LABORATORY MODELING OF EROSION POTENTIAL OF EARTHEN EMBANKMENTS
IN CONTACT WITH OPEN BEDROCK JOINTS

by

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A thesis submitted in partial fulfillment of the requirements for the degree of
MASTER OF SCIENCE in
Civil and Environmental Engineering

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ABSTRACT

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Earth dams constructed on pre-existing jointed bedrock may be susceptible to the erosion of foundation or embankment soils that come in contact with the jointed bedrock. The mechanism analyzed in this study is contact erosion of the soils due to high velocity flows in the bedrock joint. The conditions needed to be satisfied in order for contact erosion to occur are the size of the soil particles must be smaller in diameter than the bedrock joint aperture and the soil must be able to pass through an open exit. Examples of monitored and recorded events due to this phenomenon exist for multiple dams including the Quail Creek Dike failure and deterioration of the core at Mud Mountain Dam. The objective of this research is to use a laboratory model to quantify the risk that existing dams have due to contact erosion at the interface of erodible soils and naturally occurring bedrock joints.

Tests were performed using an apparatus which was designed and constructed at Utah State University. The data obtained was used to develop a relationship for predicting the initiation of erosion and the time of progression. Key variables such as soil type, soil properties,
crack aperture and flow velocity will help establish these desired correlations. Crucial values such as Critical Shear Stress and Critical Velocity are used to better understand results of testing performed and to verify results with similar research performed. Results are also compared to a Hole Erosion Test performed at Utah State University.

Conclusions of the study show how to apply the HET to data from this experiment by comparing and contrasting results from tests performed on the same material. The relationships developed are expected to be used in risk assessment toolboxes nationwide for the specific mechanism of contact erosion for existing and future dams. Further testing with the apparatus built is suggested to provide risk assessment teams with a larger database for assigning probabilities to the contact erosion mechanism in existing dams.

(230 pages)
Laboratory Modeling of Erosion Potential of Earthen Embankments in Contact with Open Bedrock Joints

Joseph T. Zaleski

Earthen dams are often built into bedrock abutments and on bedrock foundations. Bedrock joints naturally occur in bedrock materials. These bedrock joints create voids for ground water to pass through. Historically earthen dams were sometimes built in direct contact with the bedrock joints, causing a contact point between the soil of the dam and the flowing water. It has been engineering practice to place grout into exposed bedrock joints for some time now. However, soil is not always cleaned out of bedrock joints before they are grouted, which leaves a weakness for water to push through.

The purpose of this study is to understand the point at which water flowing through bedrock joints will erode soil from the earthen dam embankment. The information of how much soil is eroded away in an amount of time is also crucial to the scope of this study.

The goals of this study were accomplished by building a physical model or apparatus of an earthen dam embankment on top of a simulated bedrock joint. Different soil types were tested in the apparatus to start a database of information about erosion rates of the soil along the bedrock joint and embankment interface. These results will be used to start a database for organizations that assign probabilities of dam failures. The purpose of the study is not to indicate when dams will fail, but to help with assigning probabilities of the likelihood of a serious problem being caused from this type of mechanism presented in this study.
ACKNOWLEDGMENTS

First, I would like to thank Dr. Rice and the Corps of Engineers for the opportunity to participate in this research project. I have appreciated Dr. Rice’s patience and help throughout the entire process. It has been a very good learning experience for me.

I would also like to thank my committee members, Dr. Rice, Dr. Bay and Dr. Barr. I have appreciated the help on specific aspects of my project and the time that has been spent assisting me and helping me to understand.

I would also like to thank Ken Jewkes and John Harlow for their help in the construction of the apparatus. I would especially like to thank Ken for all the materials, tools and time. I would like to thank co-workers Jake Erickson, Nate Lowe, Craig Adams, Jorge Peralta, Tyler Coy and Nate Braithwaite. They all contributed to my project and helped me along the way.

