a total of nine formations, using the continuous sample tube system, in the coastal plains of Alabama. Six of the collected formations were tested in the EFA with a wide variety of results. These formations were then tested for traditional geotechnical index properties including a full grain size

distribution and Atterberg limits. Using the results from the EFA and geotechnical testing, qualitative correlations were determined.

The six formations sampled, erosion tested, and index tested include the Bucatunna Clay, Yazoo Clay, Demopolis Chalk, Mooreville Chalk, Prairie Bluff Chalk, and Porter's Creek Clay formations. The Demopolis Chalk, Mooreville Chalk, and Prairie Bluff Chalk formations were all scour resistant according to the testing program previously established. The Bucatunna Clay, Yazoo Clay, and Porter's Creek Clay formations all showed signs of measurable scour during testing. EFA testing of the Yazoo Clay formation was suspended due to high variability and inconsistencies in testing. Weathered planes in the Porter's Creek Clay formation heavily altered the EFA testing of the formation.

6.2 Conclusions

Determining correlations between erosion testing and geotechnical testing is difficult given the limited number of tests performed in this study. At best the correlations determined in this study should be expounded upon in future work. Figure 5-2 and Figure 5-3 indirectly validate the correlation between critical shear stress and particle size presented by Briaud (1999). Similarly, Figure 5-2 and Figure 5-3 confirm a correlation between scour rates and particle size. Figure 5-1 shows an observation between moisture content and SPT N values. EFA testing is highly variable and unique to each individual formation. This makes it increasingly difficult to quantify the results with standardized geotechnical properties. Overall the qualitative correlations confirmed or created throughout this study should be further studied to create more quantitative correlations. A summary of the research results is provided below in Table 6-1. Table 6-1 allows for easy correlations between test results and visual observations. The "Pushable" column in Table 6-1 states if a sample was

able to be automatically advanced in the EFA. The “Swelling Witnessed” column in Table 6-1 notates if swelling was witnessed during EFA testing. Table 6-1 illustrates all formations that required manual advancement into the EFA did not scour. Similarly all formations that swelled during testing exhibited scour. From the geotechnical index testing, formations containing a SPT N value greater than 60, a percent of particles passing the #200 sieve greater than 80, and insitu moisture content less than 25, and a mean particle size less than 0.003 mm will not exhibit scour. Again these correlations are qualitative but can be used to compare future results and observations.

Table 6-1: Summary of Results and Observations

Formation:	Scourable	LL	PL	PI	D ₅₀ (mm)	% Passing #200	USCS Classification	SPT N Value	Average Moisture	Pushable	Swelling Witnessed
Bucatanna Clay	Yes	68	39	29	0.033	64	MH (Sandy Elastic Silt)	9	47.9	Yes	Yes
Yazoo Clay	Yes	57	NP	NP	0.088	44	SM (Non-Plastic Silty Sand)	15	59.9	Yes	No
Demopolis Chalk	No	37	27	10	0.0021	97	CL (Lean Clay)	92	21.8	No	No
Mooreville Chalk	No	52	25	27	0.0024	92	CH (Fat Clay)	60	23.4	Yes/No	No
Prairie Bluff Chalk	No	32	19	13	0.0028	82	CL (Lean Clay with Sand)	86	17.7	No	No
Porter's Creek Clay	Yes	62	53	9	0.0082	90	MH (Elastic Silt)	30	35.7	Yes	Yes

6.3 Recommendations

Generally, the means and methods used in this study to collect and test EFA samples were successful. However, below are a few recommendations meant to improve the interpretation of test results. The three formations sampled and not tested—the Clayton formation, the Nanafalia Clay formation, and the Naheola Clay formation—should all be tested in the EFA. Also these three formations should be tested for geotechnical index properties to determine if the data obtained continues to follow the same correlations listed above. The Yazoo Clay formation should be resampled and tested to ensure that the correct

sample is gathered. The Yazoo Clay sample acquired should classify as a clay with more than fifty percent of the sample passing a number 200 sieve.

Chalk formations should be sampled with rock cores. The chalk samples were unable to be automatically advanced from the continuous sample tubes do to the stiffness of the samples. Chalk formations can be tested in the EFA by using traditional rock cores, without sacrificing resolution from the ultrasonic sensor. Future sampling of clay formations should include at least one Shelby tube sample, if possible, to accompany the continuous samples. These changes to the sampling procedure ensure every formation sampled would have a sufficient sample for shear strength testing. An unconfined compression test can be performed on a prepared rock core sample to determine the undrained shear strength. Likewise an unconfined compression test can be performed from the Shelby tube sample gathered from clay formations. This simple change in sampling technique allows future studies to obtain shear strength parameters for each of the tested formations which would add validity to correlations made with strength parameters. By obtaining shear strength parameters, the critical shear stress of the soil from EFA testing can be compared to the ultimate shear stress in compression.

The testing program should be expanded to include EFA tests with a longer duration and a higher velocity. The testing regiment should be expanded to velocities upwards of 6.0 meters per second in an attempt to determine the critical shear stress for the stiffer formations. Similarly a longer duration test should be introduced to determine if scour occurs at a lower velocity after multiple hours of exposure to relatively low shear stresses. This longer duration test can be altered to fit the format of the multiple events test performed on the Demopolis and Mooreville Chalk formations.

Finally, the testing program should be expanded beyond the nine formations sampled. This additional testing would create more data from which correlations can be drawn, with greater accuracy. This data can be compared with correlations initially created by Briaud to statistically advance the legitimacy of any quantitative correlations. In conclusion, to completely understand the scour behavior at an individual stream crossing, an extensive unique EFA study should be performed. However, with extensive EFA testing, using the ultrasonic sensor for higher resolution, there is potential for creating statistically adequate correlations for estimating scour behavior of a specific formation.

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APPENDIX A

EFA AND GEOTECHNICAL TEST RESULTS:

