ERODIBLITY TESTING OF COHESIVE SOILS

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ERODIBILITY TESTING OF COHESIVE SOILS

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ERODIBILITY TESTING OF COHESIVE SOILS

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Thomas Jackson Mobley II, son of Max. A. Mobley and Kathy W. Mobley, was born on September 21, 1984. He graduated Valedictorian from Abbeville High School in Abbeville, Alabama in May 2003. He began his undergraduate studies at Auburn University in the fall of 2003 and graduated Magna Cum Laude in August of 2007 with a Bachelor of Civil Engineering degree. His graduate studies began at Auburn University with specialties in Hydraulics, Hydrology, and bridge scour in order to pursue a Master of Science degree in Civil Engineering.

THESIS ABSTRACT

ERODIBILITY TESTING OF COHESIVE SOILS

Thomas Jackson Mobley II

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The Erosion Function Apparatus (EFA) is used to measure erosion rates for cohesive soil samples subjected to water velocities tangent to the sample surface. EFA testing was performed on soil samples taken from three bridge sites in Alabama: Talladega County, Sumter County (Sucarnoochee River), and Dallas County. Results from Talladega County varied greatly. One sample yielded a critical shear stress of 0.49 N/m^2 and produced a scour rate of 100 mm/hr at V = 2.0 m/s, while another sample yielded a critical shear stress of 0.049 N/m^2 and caused a scour rate of 120 mm/hr at V = 1.44 m/s. No erosion functions were generated for three samples, but testing indicated they were very scour resistant at V = 1.90 m/s, 1.60 m/s, and 1.00 m/s. Samples from Dallas County were tested at V = 6 m/s and were extremely resistant to scour. The internal structure of the sample from Sumter County was very complex, which made surface preparation an issue. Attempts to trim the surface flush with the EFA flume bed

greatly disturbed the sample and resulted in mechanical erosion that was not reflective of the soil's true erodibility. However, one test was successfully performed with minimal disturbance and was highly erosion-resistant at V = 1.00 m/s.

The EFA was also used to measure erosion rates for "model" soils composed of bentonite and sand. These soils were studied to allow for controlled variations of bentonite content and wet density. Samples ranged in bentonite content from 3.5% to 25% and were compacted to a wet density of either 1.7 g/cm^3 or 1.9 g/cm^3 at a water content of 20%. Critical shear stress ranged from 0.62 N/m^2 at 5% bentonite to 11.45 N/m^2 at 25% bentonite (both at 1.9 g/cm^3). An increase in density from 1.7 g/cm^3 to 1.9 g/cm^3 reduced scour rates from 84 to 34 mm/hr at V = 6 m/s.

In addition to the EFA tests, model soils were subjected to impinging jet erosion. This facilitated more rapid testing for soils with relatively smaller bentonite content (less than 8%) that were not amenable to EFA testing. Erosion rates were impossible to measure in samples with less than 8% bentonite in the EFA because of a rapid rate of increase in erosion rate with respect to time. Soil was compacted in PVC caps, and a free jet from a water column and orifice was directed at the horizontal surface of the soil. Tests showed a significant decrease in soil erodibility with increasing initial compaction and increasing bentonite content.

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Computer software used Microsoft Word 2007, Microsoft Excel 2007, Microsoft PowerPoint 2007

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CHAPTER ONE

INTRODUCTION

1.1 Background of the EFA and Bridge Scour Estimation

Scouring of riverbeds continues to be a major problem today. The National Bridge Inventory (NBI) estimates around 575,000 bridges exist in the United States, and 84 percent of them are over water (Lagasse et al. 1995). According to the American Society of Civil Engineers, over 500 bridge failures occurred in the U.S. between 1989 and 2000, with scour during flooding contributing to approximately 53 percent of those failures. On average, the federal government spends approximately $50 million per year for flood damage to highways and bridges (Lagasse, et al., 1995). Contraction scour, which occurs during this flooding, is the leading cause for bridge failure in the U.S. (Richardson et al. 1995).

Hydraulic engineering circulars published by the Federal Highway Administration (FHWA), HEC-18, and HEC-20, are currently used to predict scour for non-cohesive soils such as sand (Crim 2003). However, these methods do not consider the rate of scour development in fine-grained cohesive soils – soils with more than 50 percent of particles passing the No. 200 (.075 mm) sieve. Cohesive soils contain electrostatic and van der Waals forces binding their particles together and giving them additional scour resistance; because of these forces, scour rates can be thousands of times smaller than cohesionless soils. In order to quantify these rates, the Erosion Function Apparatus (EFA) was developed by Briaud et al. (1999, 2001a, 2001b, 2004).

The EFA can be used for any type of soil that can be sampled with a standard Shelby tube (76.2 mm diameter, ASTM-D1587), whether it is coarse grained soil such as sands or fine grained soil such as clays. The Auburn University EFA is displayed in Figure 1.1, and a detailed description of how to operate it is provided by Crim (2003).

Figure 1.1: The Auburn University EFA

The soil sample is extracted from its original Shelby tube and is pushed into a new Shelby tube to be loaded onto the EFA. As shown in Figure 1.2, 1 mm of soil is projected from the Shelby tube into the flow of water. For a fixed cross flow velocity, the time interval required to erode the 1 mm projection is recorded and the erosion rate (mm/hr) is calculated. This procedure is repeated for different layers of soil and at different cross flow velocities.

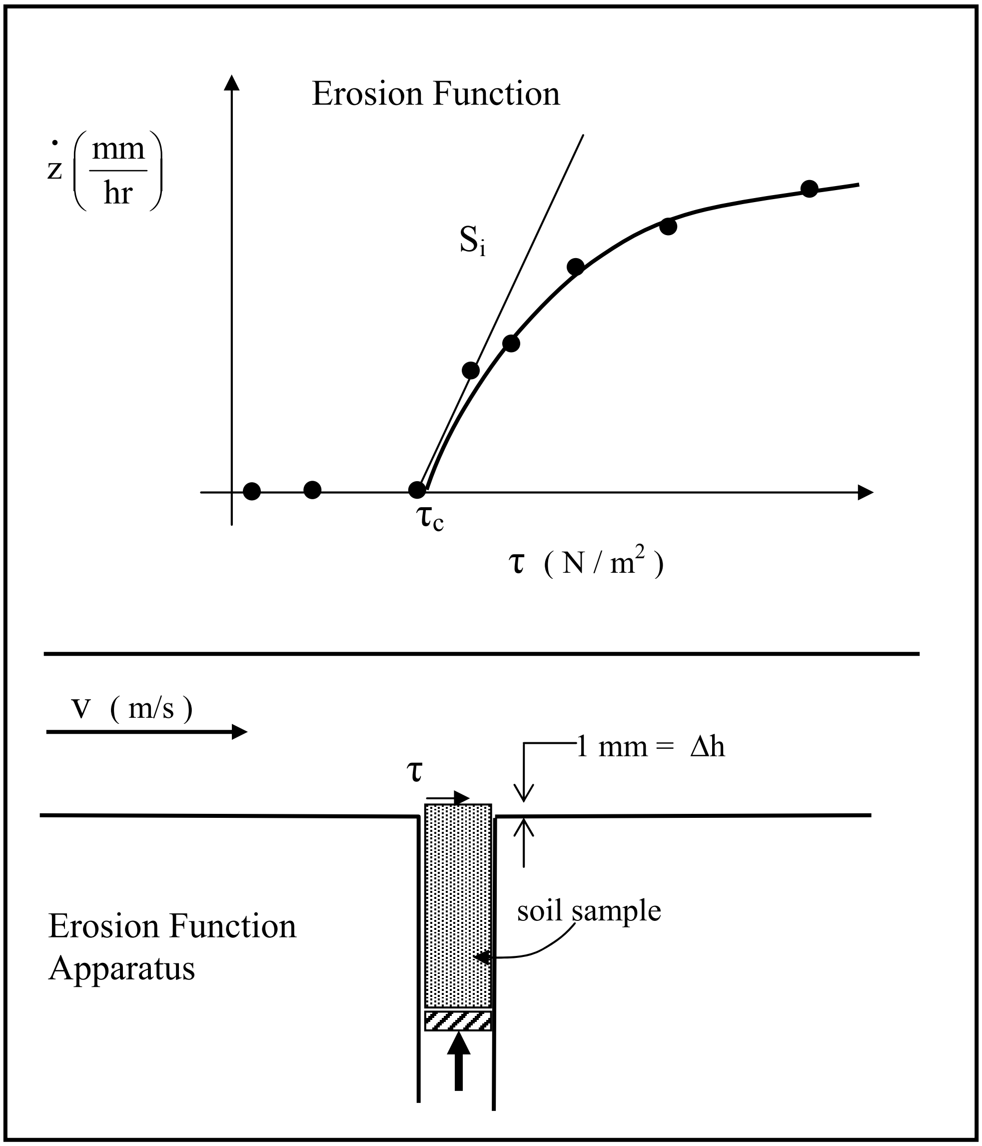


Figure 1.2: EFA and Bridge Scour Prediction

The data from this procedure is primarily used to generate the erosion function of a particular soil, which is the relationship between scour rate ($\dot{z}$) and shear stress (τ, which is related to velocity) as shown in Figure 1.2. The critical shear stress (τ_c) is the shear stress below which no scour takes place. Initial erodibility (S_i) indicates how fast the soil scours at the critical shear stress. If all plotted data show a linear trend between τ and $\dot{z}$, a straight line, with S_i = constant, can approximate the data and the erosion function is estimated by:

$$\dot{z} = S_i(\tau - \tau_c) \quad (1.1)$$

If the points do not show a linear trend, the best relationship is fit (usually a parabola) and S_i is drawn tangent to the curve beginning at τ_c (Figure 1.2).

Using the erosion function generated as outlined above, predicted rates of scour at bridge sites can be obtained. The method called SRICOS (Scour Rate in Cohesive Soils), is described in work by Briaud et al. (1999, 2001a, 2001b, 2004), where EFA data is used to predict the development of scour depth with time. EFA data has been used by Güven et al. (2002) and Curry et al. (2003) to predict scour in cohesive soils. Also, Santamaria (2003) performed scour calculations where EFA data was combined with a hydraulic model of flow through a contraction based on a 400 day hydrograph to develop a scour accumulation prediction. Ultimately, knowledge of τ_c and S_i allows estimates of scour rates in cohesive soils which is an improvement over ultimate scour methods.

1.2 Literature Review

A M.S. Thesis report titled "Erosion Functions of Cohesive Soils", written by Crim (2003), concluded that the EFA was useful when studying scour characteristics of soil. The EFA was found to be helpful in finding the erosion function, critical shear stress, and the initial erodibility of a particular soil. Crim (2003) also discovered that samples taken from different depths at the same bridge site would have different scour rates; this is very important when evaluating scour rate in stratified soils. His study agreed with earlier work performed by Briaud et al. (2001a), where correlations were found between critical shear stress, plasticity index, and compaction. Crim (2003) also confirmed correlations between critical shear stress and initial erodibility from earlier work performed by Briaud et al (2001a).

In a paper titled “Erosion Function Apparatus for Scour Rate Predictions,” Briaud et al. (2001a) outlined the steps to determine shear stresses for the EFA by using the Moody diagram to correlate the shear stress, velocity, and friction factor. The Moody diagram relates relative roughness (roughness height/conduit diameter) and the Reynolds number (equation 3.3) to obtain a friction factor *f*, which is used to calculate EFA shear stress through equation 3.2. They estimated the relative error of EFA scour rate measurement is 10%, found poor correlations between soil properties and erosion rates, and concluded that soils with higher critical shear stresses have lower initial erodiblity. Briaud et al. (2001b) extended their research in a paper titled “Multiflood and Multilayer Method for Scour Rate Prediction at Bridge Piers”, where the SRICOS method was expanded to include random velocity-time history and multilayer soil stratigraphy.

Similar to bridge scour, erosion rates in cohesive soils have been studied by simulating overland flow scenarios. A study titled “The Influence of Soil Conditions on the Resistance of Cohesive Soils against Erosion by Overland Flow” was conducted at the Technological Educational Institute of Larissa (Chouliaras et al. 2003). Surface erosion tests were performed in a laboratory flume with cohesive soils consisting of various plasticities and sand contents. The rate of soil lost was measured by the methods of “settling, filtering, and percolation in sieves with 0.063 mm aperture” (Chouliaras et al. 2003). Preston tubes were used to measure the shear stress of the flow, while the longitudinal slope and depth of flow were measured by hand. Several conclusions were reached upon further analysis of these tests:

- the most erodible soils are silty sands
- increased density reduces erodibility more for clayey soils than for silty soils
- surface shear strength of cohesive soils does not provide reliable estimates of erosion resistance
- the relationship between flow shear stress and cohesive soil erosion rates is almost linear

These results are fairly similar to conclusions drawn from bridge scour research, and are certainly applicable to EFA testing.

Another scour prediction method, similar to SRICOS, is presented in a paper called “Measurement of Scour in Cohesive Soils around a Vertical Pile - Simplified Instrumentation and Regression Analysis” (Babu et al. 2003). Experimental laboratory techniques are outlined on measuring scour around pile foundations in silty-clay soil in an oceanic environment. Model piles having a diameter of 50-110 mm were placed in soil in a wave flume approximately 30 m long and 2 m wide, where scour depths were continuously monitored with different combinations of velocities and wave characteristics (Babu et al. 2003). Good correlations were established between scour depth and flow duration, soil properties, and fluid mechanics. Overall, this study found its methods for measuring scour depths in cohesive soils to be satisfactory.

Regazzoni et al. (2008) examined the influence of some engineering parameters on the erosion of soils by applying an impinging jet for scour analysis. The study focused on the influence of two parameters affecting erosion: water content and compaction. Soil samples were compacted at five different water contents (weight of water divided by weight of solids in a given sample). Some were completely saturated, and some were maintained at their original moisture content. The soils were then submerged and subjected to jet erosion tests shown in Figure 1.3.

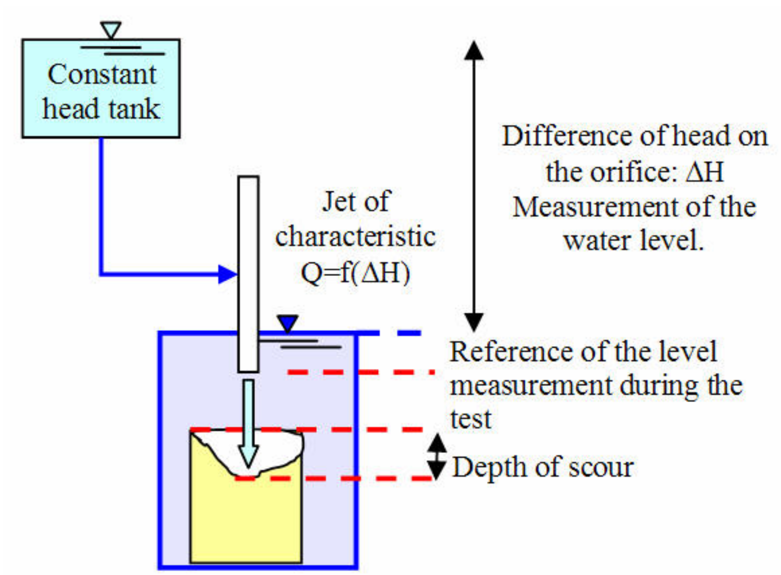


Figure 1.3: Jet Test Setup (Regezanni et al. 2008)

The optimum moisture content (OMC) is the water content at which a soil can be compacted to the maximum dry unit weight by a given compaction effort. When compared to measured scour at the OMC, rates were as much as 100 times higher in soils compacted drier than the OMC and only 10 times higher in soils compacted wetter than the OMC. This illustrates the dependence of erosion on compaction and water content. Scour is deeper in soils not at their OMC, and deeper in drier soils. Compaction energy can also affect scour. Soils compacted with higher energy had increased erodibility on the wet side of the OMC and decreased erodibility on the dry side. In summary, this shows that erosion is a function of hydraulic stress and the fabric of soil, with compaction effort and water content having a large influence.