Last, I would especially like to thank my wife, Marinda, for her love and encouragement. I also thank my family for their love, encouragement and support.

Joseph T. Zaleski
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CHAPTER 1

INTRODUCTION

1.1 Open Bedrock Joints

Bedrock joints are created by weathering and deformation processes over time. This research applies specifically to earthen dams that have been built on top of a bedrock foundation and/or have abutments into bedrock. If the joints were not treated, filled with grout or otherwise capped before the construction of the dam, then these joints remain open or filled with soil. If joints were treated before the construction of the embankment, then there could still be locations where the grout did not take due to the presence of soil. Therefore, joints are either open, or, if the trapped soil in the joints is removed, can become open. Fig. 1, depicts a natural bedrock joint from the location of the failed Quail Creek Dike.

![Fig. 1. Quail Creek Dike Natural Occurring Bedrock Joint (Von Thun et al. 1989)](image-url)
1.2 Increased Flow through Bedrock Joints

Several factors will affect the flow regime through bedrock joints including, but not limited to: increased water pressure from first filling of the reservoir, seasonal rises in the water level, inclusion of a seepage barrier in a dam, and larger flow channels forming from solutioning of the bedrock material. Increased gradients through bedrock joints beneath dams can lead to two types of erosion mechanisms through the joints. One mechanism starts with joint dilation, which is caused from increased hydraulic pressure from the reservoir and more concentrated flows around keyways and seepage barriers. Once joint dilation occurs, cracks form between bedrock and soil in the joint. The velocity of the water in the crack can cause the soil to be at risk of being eroded. The other mechanism would be a gradient increase through the soil, due to the increase of water pressure in the joint. A high gradient can lead to contact erosion of the soil. These two mechanisms are depicted in steps in Fig. 2. (Rice and Duncan 2010)

![Mechanisms of Bedrock Joint Erosion](image)

Fig. 2. Mechanisms of Bedrock Joint Erosion (Rice and Duncan 2010)
1.3 Contact Erosion

Contact erosion is defined as a fine grained soil that is susceptible to erosion through an underlying coarse grained soil layer. Two conditions must be in place for contact erosion to occur. First, the coarse soil layer must have large enough voids for the fine particles to pass through. The second condition is that the flow velocity must be strong enough to detach and carry in suspension the fine particles from the upper layer. The more cohesive the fine soil particles are the greater the flow velocity needs to be in order to detach them. The finer particles could be removed along the interface of the fine and coarse soil layers between the pores of the coarse grained layer (ICOLD 2012).

Contact erosion can occur as water flows through open bedrock joints in contact with fine embankment soil particles. High flow velocities can begin to transport fine particles away through the open joints. Under the proper conditions, an open exit and non-collapsing soils (i.e. sufficiently cohesive), a “pipe” can begin to form along the embankment and bed rock joint interface. Fig. 3 shows this scenario schematically. The initiation of erosion along this interface, through contact erosion, will allow the flow velocity to increase through the enlarged flow path. The pipe will continue to increase in size until the area of the flow path is sufficiently large to lower flow velocity and an equilibrium point is reached. This is the mechanism specifically researched in this study.

The “pipe” formed at the interface of the bedrock foundation and embankment soil layer by contact erosion is the mechanism analyzed in this study. All references to erosion through this study are referring to contact erosion.
Fig. 3. Schematic Drawing of Contact Erosion Susceptibility of Bedrock Joints