BUCATUNNA CLAY FORMATION

TEST VELOCITY: 0.6 METERS PER SECOND

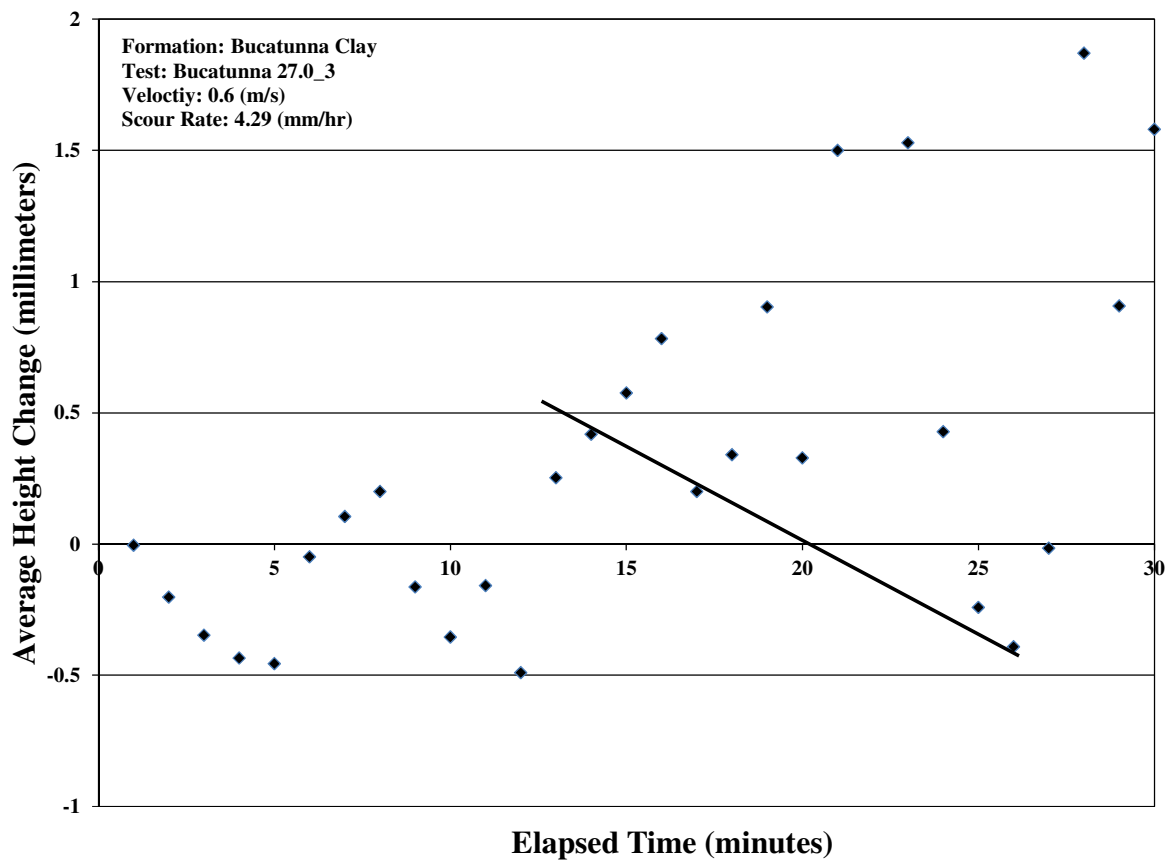


Figure A-1. "Bucatunna 27.0_3" Test Results

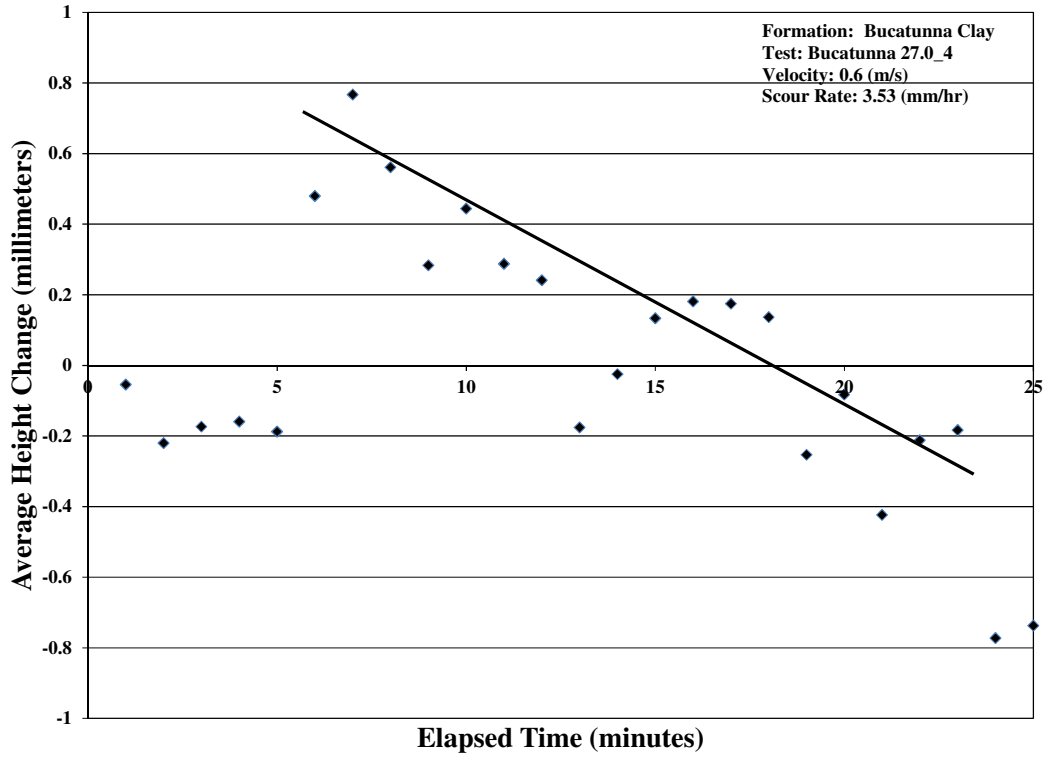


Figure A-2. “Bucatanna 27.0_4” Test Results

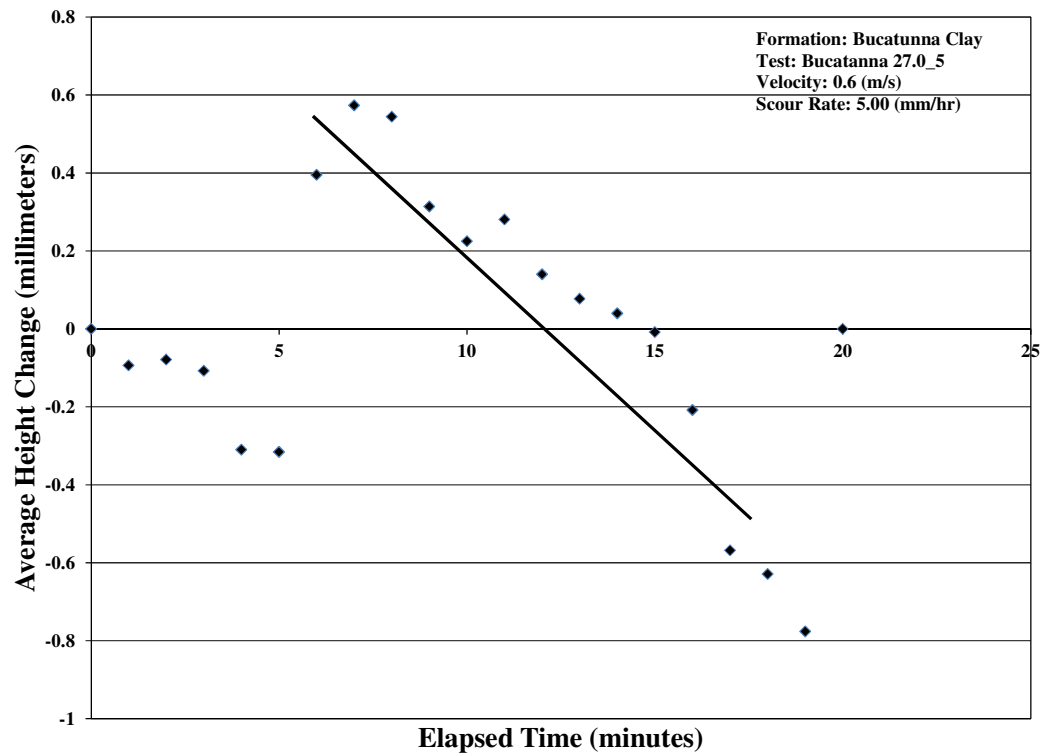


Figure A-3. “Bucatanna 27.0_5” Test Results

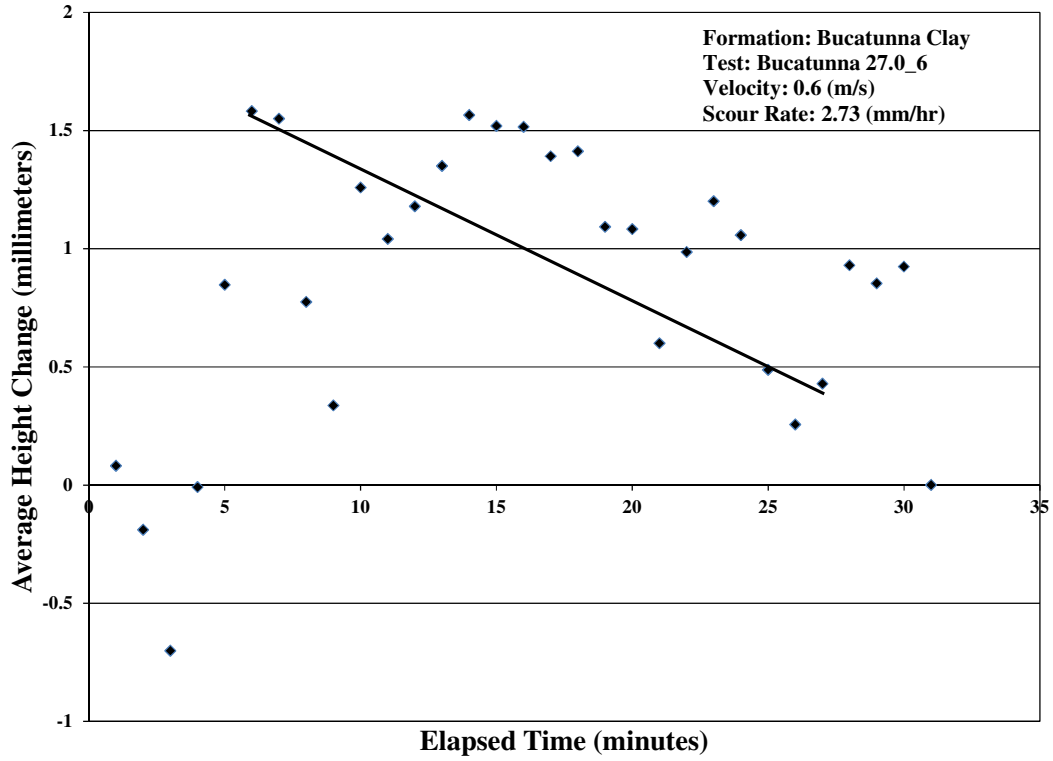


Figure A-4. "Bucatunna 27.0_6" Test Results

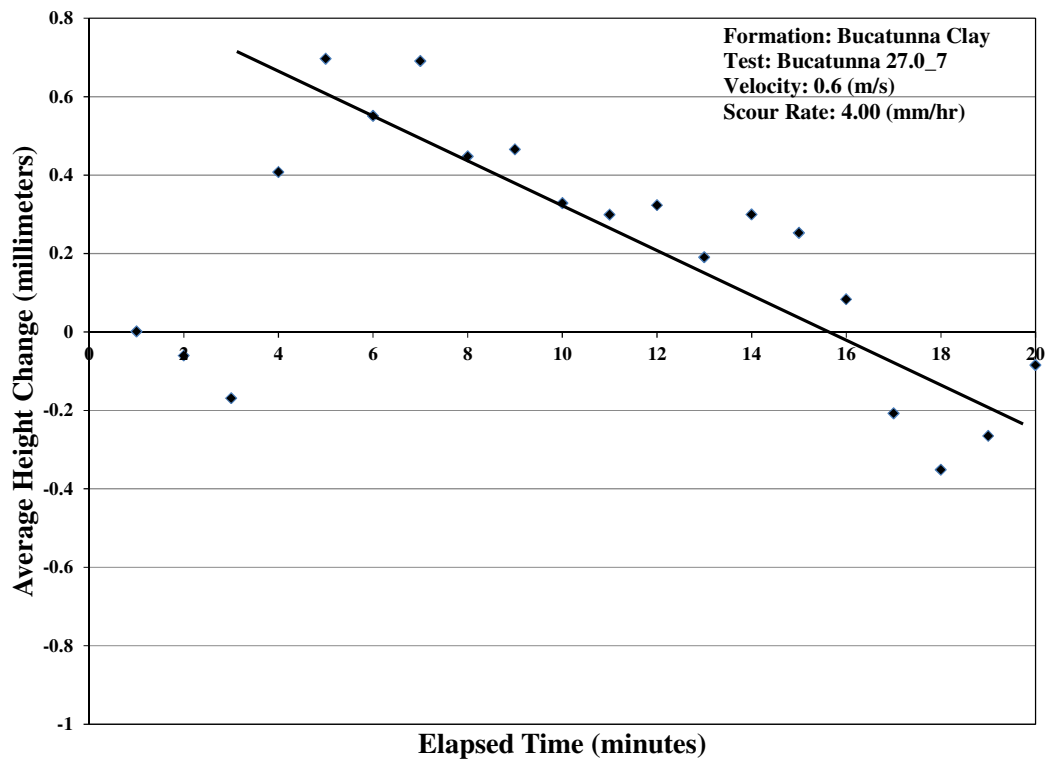


Figure A-5. "Bucatunna 27.0_7" Test Results

TEST VELOCITY: 1.0 METERS PER SECOND

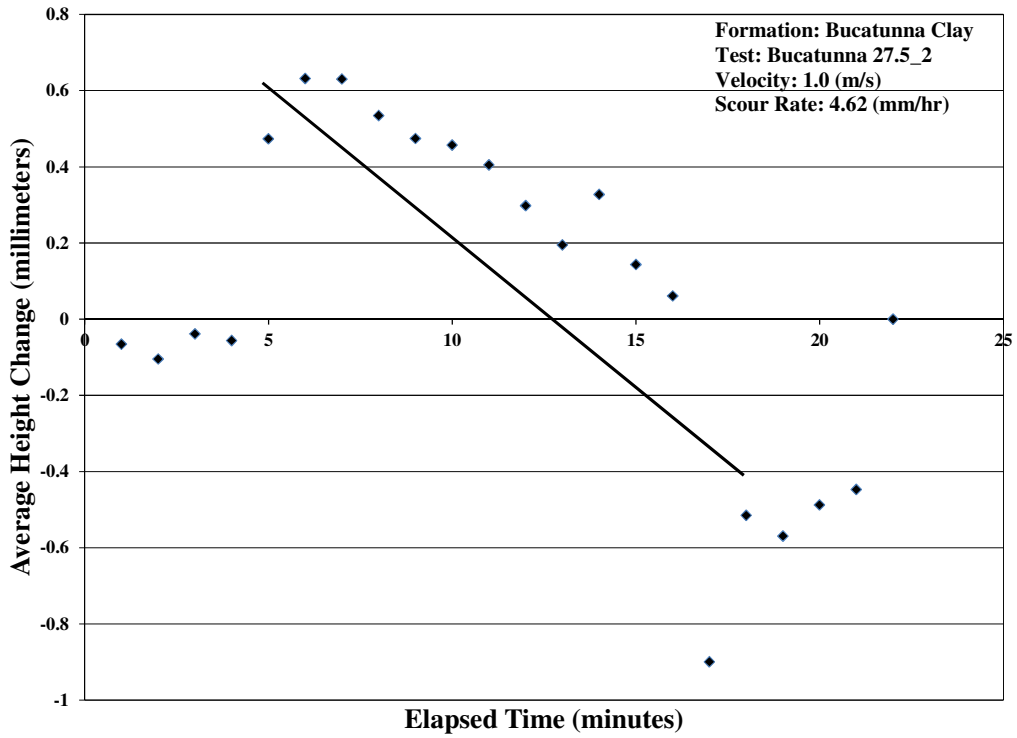


Figure A-6. "Bucatumna 27.5_2" Test Results

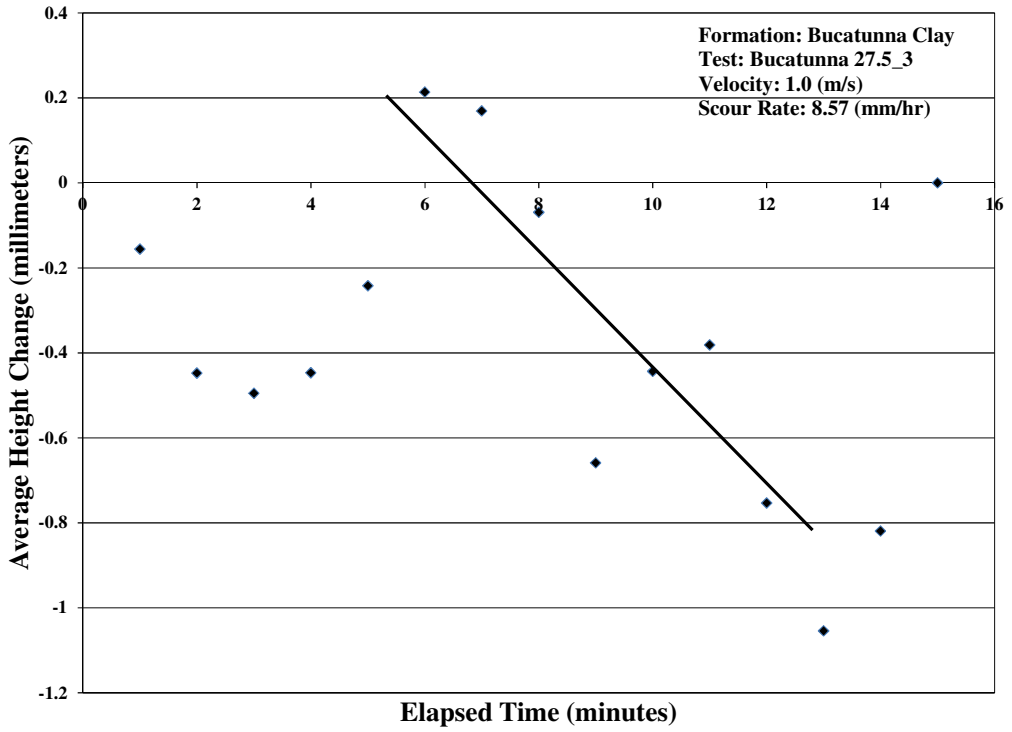


Figure A-7. "Bucatumna 27.5_3" Test Results

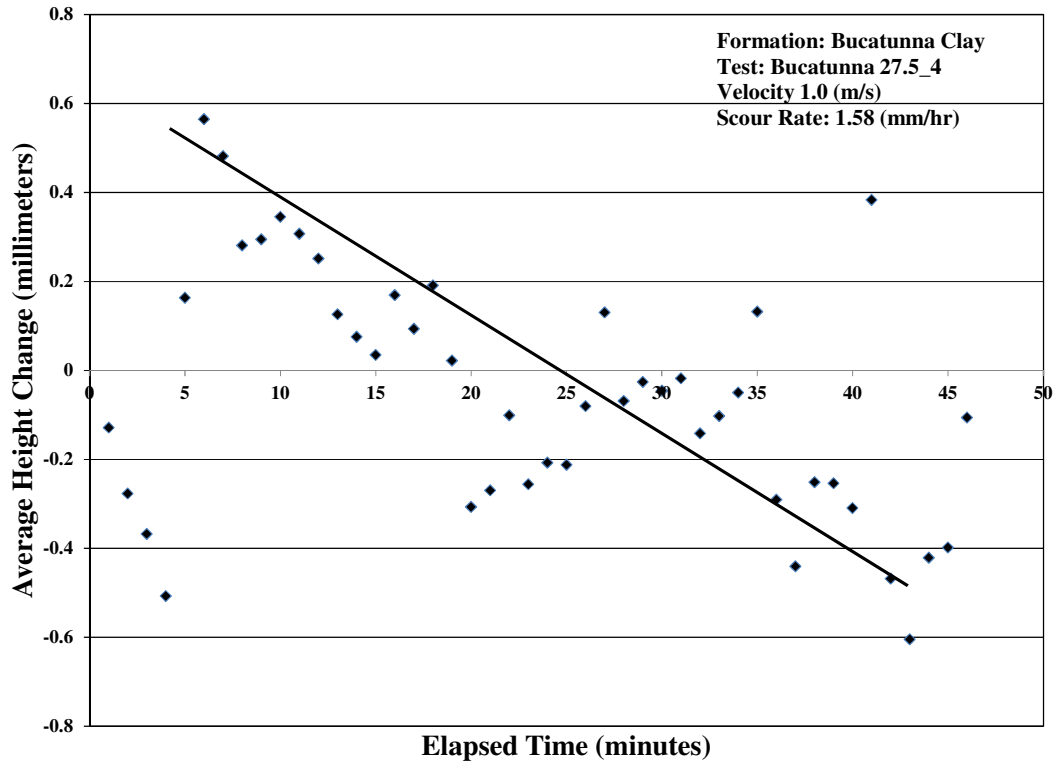


Figure A-8. "Bucatumna 27.5_4" Test Results

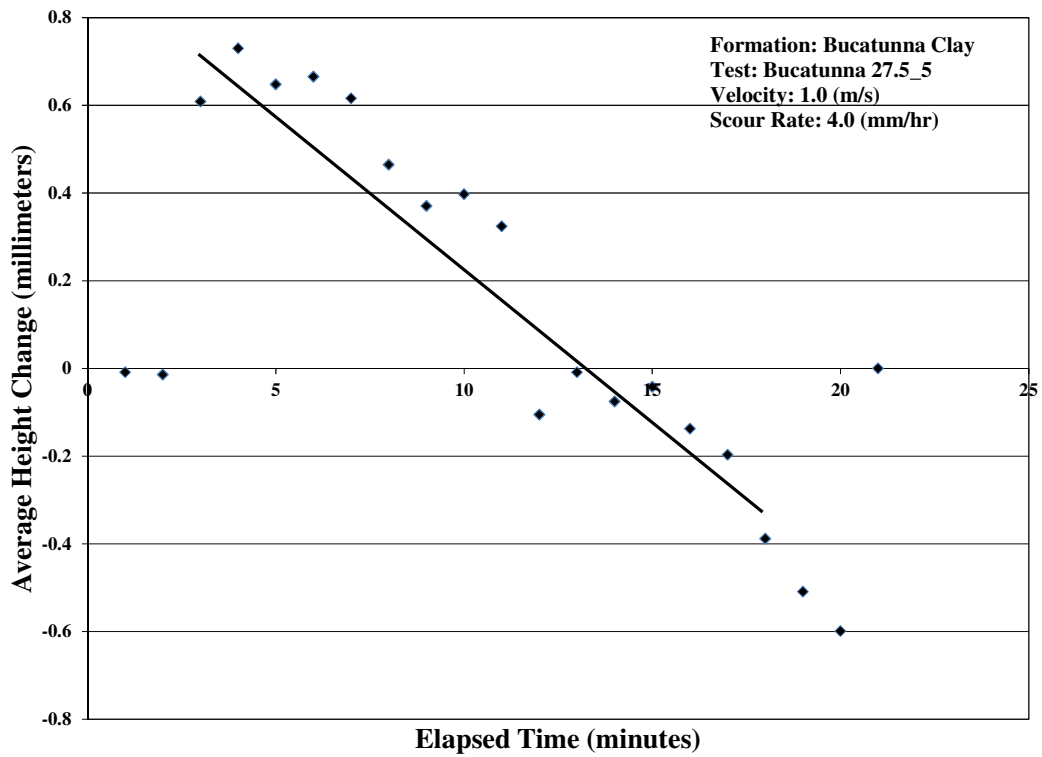


Figure A-9. "Bucatumna 27.5_5" Test Results

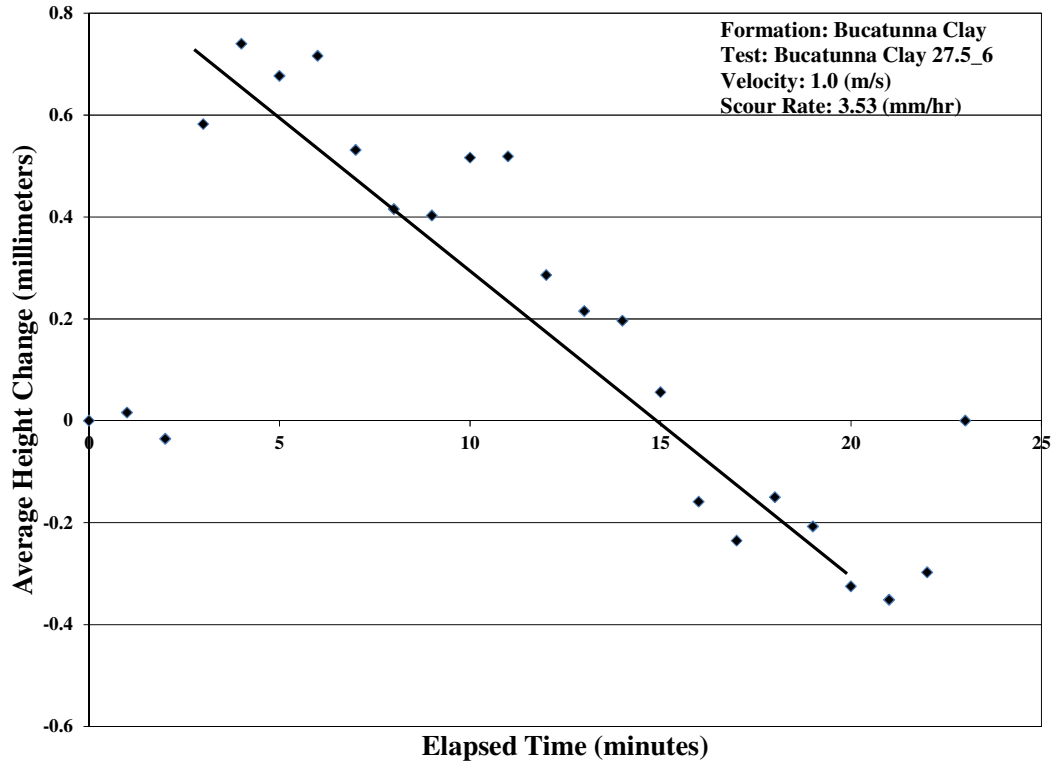


Figure A-10. "Bucatunna Clay 27.5_6" Test Results

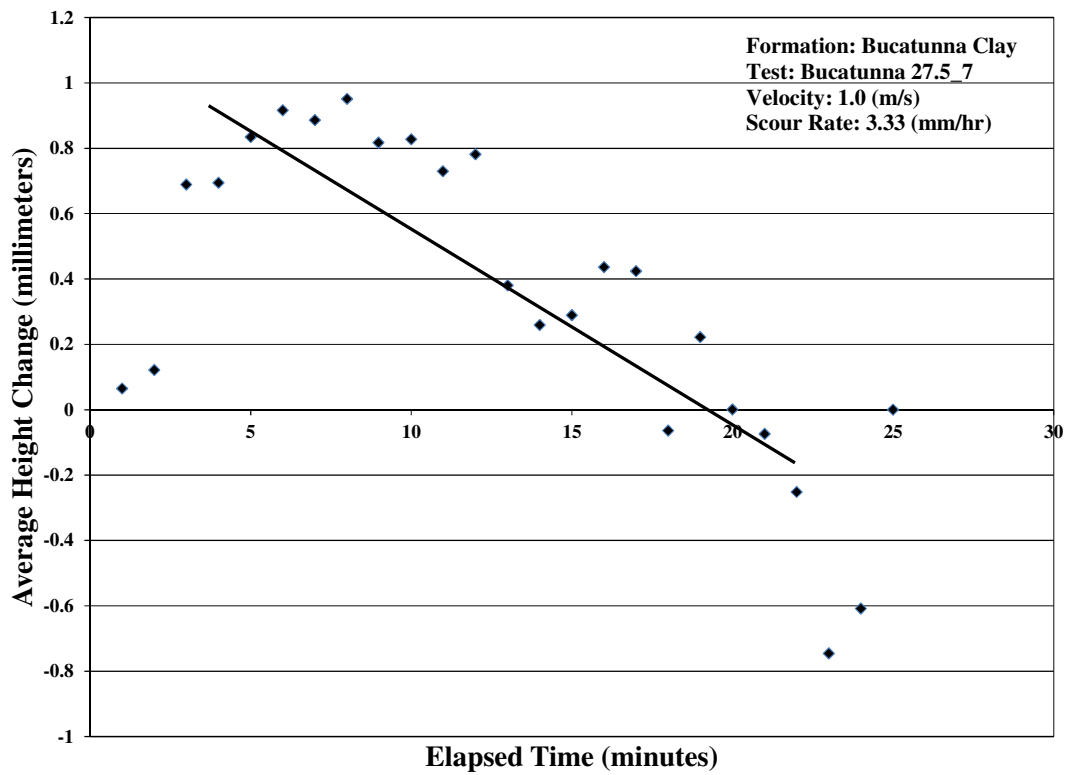


Figure A-11. "Bucatunna 27.5_7" Test Results

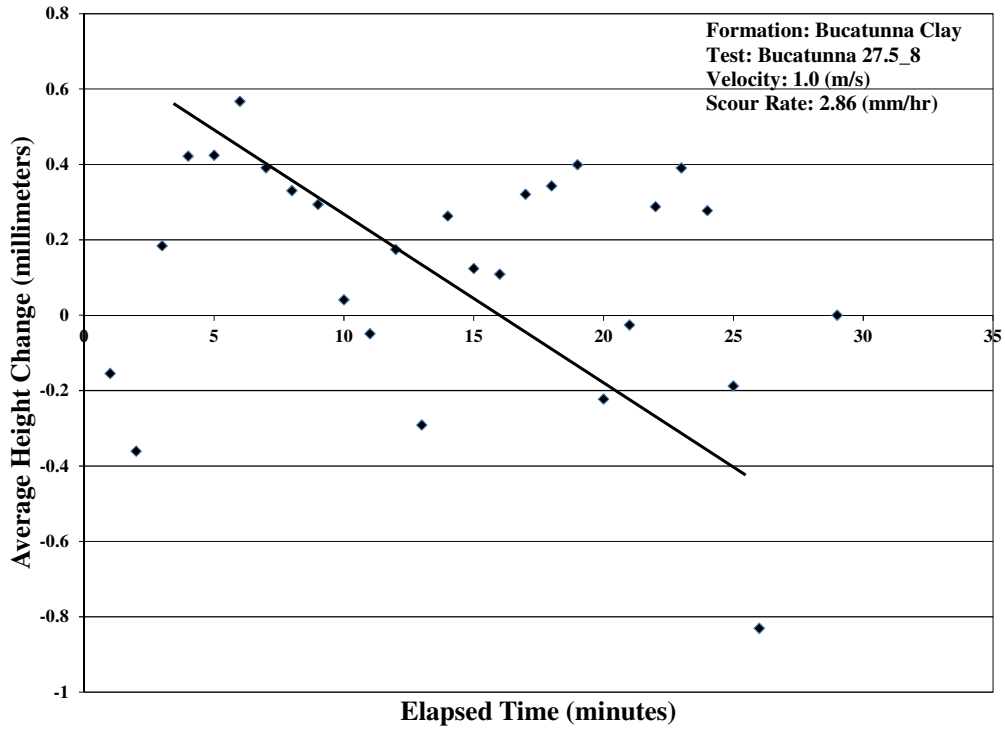


Figure A-12. “Bucatunna Clay 27.5_8” Test Results

TEST VELOCITY: 1.5 METERS PER SECOND

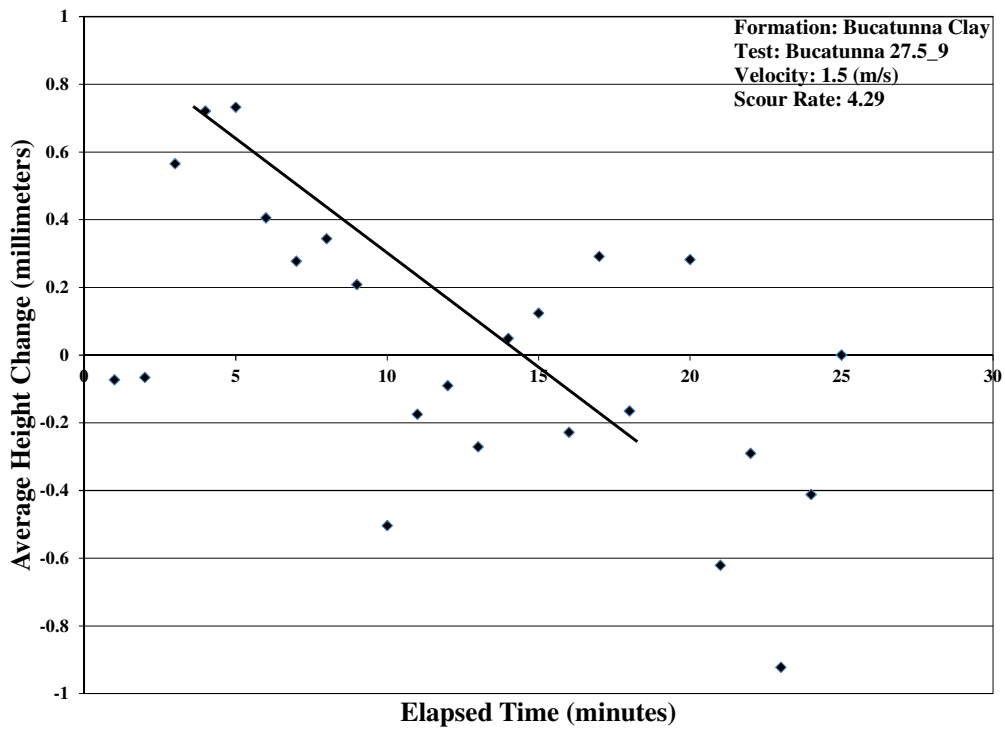


Figure A-13. “Bucatunna Clay 27.5_9” Test Results

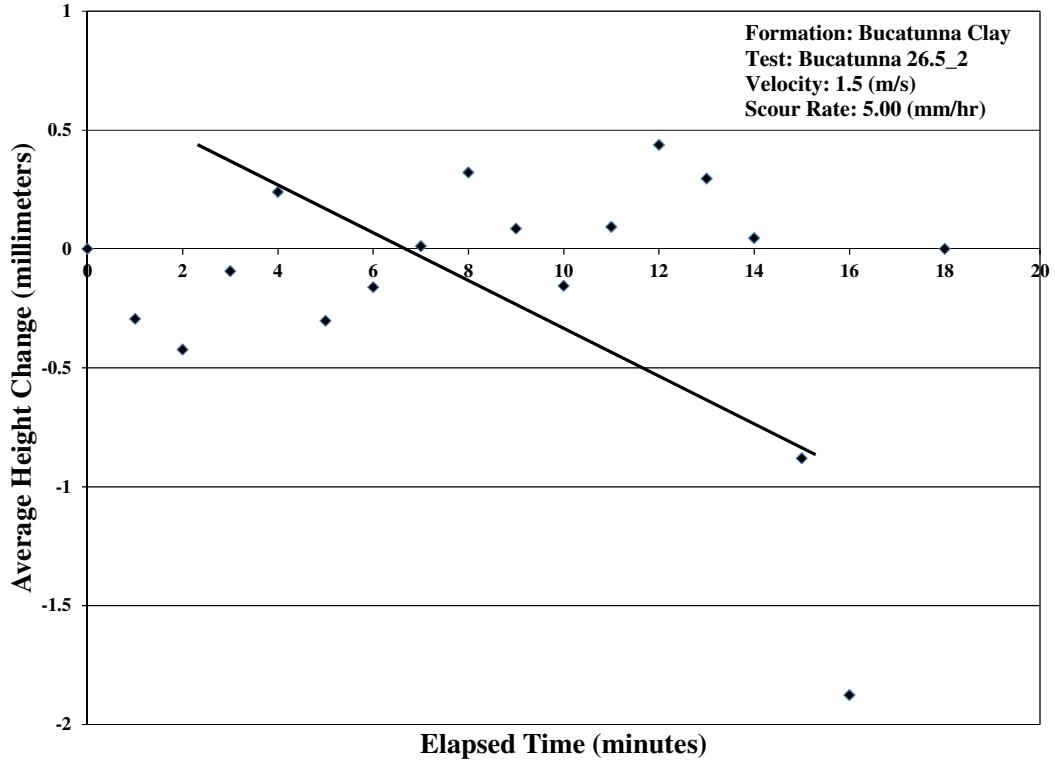


Figure A-14. "Bucatanna 26.5_2" Test Results

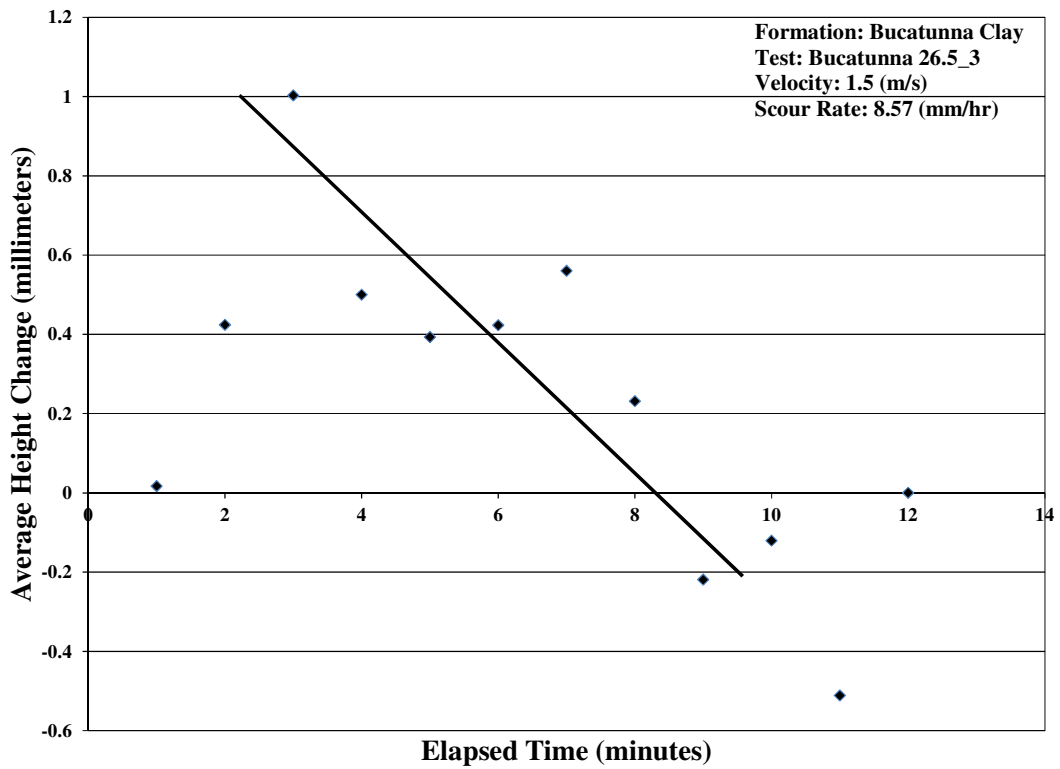