Other research has been performed on jet impingement testing to measure soil erodibility. Hanson and Cook (2004) describe a jet apparatus, procedure and analysis to determine τ_c and S_i in field measurements. An analysis of impinging jet flow on a flat surface by Phares et al. (2000) presents detailed theoretical and measured wall shear stress distributions on the flat surface. This detailed analysis and the field apparatus are different from the jet tests described in the present work in that the jet is fully submerged. The core velocity of the submerged jet dissipates as mixing takes place before the jet impinges on the flat surface. In contrast, the jet experiments of the present work are for free jets with perhaps only a centimeter of submergence where the jet impinges on the soil sample. Another difference is that the present work has a falling head column which generates a jet velocity which decreases linearly with time. The field apparatus and the analysis referred to above was for a constant head and constant jet velocity. In these two references the maximum shear stress, τ_m , is related to the jet velocity as

$$\tau_m = C_f \rho u_o^2 . \tag{1.2}$$

The coefficient, C_f, depends on the jet Reynolds number and the jet and apparatus geometry. Approximate application to the present work suggests $0.004 < C_f < 0.016$. If it is assumed that $C_f = 0.01$ for the application of u_o=3.71 m/s (y_o= 0.70 m), the maximum stress which occurs in the jet test is $\tau_m = 138\ N/m^2$. This shear stress in the EFA would correspond to a velocity, $7.4 < v < 10.5$ m/s. This velocity range exceeds velocities tested in the EFA. For the jet test it should be noted that duration is 96 s and the velocity decreases from u_o to zero. Thus it is concluded that the jet shear stresses generated in the present work are comparable with EFA generated stresses. A distribution of the impinging jet and shear stress on the soil surface is shown on Figure 1.4:

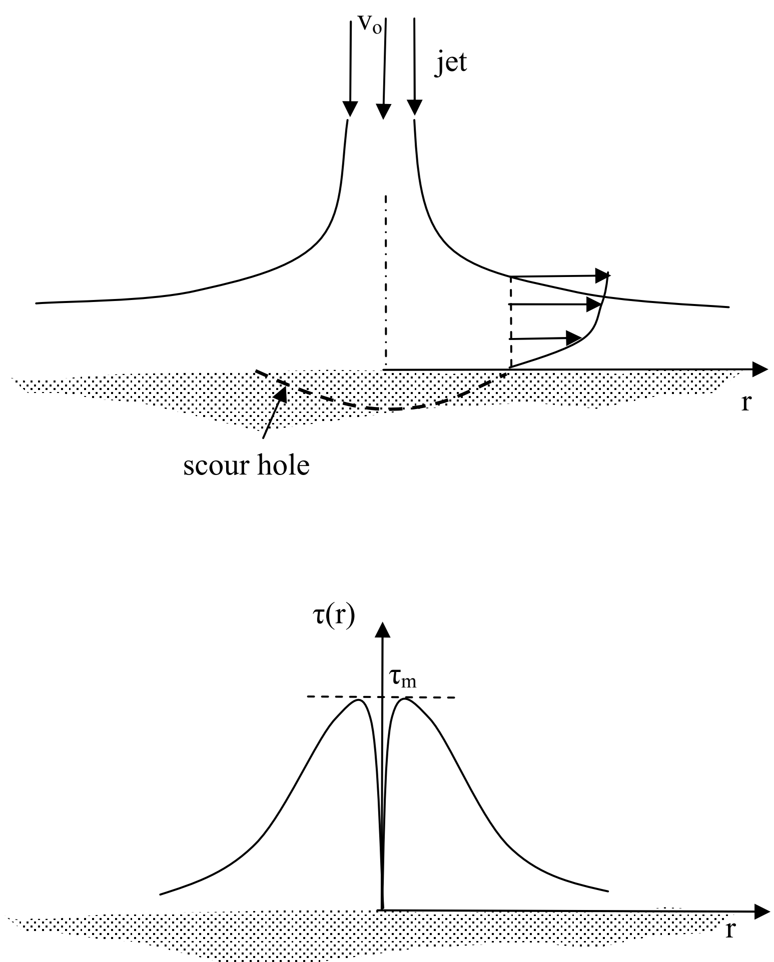


Figure 1.4: Impinging Jet and Shear Stress Distribution on the Soil Surface

1.3 Description of Study

This particular study contained three objectives. The first was to develop modifications to the EFA to accommodate sample sizes typical of cores for hard clays or soft rocks encountered at some stream crossings. Previous studies only dealt with soil samples supplied in 76.2 mm diameter Shelby tubes. Cores supplied from two separate sites in this study were 4.45 cm and 4.75 cm in diameter, and a new test tube and piston were developed to facilitate EFA testing in these samples.

The second objective was to establish an expected erosion range and variability in cohesive soils encountered at Alabama stream crossings. Different techniques were applied to measure erosion rates in these soils. Some rates were measured in the same way described in section 1.1, and new erosion-measuring methods were developed for other soils. A description of all Alabama soils and their measurements can be found in sections 2.1, 2.3, and Chapter 4.

The final objective was to develop erosion possibilities in "model" soils constructed of sand and sodium bentonite. Using various bentonite contents, correlations could be established between clay content and τ_c which, in turn, corresponded to other properties such as plasticity index. These soils were subject to both EFA testing and impinging jet testing, which differed slightly from the description in section 1.2. The jet and the samples in this study were not submerged.

CHAPTER TWO

TESTING OF SOIL SAMPLES

2.1 Sampling – Alabama Soils

One of the objectives of this study was to measure erosion rates in layers of some of the cohesive soils encountered at Alabama stream crossings, which could be used to predict scour depth for certain flood scenarios. Measurements were accomplished by performing EFA tests on soil samples provided by the Alabama Department of Transportation (ALDOT). Samples came from three different stream crossings in Alabama: Talladega County (culvert extension project for additional lanes on SR 275 (SR 21 to SR 77), Dallas County (bridge replacement along SR 8), and the Sucarnoochee River in Sumter County (additional lanes on US 80). Eight Talladega County samples provided by ALDOT were collected in the same manner as they were for Crim (2003): an ASTM standard Shelby tube with an outside diameter of 76.2 mm was driven into the ground and pulled up. Soil from Dallas County and Sumter County was sampled with cores that had smaller diameters: 4.45 cm and 4.75 cm. Boring logs containing information such as depths, soil descriptions, and standard penetration test (ASTM-D1586) blow counts (N) were also provided. Every sample was stored in a humidity controlled chamber until testing.

A modification of the EFA was made to accommodate smaller sample sizes collected with coring equipment required for hard clay soils or soft rock encountered at some stream crossings. New core injection cylinders (CIC) with inside diameters of D = 4.45 cm and D = 4.75 cm (see Figure 2.1) and a thick wall of 1.5 cm were fabricated for use with chalk cores from Dallas County and Sumter County. In total, seven tests on one core sample from Sumter County and three tests on three samples from Dallas County were completed in the EFA using the CIC. Testing was completed on five of eight samples from Talladega in an ASTM standard Shelby tube.

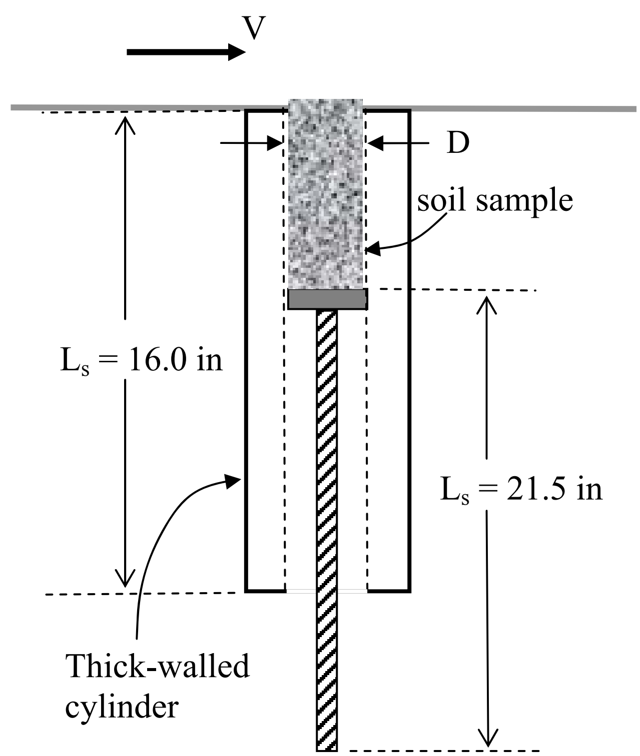


Figure 2.1: Core Injection Cylinder (CIC) Geometry

2.2 Soil Classification Testing

Soils were tested to determine grain size distribution and Atterberg limits (plastic limit and liquid limit). Grain size distribution tests were performed in accordance with ASTM-D422, and all Atterberg limit tests were done in accordance with ASTM-D4318. Plastic limit is the water content which corresponds to about 15 g of soil being rolled into a 1/8" diameter thread just before crumbling. Liquid limit is the water content when the soil begins to "flow." This corresponds to the water content at which 25 blows in the liquid limit testing device closes a gap drawn down the middle of the soil 0.5". Soils were classified according to the United Soil Classification System.

2.3 EFA Testing – Alabama Soils

Different techniques were applied to measure erosion rates for each sample. Some EFA tests were conducted in exactly the same manner as prescribed by the basic operating procedures found in Briaud et al. (2001a). The conventional method as prescribed by Briaud et al. (2001a) was to trim the soil sample within the tube to be flush with the EFA flume bed, so it could be raised into a given flow 1 mm in 0.5 mm increments. This is all controlled by the EFA computer, which also records average velocity, temperature, distance the soil sample is advanced, and elapsed time. After 1 mm was completely scoured away, another 1 mm was raised into the flow for scour measurement. This procedure was repeated over a certain time interval displayed on the computer, which ranged from 5 minutes to an hour. At the end of the test, the erosion rate ($\dot{z}$) is interpreted as the distance the soil sample is advanced (mm) divided by the test duration. Shear stresses (τ) are calculated via equation 3.2 using the average velocity during the test. This procedure is repeated at different velocities to generate an erosion function displayed in Figure 1.2, plotting τ versus $\dot{z}$. Talladega samples 78296 and 78301 were subject to this particular test.

This procedure can be very ambiguous. Scour may not occur uniformly over the sample because of several factors, such as surface preparation. As a result, the surface can become very uneven during the duration of the test. Some of the surface may have scoured more than 1 mm, and some may have not scoured at all. Whenever this occurs, it is very difficult for the operator to judge when an average of 1 mm has completely eroded.

Because of the problems posed by surface irregularity, a new scour-measuring technique was attempted, called the modified method. The EFA was operated the same, and the sample surface was trimmed as flush as possible with the top of the Shelby tube. Next, a dry paper towel was folded and its dry mass was recorded (m_{dry}). A syringe dropped water on top of the sample to fill any voids with water to the surface. The folded paper towel was placed over the surface to soak up the water in the voids, and a "wet" mass (m_{wet}) was recorded. The difference in mass between m_{dry} and m_{wet} represents the mass of water filling the voids. Using the density of water, the volume of voids was calculated. An EFA test was performed for a known time interval without raising the sample 1 mm into the flow. Newly developed voids were again filled with water, and the same procedure was repeated to calculate the increase in void volume on the surface. Hence, the difference between the calculated volumes before and after each EFA test corresponded to the volume of soil lost. This could eventually be converted into average erosion rates in terms of mm/hr. An example calculation for the modified method can be found in Appendix B.

Talladega samples 78305 and 78311 were subjected to this particular method, and comparisons were made to determine its overall effectiveness. In addition, Talladega sample 78316, the Sumter County sample, and all Dallas County samples were simply tested with 1 mm above the flume bed, but no erosion functions were generated since no measurable erosion occurred at large times with the EFA operating at maximum velocity.

2.4 Model Soils

Reduced erodibility is influenced by cohesive forces in soil which depend on the content of fine-grained particles. To quantify this dependence model soils were investigated. Model soils were fabricated with sand and a specific amount of laboratory grade sodium bentonite. These soils were subjected to conventional EFA testing as well as impinging jet testing, in which the soil was compacted in small PVC caps and placed under an orifice at the bottom of a vertical column filled with water to impinge on the soil sample. The head in these tests was not constant with a linear decrease in velocity with respect to time. Impinging jet testing was mostly for relatively small bentonite contents. With these model soils, parameters such as density, water content, and clay content could be controlled and used for data collection and comparison to Alabama soils. Attempts to find any correlations between these two testing methods were made, as well as correlations between critical shear stress, wet density, and bentonite content.

Dry, uniformly graded sand was mixed with clay (bentonite) to achieve desired clay contents and form model soils. Next, water was thoroughly mixed into the soils to achieve a desired water content of 20 percent. Clay content varied from 3 to 25 percent for EFA testing and 3.2% to 7.2% for impinging jet testing. Soils were compacted at wet densities ranging from 1.0 g/cm^3 to 1.9 g/cm^3 for impinging jet testing and 1.7 g/cm^3 or 1.9 g/cm^3 for EFA testing (Figure 2.2). These densities and clay contents provided a good range of erosion resistance for the model soils. A grain size distribution plot of the sand is displayed in Figure 5.1.

Figure 2.2: Formation and Compaction of Models Soils, Water Content = 20%

2.5 EFA Testing – Model Soils

These model soils described in section 2.4 were subject to conventional EFA testing. The soil was manually compacted into the core injection cylinder, which was already fixed in the EFA as shown in Figure 2.2. In order to determine the soil density, the mass of the total mixed soil was measured. After some soil was compacted into the EFA, the remaining soil mass was measured. The difference between the two corresponded to the total mass in the CIC. In order to determine the volume, the EFA piston was raised to be perfectly even with the CIC surface. Using the erosion computer software to track the piston height, the piston was lowered in 0.5 mm increments until the desired distance was achieved (which was usually around 7 to 8 cm). Knowing these two elements, density could be determined (1.7 g/cm^3 or 1.9 g/cm^3). It should be noted that it is not possible to assure uniform density in the entire sample, with smaller densities leaving more voids in random places.

2.6 Impinging Jet Testing – Model Soils

Model soils were subjected to impinging jet testing, in which the mixed soil was compacted in small PVC caps and placed two inches under an orifice at the bottom of a vertical column filled with water to impinge on the soil sample as shown in Figures 2.3 and 2.4. This test was developed to facilitate more rapid testing in soils of small and variable clay content. The erosion mechanism of the impinging jet is very different from EFA testing; with zero or very small clay content EFA testing is impractical. Erosion rates are impossible to determine with the EFA for bentonite contents less than 8% with a rapid rate of increase in erosion with respect to time. Soils with bentonite contents greater than 8% produced erosion rates much more uniform with respect to time.