1.4 Purpose of Research

The purpose of this research is to test contact erosion rates of different soil types along a simulated bedrock joint. The funding for this research has been provided because there has been an increased need for research to be done on contact erosion rates. One approach to evaluating the likelihood of a dam failing is through a risk assessment process. In this approach each mechanism that affects the performance of the dam is looked at separately. For some of these mechanisms, little research has been performed to provide reasonable values for assumed probabilities used in the risk assessment process. Contact erosion along the interface between bedrock foundations and soil embankments is one of the mechanisms that has had little research performed on it. There are few known quantities of erosion for this type of mechanism.
To perform this research an apparatus has been built that can be used for testing contact erosion initiation and progression rates in soils due to flow through a bedrock joint. The apparatus has been used to test several different soil types. The end goal is to create a database of contact erosion rates of an array of soil types and bedrock joint conditions. This would provide risk assessment teams with known values to compare against when assigning a probability of the likelihood of contact erosion occurring in existing dams. This research looks specifically at a smooth 3 mm-wide joint for the majority of the tests performed. Tested soils include some manufactured sands mixed with low or high plasticity clay (kaolinite or bentonite) and two natural soils from the failed Teton Dam in Idaho and the East Branch Dam in Pennsylvania.

1.5 Report Organization

This thesis includes six chapters. Chapter 1 is the introduction to the purpose of the research. Chapter 2 is a composed summary of relevant literature materials that were reviewed for improved comprehension of the thesis topic. The materials reviewed include: Seepage Barriers in Dams, Surface Erosion, Contact Erosion, Internal Erosion, Previous Testing Done on the Subject Matter, Hole Erosion Testing and Relevant Case Histories. Chapter 3 is an introduction to the testing apparatus including: the structure of the apparatus, the materials that are used to create the interior of the apparatus, and the instrumentation used in the apparatus. Chapter 4 presents the testing procedure developed for the research and specifics of the test setup and monitoring, and how to analyze a test. Chapter 5 discusses the results from the tests performed. Each test includes pictures of the resultant erosion and the data analysis. Comparisons to other erosion rate methods are also included in this chapter. Chapter 6 contains the conclusions of this research.
CHAPTER 2
LITERATURE REVIEW

2.1 Introduction

Minimal research has been performed on bedrock joint effects on the overall performance of earthen dams. A literary review of relevant research topics is provided for the following subjects:

- Seepage Barriers in Dams
- Surface Erosion
- Contact Erosion
- Internal Erosion
- Previous Testing Done on Subject Matter
- Hole Erosion Testing
- Relevant Case Histories

2.2 Seepage Barriers in Dams

Rice and Duncan 2010

The main discussion of this paper deals with the effect that seepage barriers have on the overall seepage of dams. The specific portion of this paper that relates to the presented research is an increased gradient through foundation structures as depicted in Fig. 4. Due to the seepage barrier, there is a possibility of a concentrated increase in hydraulic gradient in the foundation.

Increases in gradient, as discussed in this paper, could cause an increase in erosion through bedrock defects and could cause embankment soil to erode through bedrock joints or solution channels. As depicted in Fig. 5, when a seepage barrier is implemented, the flow through defects (the example shown in the figure is solutioned limestone channels) is limited to
the joints that are below the elevation of the seepage barrier. As demonstrated in the figure sequence, the flow becomes concentrated in the joints below the seepage barrier. This increases the gradient through these joints as more flow is forced through the remaining open joints. This process creates an increased velocity through the open joints which can increase the risk of erosion.

![Figure 4](image.png)

**Fig. 4. Increase Gradients through Foundation Structures due to Seepage Barriers (Rice and Duncan 2010)**

According to Rice and Duncan (2010), the increased gradients are able to create two types of erosion mechanisms. The first mechanism is shown in Fig. 2(a), 2(b) and 2(c). This mechanism starts with joint dilation caused by increased hydraulic presser in the joint. Joint dilation allows a crack (or hydraulic fracture) between the soil and rock to form. The flow is then able to jet through the newly formed crack, eroding as it goes because of high velocities. As soil begins to be eroded away, the velocity of the flow increases as soil continues to be removed until an equilibrium point between the flow velocity and open joint area is reached. The other mechanism illustrated in Figure 2(d), shows that an increased gradient will cause erosion to propagate from the downstream end of the joint as the high gradient initiates piping erosion at the exit point. Once the soil in the joint starts to erode at the exit point a piping sequence forms as soil is eroded farther and farther back into the soil filled joint.
Several dams in the study by Rice and Duncan (2010) have seepage barriers that extend into bedrock foundations to mitigate seepage problems through the bedrock. Other dams from the study are known to have erodible soils above bedrock foundations and thus, the seepage barrier only extends down through the soil layers. Rice and Duncan (2010) discuss how there is little head loss through open joints, which allows for sustained erodible velocities. It is suggested that these high velocities could create high exit gradients that would create a contact erosion concern. These high velocities could erode embankment soils, if they come in contact with the soils, through open bedrock joints; a similar mechanism to the one that caused the
Rice and Duncan (2010) conclude that the mechanism of seepage through bedrock joints needs to be assessed when implementing a seepage barrier in a dam constructed on a bedrock foundation. It is also suggested that an increased velocity through the foundation, due to the installation of a seepage barrier, could increase the risk of internal erosion in a dam.