Figure A-15. "Bucatanna 26.5_3" Test Results

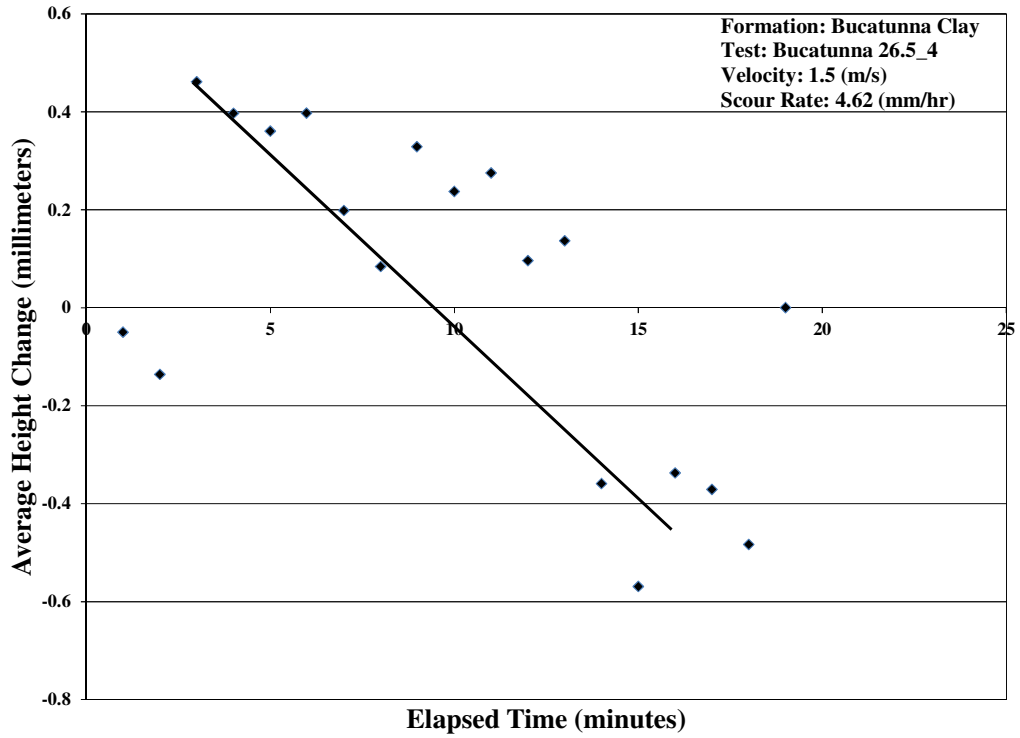


Figure A-16. "Bucatumna 26.5_4" Test Results

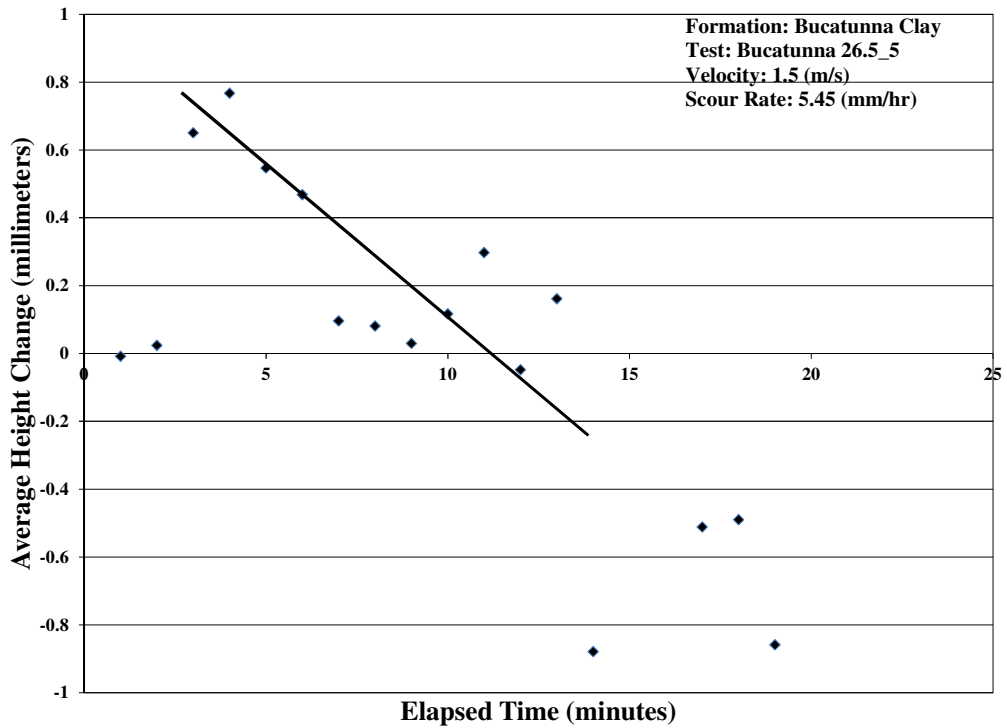


Figure A-17. "Bucatumna 26.5_5" Test Results

TEST VELOCITY: 2.0 METERS PER SECOND

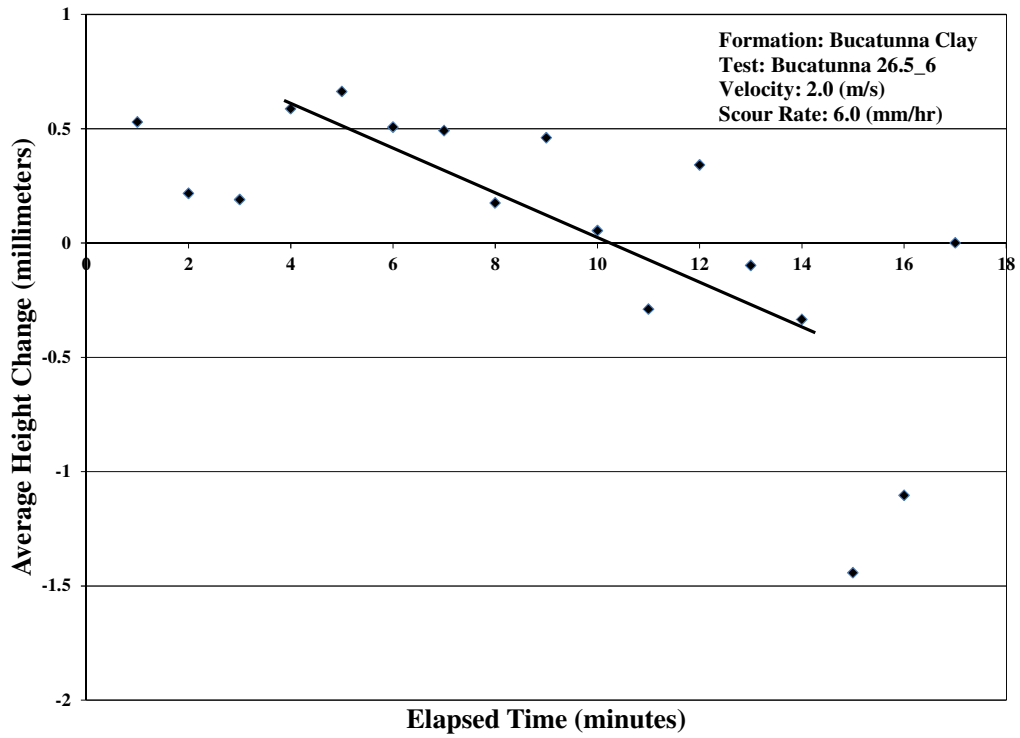


Figure A-18. "Bucatumna 26.5_6" Test Results

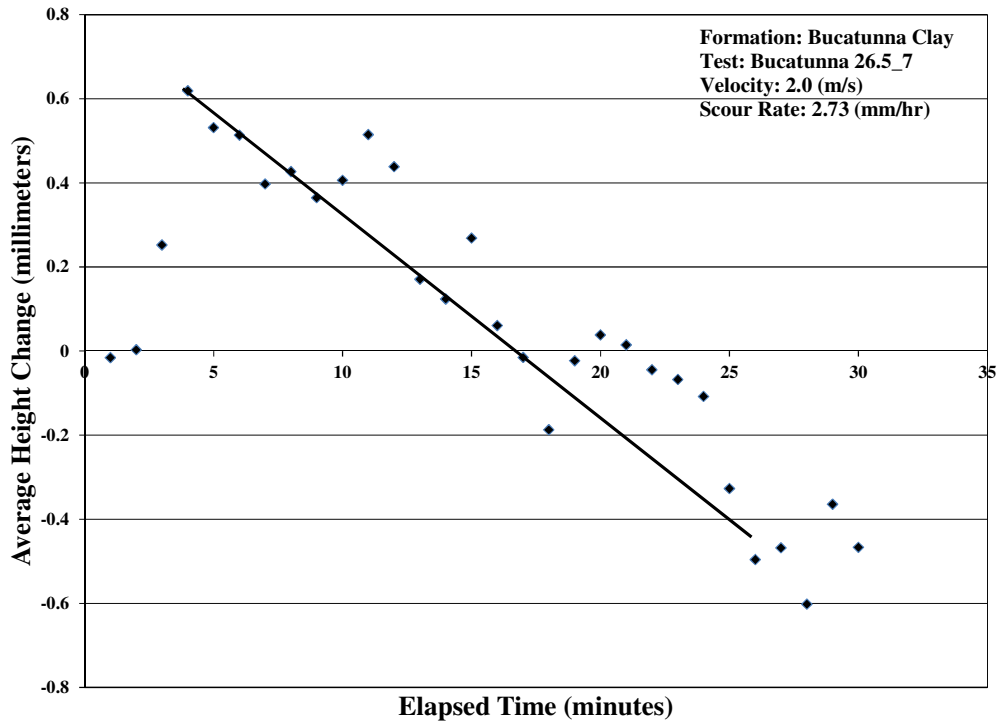


Figure A-19. "Bucatumna 26.5_7" Test Results

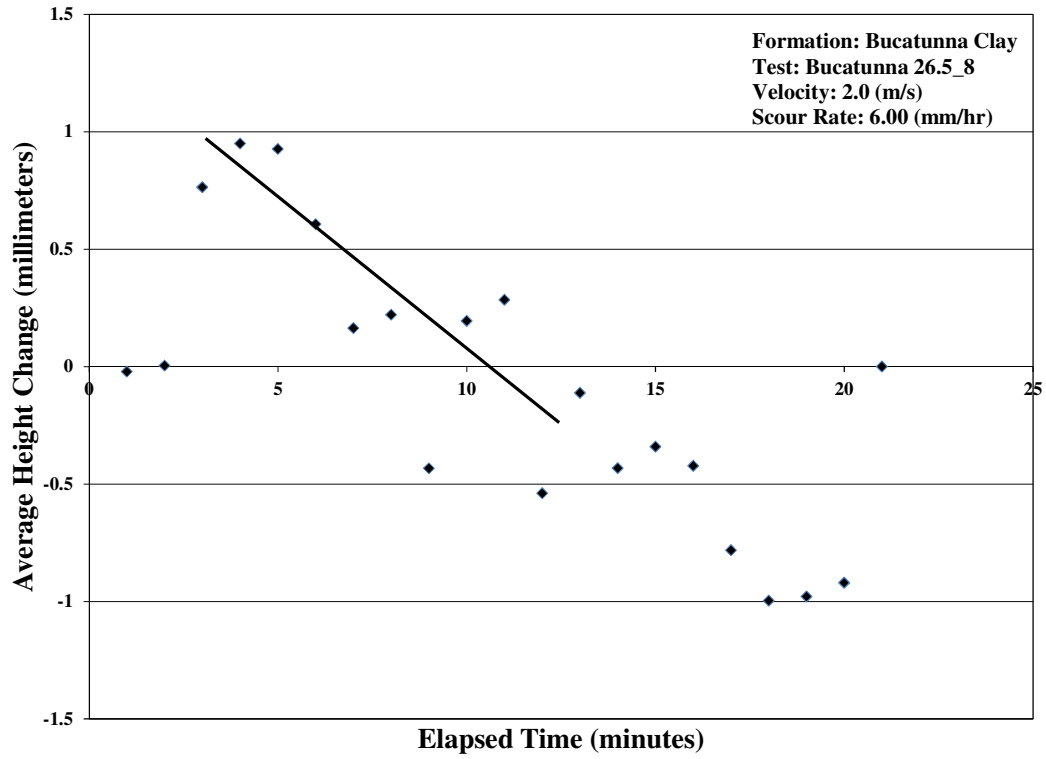


Figure A-20. "Bucatunna 26.5_8" Test Results

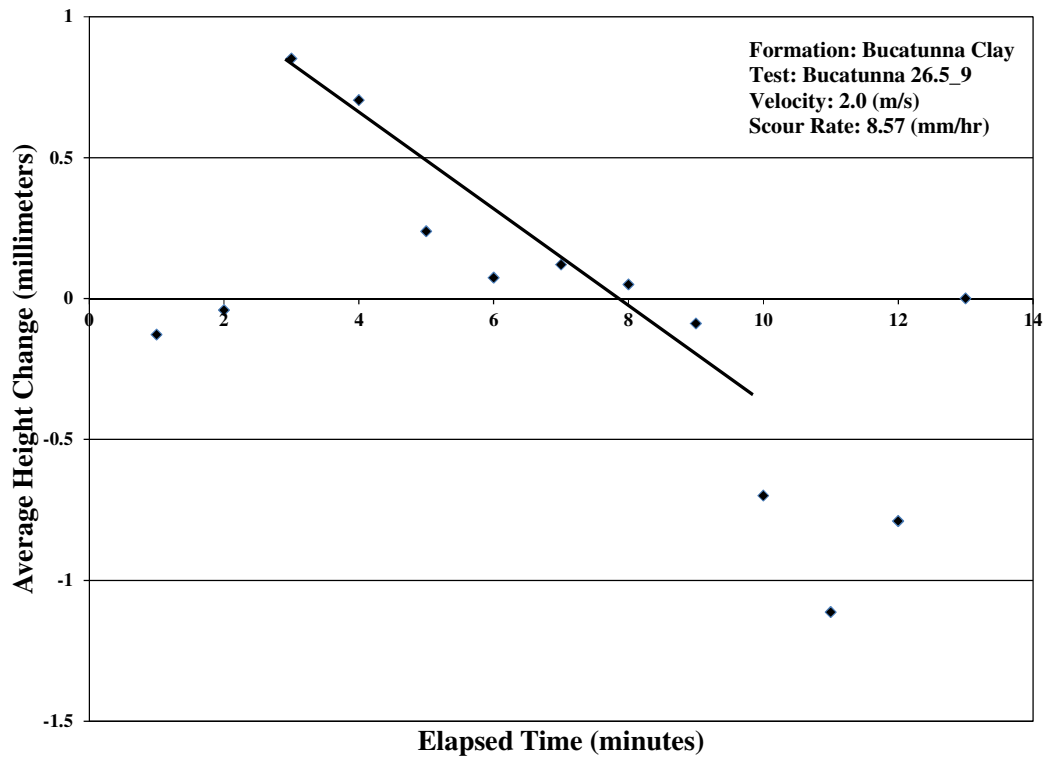


Figure A-21. "Bucatunna 26.5_9" Test Results

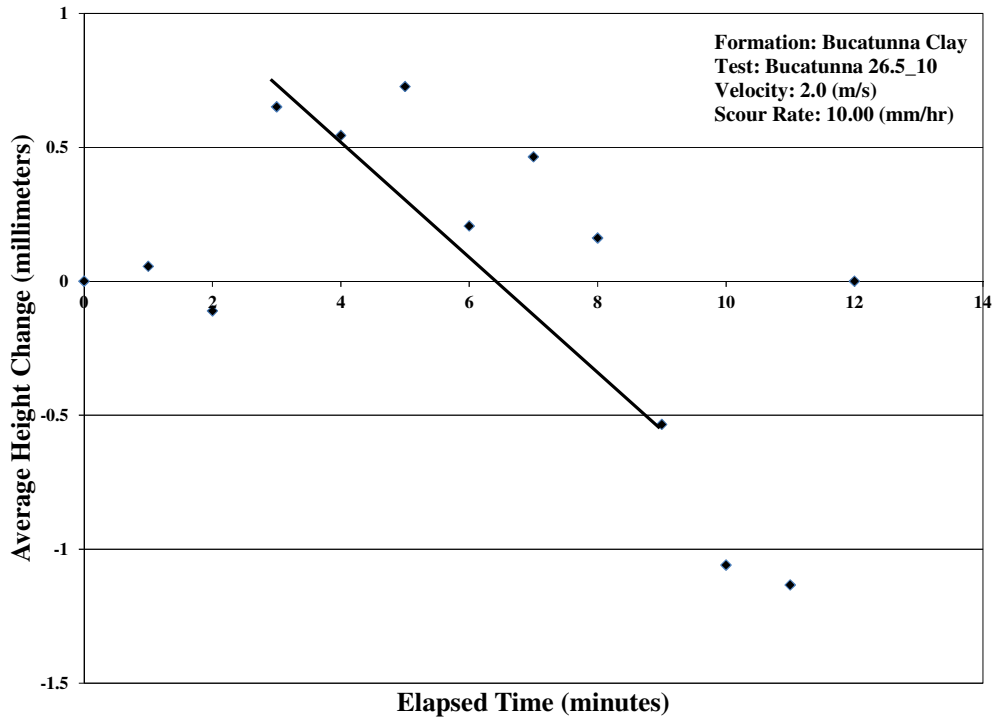


Figure A-22. "Bucatanna 26.5_10" Test Results

TEST VELOCITY: 3.0 METERS PER SECOND

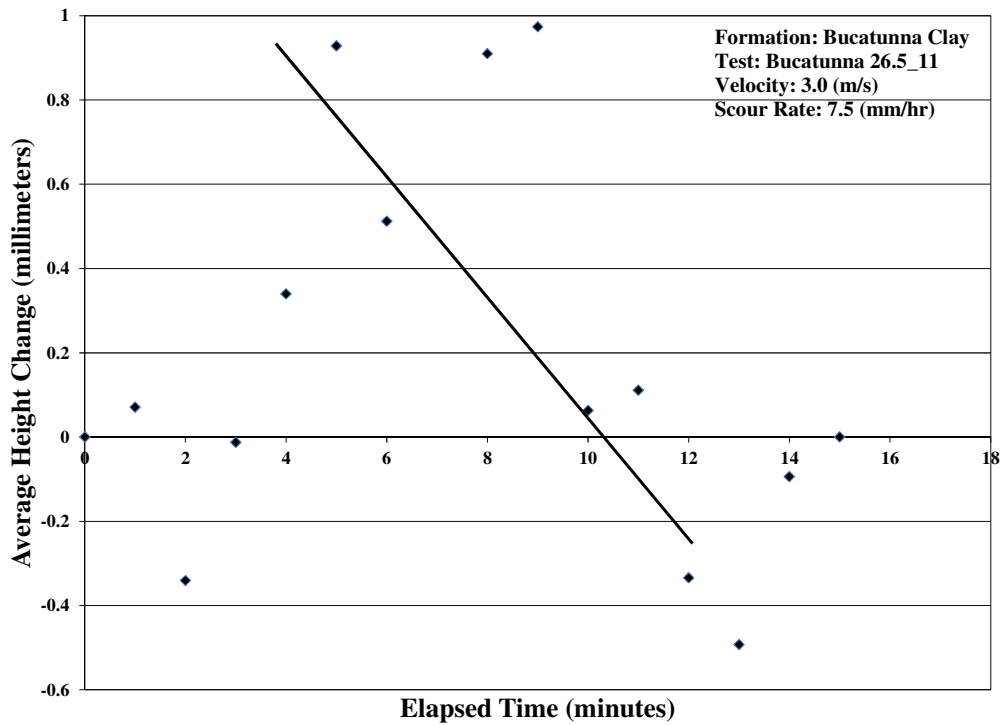


Figure A-23. "Bucatanna 26.5_11" Test Results

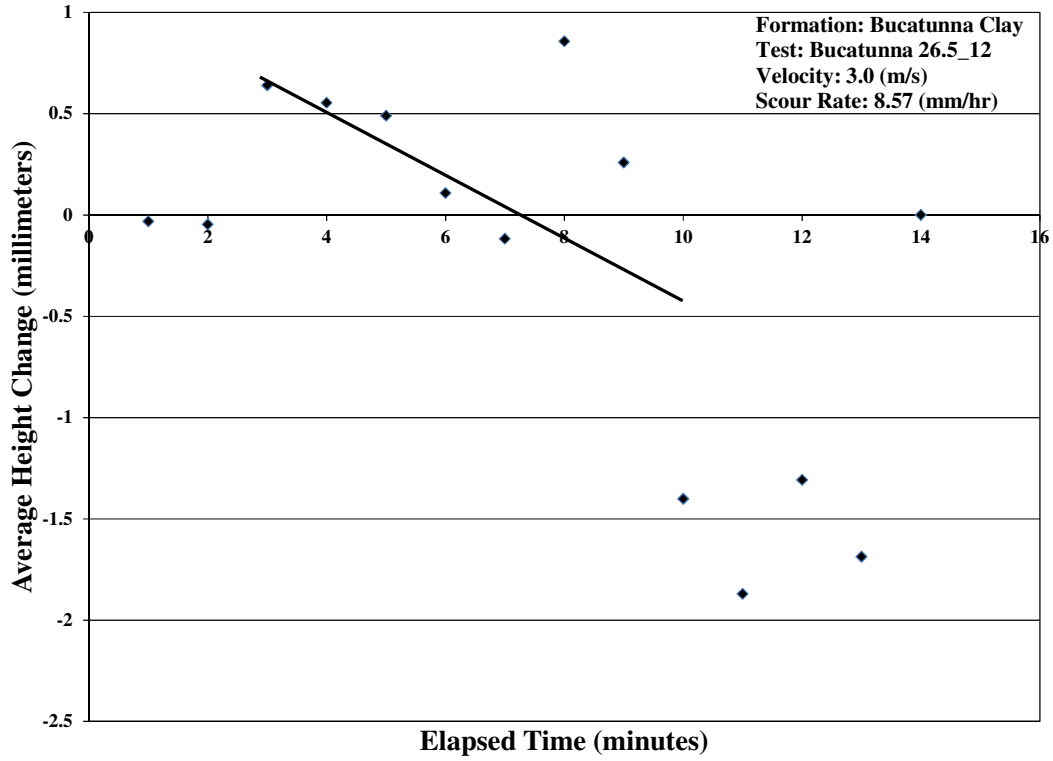


Figure A-24. "Bucatunna 26.5_12" Test Results

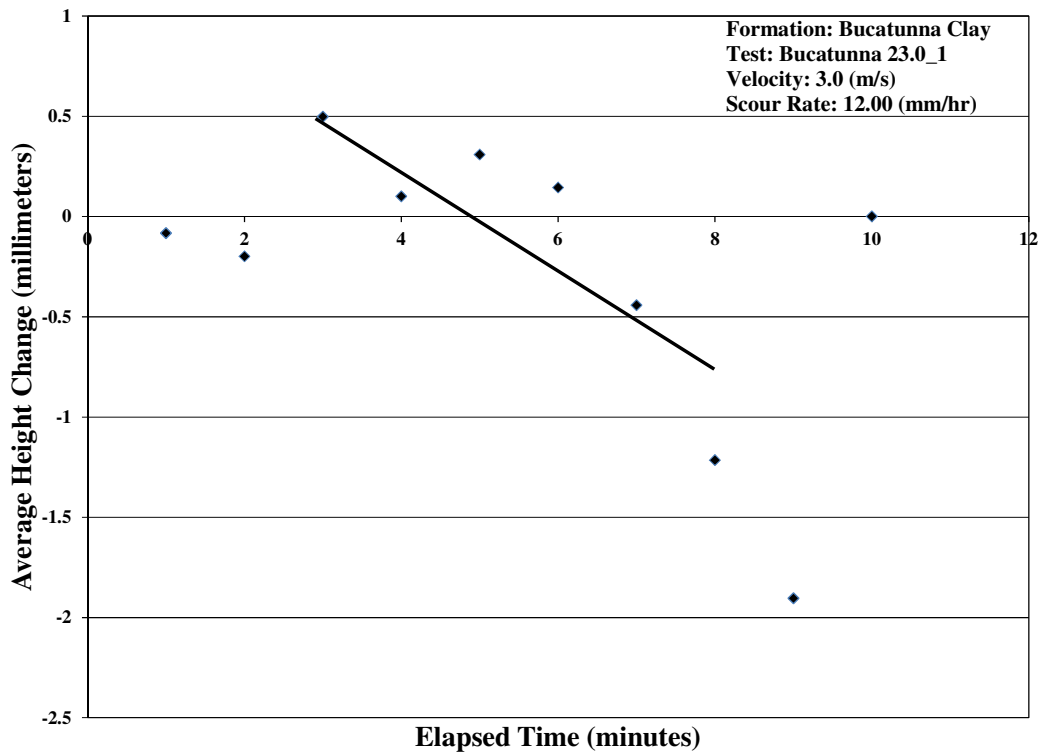


Figure A-25. "Bucatunna 23.0_1" Test Results

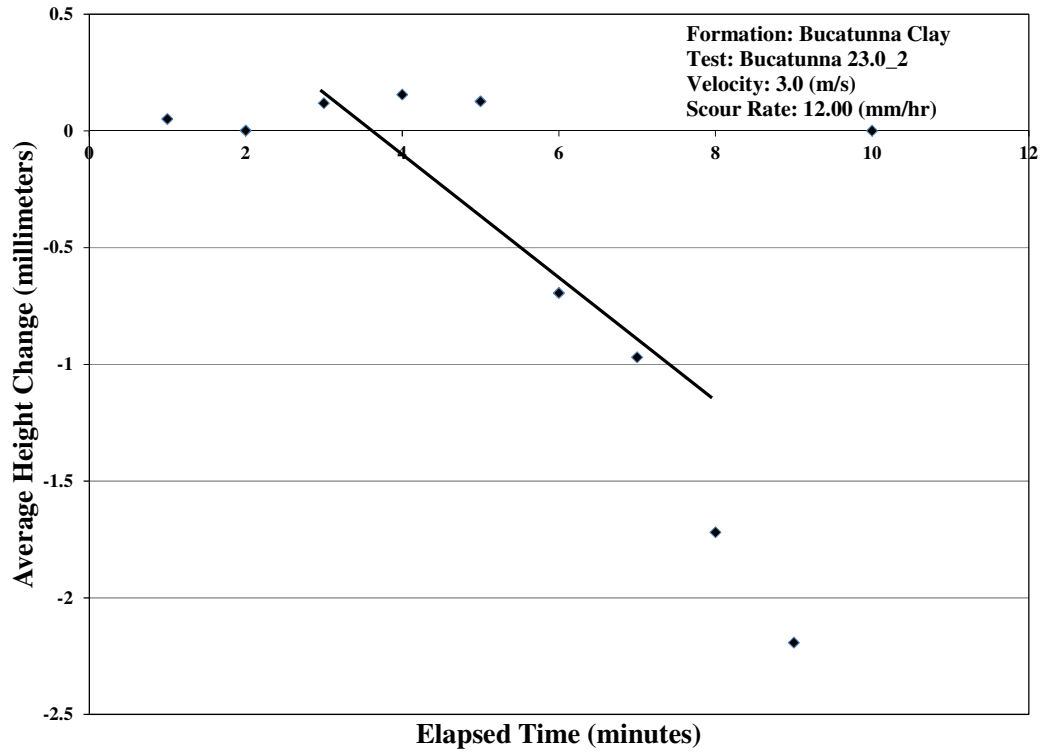


Figure A-26. "Bucatumna 23.0_2" Test Results

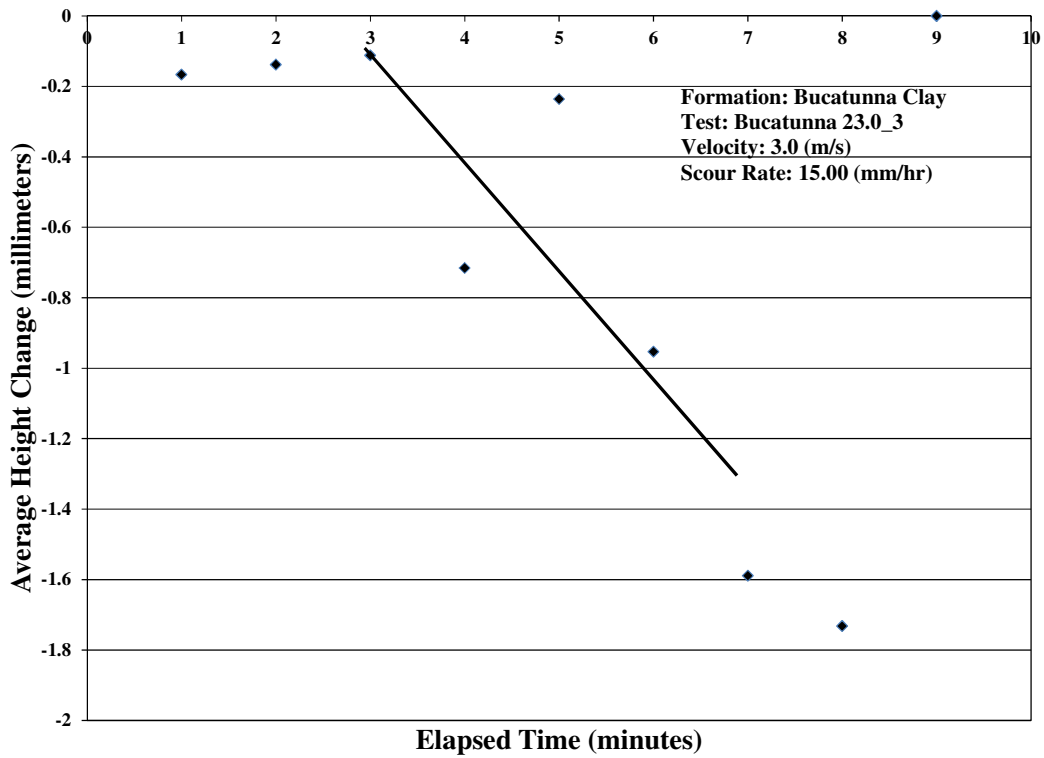


Figure A-27. "Bucatumna 23.0_3" Test Results

THRESHOLD VELOCITY TEST

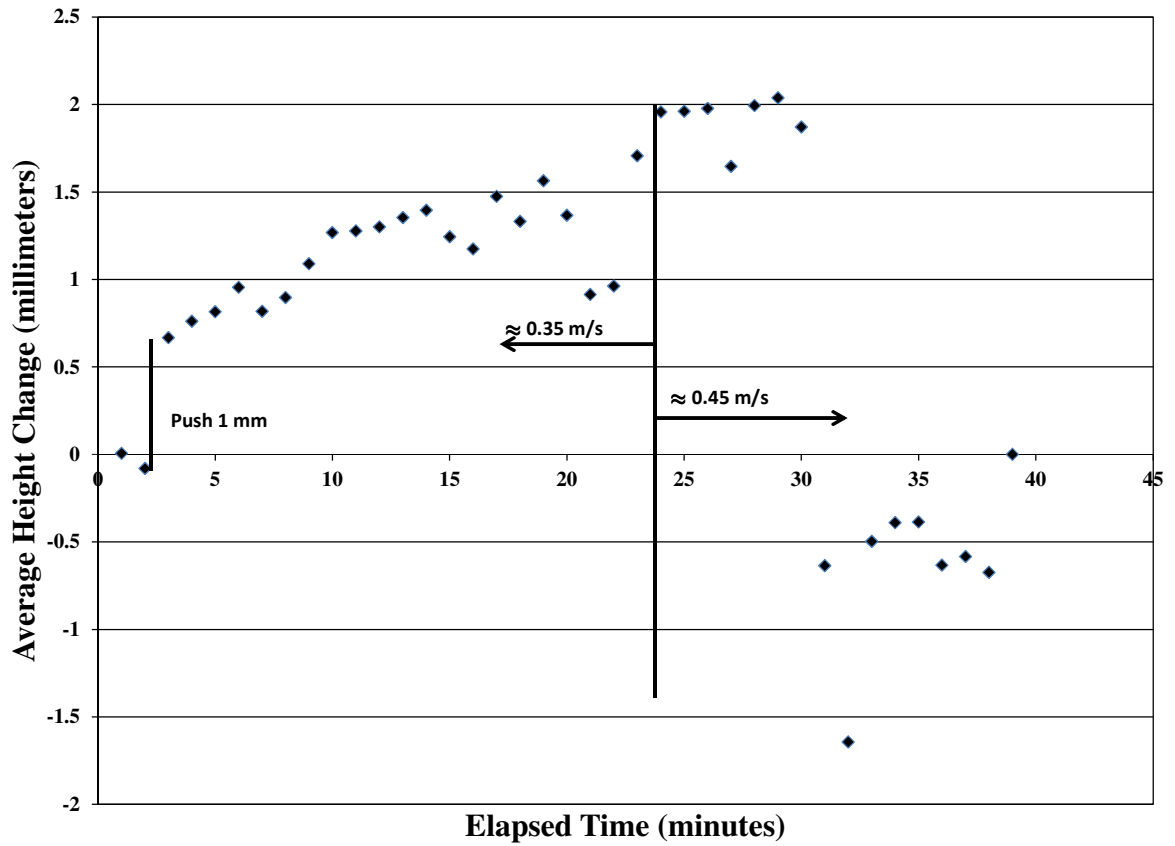


Figure A-28. "Bucatanna 23.0_4" Test Results

GEOTECHNICAL TESTING RESULTS

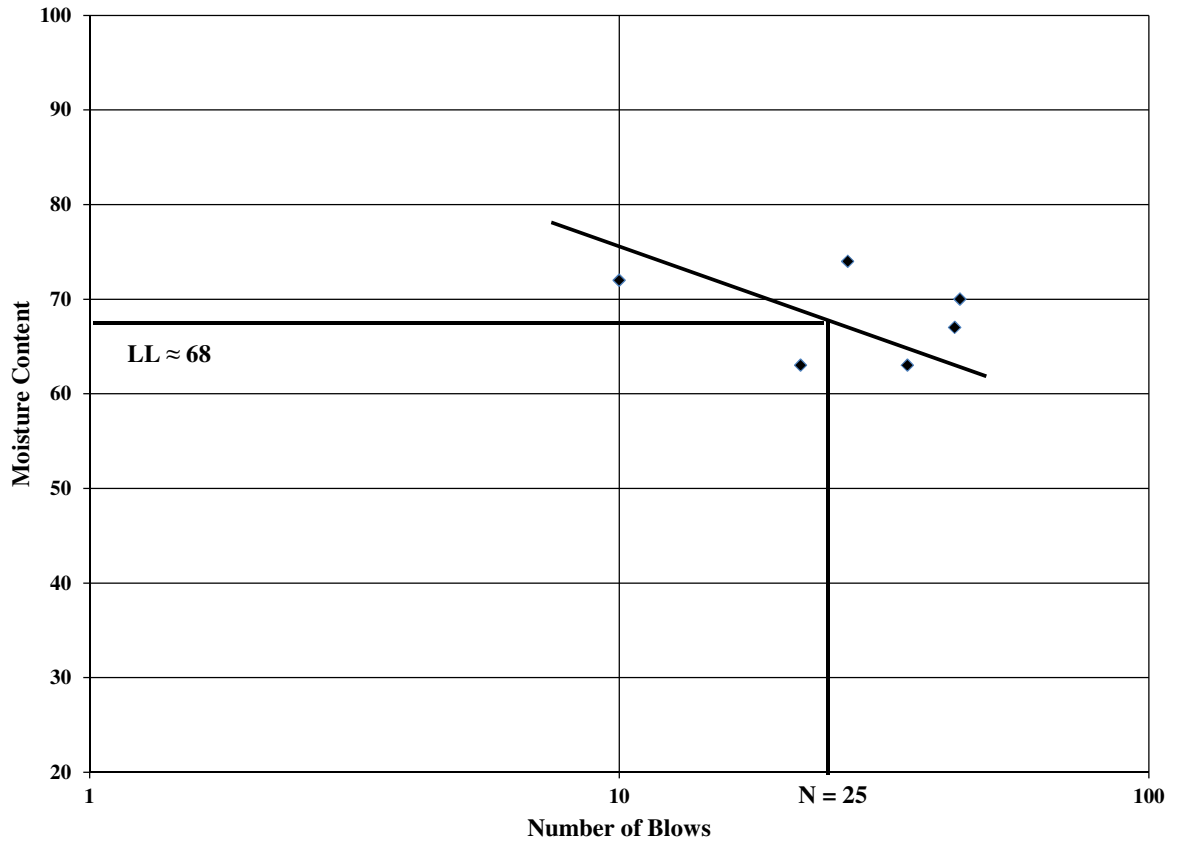


Figure A-29. Bucatunna Clay Liquid Limits Results

APPENDIX B

EFA AND GEOTECHNICAL TEST RESULTS:

YAZOO CLAY FORMATION

INITIAL SCOUR TEST

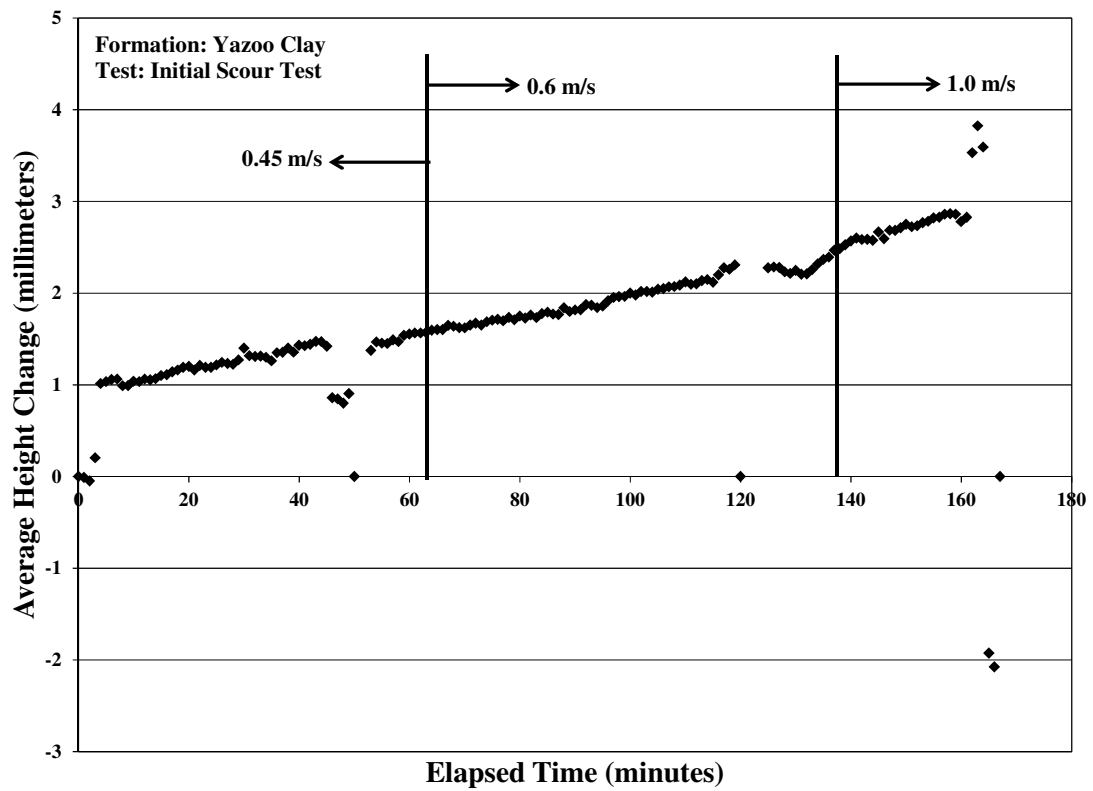


Figure B-1. "Yazoo Clay 18.5_2" Test Results

TEST VELOCITY: 1.0 METERS PER SECOND

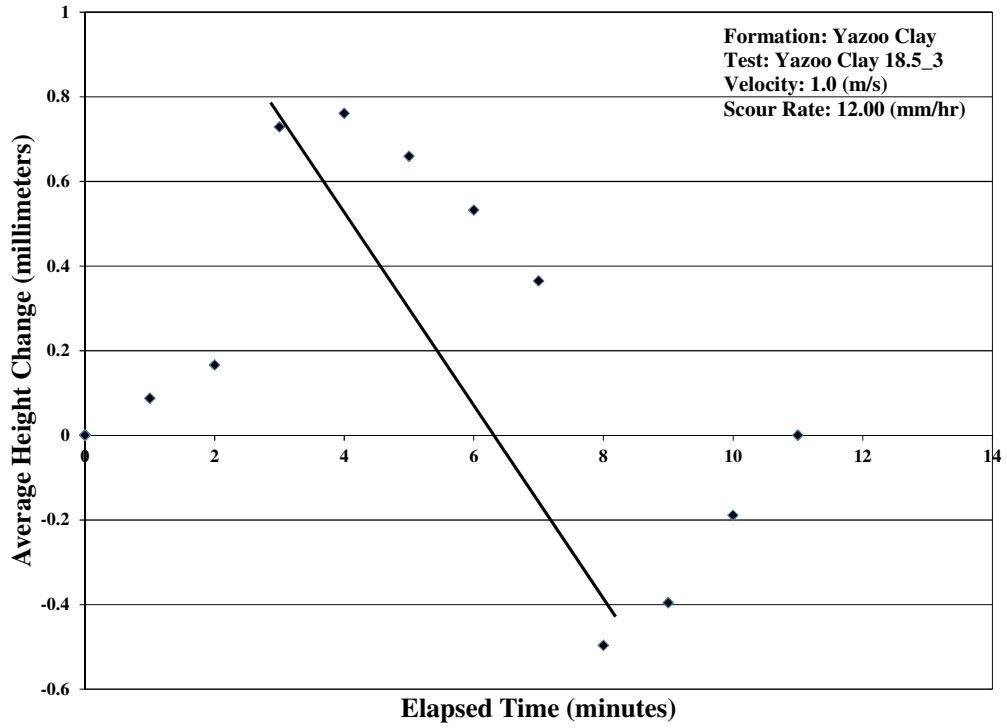


Figure B-2. “Yazoo Clay 18.5_3” Test Results

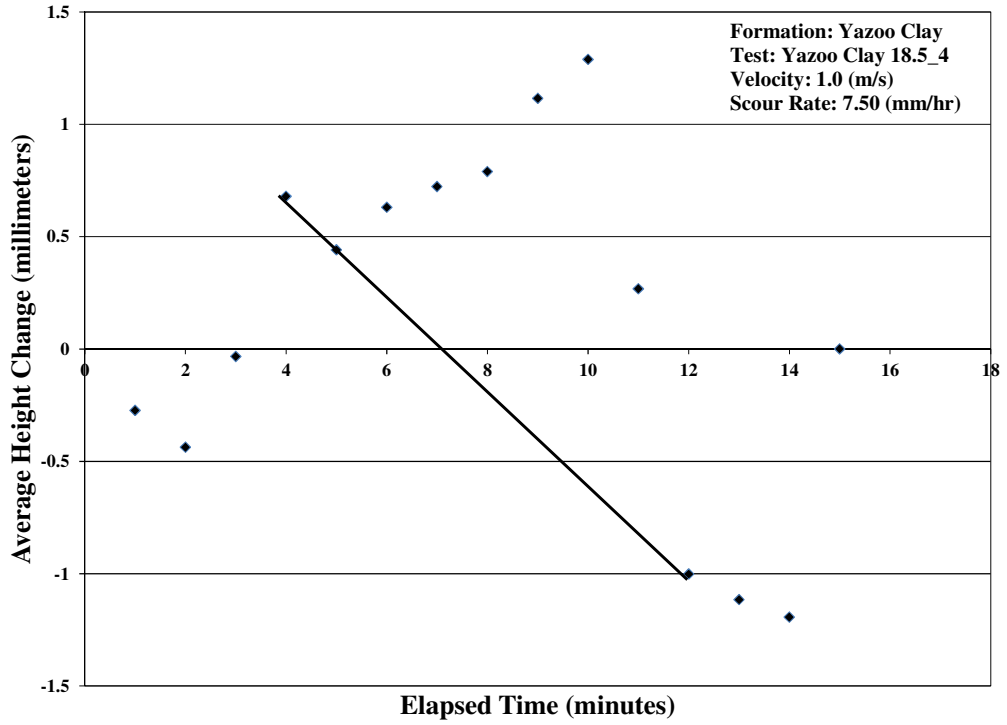


Figure B-3. “Yazoo Clay 18.5_4” Test Results

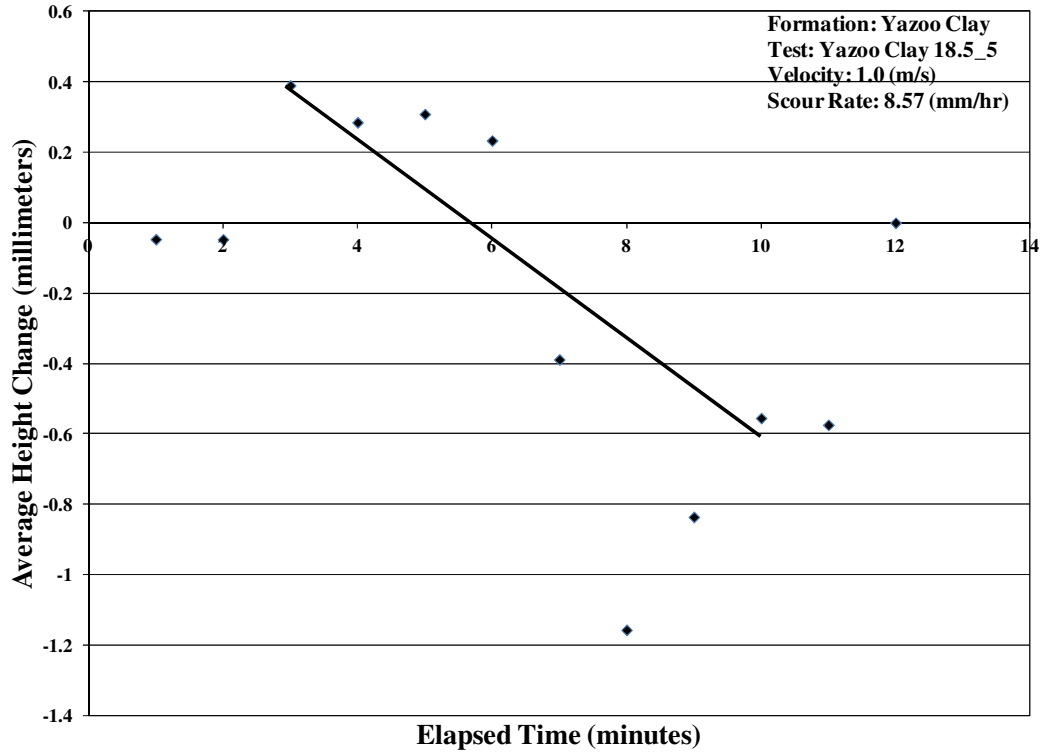


Figure B-4. "Yazoo Clay 18.5_5" Test Results

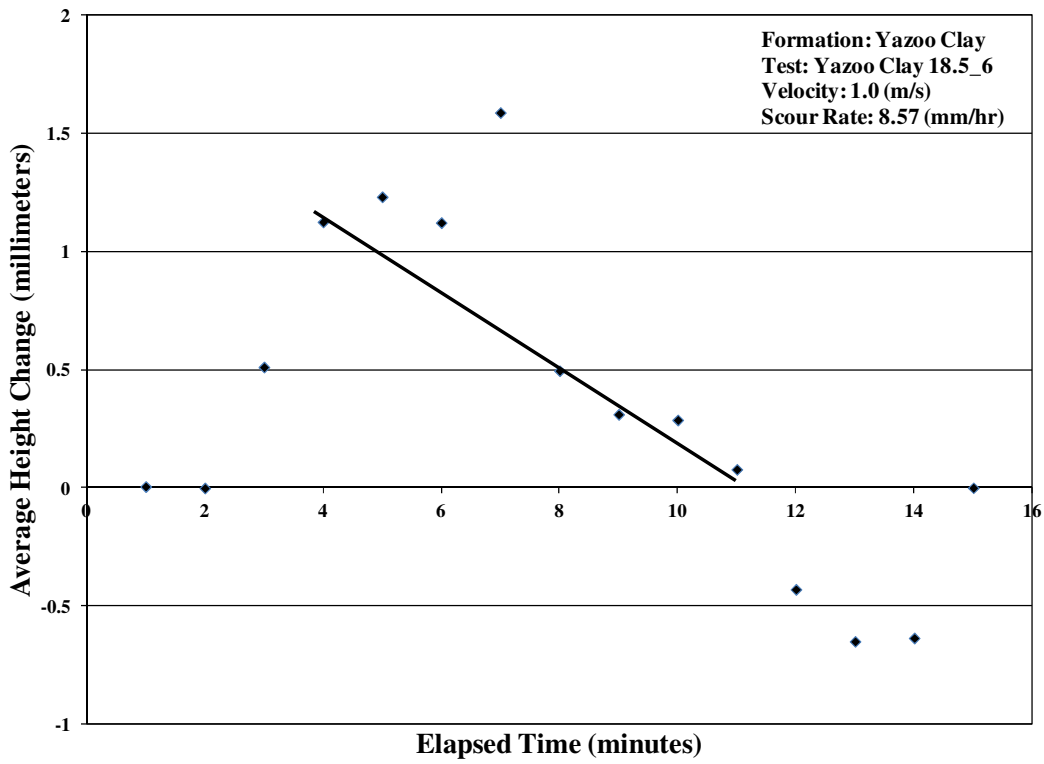


Figure B-5. "Yazoo Clay 18.5_6" Test Results

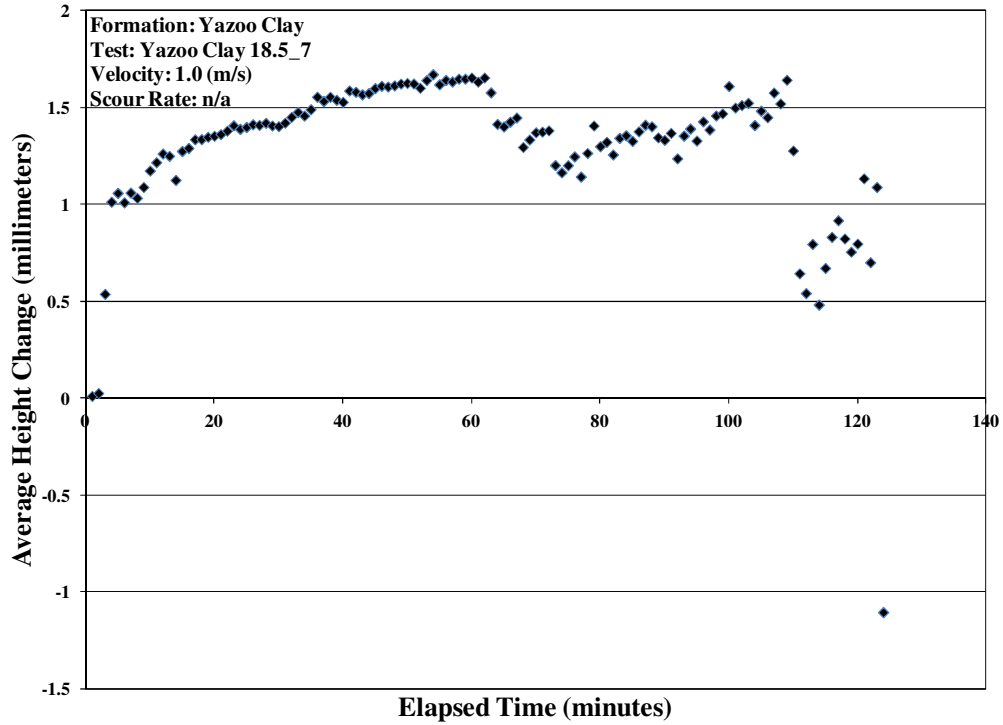


Figure B-6. “Yazoo Clay 18.5_7” Test Results

TEST VELOCITY: 1.5 METERS PER SECOND

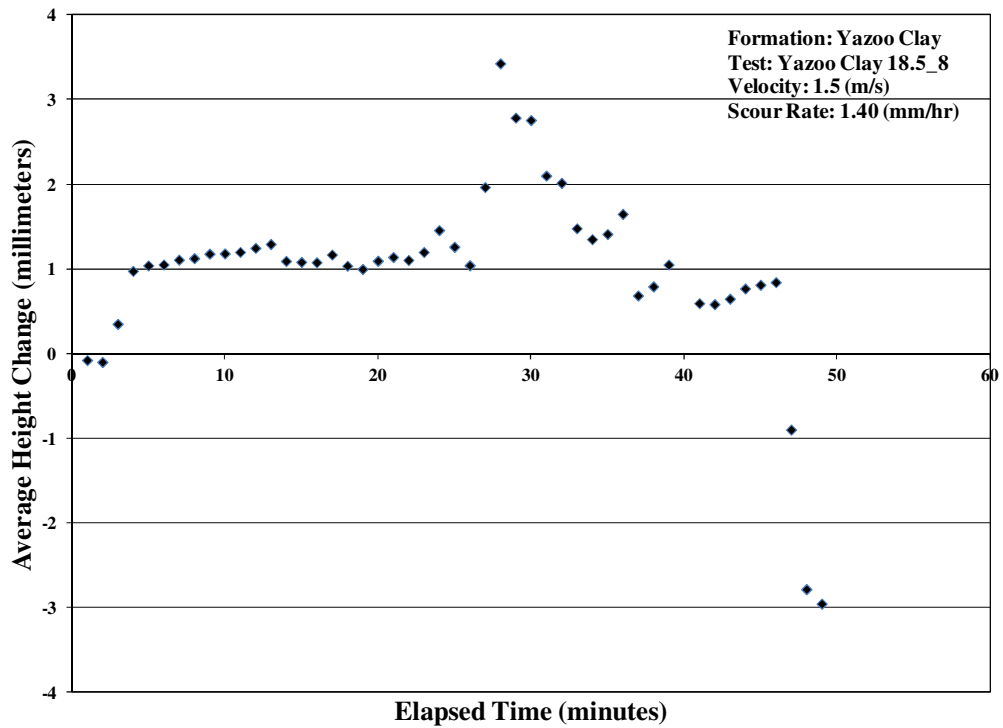


Figure B-7. “Yazoo Clay 18.5_8” Test Results

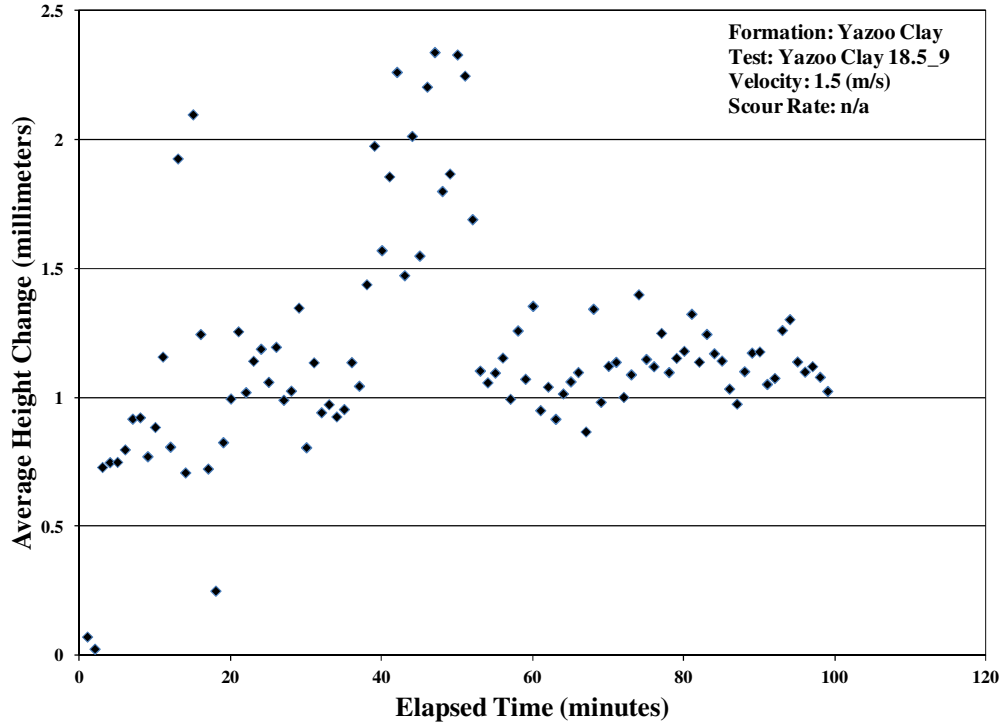


Figure B-8. “Yazoo Clay 18.5_9” Test Results

TEST VELOCITY: 2 METERS PER SECOND

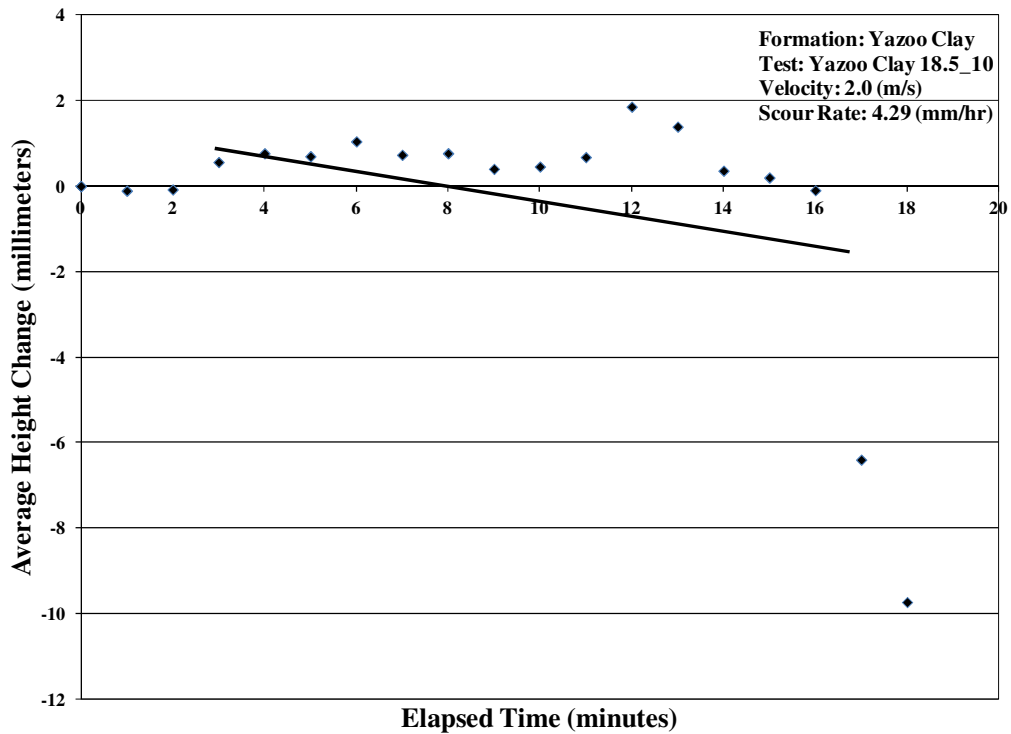


Figure B-9. “Yazoo Clay 18.5_10” Test Results

GEOTECHNICAL TESTING RESULTS

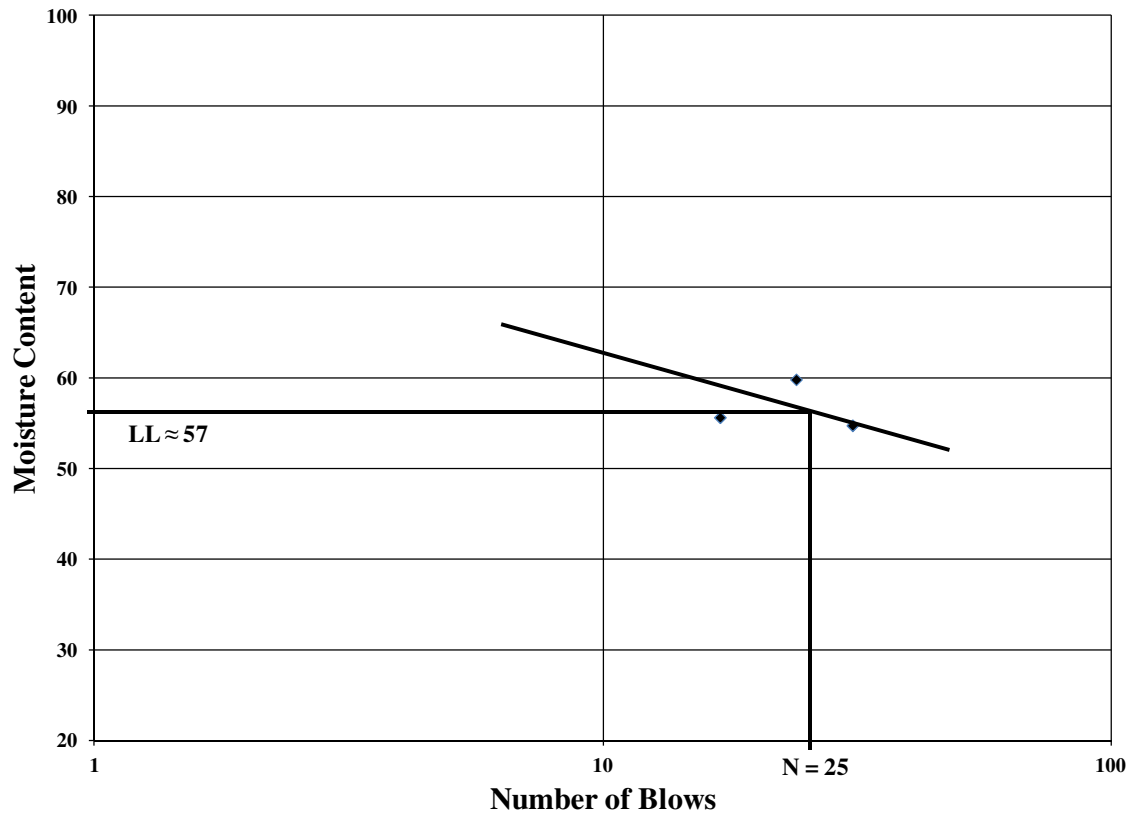


Figure B-10. Yazoo Clay Liquid Limits Results

APPENDIX C

EFA AND GEOTECHNICAL TEST RESULTS:

DEMOPOLIS CHALK FORMATION

INITIAL SCOUR TEST

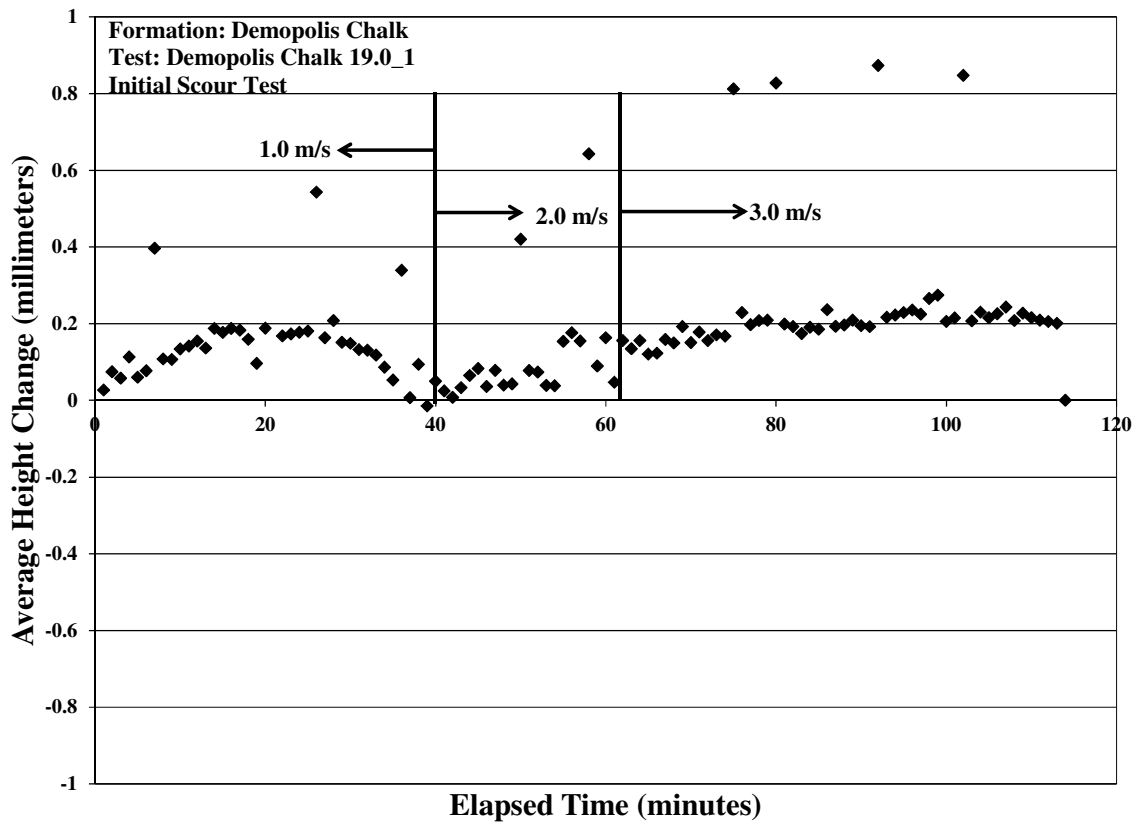


Figure C-1. "Demopolis Chalk 19.0_1" Test Results

TEST VELOCITY: 3.0 METERS PER SECOND

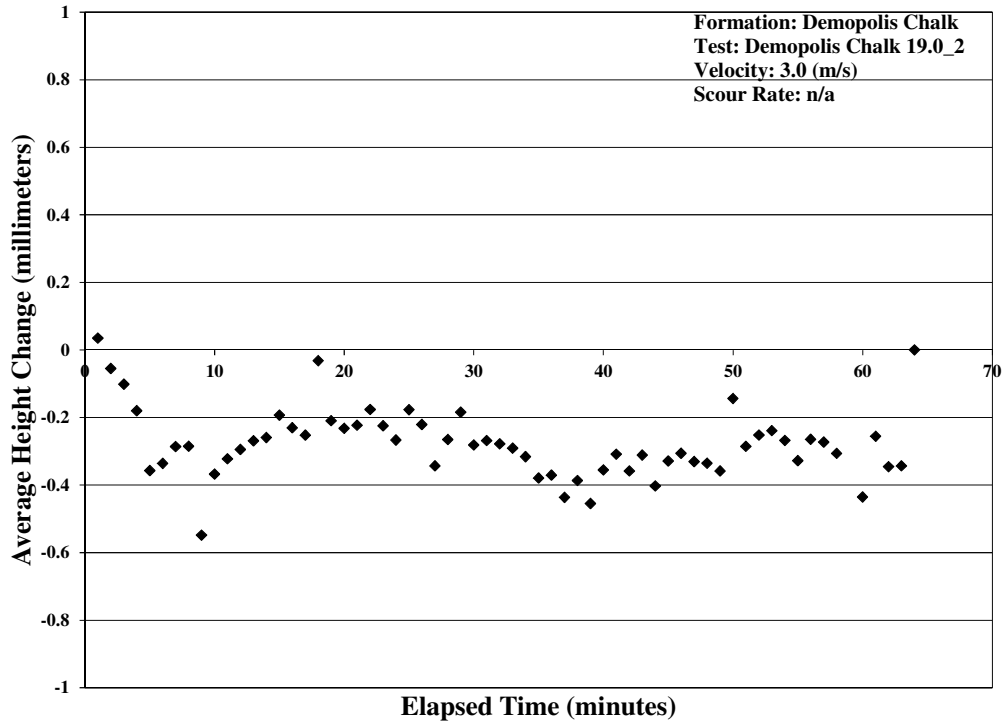


Figure C-2. “Demopolis Chalk 19.0_2” Test Results

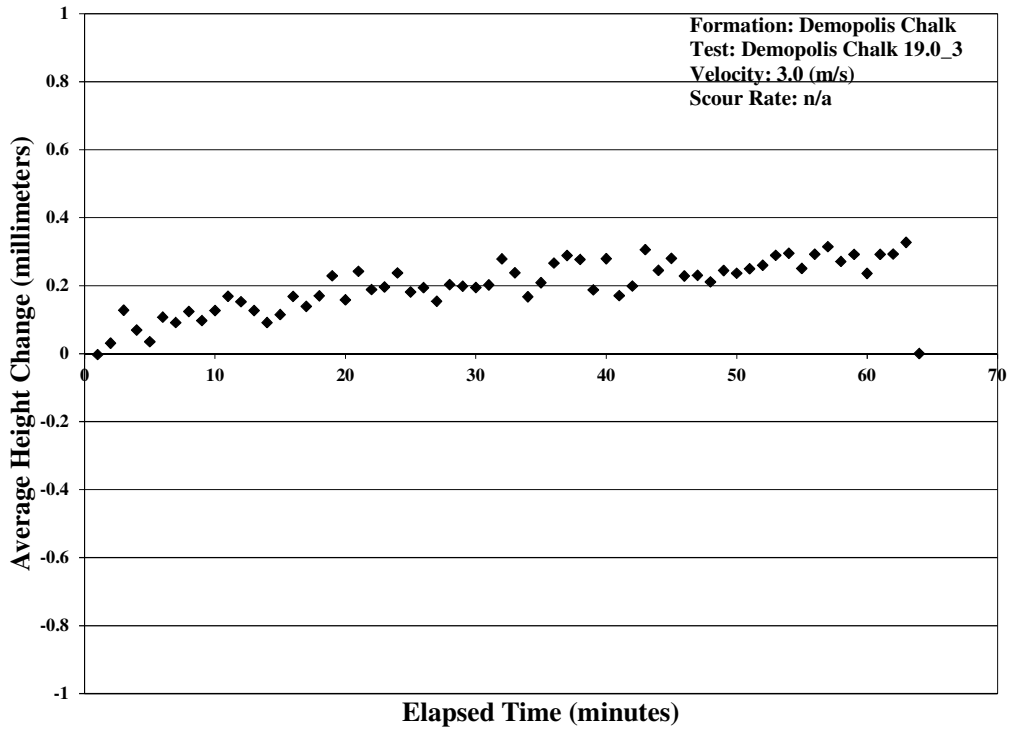


Figure C-3. “Demopolis Chalk 19.0_3” Test Results

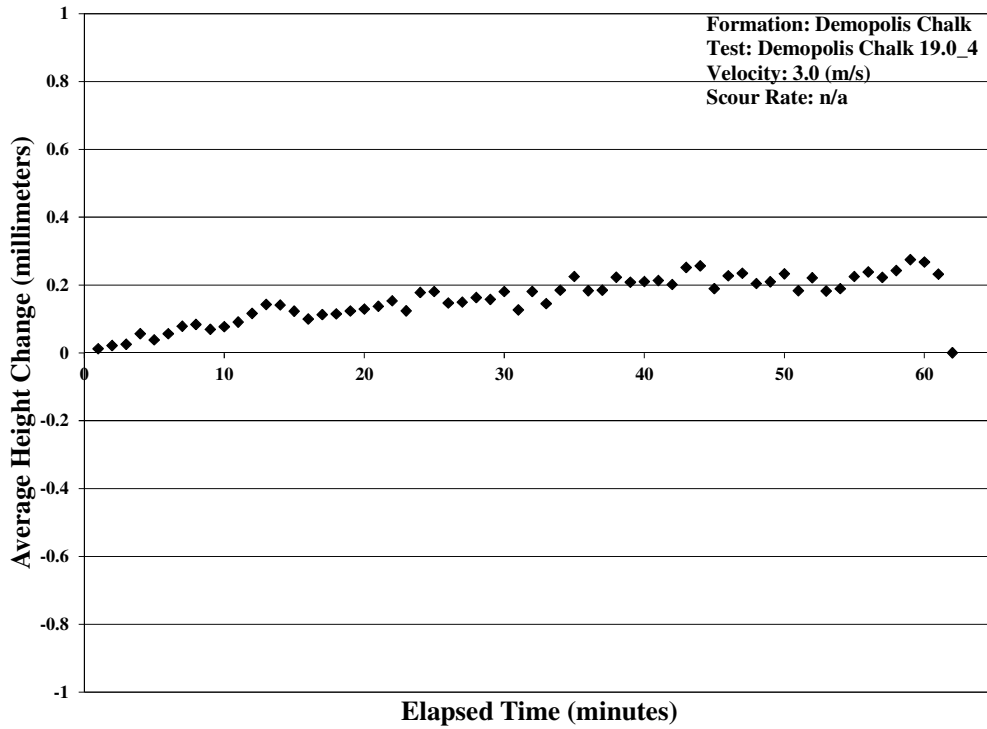


Figure C-4. “Demopolis Chalk 19.0_4” Test Results

REPEAT EVENT TEST

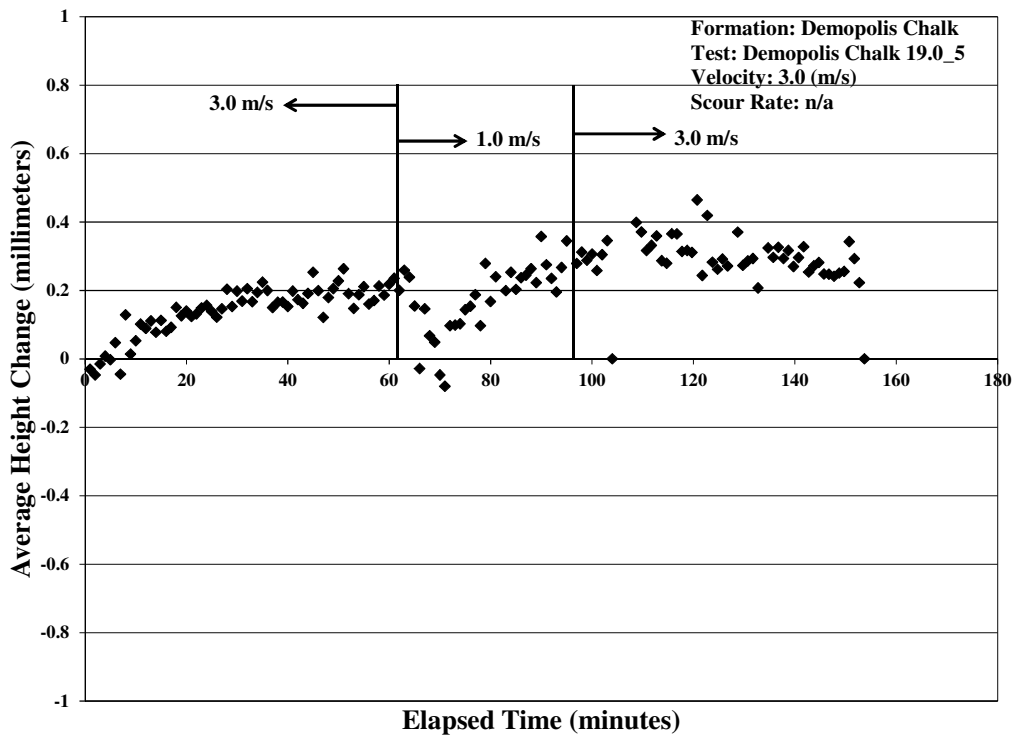


Figure C-5. “Demopolis Chalk 19.0_5” Test Results

GEOTECHNICAL TESTING RESULTS

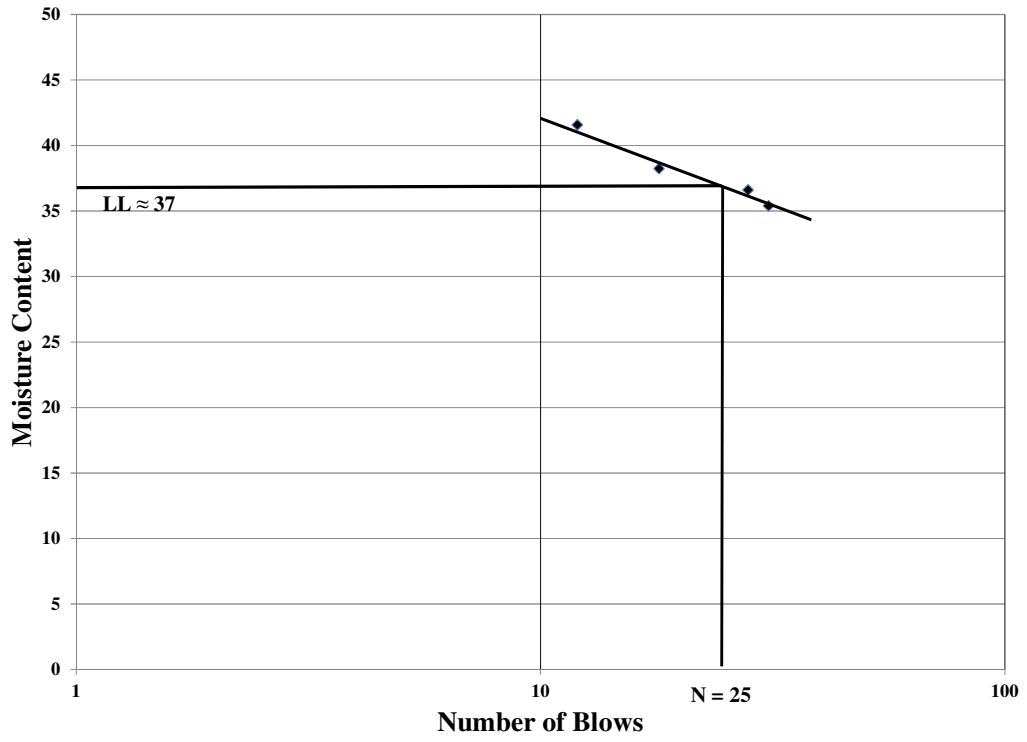


Figure C-6. Demopolis Chalk Liquid Limit Results

APPENDIX D

EFA AND GEOTECHNICAL TEST RESULTS:

MOOREVILLE CHALK FORMATION

INITIAL SCOUR TEST

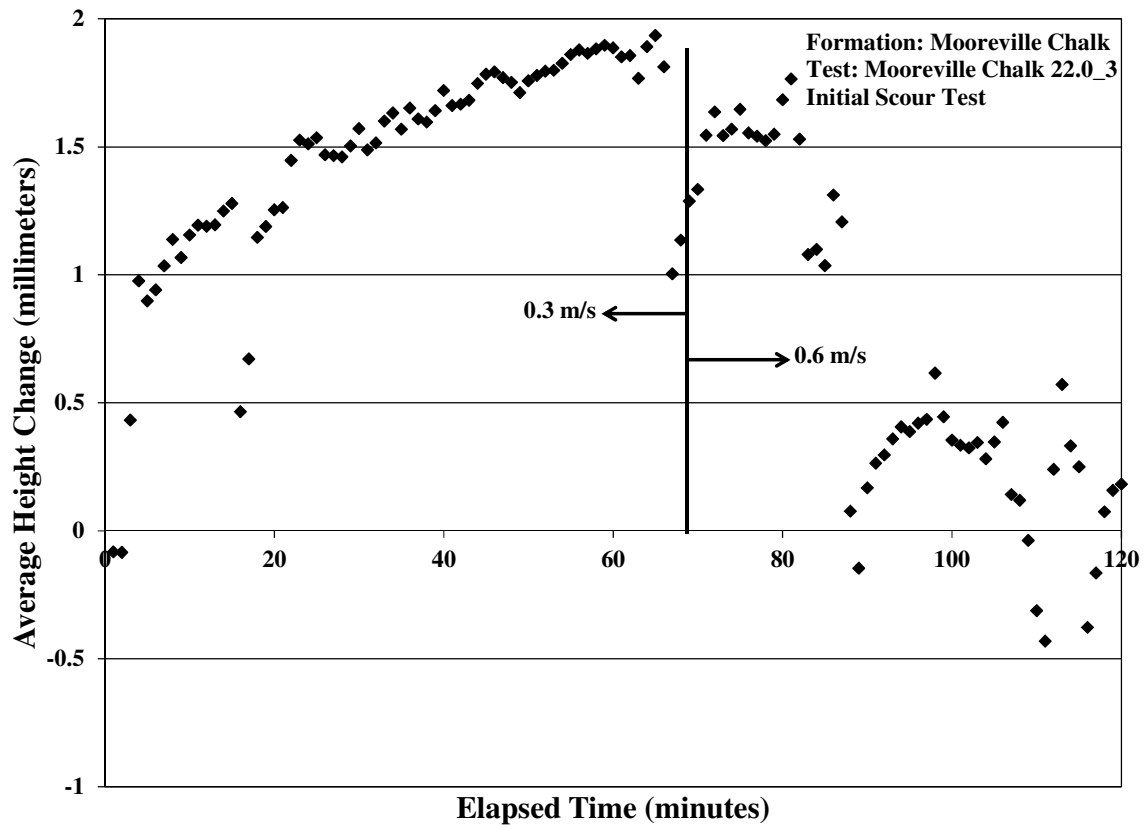


Figure D-1. "Mooreville Chalk 22.0_3" Test Results

TEST VELOCITY: 0.6 METERS PER SECOND

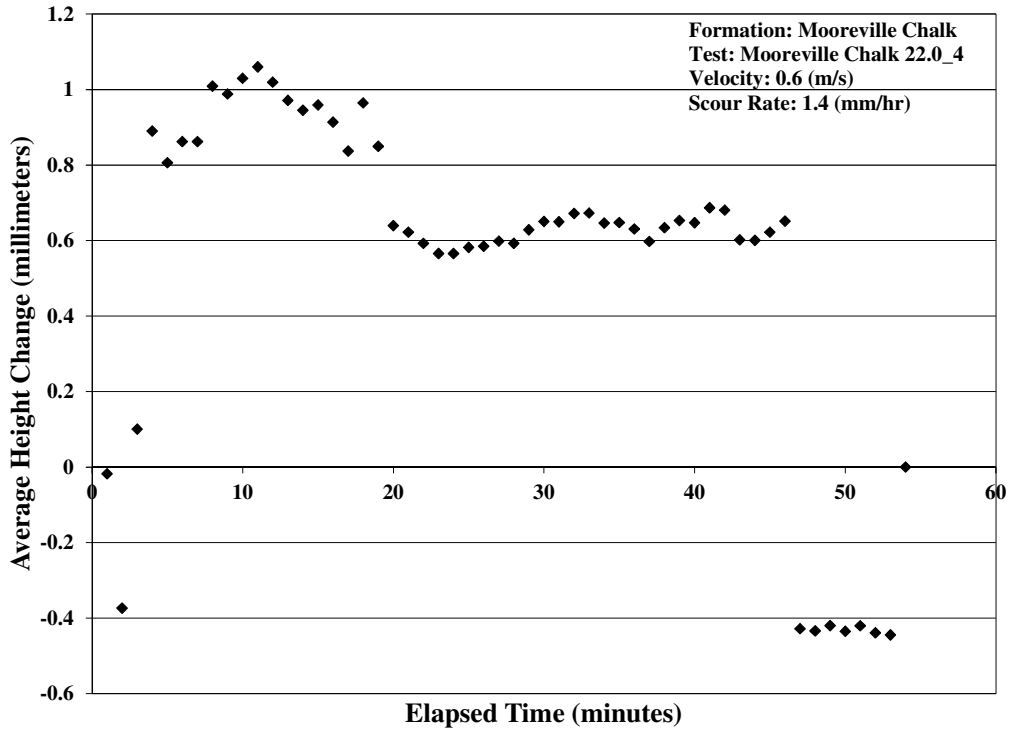


Figure D-2. “Mooreville Chalk 22.0_4” Test Results

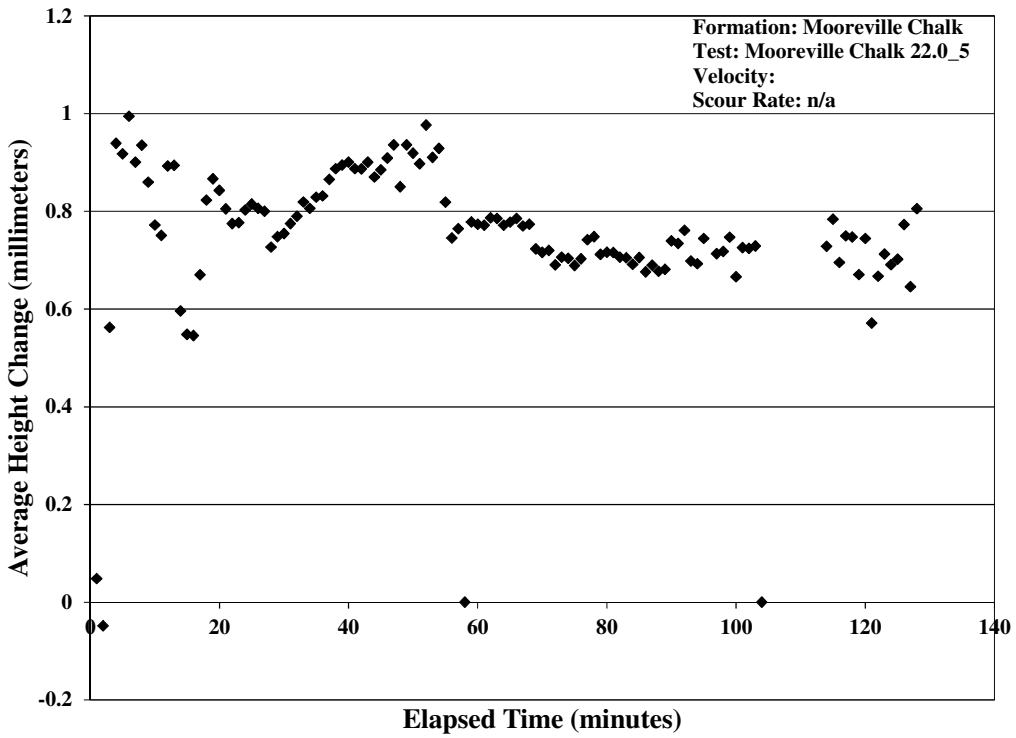


Figure D-3. “Mooreville Chalk 22.0_5” Test Results

TEST VELOCITY: 1.0 METERS PER SECOND

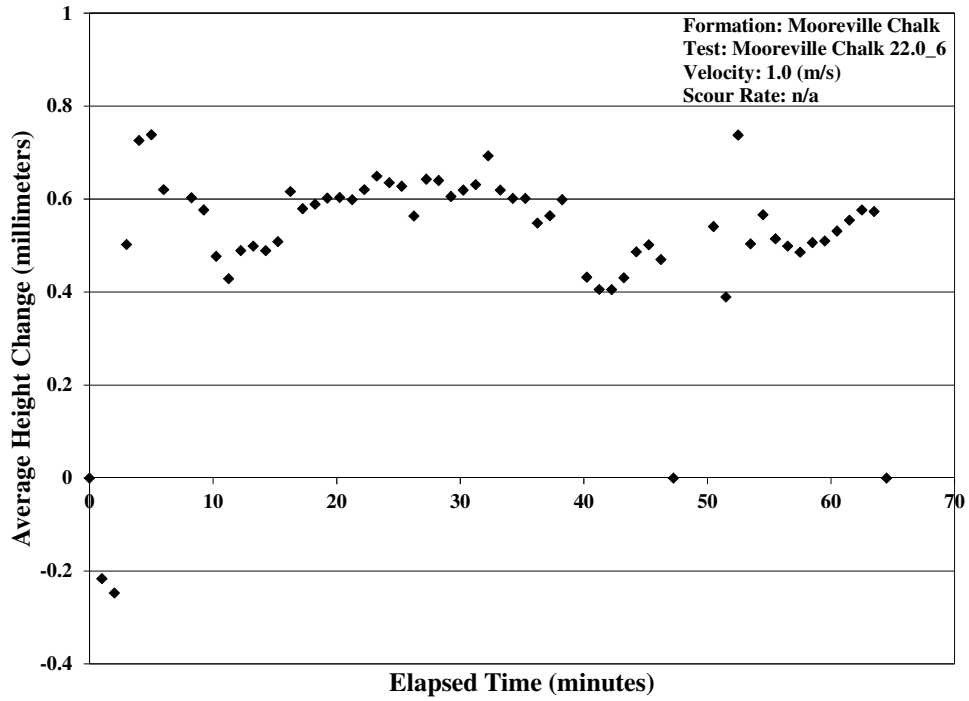


Figure D-4. “Mooreville Chalk 22.0_6” Test Results

TEST VELOCITY: 1.5 METERS PER SECOND

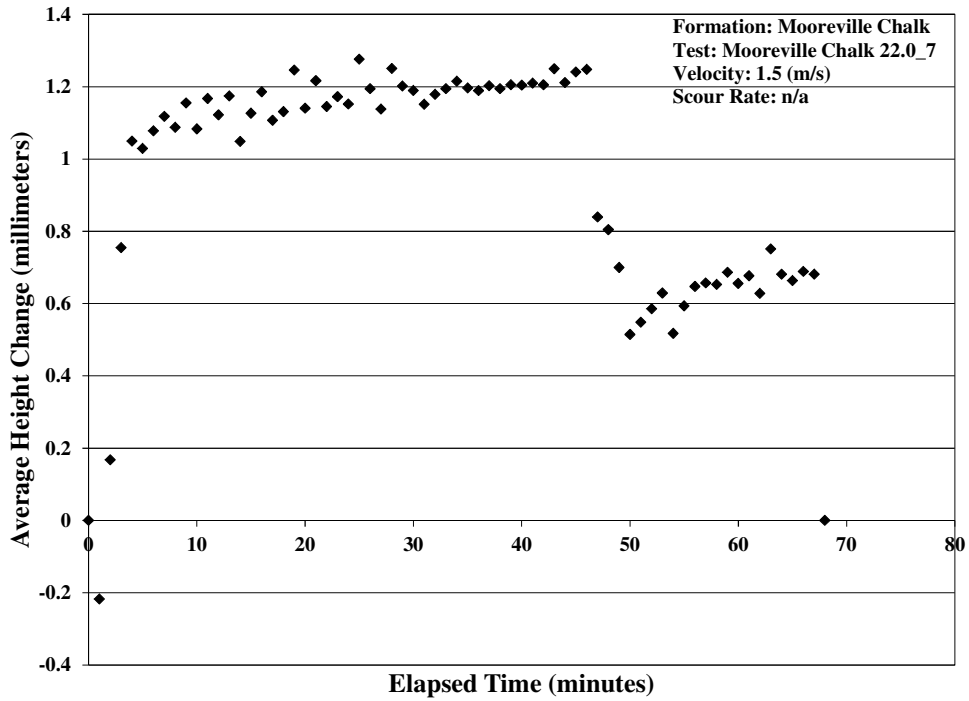


Figure D-5. “Mooreville Chalk 22.0_7” Test Results

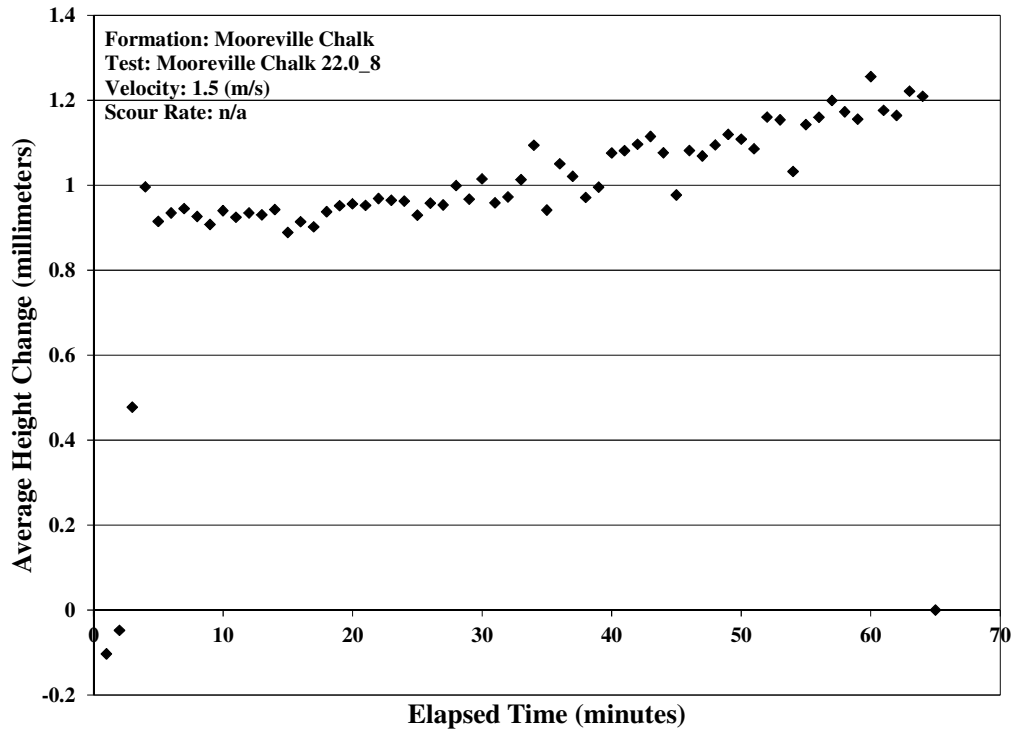


Figure D-6. “Mooreville Chalk 22.0_8” Test Results

TEST VELOCITY: 2.0 METERS PER SECOND

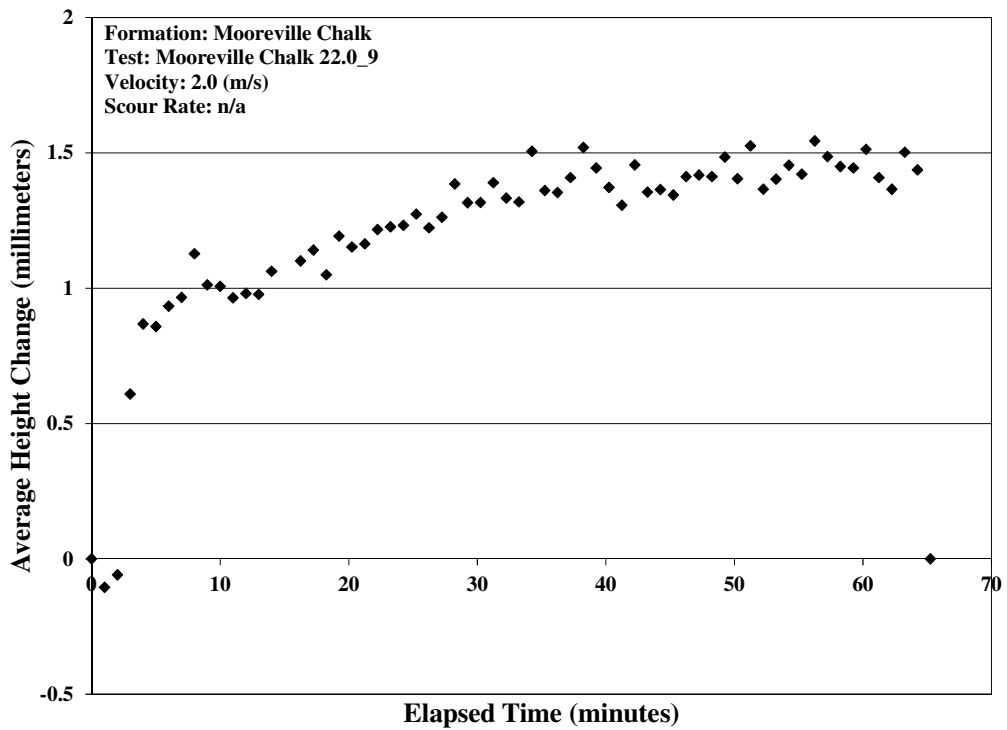


Figure D-7. “Mooreville Chalk 22.0_9” Test Results

TEST VELOCITY: 3.0 METERS PER SECOND

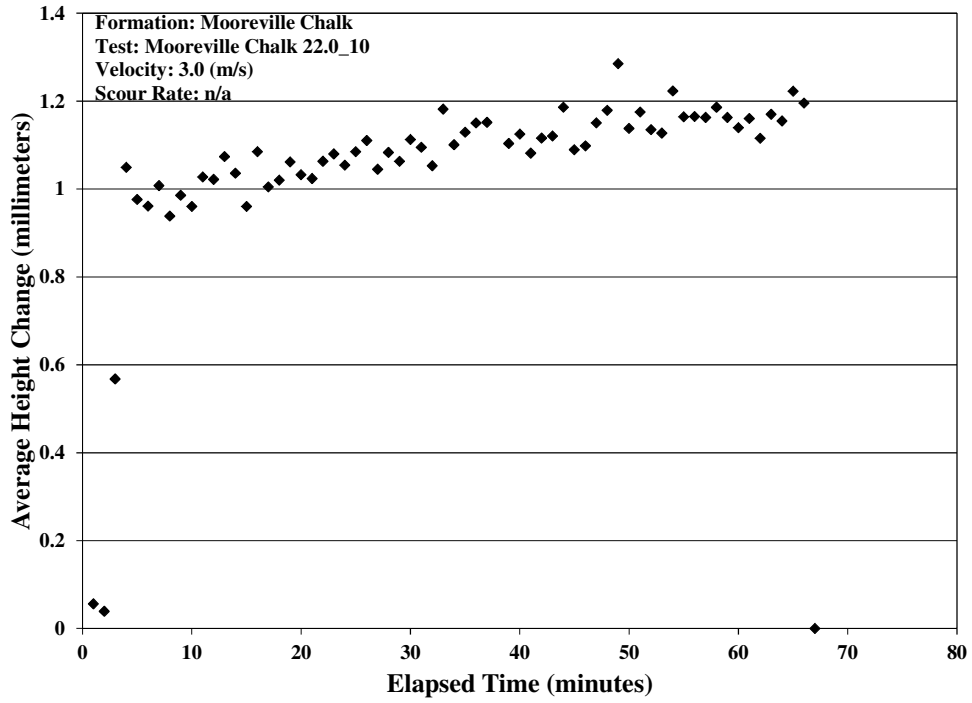


Figure D-8. “Mooreville Chalk 22.0_10” Test Results

REPEAT EVENT TEST

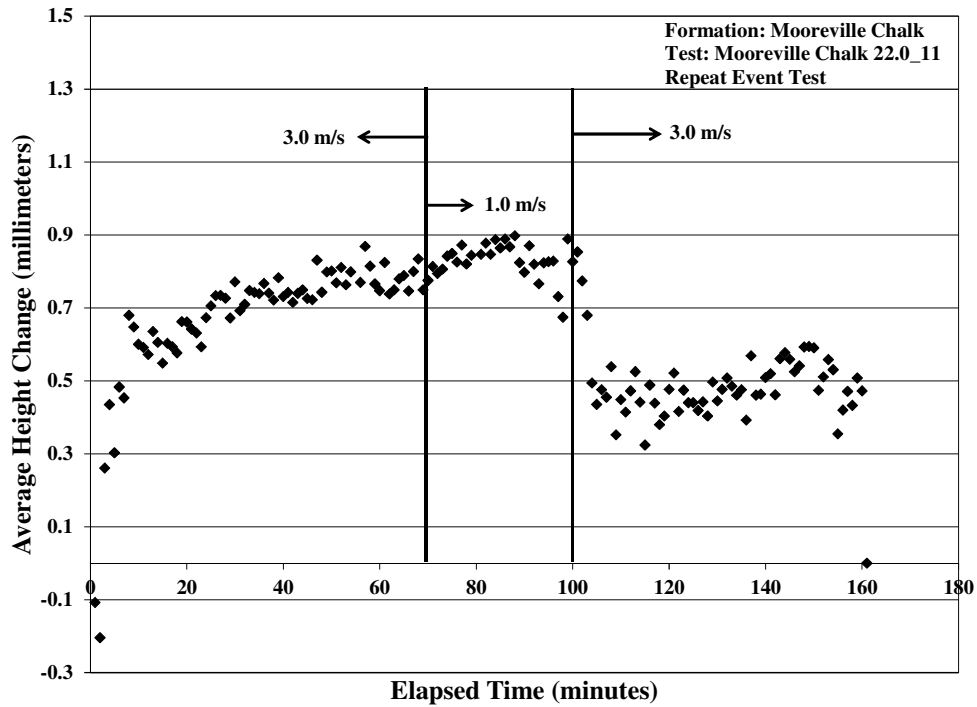


Figure D-9. “Mooreville Chalk 22.0_11” Test Results

GEOTECHNICAL TESTING RESULTS

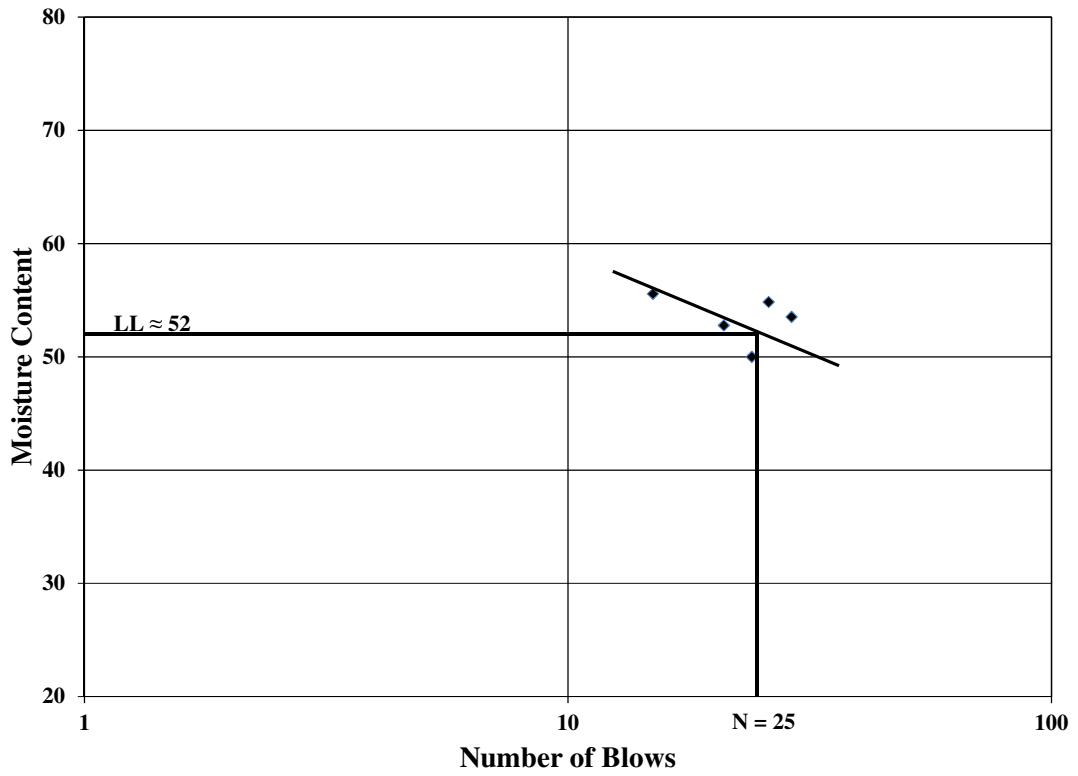


Figure D-10. Mooreville Chalk Liquid Limits Results

APPENDIX E

EFA AND GEOTECHNICAL TEST RESULTS:

PRAIRIE BLUFF CHALK FORMATION

TEST VELOCITY: 1.0 METERS PER SECOND

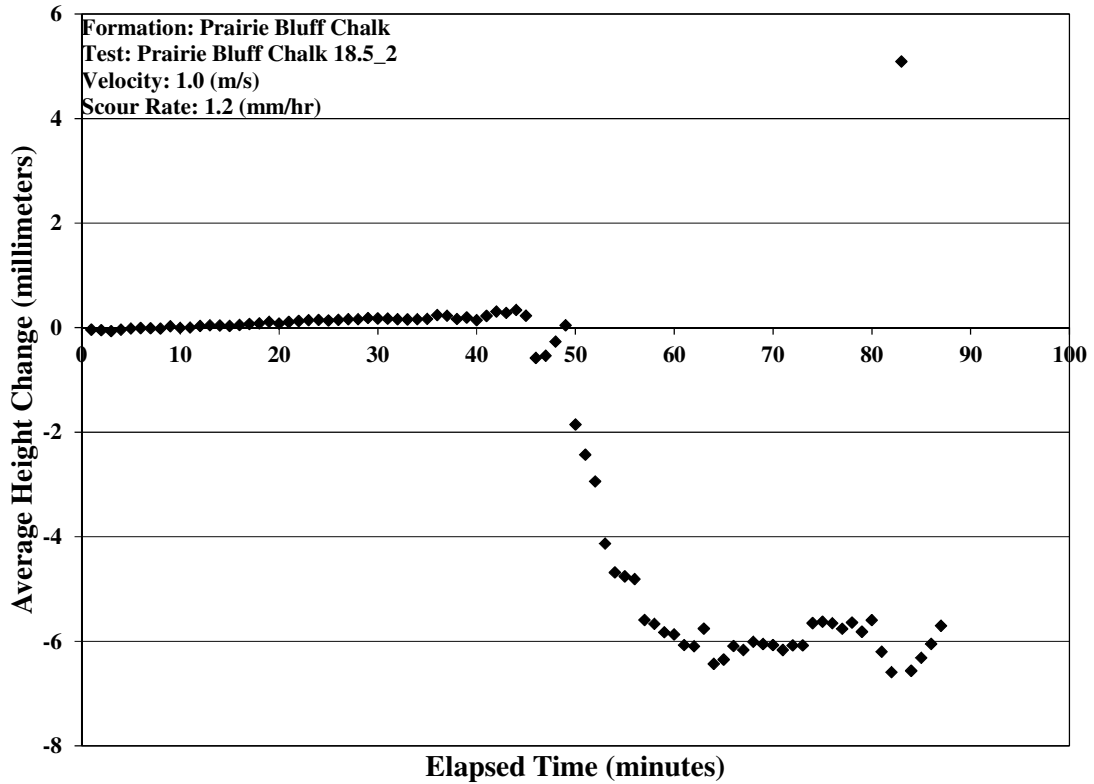


Figure E-1. "Prairie Bluff Chalk 18.5_2" Test Results

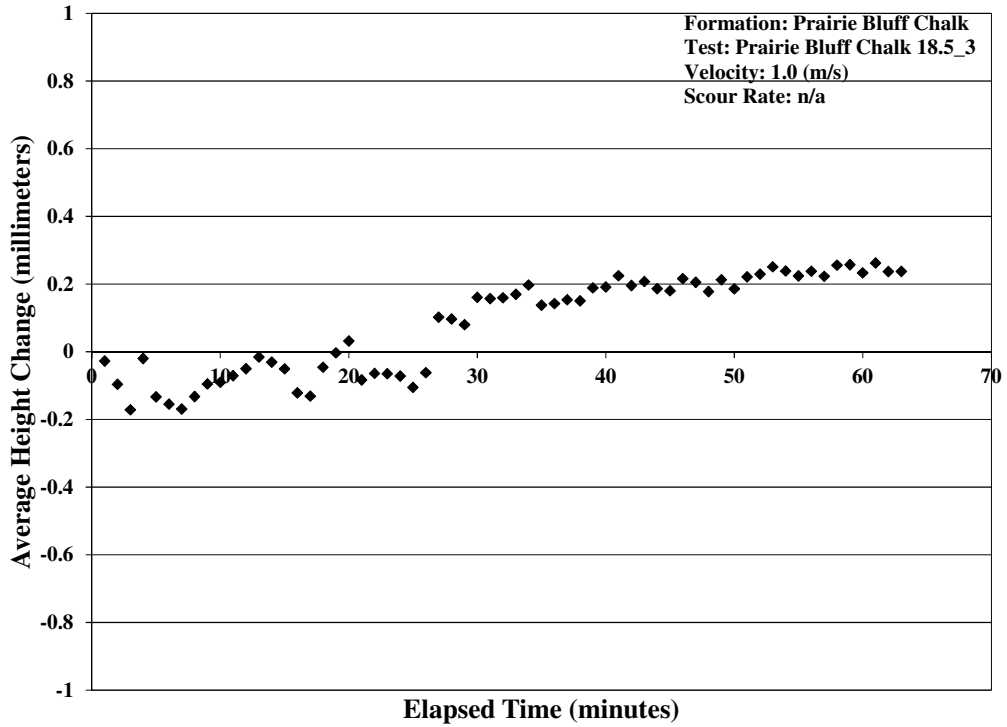


Figure E-2. "Prairie Bluff Chalk 18.5_3" Test Results

TEST VELOCITY: 1.5 METERS PER SECOND

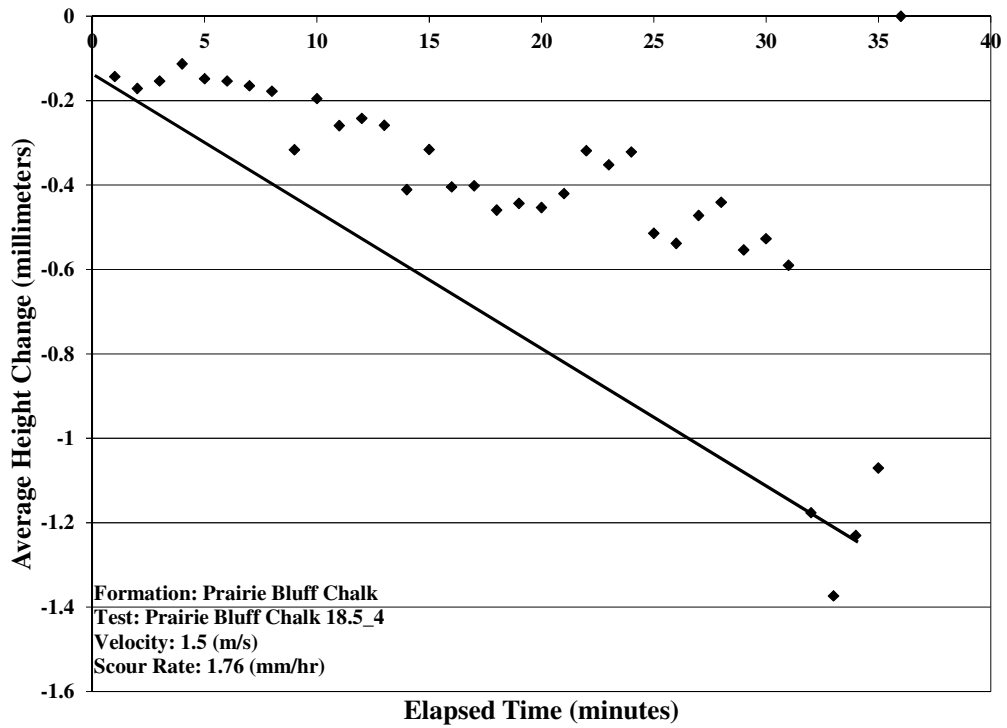


Figure E-3. "Prairie Bluff Chalk 18.5_4" Test Results

TEST VELOCITY: 2.0 METERS PER SECOND

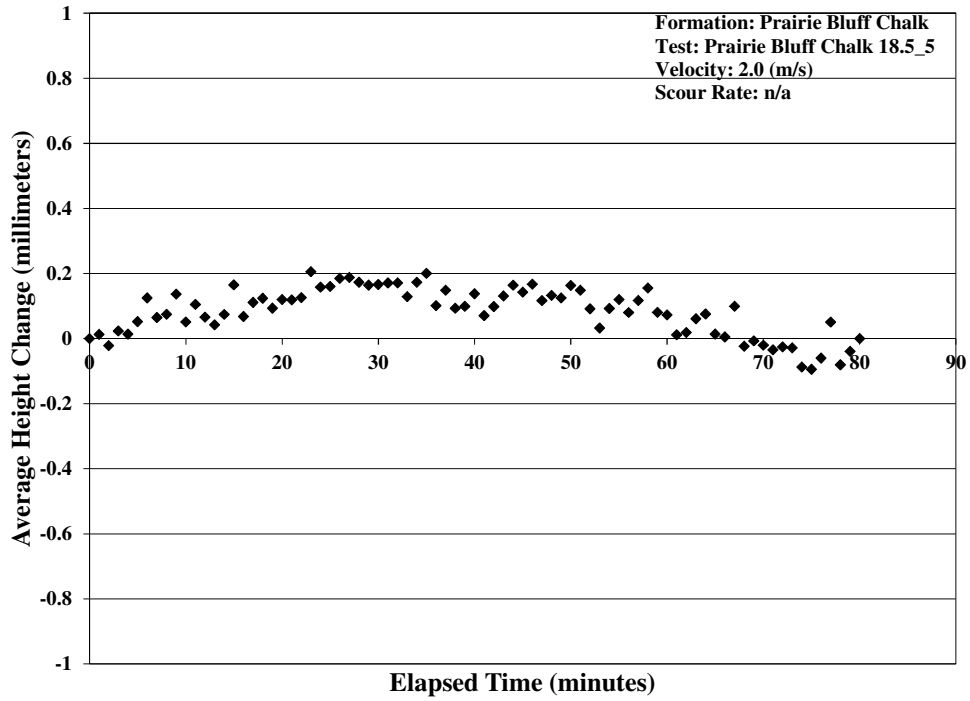


Figure E-4. "Prairie Bluff Chalk 18.5_5" Test Results

TEST VELOCITY: 3.0 METERS PER SECOND

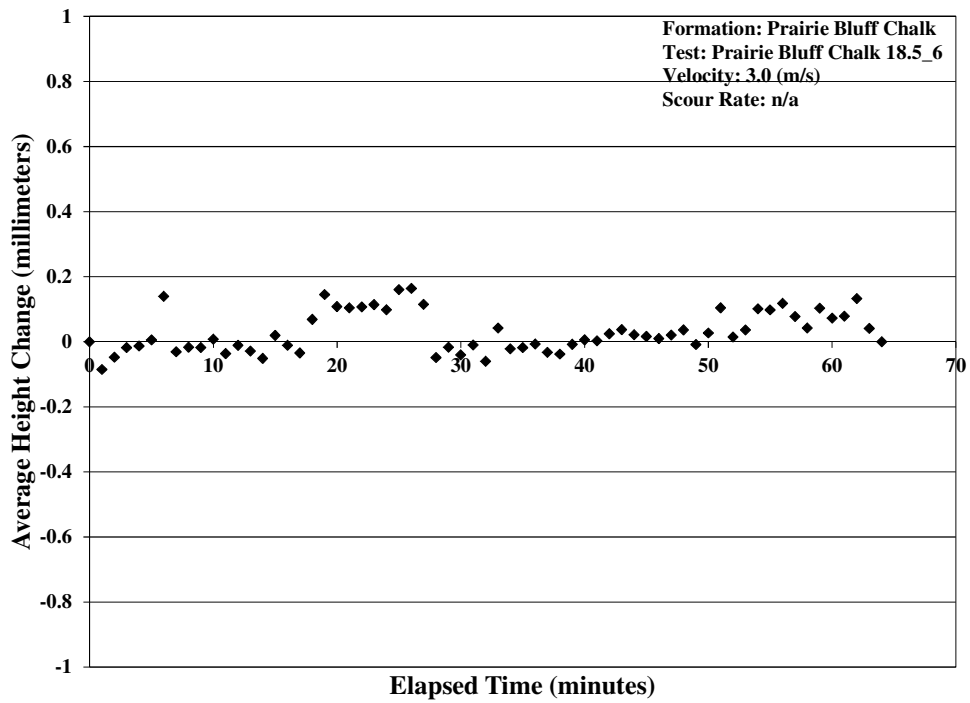


Figure E-5. "Prairie Bluff Chalk 18.5_6" Test Results

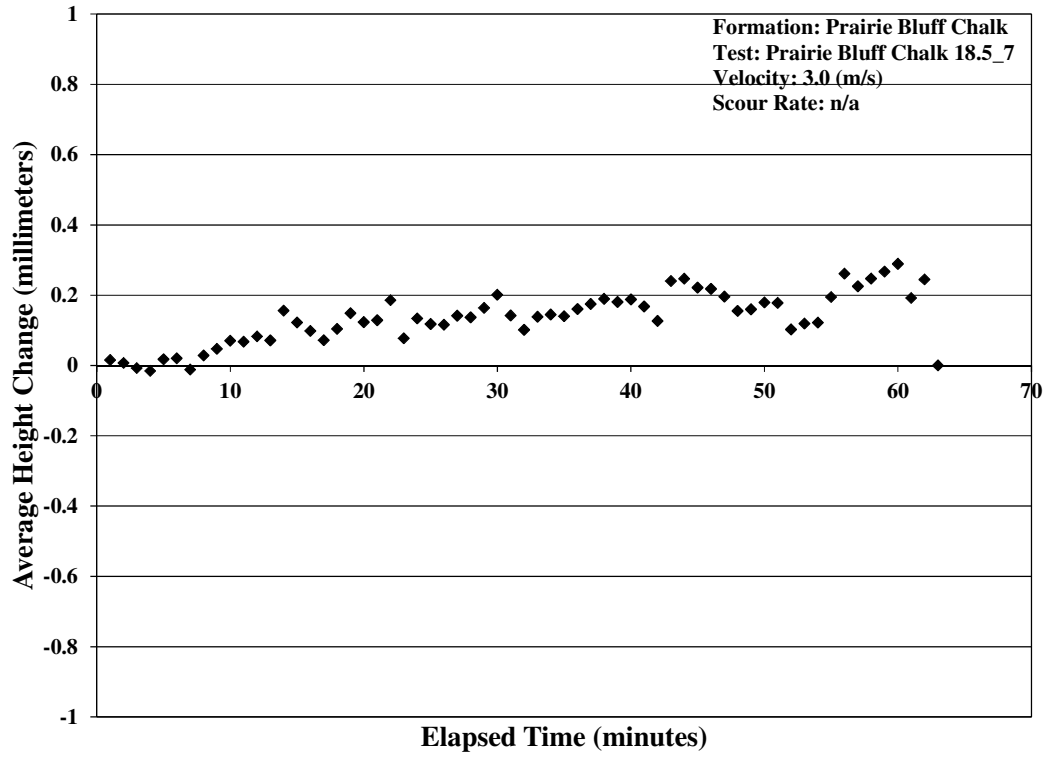


Figure E-6. "Prairie Bluff Chalk 18.5_7" Test Results

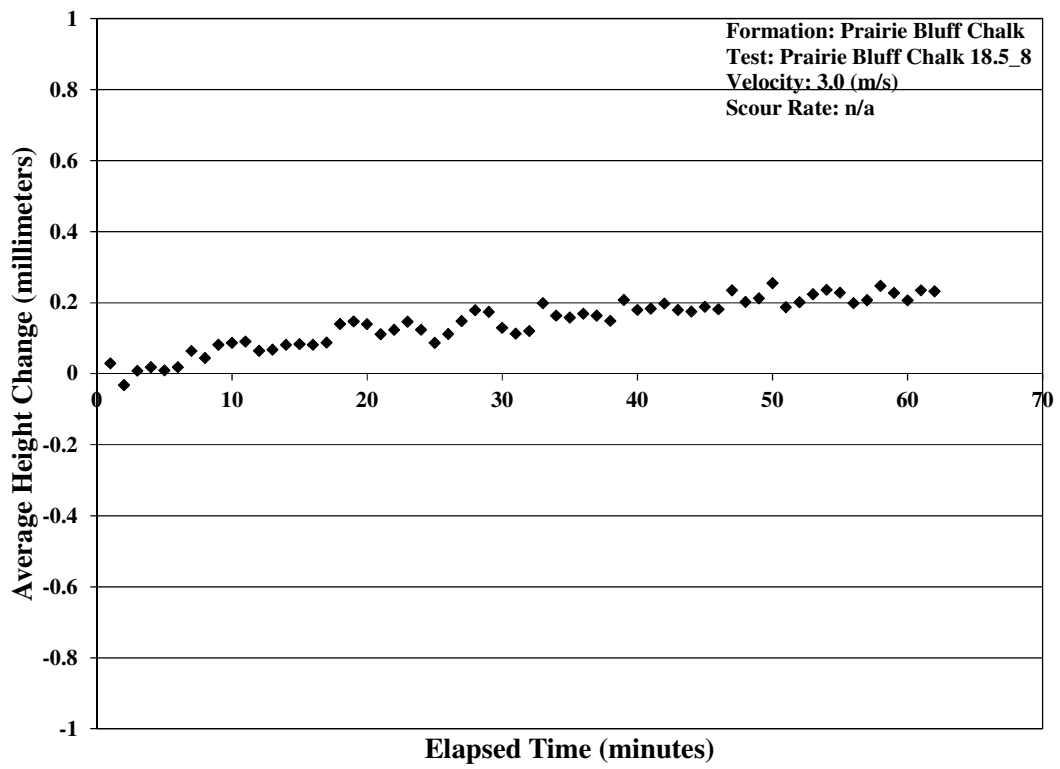


Figure E-7. "Prairie Bluff Chalk 18.5_8" Test Results

GEOTECHNICAL TESTING RESULTS

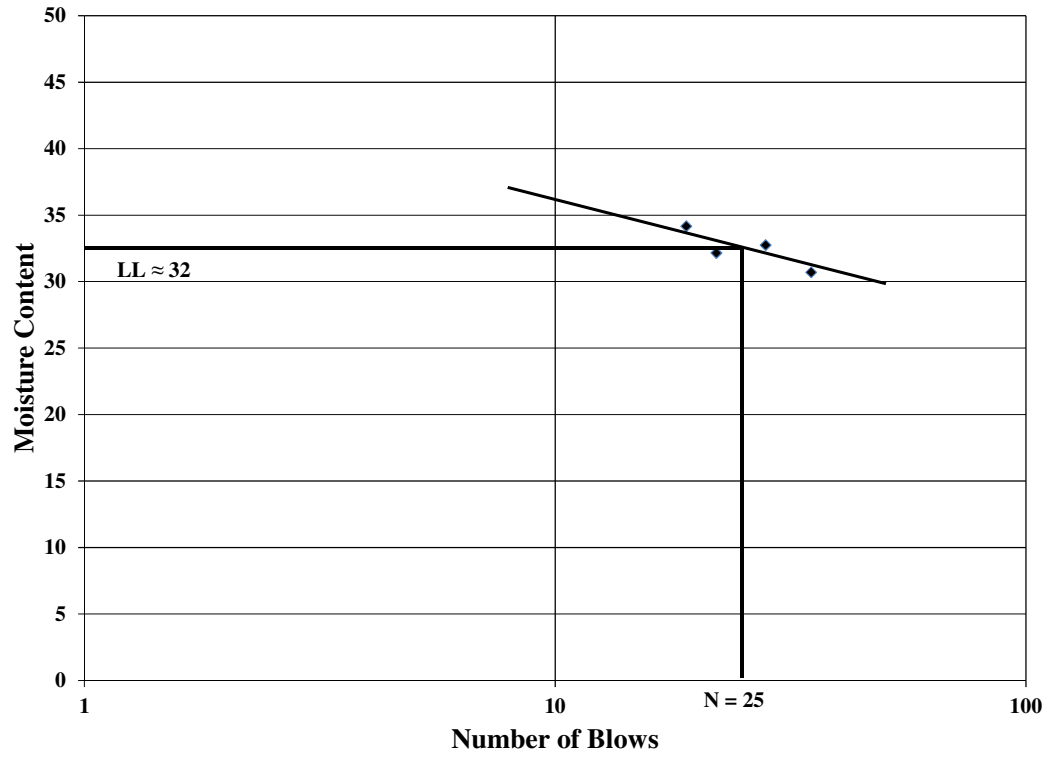


Figure E-8. Prairie Bluff Chalk Liquid Limits Results

APPENDIX F

EFA AND GEOTECHNICAL TEST RESULTS:

PORTER'S CREEK CLAY FORMATION

TEST VELOCITY: 0.3 METERS PER SECOND

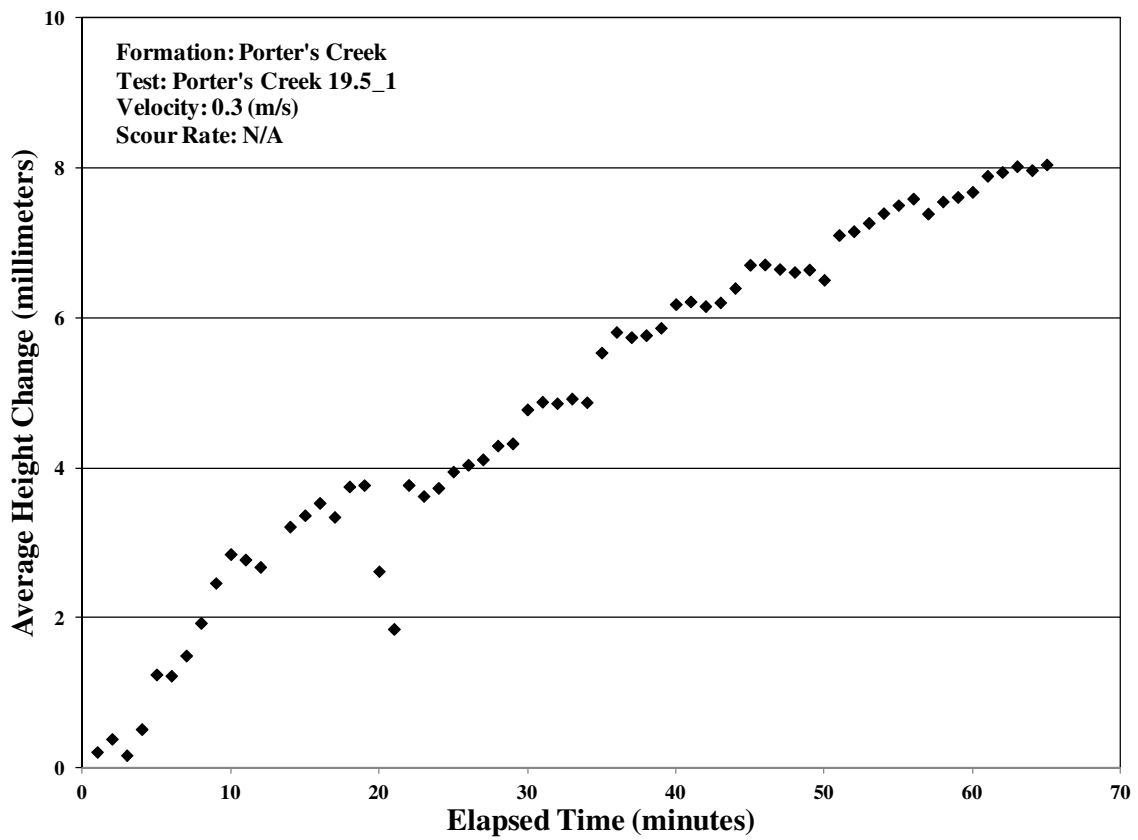


Figure F-1. "Porter's Creek 19.5_1" Test Results

TEST VELOCITY: 0.6 METERS PER SECOND

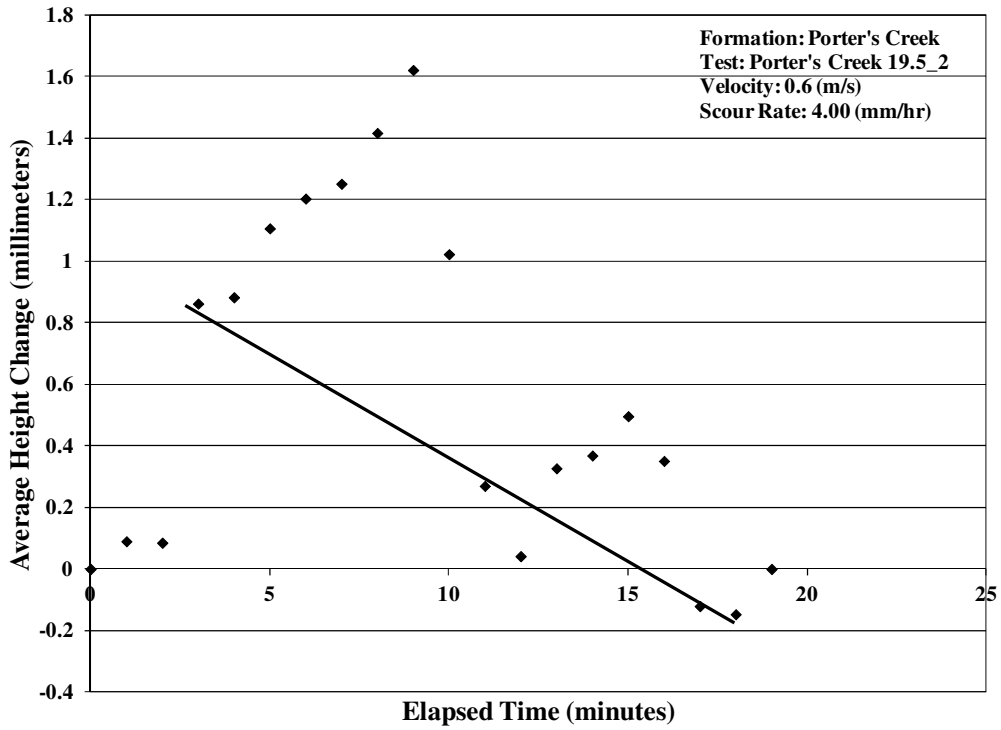


Figure F-2. "Porter's Creek 19.5_2" Test Results

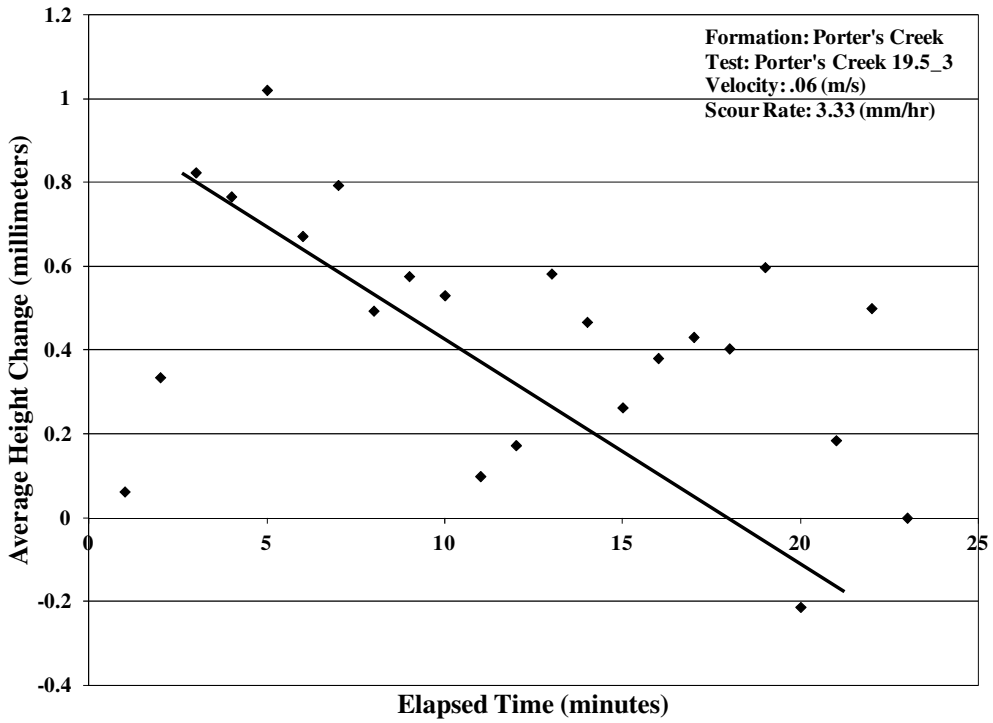


Figure F-3. "Porter's Creek 19.5_3" Test Results

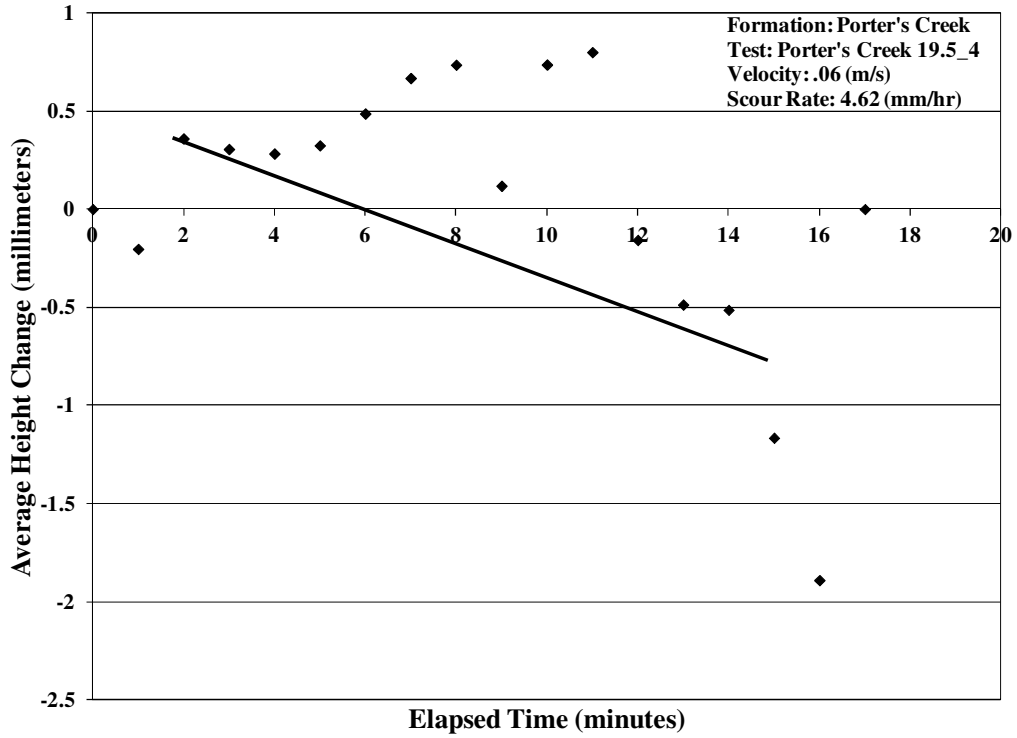


Figure F-4. "Porter's Creek 19.5_4" Test Results

TEST VELOCITY: 1.0 METERS PER SECOND

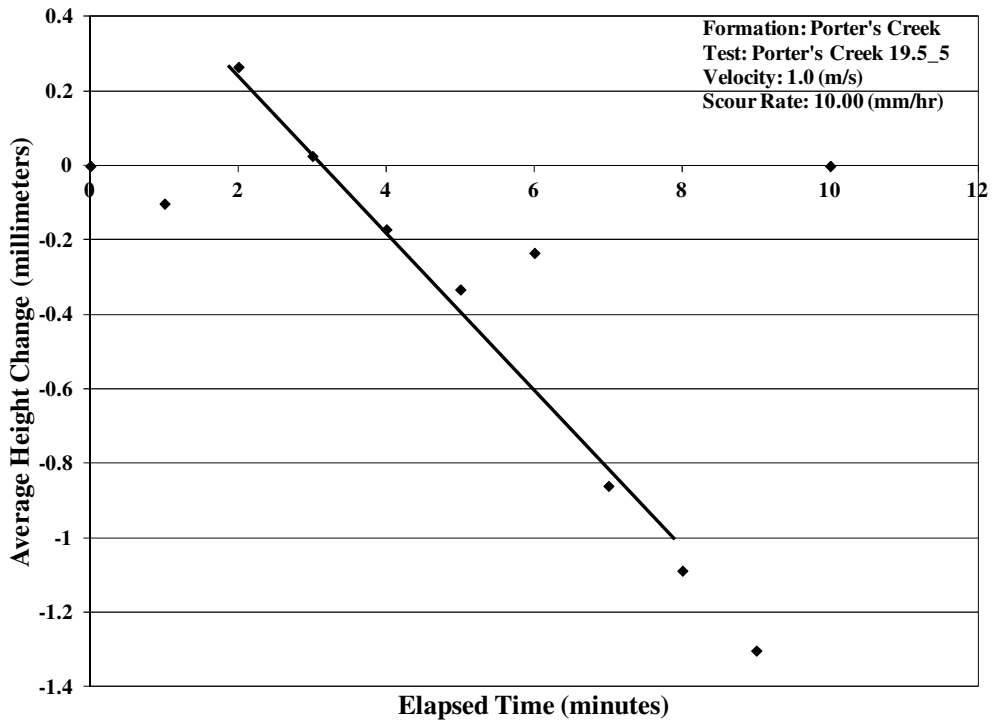


Figure F-5. "Porter's Creek 19.5_5" Test Results

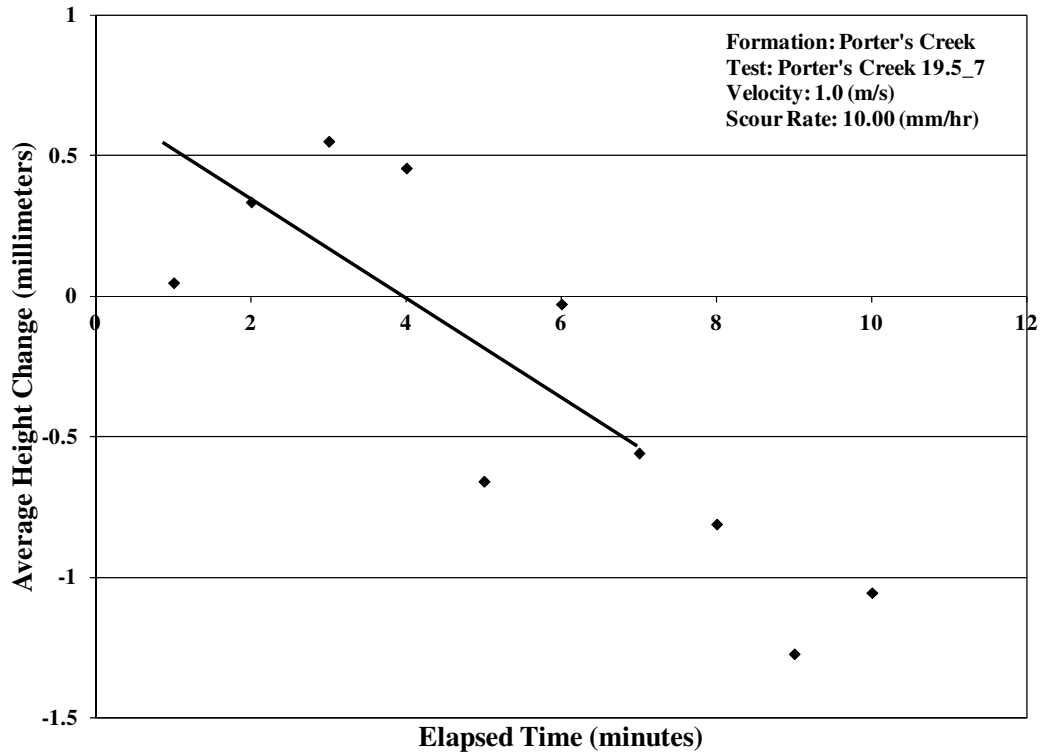


Figure F-6. "Porter's Creek 19.5_7" Test Results

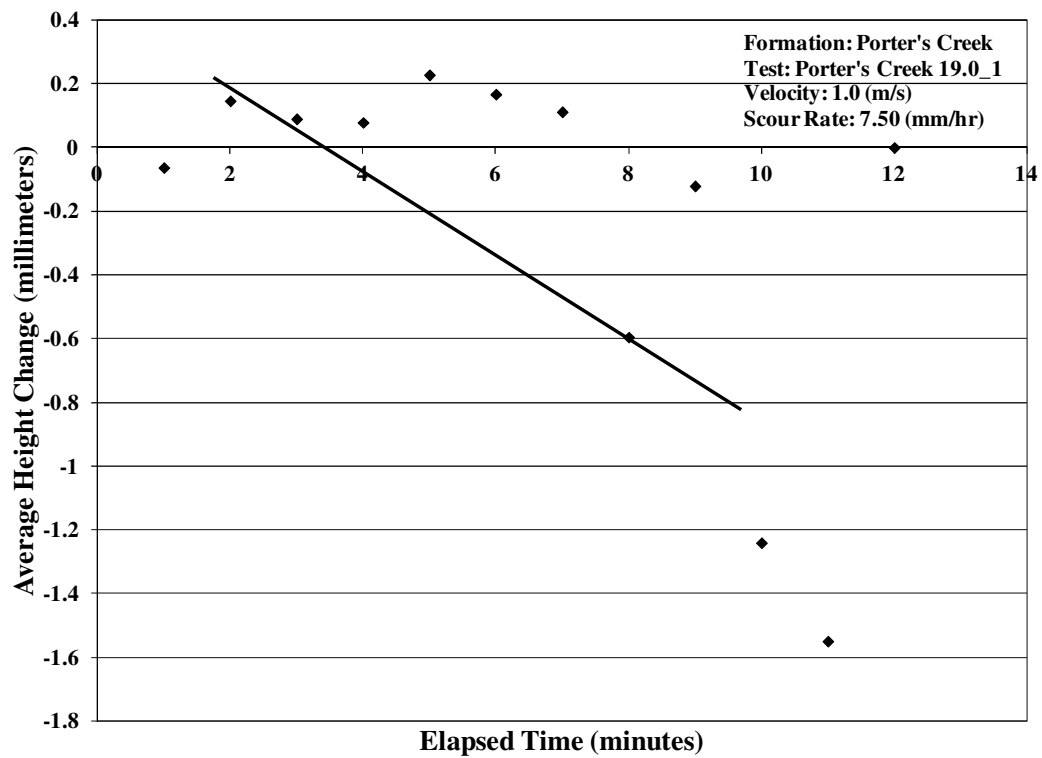


Figure F-7. "Porter's Creek 19.0_1" Test Results

TEST VELOCITY: 1.5 METERS PER SECOND

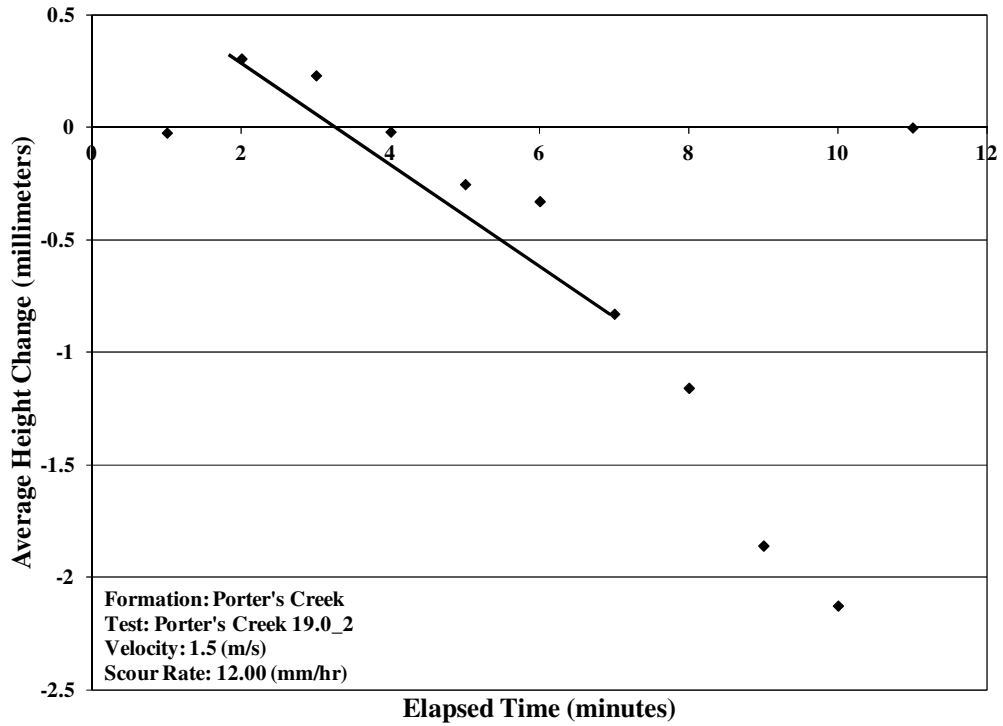


Figure F-8. "Porter's Creek 19.0_2" Test Results

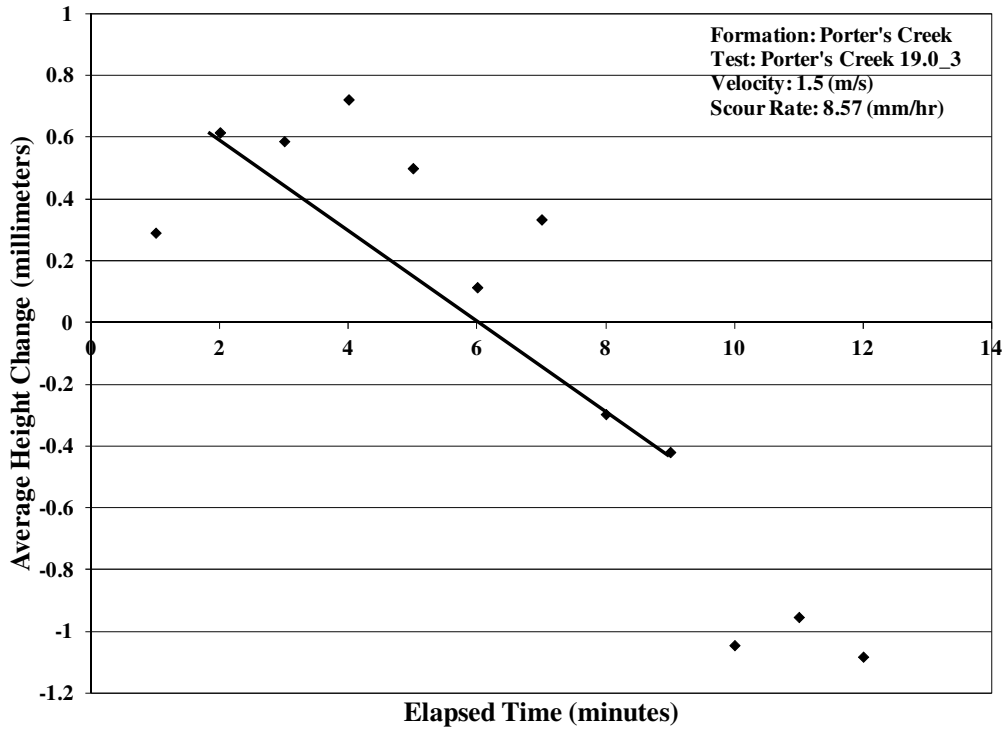


Figure F-9. "Porter's Creek 19.0_3" Test Results

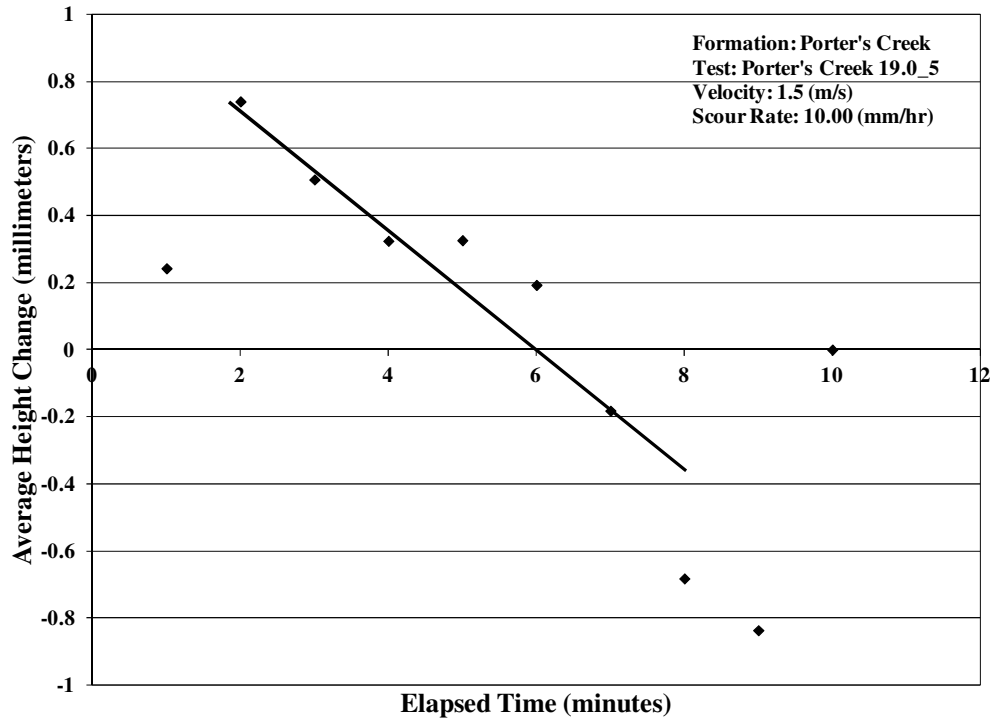


Figure F-10. "Porter's Creek 19.0_5" Test Results

TEST VELOCITY: 2.0 METERS PER SECOND

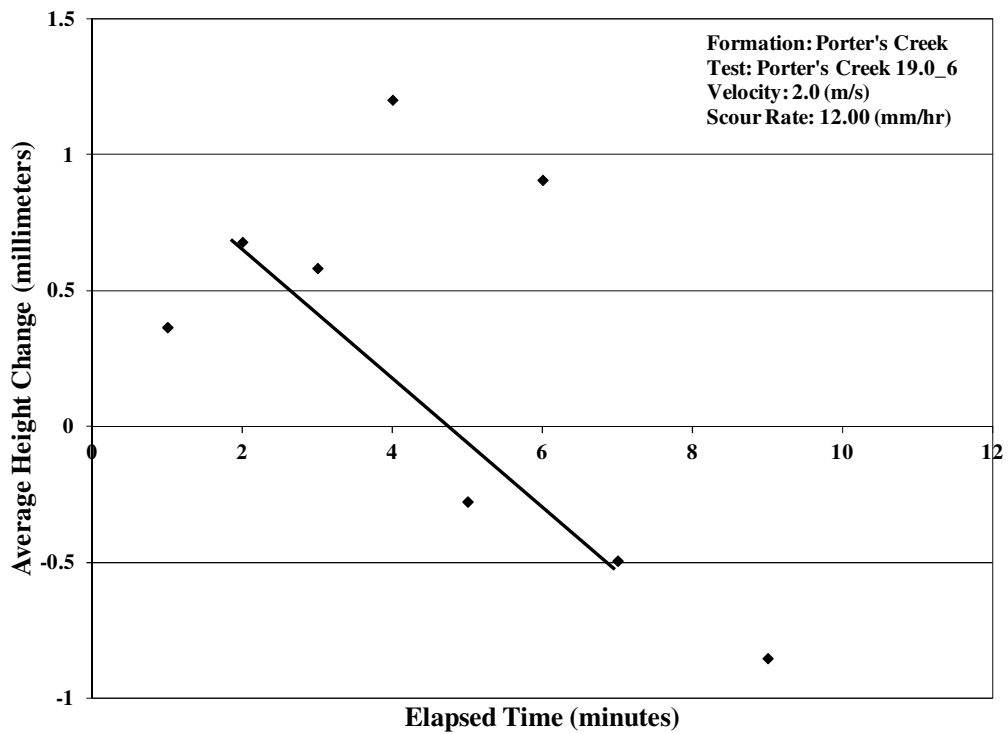


Figure F-11. "Porter's Creek 19.0_6" Test Results

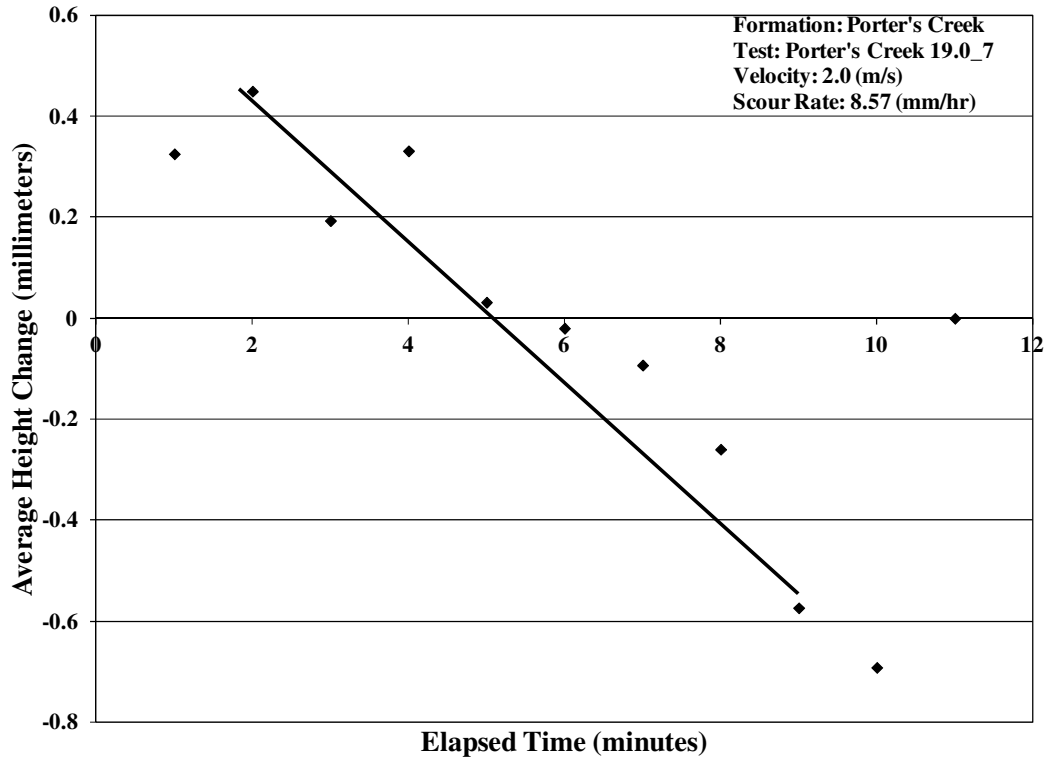


Figure F-12. "Porter's Creek 19.0_7" Test Results

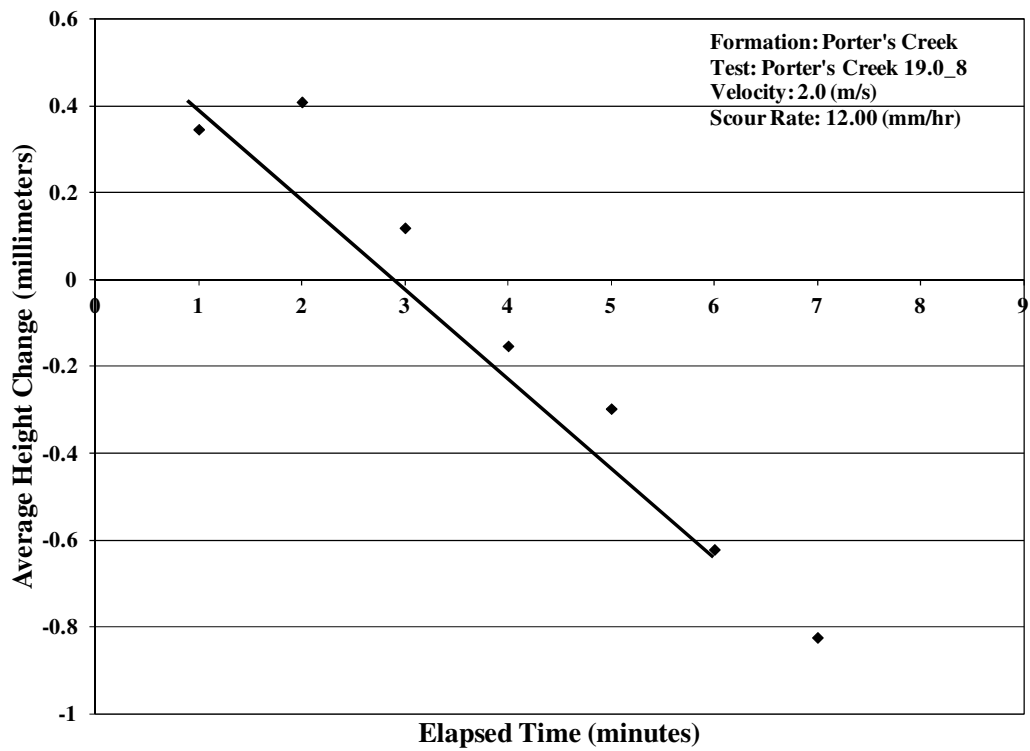


Figure F-13. "Porter's Creek 19.0_8" Test Results

TEST VELOCITY: 3.0 METERS PER SECOND

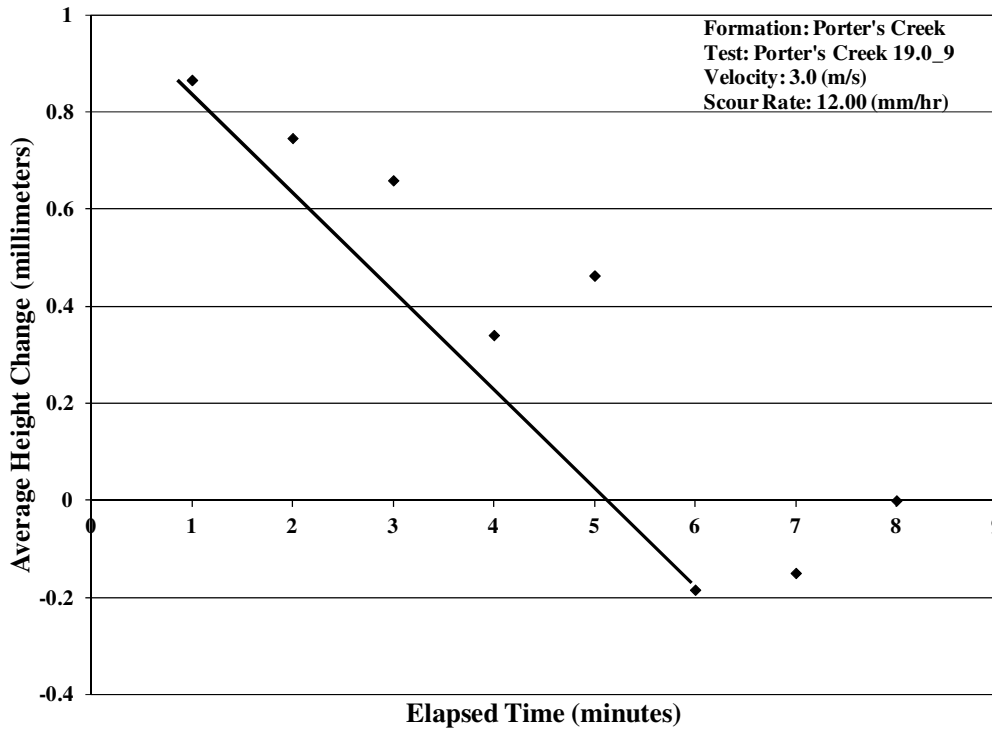


Figure F-14. "Porter's Creek 19.0_9" Test Results

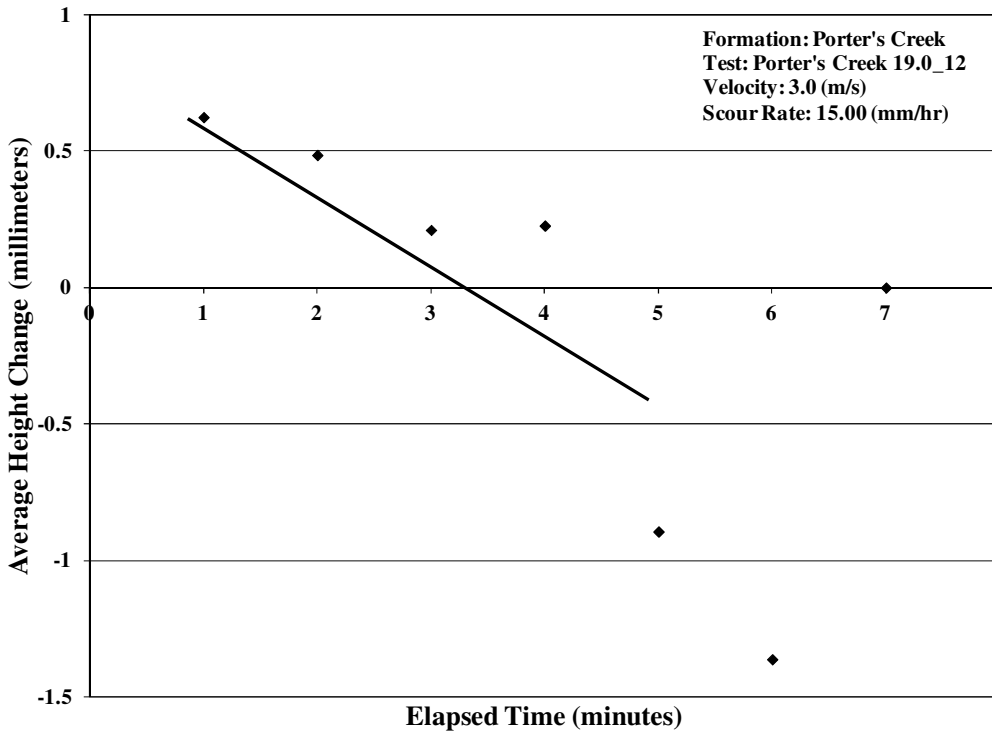


Figure F-15. "Porter's Creek 19.0_12" Test Results

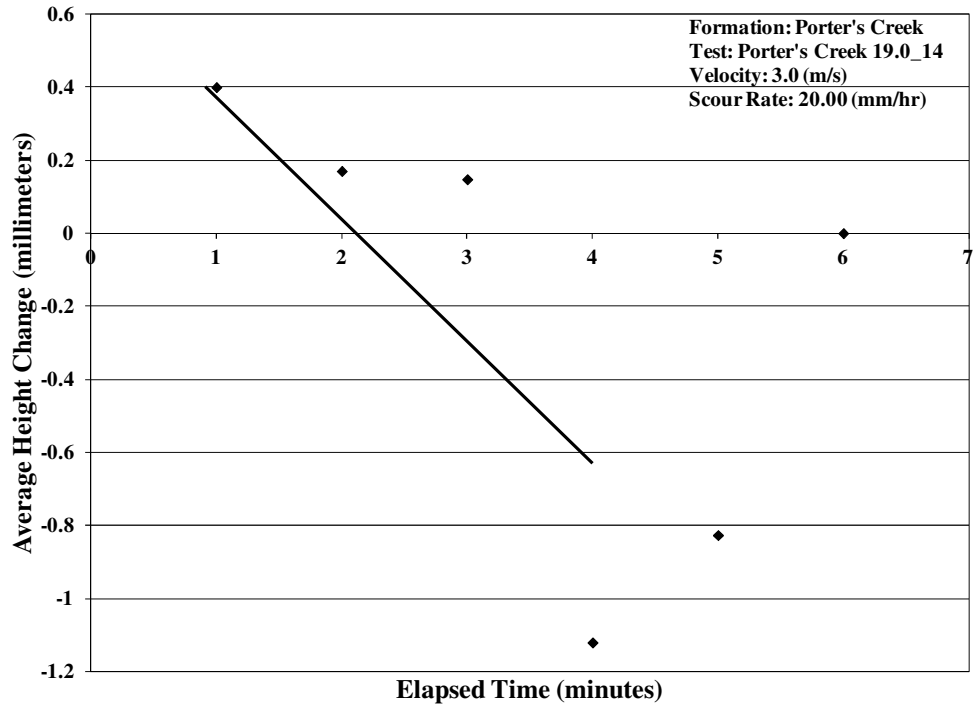


Figure F-16. "Porter's Creek 19.0_14" Test Results

GEOTECHNICAL TEST RESULTS

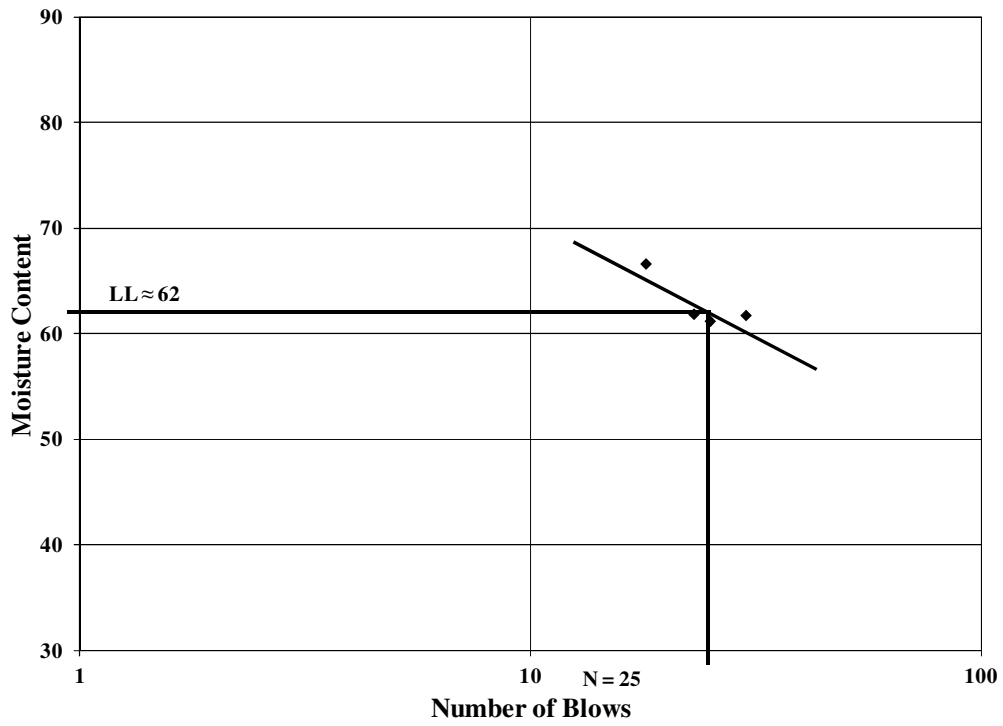


Figure F-17. Porter's Creek Clay Liquid Limits Results