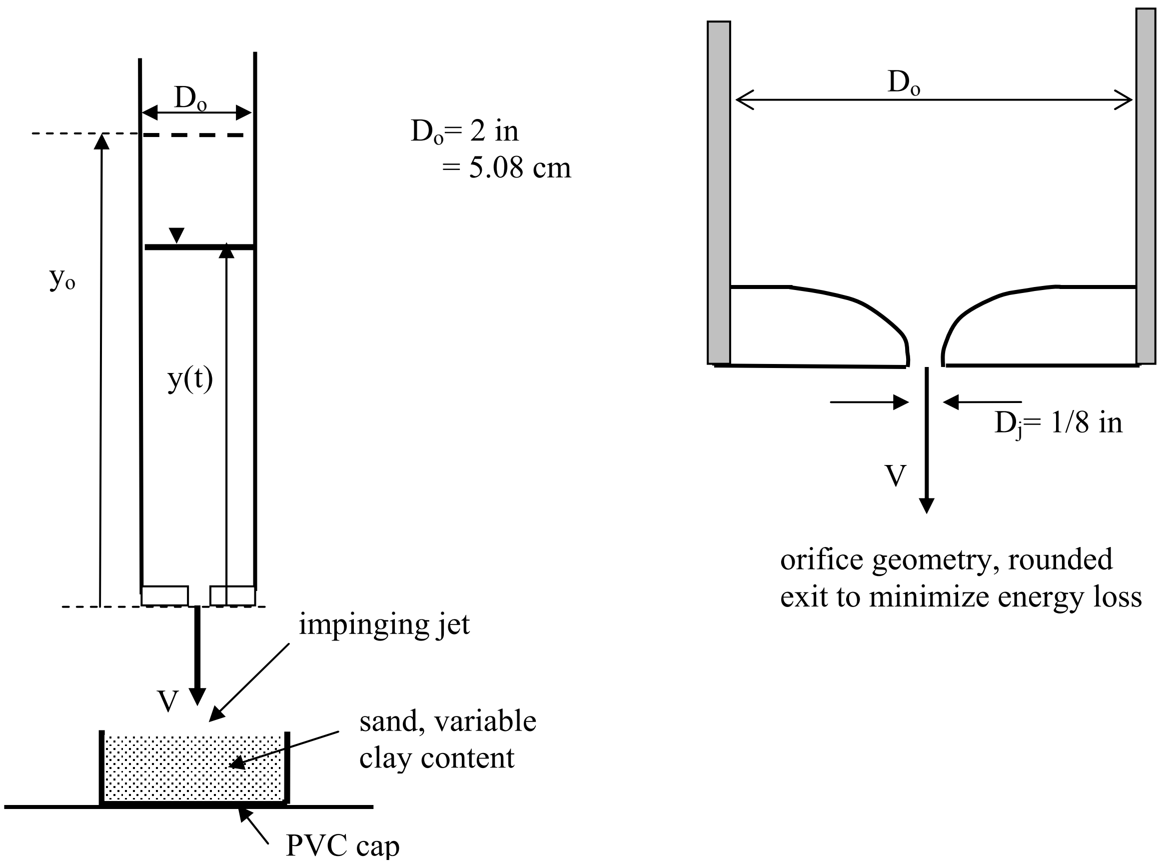


Figure 2.3: Impinging Jet Test Geometry

The soil mass compacted in the PVC cap could easily be calculated by measuring the mass of the cap and re-measuring it after the soil was added; the difference between the two masses is the total soil mass compacted into the cap. Hence, soil wet density could be obtained by dividing the soil mass by the cap volume. The soil mass lost (Δm) could then be determined by measuring the initial water content and calculating the initial dry mass. After the test, the remaining soil was extracted and dried in an oven at 275 ° F. The dry mass of the remaining sample was measured; hence, the difference between the initial dry mass and the remaining dry mass was recorded as the mass lost, or Δm. Bentonite contents for these samples ranged from 3.2% to 7.2%.

Figure 2.4: Impinging Jet Test Setup

CHAPTER THREE

THEORY AND DATA REDUCTION

3.1 Scour Rate

The scour rate ($\dot{z}$) is:

$$\dot{z} = \frac{\Delta h}{\Delta t} \tag{3.1}$$

where Δh = the height of soil projection in the EFA (Figure 1.2) and Δt = the time required to erode the projection. Units for the scour rate were mm/hr.

For irregular surface erosion the scour rate was estimated based on a measure of the total volume of eroded soil. Water was added to the irregular surface to fill the eroded volumes. This water was collected by a dry paper towel and the mass was measured using a differential gravimetric technique. The water mass divided by the water density (1.0 g/cm^3) is the volume which approximates the total volume of eroded soil. This volume was divided by the cross-sectional area of the Shelby tube or CIC to calculate a depth of erosion, which was divided by the test duration to obtain a scour rate in mm/hr. For example, if the mass of water filling the volume of soil eroded was 5 g over a 30 minute test, the corresponding volume of water is 5 cm^3. An ASTM Shelby tube with an area of 45.58 cm^2 was used; hence, the depth of eroded soil is 5 cm^3 divided by 45.58 cm^2, which is 0.11 cm. The test occurred over 30 minutes, so this depth would be multiplied by 2 to convert it into a final scour rate of 0.11 cm/hr, or 1.1 mm/hr.

3.2 Shear Stress

The test section of the EFA is a rectangular conduit with width b = 10.16 cm and a = 5.08 cm. Shear stress in the EFA is:

$$\tau = \frac{\rho f V^2}{8} \tag{3.2}$$

where τ = shear stress (N/m^2), ρ = density of water at room temperature (1000 kg/m^3), V = average conduit velocity, and f = friction factor. Theoretically, f is based on the Moody diagram, relative roughness (roughness height/conduit diameter), and the Reynolds number, which is calculated as:

$$\mathrm{Re} = \frac{VD}{\upsilon} \tag{3.3}$$

where D = hydraulic diameter of the pipe and υ = kinematic viscosity of water (10^{-6} m^2/s at 20 ° C). Hydraulic diameter, D, is calculated as 4 times hydraulic radius, R. Hydraulic radius is defined by flow area divided by wetted perimeter. In terms of EFA flow, this is represented as:

$$R = \frac{ab}{2(a+b)} \tag{3.4}$$

where a and b = the dimensions of the rectangular conduit. When converted to hydraulic diameter, equation 3.4 multiplied by 4 is:

$$D = \frac{2ab}{(a+b)} \tag{3.5}$$

The EFA flume is considered smooth. For turbulent flow in smooth conduits, the following are approximate descriptions of friction factor and Reynolds number dependence. For Re > 3000:

$$\frac{1}{\sqrt{f}} = 2.0 \log (\mathrm{Re}\sqrt{f}) - 0.8 \tag{3.6}$$

When Re < 10^5, equation 3.6 can be approximated as:

$$f = \frac{0.316}{\mathrm{Re}^{1/4}} \text{ (Henderson, 1966)} \tag{3.7}$$

which is known as the Blasius equation. As noted by Crim (2003), equations 3.6 and 3.7 can be used to estimate the friction factor without having to use the Moody diagram, even though iteration may be required (Henderson, 1966). For example, the EFA flume is approximately 0.00516 m^2, which corresponds to a hydraulic diameter of 0.0677 m using equation 3.5:

$$\mathrm{D} = \frac{2(.1016)(.0508)}{(.1016 + .0508)} = 0.0677 \text{ m}$$

The corresponding Reynolds number at a critical velocity of 1 m/s is calculated using equation 3.3:

$$\mathrm{Re} = \frac{(1 \text{ m/s})(.0677 \text{ m})}{10^{-6} \text{ m}^2/\text{s}} = 67,700$$

Since Re < 10^5, the Blassius equation may be used:

$$f = \frac{0.316}{67700^{1/4}} = .0196$$

The corresponding value of τ_c is 2.45 N/m^2 using equation 3.2:

$$\tau_c = \frac{(1000 \text{ kg/m}^3)(.0196)(1 \text{ m/s})^2}{8} = 2.45 \text{ N/m}^2$$

In reality, a stream or river is going to be much bigger in area, and hydraulic diameter will be calculated differently in an open channel than a closed conduit. Figure 3.1 depicts the geometry of a rectangular open channel:

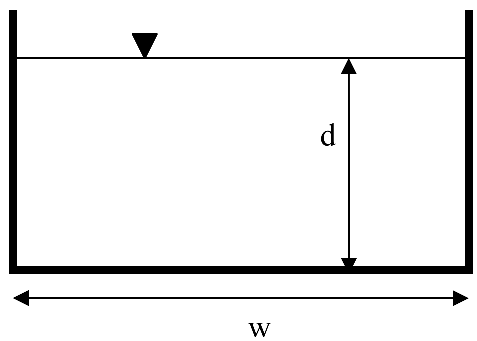


Figure 3.1: Geometry of Open-Channel Rectangular Flow

where d = flow depth and w = channel width. Using this new geometry, R is calculated as:

$$R = \frac{dw}{w + 2d} \tag{3.8}$$

Multiplying equation 3.8 by 4 to convert to hydraulic diameter yields equation 3.9:

$$D = \frac{4dw}{w + 2d} \tag{3.9}$$

These two equations only represent rectangular channels, and other cross-sectional shapes may apply (trapezoidal, triangular, etc.). Nevertheless, a bigger flow area which can significantly reduce f represents an entirely different scenario than EFA flow. For example, a river with dimensions w = 10.16 m and d = 5.08 m (flow area of 51.6 m^2) composed of the same soil in the EFA example will have a hydraulic diameter of 10.16 m using equation 3.9. This will result in a Reynolds number of 10.16×10^6, over 100 times greater than the previous calculation. Through trial and error calculations of equation 3.5, the friction factor for this scenario is found to be 0.0081, less than half the EFA flume factor. The corresponding shear stress is 1.01 N/m^2, only 41% of the EFA critical shear stress. Assuming the critical shear stress remains the same in the river, the corresponding critical velocity would be 1.56 m/s, over 3/2 times the EFA value. Thus, EFA velocities producing critical shear stress will be much lower in the EFA than in streams and rivers.

For this particular study, the friction factor was simply assumed to equal 0.01. Previous results from Crim (2003) produce a low variation in the calculation of f which can be ignored, and shear stress values calculated with an assumed friction factor of 0.01 will provide conservative estimates of τ and τ_c that will produce a higher factor of safety.

3.3 Critical Shear Stress & Initial Erodibility

Critical shear stress (τ_c) is the point in the erosion function at which scour first occurs in the soil. This critical shear stress was found strictly by observation during an EFA test. Flow was gradually increased during a test until measurable erosion began to occur. If some erosion initially occurred at the start of test (first 5-10 minutes) but did not occur for the remaining duration, the rate was interpreted as 0 mm/hr and τ_c had yet to be reached. The initial erosion was attributed to soil disturbance caused by surface preparation.

Initial erodibility (S_i) is the measure of how fast scour initially occurs just after the critical shear stress is reached. Crim (2003) and Briaud et al. (2001a) found good correlations between S_i and pockets of loose soil in the sample. Initial erodibility could not be calculated in Alabama soils with difficulty surface preparation that prevented erosion functions from being generated. It was neglected for EFA model soil testing.

3.4 Impinging Jet Erosion

For impinging jet testing, the jet velocity is calculated based upon Bernoulli's equation:

$$\frac{p_1}{\gamma} + \frac{u_1^2}{2g} + z_1 = \frac{p_2}{\gamma} + \frac{u_2^2}{2g} + z_2 + h_L \tag{3.10}$$

where p = water pressure (N/m^2), g = acceleration of gravity (9.81 m/s^2), u = velocity (m/s), z = elevation head (m), and h_L = orifice head loss (m). Neglecting head loss through the orifice, this equation applied directly to the test setup described in Chapter 2 is rearranged to equation 3.11:

$$u = \sqrt{2g(z_2 - z_1)} \tag{3.11}$$

The term $(z_2 - z_1)$ can be substituted by a single variable, y (water depth in the column), for this setup.

Equation 3.12 represents the conservation of volume principle specifically for this experiment:

$$\frac{\pi D_o^2}{4}\frac{dy}{dt} = -u\frac{\pi D^2}{4} \tag{3.12}$$

Simplifying equation 3.12 yields 3.13:

$$\frac{dy}{dt} = -u\left(\frac{D}{D_o}\right)^2 \tag{3.13}$$

Taking the derivative of velocity with respect to time in 3.11 yields 3.14:

$$\frac{dy}{dt} = \frac{u}{g}\frac{du}{dt} \tag{3.14}$$

Setting 3.13 and 3.14 equal to each other:

$$\frac{du}{dt} = -g\left(\frac{D}{D_o}\right)^2 \tag{3.15}$$

Separating and integrating 3.15 with the initial condition $u = u_o$ when $t = 0$ produces 3.16:

$$u = u_o - g\left(\frac{D}{D_o}\right)^2 t \tag{3.16}$$

The initial jet velocity from equation 3.11 is $u_o = \sqrt{2gy_o}$. When the initial depth, y_o, is 70 cm, $u_o = 3.71$ m/s. From 3.16, after setting $u = 0$ and $t = t_o$, the drain time is $t_o = \frac{u_o}{g}\left(\frac{D_o}{D}\right)^2$.

With the linear jet velocity variation with time, a total impulse applied to the soil during the impinging jet test can be calculated. From momentum analysis the force at the water-soil interface is

$$R = \rho Q u = \rho A u^2 \tag{3.17}$$

The total impulse is

$$I_j = \int_{t=o}^{t_o} R\,dt \tag{3.18}$$

Substituting equation 3.16 into 3.17 and integrating 3.18 will yield equation 3.19:

$$I_j = \rho A \int_{t=0}^{t_o} \left[u_o - g\left(\frac{D}{D_o}\right)^2 t \right]^2 dt$$

$$I_j = \left[\frac{\rho A}{3g\left(\frac{D}{D_o}\right)^2} \left[u_o - g\left(\frac{D}{D_o}\right)^2 t \right]^3 \right]_{t=0}^{t_o}$$

$$I_j = \rho A {u_o}^2 \frac{t_o}{3} \tag{3.19}$$

This equation was applied for total impulse calculations in these experiments with units in N-s.

CHAPTER FOUR

TEST RESULTS – ALABAMA SOILS

4.1 Talladega Sample Results

Every Talladega County sample in this study came from the same depth. The top of each sample was at an elevation of 0.31 m, and the bottom of each sample was at an elevation of 0.91 m. Soil properties performed on four of the eight samples are presented in Table 4.1. Some are very consistent, with every sample having around 15% of its particles smaller than the number 200 sieve (0.075 mm). The plasticity index (liquid limit – plastic limit) for each sample is around 6%. From these two properties, every sample is classified as clayey sand (SC) from the United Soil Classification System.