2.3 Surface Erosion

Briaud 2008

This report discusses the parameters used to find the erosive rate of a soil. Three different parameters are discussed in the report: the erodibility of the soil or rock, the flow velocities that are passing through the system and the obstacles in the geometry. The two topics that most readily apply to the study of this thesis are the erodibility of the soil or rock being used and the velocities of the flow passing through the system.

The point of erodibility of a soil particle or rock is dependent on a number of different factors. The cohesion of the soil can help the soil to not erode as quickly as a soil particle of the same size with no cohesion. The size of non-cohesive particles determines the point at which they are eroded. Soil particles and rocks will be removed when the shear stresses on the material exceed the critical shear stress or the point of initiation of erosion. Fig. 6, depicts the stresses acting on a soil particle in a river. As shown, the flow rate must reach a certain point in order to erode the particle. This point of erosion marks the critical shear stress and the critical velocity.

As mentioned, the velocity of the flow is an important factor for causing erosion. Briaud (2008) discusses how the velocity at the bottom of a river is less than the velocity at the top of
the river. This is due to the hydraulic shear stress that is resisting the flow along the contact between the materials on the bottom of the river and the water. Therefore, the hydraulic shear stress on a surface is proportional to the change in the velocity of the flow with distance away from the surface. This relationship is demonstrated in Fig. 7.

Two graphs, created by Briaud (2008), relating the results of the testing performed in this study are to be used to help interpret the results of this thesis. These graphs are based on data obtained by Briaud as well as other researchers and compare both critical velocity (Fig. 8) and critical shear stress (Fig. 9) to the median grain size of materials being tested. This comparison is used to help validate the results from the study of this thesis. These graphs are shown in the Comparison to Other Research section of the Analysis and Results chapter.
2.4 Contact Erosion

ICOLD 2012

This bulletin focuses on the mechanisms that comprise the broader category of internal erosion in dams. It discusses terms for the different types of internal erosion: piping, concentrated leak erosion, contact erosion, and suffusion. The mechanism that most closely fits the topic of this research is contact erosion.

The ICOLD bulletin (2012) defines contact erosion as the erosion of a layer of fine soil (sands, clays and silts) into the voids in an underlying coarser layer (primarily gravels) or a void such as a bedrock joint. As discussed in this publication, two conditions must exist in order for contact erosion to occur. First, the coarse layer must have an exposed, open matrix to the fine soil layer. This will allow for the fine soil particles to be eroded away through the coarse soil layer. Second, a sustained velocity must be maintained, which can be erosive to fine soil particles.

The ICOLD committee (2012) discusses how contact erosion can lead to failures in dams.
Fig. 8. Braïud’s Critical Velocity Versus Mean Grain Size

Fig. 10, shows the possible failure modes that could be introduced from the contact erosion mechanism. As depicted, the primary flow of water is down through the foundation of the dam in all of the failure modes. The flow then reaches a velocity that starts to erode the fine soil in the embankment in the most vulnerable location. Erosion propagation then naturally develops into the seepage problems depicted in a), b), c) and d) of Fig. 10.

Contact erosion can create softening of the overburden material to the point of creating a sinkhole as shown in a) of the figure. Piping along the interface as depicted in b) of the figure is a continuation of contact erosion occurring along the embankment and bedrock interface. The piping could progress all the way through the base of the embankment. Sliding could also occur as the point of contact erosion weakens the failure plane of the downstream slope of the
Fig. 9. Braud’s Critical Shear Stress Versus Mean Grain Size

earthen dam embankment as shown in c) of the figure. Natural filtering could cause the bedrock joints to become clogged, possibly causing a new flow path to form up through the downstream toe of the embankment as depicted in d) of the aforementioned figure. Contact erosion may not lead to failures in dams but could act as a catalyst to failing the dam by another mechanism.

The ICOLD committee (2012) summarizes the findings of different research performed to find the velocities at which point different types of soil particles begin to erode. The following equation solves for the Darcy velocity ($U_{crit}$, m/s) for fine cohesionless sand below a coarse soil layer:

```
\tau_c = 0.006 (D_{50})^{-2}
```

```
\tau_c = 0.05 (D_{50})^{-0.4}
```

Legend:
Fig. 10. Possible Failure Modes due to Contact Erosion (ICOLD 2012)

\[
U_{\text{crit}} = 0.65 n_D \sqrt{\frac{\rho_s - \rho_w}{\rho_w}} \cdot g d_{50}
\]  

(2.1)

Where, \( n_D \) is the porosity of the coarse layer, \( \rho_s \) (kg/m\(^3\)) is the density of the sand particles, \( \rho_w \) (kg/m\(^3\)) is the water density, \( g \) (m/s\(^2\)) is the gravity acceleration and \( d_{50} \) (m) is the median diameter of the sand grading curve.

ICOLD (2012) also presents work done by Guidoux et al. (2010) in experimentation with contact erosion for clays and silts. The following equation is an equation used by Guidoux et al. (2010) to find the effective diameter \( (d_H, \text{ m}) \) of clays and silts:

\[
d_H = \left( \frac{\sum_{j=1}^{m} F_j}{d_j} \right)^{-1}
\]

(2.2)

Where, \( F_j \) is the percentage of the diameter and \( d_j \) is the diameter on the gradation curve of the soil. The effective diameter is then used to solve for the Darcy velocity \( (U_{\text{crit}}, \text{ m/s}) \) in the following equation for clays and silts:
where, \( n_D \) is the porosity of the coarse layer, \( \rho_s \) (kg/m\(^3\)) is the density of the sand particles, \( \rho_w \) (kg/m\(^3\)) is the water density, \( g \) (m/s\(^2\)) is the gravity acceleration, \( d_H \) (m) is the effective diameter from Equation (2.3) and \( \beta \) is an empirical factor found through experimentation by Guidoux et al. (2010), which value is \( 5.3 \times 10^{-9} \) m\(^2\).

The results of Guidoux et al. (2010) research and others results on velocities needed to initiate contact erosion are summarized in one comparative figure, which is included in Fig. 11. As discovered in the research performed, sands begin to erode at lower velocities than cohesive soils. A key factor to be observed in earthen dams is which types of embankment soils are in contact with open bedrock joints. This factor will greatly contribute to the amount of erosion that can occur.

\[
U_{crit} = 0.65 n_D \sqrt{\frac{\rho_s - \rho_w}{\rho_w}} g d_H \left(1 + \frac{\beta}{d_H^2}\right)
\]

(2.3)

**Fig. 11.** Research on Velocities Needed to Erode Soil Particles (ICOLD 2012)
Guidoux et al. 2010

This paper describes a laboratory study to investigate contact erosion of fine soils. The authors of this study devised an apparatus to test a layer of fines over a coarse soil layer. The apparatus is shown schematically in Fig. 12. The apparatus was set up to allow the flow to pass through a coarse soil layer that is above a base soil layer. The base soil layer is made up of poorly graded or gap graded soils with significant portions of fines. If the water flows through the coarse layer at a high enough velocity it is able to remove fine particles and transport them through the voids in the coarse layer and out of the open exit of the apparatus.