Table 4.1: Talladega Soil Sample Properties

Sample Identification Number	Water Content %	Liquid Limit %	Plastic Limit %	Plasticity Index LL - PL	Dry Density g/cm^3	% < No. 200 Sieve
78296	26.4	27.0	21.1	5.9	1.635	15.2
78301	13.7	22.7	17.0	5.7	1.817	14.2
78311	18.8	28.4	22.2	6.2	1.658	14.9
78316	12.7	24.5	18.0	6.5	1.92	N/A

Different methods were applied to different samples to measure erosion. However, there was a common erosion pattern for every tested sample. Much silty, clayey emission occurred around the edges, and erosion began to occur upstream on the surface, usually with a large spall suddenly being removed from the sample as shown in Figure 4.1:

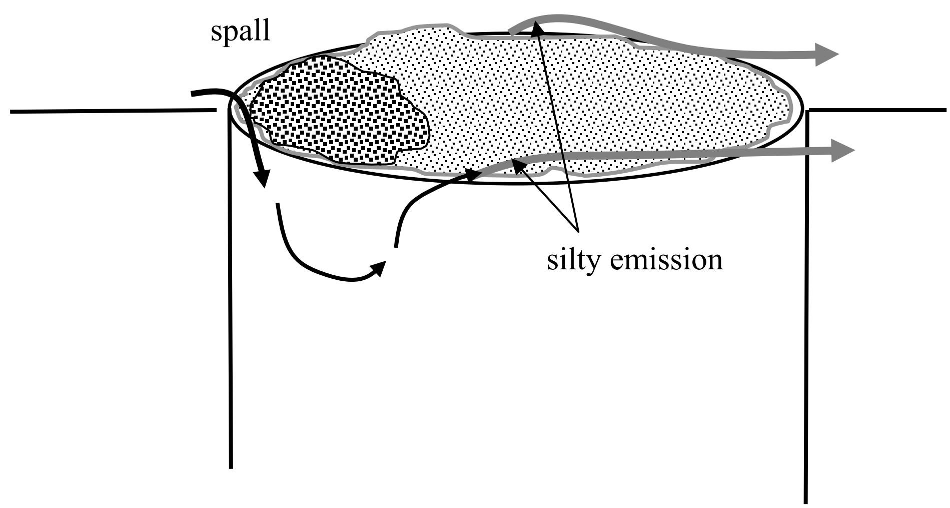


Figure 4.1: Flow Path for Silty Emission from Lateral Boundaries

Talladega samples 78296 and 78301 were tested and the soil eroded measured using conventional EFA procedures described in section 2.3. There was no difficulty in preparing a smooth, flat surface to be exposed to uniform shear stress. Soil was projected into a given velocity for erosion in 1 mm increments. After the test, the average velocity was determined, and the erosion rate was interpreted as the total eroded height divided by the test duration. This procedure was repeated at different velocities to generate an erosion function displaying shear stress (N/m^2) versus erosion rate (mm/hr). Results for sample 78296 are shown on Figure 4.2 and Table 4.2, and sample 78301 is shown in Figure 4.3 and Table 4.3:

Table 4.2: Talladega Sample 78296 EFA Test Results

Velocity, m/s	Shear Stress, N/m^2	Eroded Depth, mm	Test Duration, minutes	Erosion Rate, mm/hr
0.491	0.301	0	60	0
0.786	0.772	3	60	3
1.334	2.224	22	60	22
1.557	3.030	18	60	18
1.995	4.975	25	15	100

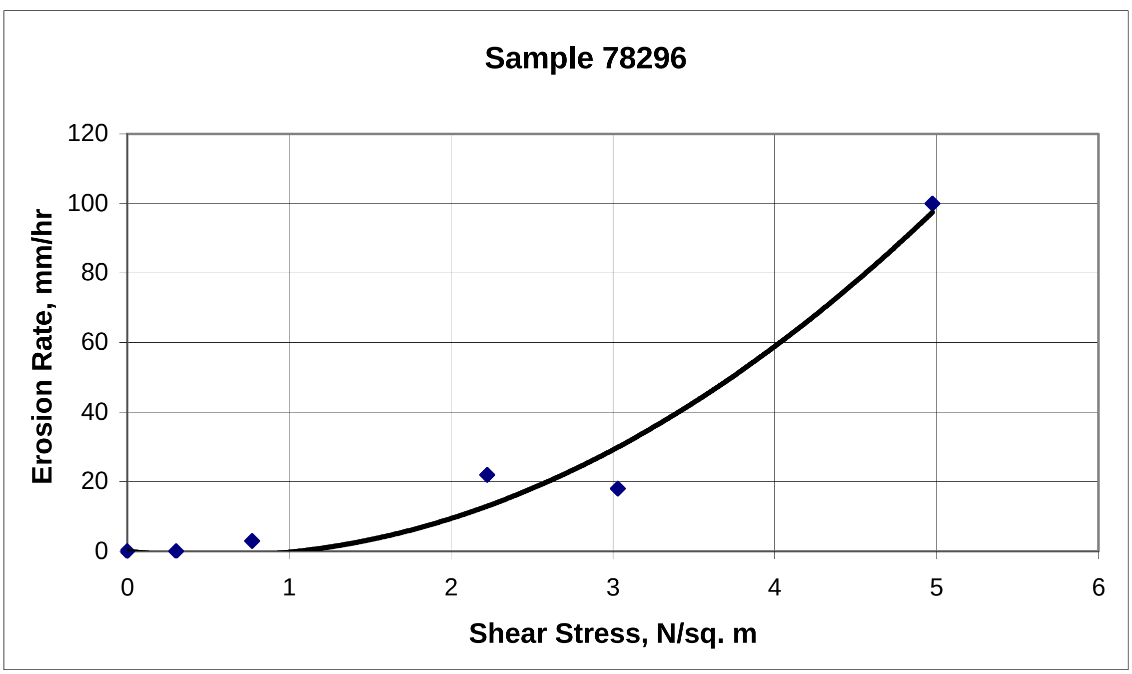


Figure 4.2: Talladega Sample 78296 EFA Test Results

Table 4.3: Talladega Sample 789301 EFA Test Results

Velocity, m/s	Shear Stress, N/m^2	Eroded Depth, mm	Test Duration, minutes	Erosion Rate, mm/hr
0.198	0.049	0	60	0
0.507	0.321	5	60	5
0.730	0.666	6	60	6
1.170	1.710	21	60	21
1.468	2.694	20	10	120

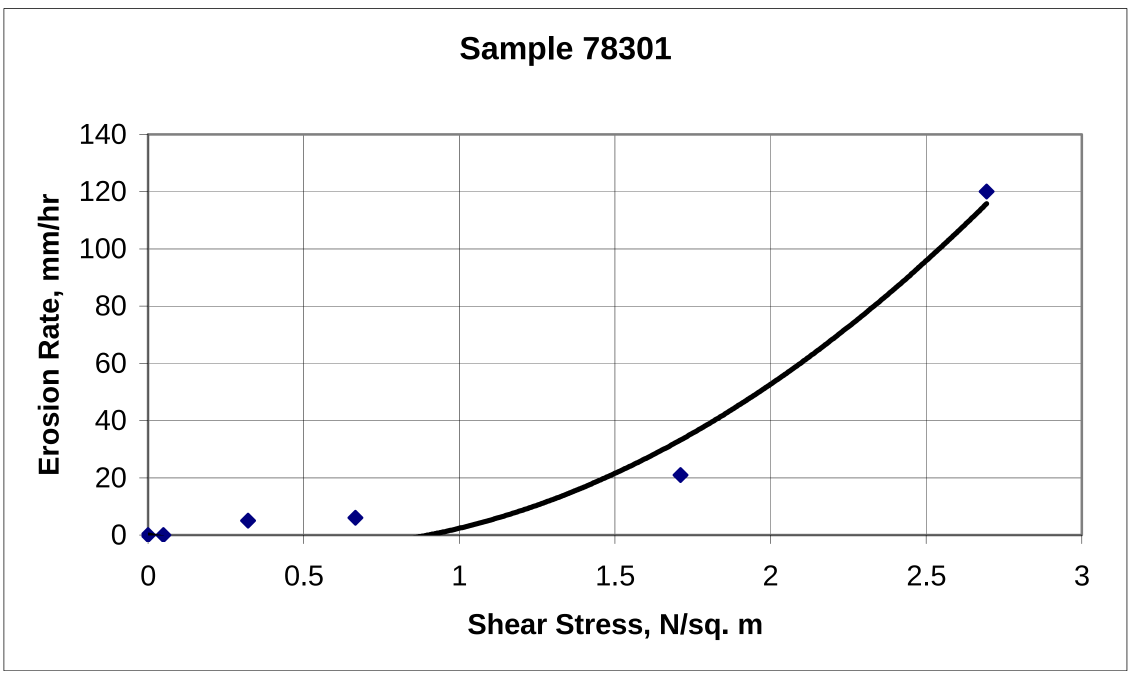


Figure 4.3: Talladega Sample 78301 EFA Test Results

Both samples appear to display a non-linear relationship between shear stress and erosion rates. They show erosion increasing at a growing rate with respect to shear stress. Sample 78296 shows a significant jump in erosion from 3 N/m^2 to 5 N/m^2, and sample 78301 shows a large jump between 1.7 N/m^2 and 2.7 N/m^2. Critical shear stress is almost ten times lower in sample 78301 than 78296, and significantly higher erosion rates occur at lower shear stresses for 78301. This is most likely related to water content. Sample 78296 has a water content of 26%, twice the water content of 78301 at 13%. The strength of cohesive soils is a strong function of water content, and this would explain why 78301 was significantly lower in scour resistance.

The soil eroded for samples 78305 and 78311 was measured using the modified method described in section 2.3 and Appendix B. There was much difficulty in preparing a smooth, flat surface, so water was dropped into any initial surface voids and collected with a dry paper towel. The masses of the dry and wet paper towel were recorded, and the difference corresponds to the mass of water filling the voids. Using the density of water (1 g/cm^3), the volume of the voids could be calculated. An EFA test was performed without projecting the sample 1 mm into the flow, and the procedure was repeated to measure any new voids caused by erosion. The difference between void volume before and after an EFA test corresponds to the mass of soil eroded. Hence, erosion rates can be calculated and converted to mm/hr.

Results for samples 78305 and 78311 are displayed in Table 4.4 and 4.5. It should be noted that a new sample surface was not prepared for each test at the same velocity and shear stress. For example, the sample surface at the end of a 10 minute test was used at the beginning of a 30 minute test.

Table 4.4: Sample 78305 EFA Test Results

Test Duration, minutes	Surface void volume increase, mm^3	Erosion Rate, mm^3/hr	Erosion Rate, mm/hr
10	498	2988	0.73
30	461	922	0.23
60	1004	1004	0.25

Sample 78305, V = 0.70 m/s, τ = 0.61 N/m^2

Test Duration, minutes	Surface void volume increase, mm^3	Erosion Rate, mm^3/hr	Erosion Rate, mm/hr
10	209	1254	0.31
30	403	806	0.20
60	587	587	0.15

Sample 78305, V = 1.20 m/s, τ = 1.8 N/m^2

Test Duration, minutes	Surface void volume increase, mm^3	Erosion Rate, mm^3/hr	Erosion Rate, mm/hr
10	4454	26724	6.56
30	1407	2814	0.69
60	134	134	0.03

Sample 78305, V = 1.90 m/s, τ = 4.5 N/m^2

Table 4.5: Sample 78311 EFA Test Results

Test Duration, minutes	Surface void volume increase, mm^3	Erosion Rate, mm^3/hr	Erosion Rate, mm/hr
10	4380	26280	6.45
30	514	1028	0.25
60	1381	1381	0.34

Sample 78311, V = 0.80 m/s, τ = 0.64 N/m^2

Test Duration, minutes	Surface void volume increase, mm^3	Erosion Rate, mm^3/hr	Erosion Rate, mm/hr
10	4368	26208	6.44
30	3473	6946	1.71
60	6137	6137	1.51

Sample 78311, V = 1.60 m/s, τ = 3.2 N/m^2

Every test for the two samples shows an overall decrease in erosion rate with respect to time. For example, sample 78305 at V = 1.90 m/s produces an erosion rate of 6.56 mm/hr during the first 10 minute test, but it is only 0.03 mm/hr during the last 60 minute test. This is an illustration of mechanically induced erosion that results from surface preparation, with equilibrium (zero erosion) eventually being achieved. Sample 78311 erodes in the same manner, but has somewhat significant erosion at 1.51 mm/hr with V = 1.60 m/s.

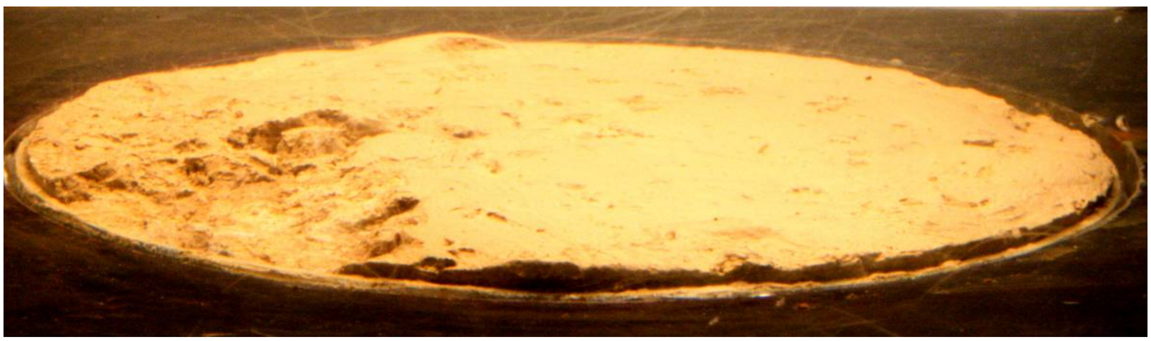
The measured erosion rates for 78305 and 78311 are believed to be authentic. Observations were made and recorded when tests were in progress, and it was observed that very little, if any, erosion occurred for these samples (other than some mechanical erosion from surface preparation). Despite potential error from evaporation or water curvature on the surface, the measurements made using the paper towel and syringe are believed to be accurate because they verify a common pattern of erosion occurring initially from surface preparation and eventually achieving equilibrium. If it is desired that this technique be used, it should be done with stiff silty or clayey soils to keep water infiltration within the sample at a minimum. Water will remain in the surface voids for easy collection by the paper towel.

It is concluded that the conventional method used for samples 78296 and 78301 is better than the modified method used on 78305 and 78311 for developing shear stress-erosion rate curves. Using the conventional method, a larger portion of the sample will be subjected to direct shear, which will provide much greater information regarding the erosive nature of the soil. Both of these methods simulate flood conditions for riverbeds very well, but the collected data shows that the conventional method prescribed by Briaud will provide much more conservative scour estimates for design purposes.

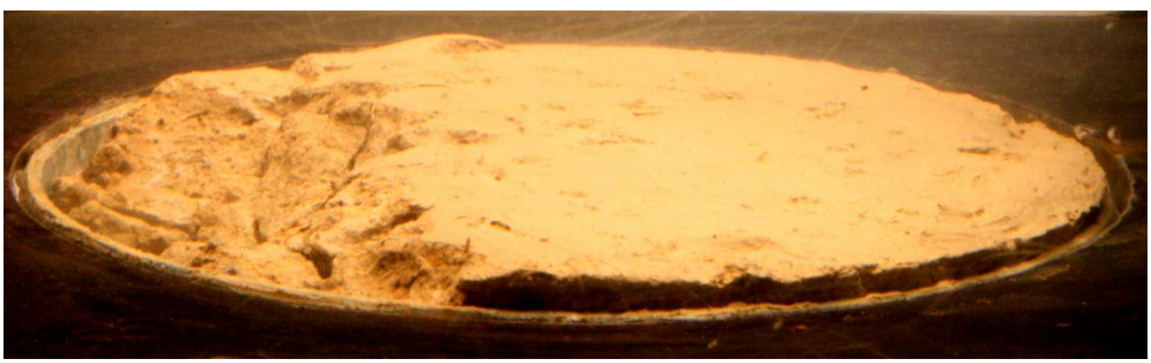
In addition, two EFA tests were performed on Talladega sample 78316 with the sample projected 1 mm into the flow. These two tests are referred to as 316A and 316B, which were performed on the top 20 cm segment of the supplied sample (if z = 0 at the top of the Shelby tube, then this test was performed on z = 0 – 20 cm). There was much difficulty in surface preparation for test 316A and 316B, and this resulted in soil disturbance and surface irregularities which made it impossible to interpret exact erosion rates. From observation, about half of the original volume above the flume bed eroded during an 87 minute test at an average velocity of 1.30 m/s.

Test 316B was conducted for approximately 4.60 hours. Velocity was set to 0.60 m/s for the first 3.17 hours and 0.95 m/s for the final 1.43 hours. At 2.25 hours, a large spall suddenly flew out of the upstream 1/3 end, as shown in Figure 4.4. Along with the same silty emission from the edges, this void grew larger at V = 0.95 m/s for the final 1.43 hours. After testing, this sample was ejected 5 cm above the Shelby tube surface for examination. Around a pocket of small pebbles, the eroded pathways were obvious at the leading edge of the sample, as shown in Figure 4.5. In contrast, the downstream 2/3 of the sample surface showed very little erosion over the total duration of 4.60

Test 316B, Test Start



Δt = 2.25 Hours, V = 0.65 m/s



Δt = 4 Hours, v = 0.90 m/s

Figure 4.4: Talladega EFA Test 316B

Figure 4.5: Talladega EFA Test 316B, Test End

<u>4.2 Sumter County (Sucarnoochee River) Sample Results</u>

A total of seven tests were completed on the core sample from the Sucarnoochee River Bridge in Sumter County. The Sucarnoochee sample is not highly compact (blow count = 39) and has the following properties:

- Liquid Limit = 72
- Plastic Limit = 43
- Plasticity Index = 29
- Water content = 41.2
- 98 percent smaller than the No. 200 sieve (0.075 mm)

Based on this data, this core is classified as high plasticity clay (CH) by the United Soil Classification System. Core diameter equaled 4.763 cm, so samples were placed in the

4.75 mm diameter CIC with little to no difficulty. Tests termed Sucarnoochee01 through Sucarnoochee05 were completed on the upper segment of the core (elevation = 26.5 ft – 27.0 ft), while Sucarnoochee06 and Sucarnoochee07 were done on the bottom portion of the core (elevation = 30.0 ft – 31.5 ft).

To avoid any mechanically induced erosion (flaking), initial erosion was allowed to happen, and equilibrium was established soon thereafter. Next, velocity was increased to result in larger scale failures of the soil surface. For every test, critical velocity and corresponding critical shear stress were in the range of 0.30 m/s < V < 0.70 m/s and 0.22 $N/m^2 < \tau < 1.20\ N/m^2$.

Actual erosion functions for these samples could not be developed. The soil structure was complex; it was composed of horizontal flakes or horizontal planes that separated with the slightest disturbance (see Figure 4.6 and 4.7). As a result, it was nearly impossible to cut and trim a smooth surface to be flush with the flume bed.

Figure 4.6: Sumter County Sample Structure and Horizontal Layering

Figure 4.7: Sumter County Surface Preparation Difficulties

Hence, a flat surface exposed to a uniform shear stress could not be created, and this is essential to the development of an erosion function for any soil sample. Most of the failures in these tests resulted in large chunks of disturbed soil suddenly leaving the sample, which is not a true representation of the erosive nature of the sample.

However, the test for Sucarnoochee07 was an exception; the preparation of a flat surface with minimal mechanical disturbance was possible, as shown in Figure 4.8.

Figure 4.8: Sucarnoochee07 Prepared Surface at 1 mm Protrusion

The test lasted approximately 2 hours and 15 minutes; during the whole test the sample remained approximately 1 mm above the bed. Some initial erosion occurred at 0.34 m/s, but equilibrium was soon achieved. More erosion occurred at 0.73 m/s, but some soil was still intact at 1 mm. When the velocity was increased to 0.99 m/s, complete and sudden erosion occurred. This is illustrated in Figure 4.9.

Time = 24 minutes, V = 0.32 m/s

Time = 2 hours 10 minutes, V = 0.74 m/s

Time = 2 hours 14 minutes, V = 0.99 m/s

Figure 4.9: Sucarnoochee07 EFA Testing

4.3 Dallas County Sample Results

Conventional EFA testing was performed on Mooreville chalk core samples 78935, 78936, and 78956. Soil properties are displayed in Table 4.6, and these samples should be classified as silty sand (SM).

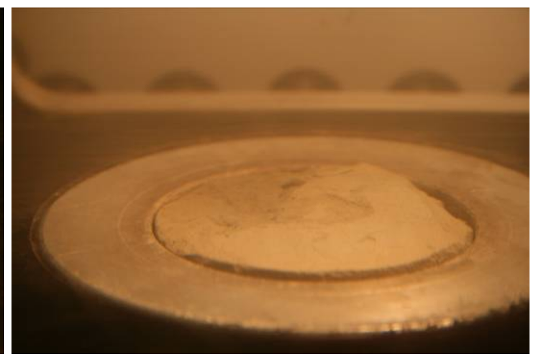
Table 4.6: Mooreville Sample Properties

Sample Identification Number	Liquid Limit %	Plastic Limit %	Plasticity Index LL - PL	Dry Density g/cm^3	% < No. 200 Sieve
78935	35.2	32.6	2.6	1.81	9.4
78956	31.1	30.1	1.0	1.73	10.8

Surface preparation was not an issue. From observation, it was easy to assume that these samples would not erode easily. The valve was initially turned to produce the highest possible EFA velocity (approximately 6 m/s), where it would remain for the entire test duration. The tests yielded minimal erosion. Chalk core 78935 was tested for roughly two hours, where only about 1/3 of the surface above the flume bed suddenly seperated and eroded after an hour of testing; there was no evidence of any overall degradation. Sample 78936 did not show any signs of erosion for two hours, so its assumed rate was 0 mm/hr. Small degradation occurred for sample 78956. During the first hour of testing, there was roughly 1 mm of degradation, but no erosion occurred during the second hour. Based on these results, it is safe to assume V_c and τ_c are close to the highest velocity and shear stress the EFA could yield, 6 m/s and 45 N/m^2, for every Mooreville chalk sample from Dallas County.

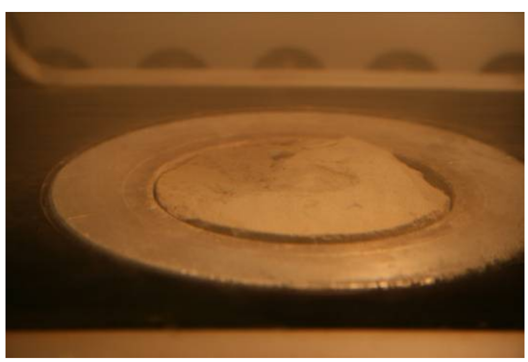
78935, Δt = 1 Hour

78936, Test End, Δt = 2 Hours



78956, Δt = 0

78956, Δt = 1 Hour



78956, Δt = 2 Hour

Figure 4.10: EFA Testing on Dallas County Samples

CHAPTER FIVE

TEST RESULTS – MODEL SOILS

5.1 EFA Test Results

EFA testing of model soil samples offered a clear advantage of controlled characteristics when compared to the natural soil samples. Ninety-eight percent of the sand was smaller than 0.84 mm, and zero percent was retained above the No. 4 sieve (4.76 mm) as shown in Figure 5.1 and Table 5.1. This sand contained a uniformity coefficient of 2.06 and was classified as SP – poorly graded sand. When clay particles were added, surface preparation became extremely easy when compared to natural soils from Alabama stream crossings. The moist soil was compacted uniformly into the CIC and trimmed flush with the flume bed to limit soil disturbance that would induce flaking. Also, unlike the natural soils, erosion rates were not influenced by heterogeneity and large inclusions retained above the No. 4 Sieve. They were very uniform and equilibrium was achieved quickly once testing began (see Figure 5.6).

Model soils were compacted to two densities: 1.7 g/cm^3 and 1.9 g/cm^3. At a density of 1.7 g/cm^3, soil samples contained the following bentonite contents (by mass): 5%, 8.5%, 15.0%, 20.8%, and 25%. Samples compacted to a density of 1.9 g/cm^3 contained bentonite contents of 3.5%, 5%, 8.5%, 12.5%, 15.0%, 20.8, and 25.0%. The significance of these tests lay in comparisons made between the various bentonite contents and their erosion functions.

As shown in Figures 5.2 and 5.3, the most consistent erosion functions were generated by soils with 20% and 25% bentonite compacted at 1.9 g/cm^3. From observation, 8% appeared to be the “boundary” between model soils which behaved in a cohesive or less cohesive manner. Soils initially compacted with clay contents greater than 8% were more amenable to EFA testing; from observation, degradation occurs much more uniformly in them. Samples containing less than 8% clay produced erosion that increased greatly with time; scour holes develop on the surface which grow larger and rapidly accelerate erosion. This acceleration may distort the interpretation of erosion into numerical values, since theoretically the erosion rate is always increasing. Nevertheless, these values were interpreted as best as possible from observation and included.

Both densities reveal some inconsistencies where higher clay contents erode faster at almost the same shear stress. For example, model soil containing 15% eroded at a rate of 66 mm/hr at 17.7 N/m^2, while model soil containing 8.5% bentonite eroded at a rate of 36 mm/hr at 18.1 N/m^2. This is much more prevalent in soils compacted at 1.7 g/cm^3. This is likely because samples compacted at 1.7 g/cm^3 are much less uniform in compaction. With less compaction, more voids will appear in random spots throughout the sample and increase erosion rates, even at lower velocities. However, the repeatability of tests with different samples is very high. For two different soils both containing 8.5% clay at 1.70 g/cm^3, critical shear stresses were 1.112 N/m^2 and 0.979 N/m^2, which is only a 12% difference. Also, for these same samples, an erosion rate of 12 mm/hr was measured at velocities of 2.87 m/s and 2.61 m/s, which differ by 9%.

Figures 5.4 and 5.5 illustrate the significance of density in these EFA tests. For soil that contained 25% clay at 6.0 m/s, the measured erosion rate for the 1.70 g/cm^3 soil was 84 mm/hr, while the 1.90 g/cm^3 soil eroded at a rate of 34 mm/hr. An increase of 0.2 g/cm^3 resulted in a 60% decrease in erosion. For soil composed of 20% clay, this same increase in density resulted in a reduction of almost half the rate at 1.70 g/cm^3 (47.6%).

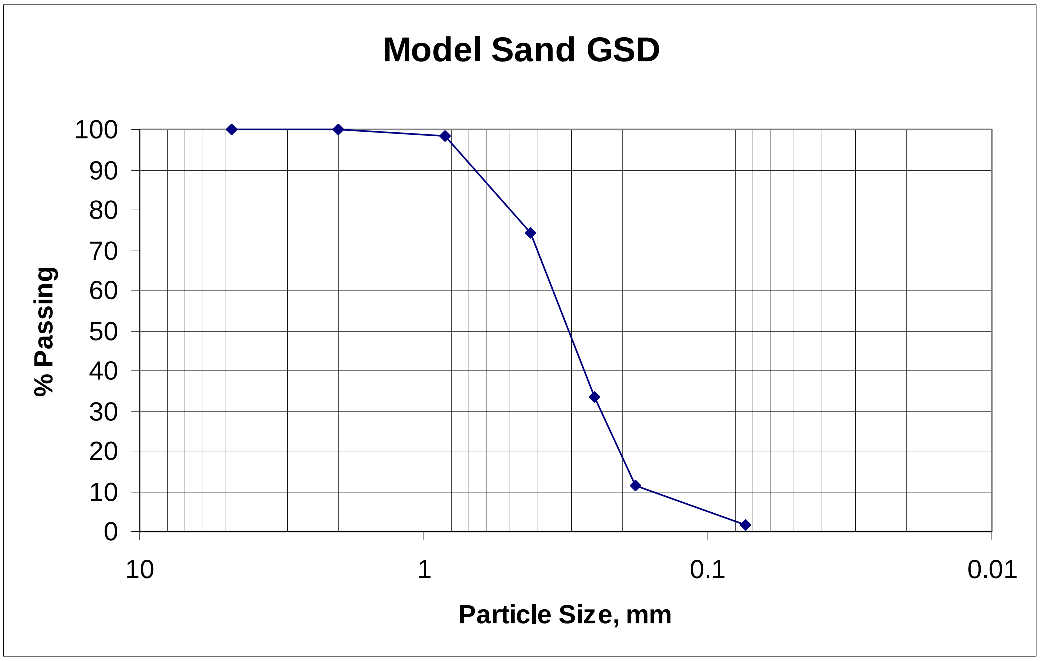


Figure 5.1: Model Sand Grain Size Distribution

Table 5.1: Model Sand GSD

Sieve Size, mm	4.76	2	0.84	0.42	0.25	0.18	0.074
% Passing	100	99.9	98.4	74.1	33.6	11.5	1.6