The results found by Guidoux et al. (2010) lead to the modification of the equations for contact erosion of sand particles to apply to contact erosion of fines. These are Equations 2.2 and 2.3 presented in the discussion of the ICOLD 2012 paper earlier in this chapter. As explained in the review of the ICOLD (2012) paper, the effective grain diameter, solved for using Equation 2.2, accounts for the fines in a poorly graded or gap graded soil (manufactured soils used in this study will be gap graded). The effective grain diameter is then used to calculate the critical Darcy velocity, solved for using Equation 2.3.
Guidoux et al. (2010) conclude that a successful new empirical expression for velocity has been created for contact erosion of silts and sand/clay mixtures. This new expression is effective for gap graded and poorly graded soils. The previous method developed for sands is applicable to well graded materials. Both of these relationships will be used through the duration of the study presented in this thesis.

2.5 Internal Erosion

Fell et al. 2003

This publication discusses the length of time it takes for dams to fail due to internal erosion. The time element is defined from evidence of the initiation of internal erosion problems to the actual breech of the dam. The two pertinent mechanisms discussed in this publication are: 1) internal erosion through the foundation and 2) internal erosion from the embankment down into the foundation.

According to Fell et al. (2003), embankment soil must be able to sustain a pipe, in the erosive process, to ultimately lead to failure. This holds true for piping through the embankment and also for piping of the embankment into the foundation voids. With the ability to sustain a pipe, seepage erosion can occur through an open jointed foundation because of the high, sustained flow that it allows. Internal erosion from the embankment soil eroding into the open joints can also occur. This form of internal erosion starts at the tail of the dam and propagates back in the upstream direction as a pipe is formed.

The process of the embankment soil eroding down into open bedrock joints should occur slowly, according to this research. This is assuming that the joint is narrow and small. As the scale of the joint increases, the time for this mechanism to develop could decrease. Once a pipe has formed through the entire width of the dam, the failure process becomes very rapid as a clear flow path through the embankment soil has been formed along the base of the dam.
Previous work done by Foster and Fell (1999, 2000) suggests that fine cohesive soils, solutioned features in the foundation, untreated bedrock joints, or a rigid structure in the dam are all most likely to support a pipe in a dam embankment. The piping process can be limited by natural processes, such as a natural filter forming, in which the flow could reach an equilibrium point before the breach of the dam or the flow could be limited enough to stop erosion.

The overall conclusion of Fell et al. (2003) is that backwards erosion occurs slowly through open bedrock joints, but is also hard to detect. Piping through the base of the embankment along the bedrock joints could take years to develop and then fail very quickly as the pipe nears forming through the entire width of the dam. Piping can be detected by changes in pore pressure, evidence of seepage or differential settlement of the embankment, but these parameters could be minimal. Therefore, it is crucial that the aforementioned values, changes in pore pressure, evidence of seepage and differential settlement of the embankment, be closely monitored to observe the initiation of the internal erosion mechanism. This mechanism might not cause the failure of the dam, but could facilitate other failure mechanisms which will ultimately fail the dam.

Foster et al. 2000

The primary purpose of this research was to estimate the probability of failure of dams by assigning probabilities to the various failure mechanisms that are specific to existing dams. The values assigned to each mechanism and treatment, are derived from case studies of dams that have either failed or had an accident. An accident refers to an incident that occurs on a dam that could have led to failure had mitigative measures not been taken. The dam failures and accidents studied are broken up into three categories; piping through the embankment, piping through the foundation and piping from the embankment into the foundation. The
category that will be summarized, which most closely fits the research topic at hand, is piping from the embankment into the foundation.