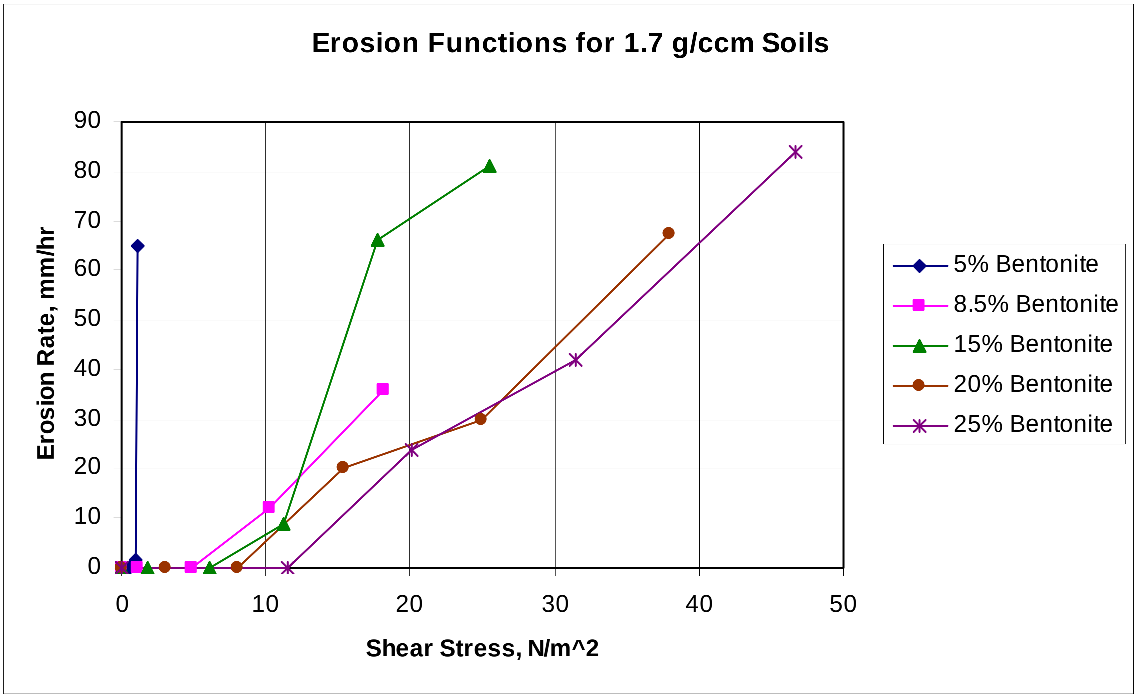


Figure 5.2: Erosion Functions for Model Soils Compacted to 1.7 g/cm^3

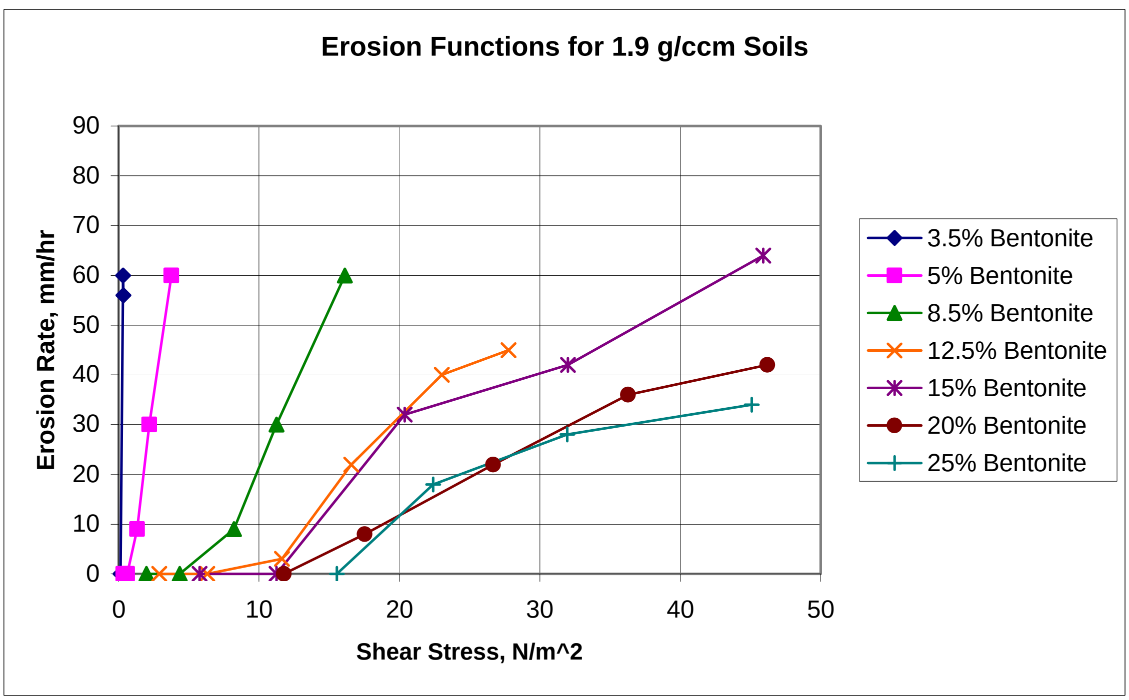


Figure 5.3: Erosion Functions for Model Soils Compacted to 1.9 g/cm^3

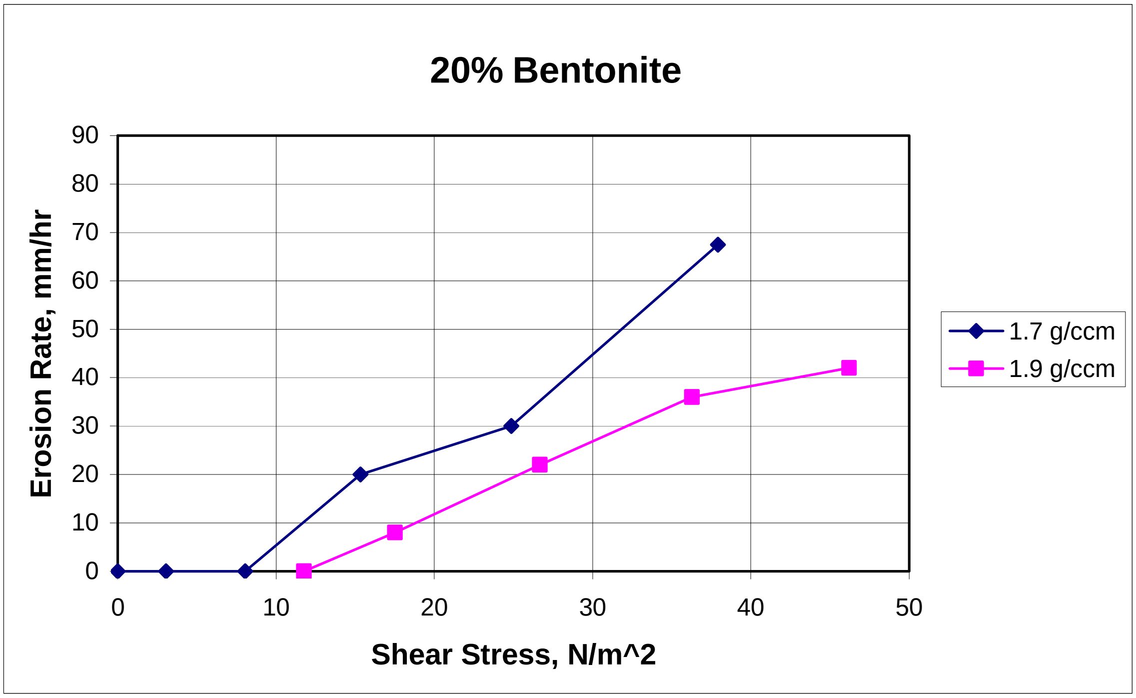


Figure 5.4: Comparison of Erosion Function Densities at 20% Bentonite

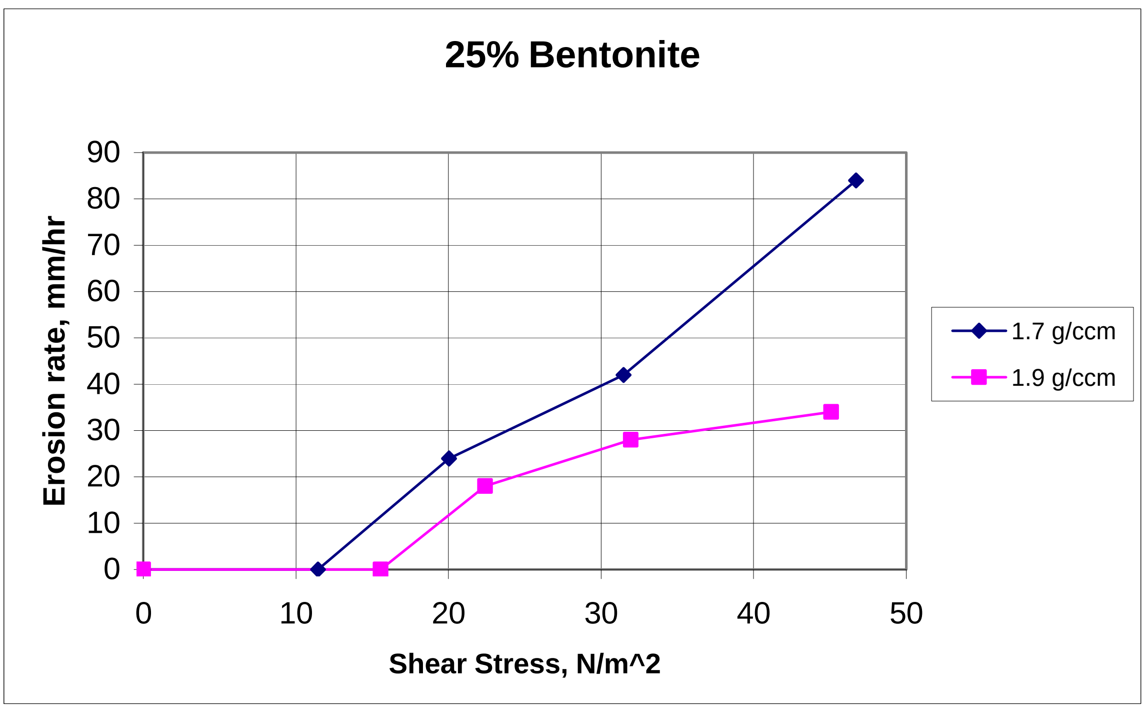


Figure 5.5: Comparison of Erosion Function Densities at 25% Bentonite

Test Start

$\Delta t = 2$ min

$\Delta t = 4$ min

$\Delta t = 5$ min

Figure 5.6: EFA Testing for 15% Bentonite, $\rho = 1.9$ g/cm^3, V = 6 m/s

There is an overall linear relationship between τ_c and clay content for both densities, as shown in Figure 5.7. As expected, higher density yields higher values of τ_c for a given clay content. It should be noted that if data points for 5% bentonite are excluded, the slopes of both trends are close (Figure 5.8). The basis for this exclusion is that model soils with 5% bentonite erode in a less cohesive manner. Also, a linear increase in for clay content corresponds to a linear increase between τ_c and plasticity index (see Table 5.2) and activity for a given soil.

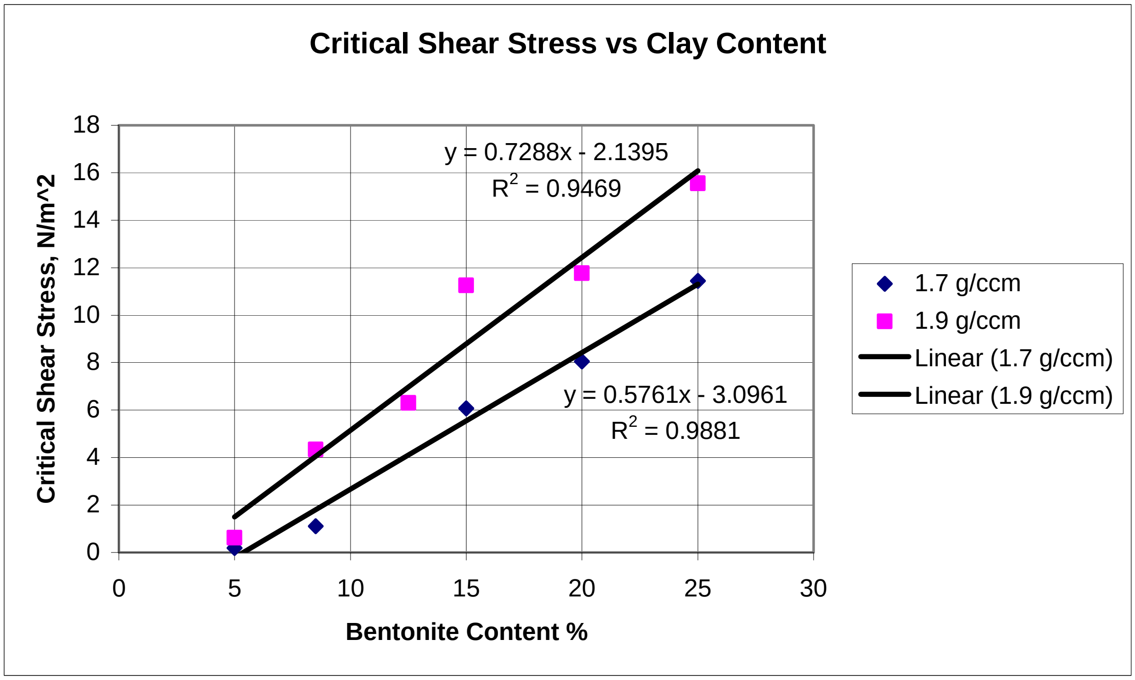


Figure 5.7: Correlations between Critical Shear Stress and Bentonite %

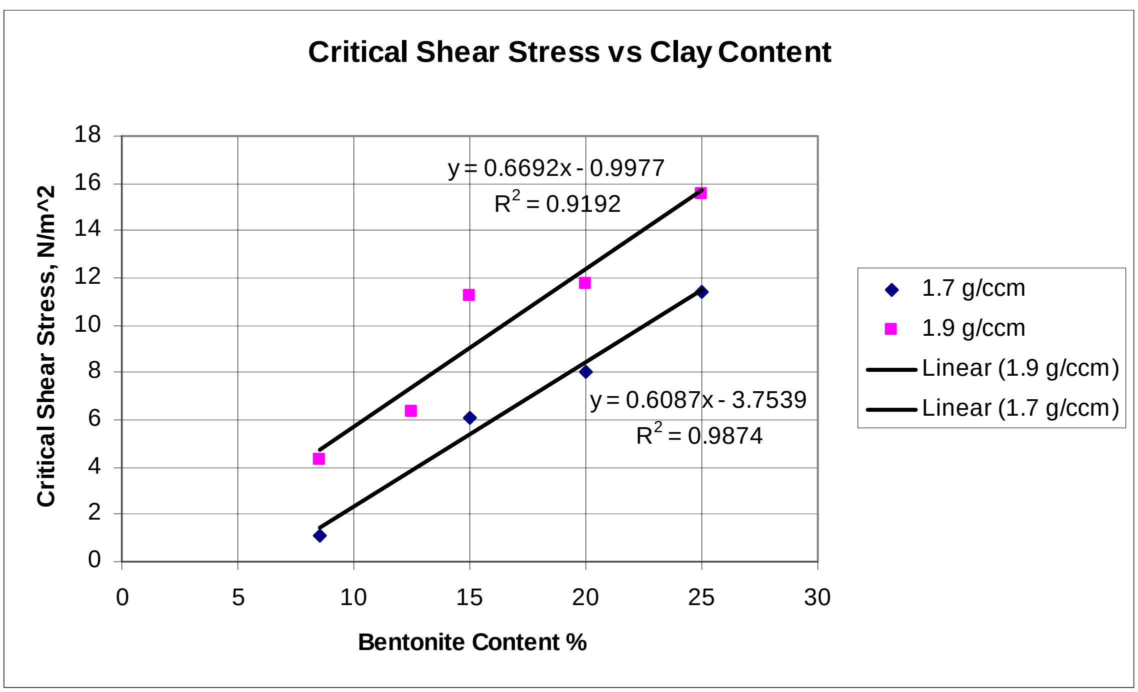


Figure 5.8: Modified Correlations between Critical Shear Stress and Bentonite %

Table 5.2: Model Soil Atterburg Limits

Clay Content	Plastic Limit	Liquid Limit	Plasticity Index
8.5% Bentonite	35.3	44.9	9.8
12.5% Bentonite	43.0	55.0	12.0
15% Bentonite	58.9	32.0	26.9
20% Bentonite	42.7	76.6	33.9
25% Bentonite	35.1	90.6	55.5

5.2 Jet Test Results

Density and bentonite content were varied for samples tested with the impinging jet apparatus. These are tabulated in Table 5.3. One will notice the "upper limit" in these tests – the density and clay content that yielded zero erosion – was 1.841 g/cm^3 and 7.2% bentonite. The sample with 5.2% bentonite compacted at 1.942 g/cm^3 was almost impenetrable, losing only 1.3% its original mass. Similar to EFA erosion, there are some slight inconsistencies in the data; for example, soils containing 5% and 7% clay compacted at almost the same density (1.6 g/cm^3) have almost identical erosion. Again, this is due to less uniformity in the sample when it is not thoroughly compacted; more voids may appear in the center, just under the trajectory of the jets, for the 5% clay sample. For every test the initial velocity was 3.706 m/s, the average velocity was 1.853 m/s, and the drain time was 96 seconds. This corresponds to a maximum shear stress of 138 N/m^2 and an average shear stress of 34.4 N/m^2.

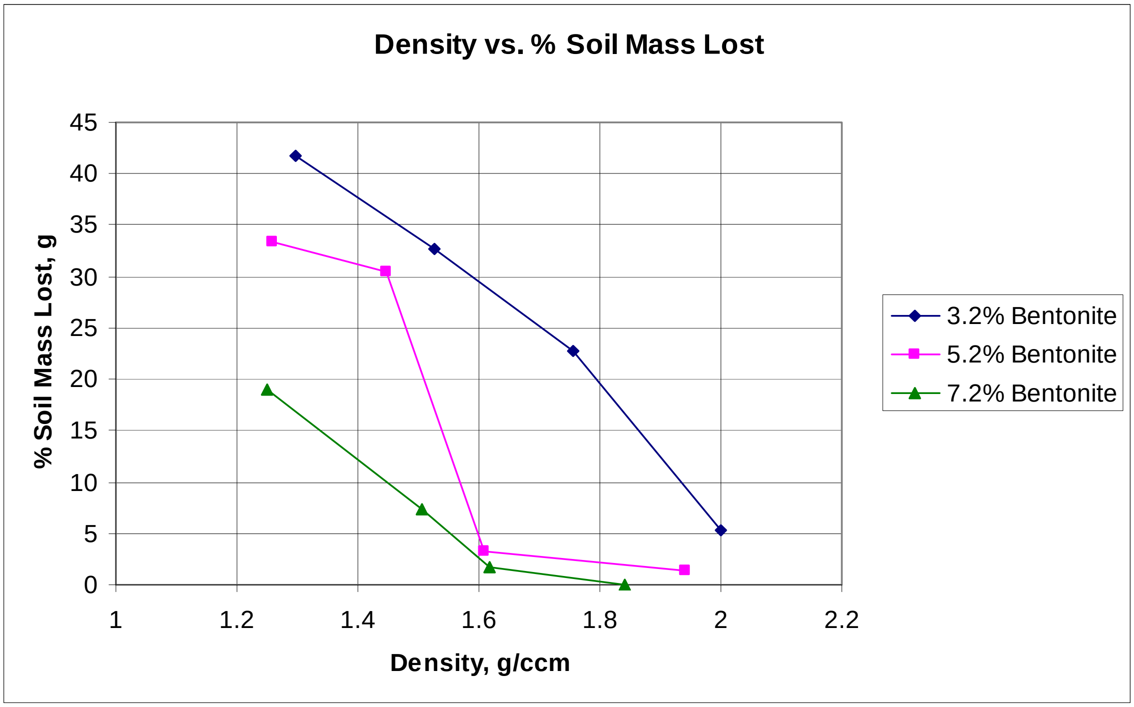


Figure 5.9: Jet Erosion Variation With Density

Table 5.3: Jet Erosion Variation with Density

3.2% Bentonite		5.2% Bentonite		7.2% Bentonite	
Density (g/cm^3)	% Soil Mass Lost	Density, (g/cm^3)	% Soil Mass Lost	Density (g/cm^3)	% Soil Mass Lost
1.296	41.8	1.259	33.3	1.251	19
1.527	32.7	1.447	30.4	1.506	7.4
1.755	22.8	1.609	3.3	1.619	1.7
2	5.2	1.942	1.3	1.841	0

Tests were also conducted to quantify erosion for different impulses. For each test, a different value of y_o was used; these value ranged from 10 cm to 70 cm. One trial used soil composed of 2.9% bentonite, and two trials used 20% kaolin, different clay powder with much smaller activity. Every bentonite sample was compacted to a wet density of 1.50 g/cm^3 with w = 20%, and every kaolin sample was compacted to 2 g/cm^3 at w = 15%. Total impulse was calculated in accordance with equation 3.19. The largest impulse (2.12 N-sec) corresponds to an average velocity of 1.855 m/s and an average shear stress of 34.4 N/m^2. The smallest impulse corresponds to an average velocity of 0.70 m/s and an average shear stress of 4.9 N/m^2.

The results displayed in Figure 5.10 and Table 5.4 show an overall linear increase between Δm and impulse for every trial (after critical shear stress is reached). The two kaolin samples illustrate the reproducibility of impinging jet experiments, which can vary significantly. Both samples were impenetrable at the same impulse, and they both eroded significantly less than the bentonite sample. Results were significantly different, though. At 2.12 N-sec, for example, erosion is 20.5 g for one test and 10.4 for another, which represents a 100% increase. For a given soil sample, multiple tests should be performed and an average eroded mass.

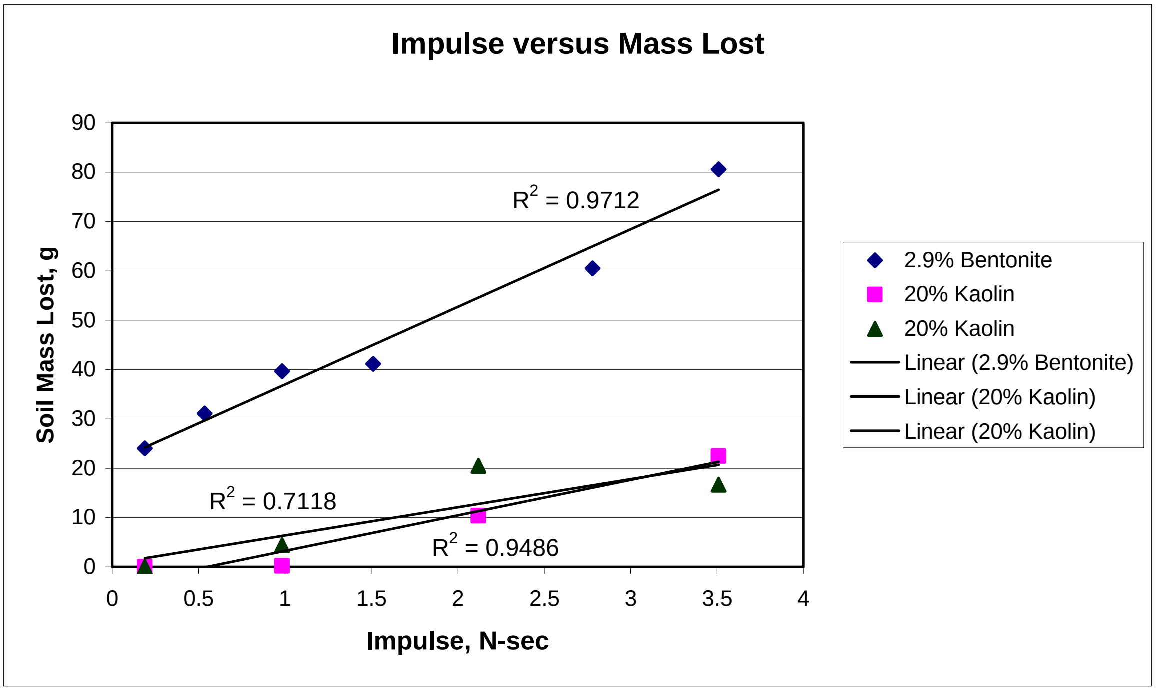


Figure 5.10: Jet Erosion Variation with Impulse

Table 5.4: Jet Erosion Variation with Impulse

2.9% Bentonite		20% Kaolin 1		20% Kaolin 2	
I (N-sec)	Δm, g	I (N-sec)	Δm, g	I (N-sec)	Δm, g
3.51	80.6	3.51	22.5	3.51	16.7
2.78	60.5	2.12	10.4	2.12	20.5
1.51	41.2	0.983	0.2	0.983	4.4
0.983	39.7	0.189	0	0.189	0
0.535	31.1	N/A	N/A	N/A	N/A
0.189	24	N/A	N/A	N/A	N/A

CHAPTER SIX

CONCLUDING REMARKS

Overall, the EFA can be very effective when determining scour rates in different types of soil, but only to a certain extent. The erosion of soil particles after their interaction with water flow can be difficult to predict, as evidenced by some of the model soils and Talladega samples. Nevertheless, the EFA can provide a good idea of when soils from a certain location will begin to scour, even though flow in the EFA quite differs from flow in an actual river or stream with different factors such as friction and debris. Particles in flow will give water a higher density, which can increase erosion at lower velocities.

For Alabama soils, a concern with EFA testing was surface preparation. The structure of the Sumter County and Talladega County samples was very problematic when attempting to prepare a smooth flat surface to be flush with the EFA flume. Surface preparation can easily disturb the soil and result in EFA erodibility that is not truly representative of field conditions. For samples where surface preparation was not an issue, results varied. Talladega samples 78305 and 78311 hardly eroded at V= 1.90 m/s and 1.60 m/s, respectively. In stark contrast, samples 78296 and 78301 produced scour rates of 100 mm/hr and 144 mm/hr, respectively, at V = 2.00 m/s and 1.44 m/s. The only test from Sumter County that was performed with minimal soil disturbance was Sucarnoochee07, which was highly erosion-resistant up to V = 1 m/s. Three samples from Dallas County (78935, 78936, and 78956) were tested at the highest EFA velocity ($V = 6$ m/s) for 2 hours, and little erosion occurred. Sample 78935 suddenly lost about one-third of its surface after one hour of testing, sample 78936 did not erode at all, and sample 78956 had 1 mm of degradation during the first hour and zero erosion the second hour.

Figures 5.2 and 5.3 display the erosion function for tests performed on model soils. Overall, higher bentonite contents and densities lead to higher values of τ_c and lower values of $\dot{z}$ at the same shear stress. For soils containing 5% and 25% bentonite compacted at 1.9 g/cm^3, τ_c ranged from 0.62 N/m^2 to 15.55 N/m^2. For these same two contents at 1.7 g/cm^3, τ_c ranged from 0.46 N/m^2 to 11.45 N/m^2. However, there are a few inconsistencies. For example, a model soil of 8.5% bentonite at 1.70 g/cm^3 exhibits a scour rate of 36 mm/hr at $\tau = 18.15$ N/m^2, while a model soil of 15% bentonite at 1.70 g/cm^3 erodes at 66 mm/hr at $\tau = 17.76$ N/m^2. Naturally, more inconsistencies will occur at 1.70 g/cm^3 than 1.90 g/cm^3, because at smaller densities more voids will appear in random spots throughout the sample. Figure 5.7 and 5.8 show that an increase in clay content for a given soil produced a linear increase in τ_c. Density can dramatically reduce scour rates in cohesive soils, as evidenced by Figures 5.4 and 5.5.

Impinging jet tests on model soils produced similar results. Overall, higher densities and clay contents yielded smaller erosion rates, but there were also some inconsistencies. For example, soil containing 5.2% and 7.2% bentonite lost almost the same mass of soil at the same density (1.6 g/cm^3). Again, this is due to less uniformity in the sample at a smaller density; more voids will appear in random places, with more possibly being located right under the trajectory of the jet for the 5% bentonite sample.

Soil at a bulk density of 1.84 g/cm^3 at 7.2% bentonite was impenetrable, while soil at 1.30 g/cm^3 containing 3.2% bentonite lost 41.8% of its original mass. For different impulses, erosion behaves in a linear fashion after τ_c is reached. The reproducibility of these experiments can differ greatly.

Overall, this report serves as a continuation for previous research performed on the EFA and its usefulness, as well as developing new methods for producing and measuring erosion.

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APPENDIX A

EFA FLOWMETER CALIBRATION

In the EFA, a paddlewheel flow meter (see Figure A.1) is used to measure the velocity, V. This velocity is intended to be the average velocity occurring in the rectangular test section of the apparatus. According to a published article by Blue White Industries, Ltd, paddlewheel flow meters are very popular; they have several distinct advantages over other types of flow meters and can achieve a high level of accuracy at a low level of cost. Per this article, one of the only accuracy challenges involving paddlewheel flow meters involves a flow of less than 1 ft/s. Any flow that carries dirt, pebbles, or rocks (which can result from an EFA test) can also damage the paddlewheel and inhibit accuracy. For this reason, EFA calibration tests were performed to investigate the accuracy of the EFA owned and operated by the Civil Engineering Department of Auburn University. A velocity of 1 m/s or less was considered in these calibration tests.

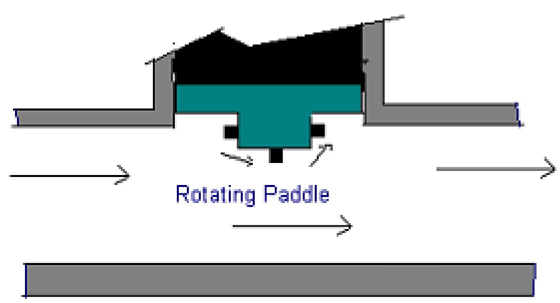

Figure A.1: Typical EFA Flowmeter (flowmeterdirectory.com)

In this report, the flow meter velocity, V, is compared to a measured velocity, V_m. The flow meter velocity is the velocity indicated on the monitor based on the signal from the flow meter and the EFA software. The measured velocity was calculated as $V_m = Q/A$, where A = area of the test section and Q = volumetric flow rate, which was determined based on direct collection of water volumes and measured time intervals. This collection was accomplished by holding a medium-sized pale under the pipe outlet to the reservoir (see Figure A.2) for a specific amount of time, which was recorded by a stopwatch. After measuring the collected weights of water, the specific weight of water was assumed to be 62.4 m^3/s and used to calculate the corresponding water volumes. Hence, Q was the collected water volume divided by the stopwatch reading. This procedure was repeated three times for a specific flow meter velocity. The EFA software requires selection of a logging rate; for a five second logging rate, the flow meter signal is averaged over a five second interval. For this flow meter velocity, a logging rate of five seconds was used.