The mechanisms, treatments and observations used for weighted values for piping from the embankment into the foundation are: filters, slope of foundation cutoff trench, foundation type, erosion-control measure of core foundation, grouting of foundation, soil geology types, rock geology types, core geological origin, core soil type, core compaction, foundation treatment, observations of seepage, frequency of monitoring and surveillance of the dam. Specific probability weights are assigned according to the overall effectiveness of treatments implemented in the dam in accordance with the geology that the dam is set in. Each weight is multiplied with all other weights to create a comparison with the “average dam failure” within the category of piping from the embankment into the foundation. The average dam failure due to piping from the embankment into the foundation is equivalent to 1.0, so an overall weighted value above 1.0 will result in a dam that will fail faster than the average dam, while an overall value less than 1.0 will result in a dam that will fail slower than the average dam. This process is also used for the conditions of piping through the embankment and piping through the foundation, respectively. The probabilities of all three categories are added up to create the total probability of dam failure due to piping. This value can also be compared with the overall probability of the average dam failure.

The time to failure varied greatly from dam to dam as the soil from the embankment eroded down into the foundation. The Teton dam failure occurred very rapidly and only showed signs of internal seepage erosion problems 4 hours before failure. The Quail Creek Dike showed signs of piping throughout its existence until failure, which was just over a three and a half year period. Other dams that had accidents showed longer, drawn out seepage flows that were detected and stopped by the lowering of the reservoir. The authors state that the dams that
had accidents were built on bedrock joints that were small enough to limit the flow and thus
slow down the internal erosion process enough to prevent the complete failure of these dams.
All of the recorded failures, due to the embankment eroding into the foundation, happened
within 12 hours of initial observed problems and as fast as within 3 hours of initial problems
observed. This can be a very rapidly occurring mechanism.

2.6 Previous Testing Done on Subject Material

Goodman and Sundaram 1980

Laboratory testing was performed by Goodman and Sundaram (1980) to see how much
soil is washed through an open bedrock joint. They used an apparatus surrounding a fractured
rock specimen that has a cross section of 1 inch. The aperture of the fracture in the rock was
adjustable. The fractured rock specimen was then surrounded by the soil specimen and placed
in an acrylic tubing device. The various rock specimens used were smooth or fractured. Cyclic
loading techniques were used to control flow through the experiment in order to simulate
realistic seasonal conditions experienced by dams.

From experimentation, it was found that erosion occurred in the soil in contact with the
joint and closely surrounding the joint. The erosion happened at the beginning of each water
cycle performed in a test. The flow was muddy at the commencement of the cycle and then
became clear shortly thereafter, but would then become muddy again with the start of another
cycle. This indicates that the erosion was happening in stages and not all at once. When the
 aperture of the joint was smaller than the smallest grain size of material, no erosion occurred.

A conclusion drawn from this experiment is that seepage erosion through bedrock joints
does not occur all at once, but over time and due to cyclic increases in gradient. This paper also
concludes that a design standard needs to be made for embankment soils placed in contact with
bedrock joints as is done in other standards of a similar nature. As stated, the erosion of the soil took place when the pressure was increased with each cycle. As the water pressure increased, the diameter of the soil particles removed increased in size. The authors of this paper referred to their work as preliminary work on this subject matter.

Fetzer et al. 1981

This publication compiles one discussion by Claude A. Fetzer (1981) and one discussion by Reginald A. Barron (Fetzer et al. 1981) in reply to the report discussing experimentation performed by Goodman and Sundaram (1980). The publication is completed by the closure of Goodman and Sundaram (Fetzer et al. 1981).

Fetzer et al. (1981) suggest that criteria used to ensure that fines from the embankment of a dam do not wash out through a gravel foundation could be adapted to the allowable diameter size of embankment materials that can be placed against open bedrock joints. Fetzer et al. (1981) also suggest that appropriate precautionary treatment of bedrock joints, including the cleaning out of joints and inserting grout, would be sufficient to stop erosion through bedrock joints from occurring.