Figure A.2: EFA Calibration Water Collection

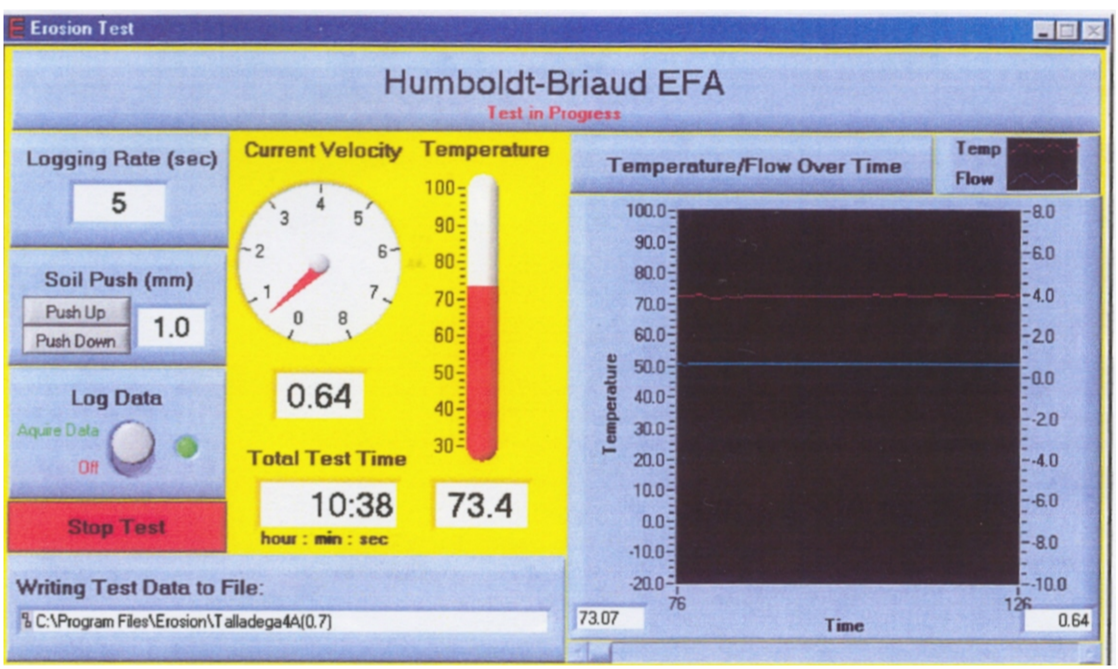

Figure A.3: Computer Screen Example during an EFA Test

The range of velocities considered in the calibration tests was $0 \leq V \leq 1.0$ m/s. For $V > 0.4$ m/s, there was less than a 10 percent difference between V and V_m and the difference decreased as velocities increased. For $V < 0.4$ m/s, however, there was a significant difference between V and V_m, especially around 0.3 m/s. This primarily depended on which direction the EFA valve was opened; if the valve was initially opened and gradually turned to $V = 0.3$ m/s, the difference was as much as 70 percent. When the valve was turned to $V = 1.0$ m/s and gradually lowered to 0.3 m/s, the difference was around 5 percent. Based on these results, direct measurement of V is recommended for EFA applications when $V < 0.4$ m/s. For this report, however, V_m was never obtained when $V < 0.4$ m/s. This was deemed unnecessary because only two measurements were made for $V < 0.4$ m/s with model soils, and zero measurements were made for $V < 0.4$ m/s with Alabama soils. The reported results and recommendations are specific for the EFA in the hydraulics laboratory of the Civil Engineering Dept. at Auburn University.

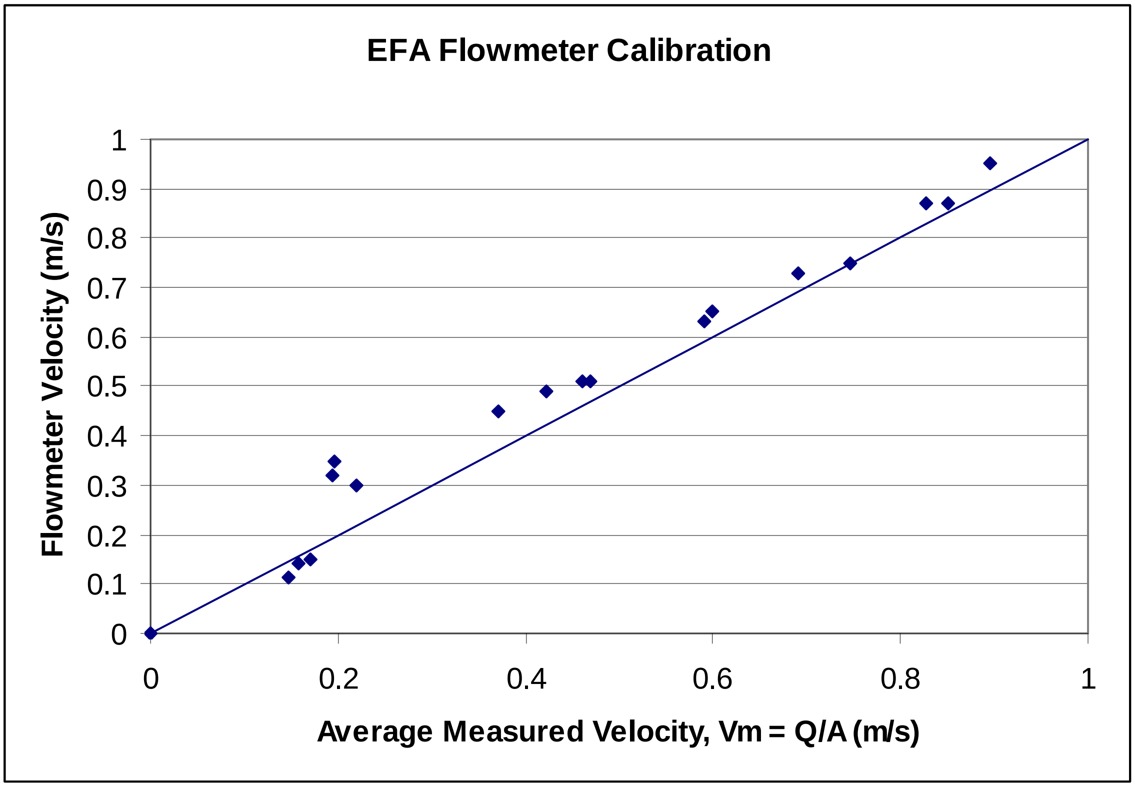

Figure A.4: EFA Flowmeter Calibration

Table A.1: EFA Flowmeter Calibration: May 29, 2007

Flow meter ,V	V_m Test 1	V_m Test 2	V_m Test 3	Average V_m
0.115 m/s	0.15 m/s	0.148 m/s	0.145 m/s	0.148 m/s
0.32 m/s	0.191 m/s	0.197 m/s	0.195 m/s	0.194 m/s
0.49 m/s	0.43 m/s	0.42 m/s	0.42 m/s	0.423 m/s
0.63 m/s	0.59 m/s	0.59 m/s	0.59 m/s	0.59 m/s
0.75 m/s	0.75 m/s	0.74 m/s	0.74 m/s	0.747 m/s
0.87 m/s	0.87 m/s	0.85 m/s	0.83 m/s	0.85 m/s

Table A.2: EFA Flowmeter Calibration: June 2, 2007

Flow meter ,V	V_m Test 1	V_m Test 2	V_m Test 3	Average V_m
0.15 m/s	0.17 m/s	0.17 m/s	0.17 m/s	0.17 m/s
0.30 m/s	0.22 m/s	0.22 m/s	0.21 m/s	0.22 m/s
0.45 m/s	0.40 m/s	0.33 m/s	0.38 m/s	0.37 m/s
0.51 m/s	0.45 m/s	0.47 m/s	0.49 m/s	0.47 m/s
0.73 m/s	0.65 m/s	0.76 m/s	0.65 m/s	0.69 m/s
0.95 m/s	0.97 m/s	0.80 m/s	0.92 m/s	0.90 m/s

Table A.3: EFA Flowmeter Calibration: June 5, 2007

Flow meter ,V	V_m Test 1	V_m Test 2	V_m Test 3	Average V_m
0.14 m/s	0.15 m/s	0.16 m/s	0.15 m/s	0.153 m/s
0.35 m/s	0.19 m/s	0.19 m/s	0.20 m/s	0.197 m/s
0.51 m/s	0.46 m/s	0.46 m/s	0.46 m/s	0.46 m/s
0.65 m/s	0.60 m/s	0.60 m/s	0.60 m/s	0.60 m/s
0.87 m/s	0.84 m/s	0.83 m/s	0.81 m/s	0.83 m/s

Table A.4: EFA Flowmeter Calibration: June 12, 2007

Valve initially opened and turned to V= 0.3 m/s

Flow meter ,V	V_m Test 1	V_m Test 2	V_m Test 3	Average V_m
0.30 m/s	0.17 m/s	0.17 m/s	0.17 m/s	0.17 m/s
0.20 m/s	0.16 m/s	0.16 m/s	0.156 m/s	0.159 m/s

Valve opened and initially turned to V = 1 m/s and then lowered to 0.3 m/s

Flow meter ,V	V_m Test 1	V_m Test 2	V_m Test 3	Average V_m
0.27 m/s	0.27 m/s	0.27 m/s	0.27 m/s	0.27 m/s
0.351 m/s	0.33 m/s	0.33 m/s	0.32 m/s	0.327 m/s

APPENDIX B

THE MODIFIED METHOD OF MEASURING EFA EROSION RATES

Talladega sample 78316 was tested for 10 minutes in the EFA at V = 0.80 m/s. Before testing began, the surface of the soil sample was trimmed flush with the EFA flume bed. The volume of any initial voids on the surface was measured by performing the following task.

First, the mass of five dry paper towels was recorded:

$$M_{dry\ 1} = 0.863\ g$$
$$M_{dry\ 2} = 0.984\ g$$
$$M_{dry\ 3} = 1.028\ g$$
$$M_{dry\ 4} = 0.858\ g$$
$$M_{dry\ 5} = 1.004\ g$$

Next, water filled any voids on the soil with a syringe. This water was soaked up by the dry paper towels, and each wet mass was recorded.

$$M_{wet\ 1} = 2.035\ g$$
$$M_{wet\ 2} = 2.397\ g$$
$$M_{wet\ 3} = 2.226\ g$$
$$M_{wet\ 4} = 2.011\ g$$
$$M_{wet\ 5} = 2.262\ g$$

The two measurements are subtracted, and the difference between the two is the water mass used to fill any voids.

$$M_{wet\ 1} - M_{dry\ 1} = 2.035\ g - 0.863\ g = 1.172\ g$$
$$M_{wet\ 2} - M_{dry\ 2} = 2.397\ g - 0.984\ g = 1.413\ g$$
$$M_{wet\ 3} - M_{dry\ 3} = 2.226\ g - 1.028\ g = 1.198\ g$$
$$M_{wet\ 4} - M_{dry\ 4} = 2.011\ g - 0.858\ g = 1.153\ g$$
$$M_{wet\ 5} - M_{dry\ 5} = 2.262\ g - 1.004\ g = 1.258\ g$$

The average difference between the two is 1.239 g. Next, this value is divided by the specific weight of water, 1 g/cm^3, which equals 1.239 cm^3. Hence, this is the volume of voids initially on the soil surface.

Next, an EFA test is performed for 10 minutes at V = 0.80 m/s. Any new voids that developed from eroded soil are measured using the exact same procedure. The new void volume after the test is measured to be 5.615 cm^3. This value is converted into an erosion rate by subtracted the original void volume from it, dividing by the Shelby tube cross-sectional area (40.92 cm^2) and multiplying by a factor of 60 to convert the overall rate in terms of mm/hr:

$$\frac{5.615\ \text{cm}^3 - 1.239\ \text{cm}^3}{40.92\ \text{cm}^3}\left(\frac{10\ \text{mm}}{1\ \text{cm}}\right)\left(\frac{60\ \text{min}}{10\ \text{min}}\right) = 6.4\ \text{mm/hr}$$

This procedure is repeated with different time intervals and velocities.

APPENDIX C

EFA MODEL SOIL TEST RESULTS

Table C.1: Model Soil EFA Test Results, ρ = 1.7 g/cm^3

<u>5% Bentonite</u>

Average Velocity (m/s)	Shear Stress (N/m^2)	Eroded Depth, mm	Test Duration, minutes	Erosion Rate (mm/hr)
0.385	0.185	0	60	0
0.608	0.462	0	60	0
0.917	1.051	1.5	60	1.5
1.152	1.659	62	57	65

<u>8.5% Bentonite</u>

Average Velocity (m/s)	Shear Stress (N/m^2)	Eroded Depth, mm	Test Duration, minutes	Erosion Rate (mm/hr)
0.943	1.112	0	60	0
1.970	4.851	0.5	60	0.5
2.870	10.296	6	30	12
3.810	18.145	9	15	36

Table C.1 cont'd: Model Soil EFA Test Results, $\rho = 1.7$ g/cm^3

<u>15% Bentonite</u>

Average Velocity (m/s)	Shear Stress (N/m^2)	Eroded Depth, mm	Test Duration, minutes	Erosion Rate (mm/hr)
1.196	1.788	0	60	0
2.203	6.067	0	60	0
3.000	11.25	9	60	9
3.769	17.757	11	10	66
4.516	25.492	9.5	7	81

<u>20% Bentonite</u>

Average Velocity (m/s)	Shear Stress (N/m^2)	Eroded Depth, mm	Test Duration, minutes	Erosion Rate (mm/hr)
1.569	3.042	0	60	0
2.538	8.052	0	60	0
3.503	15.339	20	60	20
4.460	24.865	15	30	30
5.507	37.909	9	8	67.5

Table C.1 cont'd: Model Soil EFA Test Results, ρ = 1.7 g/cm^3

25% Bentonite

Average Velocity (m/s)	Shear Stress (N/m^2)	Eroded Depth, mm	Test Duration, minutes	Erosion Rate (mm/hr)
3.026	11.446	0	60	0
4.003	20.030	24	60	24
5.017	31.463	21	30	42
6.113	46.711	7	5	84

Table C.2: Model Soil EFA Test Results, ρ = 1.9 g/cm^3

3.5% Bentonite

Average Velocity (m/s)	Shear Stress (N/m^2)	Eroded Depth, mm	Test Duration, minutes	Erosion Rate (mm/hr)
0.341	0.145	1	120	0.5
0.512	0.328	40	40	60
0.648	0.340	14	15	56

5% Bentonite

Average Velocity (m/s)	Shear Stress (N/m^2)	Eroded Depth, mm	Test Duration, minutes	Erosion Rate (mm/hr)
0.515	0.332	0	60	0
0.704	0.620	0	60	0
1.030	1.326	7.5	30	15
1.320	2.178	24	40	36
1.735	3.763	18	30	36

Table C.2 cont'd: Model Soil EFA Test Results, $\rho = 1.9\ g/cm^3$

8.5% Bentonite

Average Velocity (m/s)	Shear Stress (N/m^2)	Eroded Depth, mm	Test Duration, minutes	Erosion Rate (mm/hr)
1.255	1.969	0	60	0
1.864	4.343	0	60	0
2.562	8.209	9	60	9
3.000	11.250	5	10	30
3.590	16.110	10	10	60

12.5% Bentonite

Average Velocity (m/s)	Shear Stress (N/m^2)	Eroded Depth, mm	Test Duration, minutes	Erosion Rate (mm/hr)
1.520	2.888	0	60	0
2.246	6.306	0	60	0
3.050	11.628	3	60	3
3.640	16.562	22	60	22
4.291	23.016	20	30	40
4.713	27.765	30	40	45

Table C.2 cont'd: Model Soil EFA Test Results, $\rho = 1.9$ g/cm^3

15% Bentonite

Average Velocity (m/s)	Shear Stress (N/m^2)	Eroded Depth, mm	Test Duration, minutes	Erosion Rate (mm/hr)
2.148	5.767	0	60	0
3.000	11.250	0	60	0
4.036	20.362	16	30	32
5.060	32.005	14	20	42
6.060	45.909	16	15	64

20% Bentonite

Average Velocity (m/s)	Shear Stress (N/m^2)	Eroded Depth, mm	Test Duration, minutes	Erosion Rate (mm/hr)
3.068	11.766	0	60	0
3.743	17.513	8	60	8
4.620	26.681	22	60	22
5.388	36.288	36	20	36
6.087	46.208	42	20	42

Table C.2 cont'd: Model Soil EFA Test Results, ρ = 1.9 g/cm^3

<u>25% Bentonite</u>

Average Velocity (m/s)	Shear Stress (N/m^2)	Eroded Depth, mm	Test Duration, minutes	Erosion Rate (mm/hr)
3.527	15.550	0	60	0
4.233	22.398	18	60	18
5.056	31.954	28	60	28
6.006	45.090	34	60	34