Barron (Fetzer et al. 1981) claim that filter material criteria would have to be very conservative in order to be applicable to joint openings. Barron (Fetzer et al. 1981) suggest that the largest problem with open bedrock joints is seepage occurring parallel to the embankment surface. He proposes that piping can occur along the bedrock joints as high velocities erode away embankment soils and carry them out of the open exit provided by the open joint. Barron (Fetzer et al. 1981) also discuss the treatment of bedrock joints with cleaning and the incision of grout as an overall solution for prevention.

The closure provided by Goodman and Sundaram (Fetzer et al. 1981) confront the argument that the criteria used for gravel packs would not be adequate to be adapted to open
bedrock joints in contact with embankment soils. They also deemed it important that criteria for embankment compaction against bedrock foundations containing joints be conservative. The original authors conclude that further research is needed to better comprehend the occurrence of seepage erosion through open bedrock joints.

### 2.7 Hole Erosion Testing

**Wan and Fell 2004**

This publication discusses two laboratory tests for measuring the erosion rates from coarse to fine grained soils: the Hole Erosion Test (HET) and the Slot Erosion Test (SET). The primary purpose of the study was to assess changes in erosion rates of the soils caused by varying two parameters, compaction effort and moisture content.

The HET is performed by placing a small hole in the center of a soil sample compacted in a standard mold and running water through the hole. The Hole Erosion Test is primarily used to estimate concentrated leak erosion rates. The test can be run in a short amount of time and can give a good estimate of erosion rates for materials. The test is designed to have a constant pressure head across the system to promote a steady increase of erosion. The testing apparatus used by Wan and Fell (2004) is shown in Fig. 13.

The SET is performed by taking a large, long soil sample with a small slot along the length of one side of the soil specimen and running water through the hole of the sample. This test is also predominantly used for testing concentrated leak erosion rates. The SET has a larger soil sample, which was the purpose behind creating the test. It was determined that the larger soil sample might provide less end effects than the HET might cause. The SET apparatus used by Wan and Fell (2004) is depicted in Fig. 14.
The analysis portion of this study included the equations that were used to calculate the hydraulic shear stress from the test data. The equation that Wan and Fell (2004) used to find the hydraulic shear stress, \( \tau_t \) (Pa) was:

\[
\tau_t = \rho_w g s_t \frac{\phi_t}{4}
\]

where \( \rho_w \) (kg/m\(^3\)) is the unit weight of water, \( g \) (m/s\(^2\)) is the acceleration due to gravity (used 9.8), \( s_t \) is the hydraulic gradient across the soil sample at time \( t \) and \( \phi_t \) (m) is the diameter of the preformed hole at time \( t \). Other analysis was performed to find the erosion rate. This analysis is not included because the hydraulic shear stress value is more applicable to the study performed.
It was found that coarse soils eroded rapidly and at a significantly lower shear stress. Teton Core material, primarily silt material, also eroded quickly at a low shear stress. However, most of the finer materials eroded at higher shear stresses and initiated at higher water velocities. Compaction effort and water content also had significant impacts on the results of tests. All soils eroded more quickly at lower compactions and lower than optimum water contents. For this purpose, 95% standard maximum dry density, along with optimum water content were used for the primary results of the HET and the SET.

The HET and SET were found to give very similar results, to the point of being interchangeable. Therefore, due to feasibility, economical, and functionality purposes Wan and Fell (2004) suggest that the HET be used instead of the SET to assess erosive behaviors of particular soils.
Wahl et al. 2008

This study was performed by the Bureau of Reclamation. The purpose of this study was to rebuild the Hole Erosion Test (HET) and use it to compare results from Wan and Fell’s (2004) results (Wan and Fell 2004). The other purpose of this study was to compare the results from the HET to a submerged Jet Erosion Test (JET). The idea behind performing these tests was to establish relationships that will aid in modeling the erosion of dam embankments and the eventual breach of these dams.

The HET performed in this test is designed after the HET performed by Wan and Fell that was previously reviewed. There were slight modifications made to the test setup and type of equipment used to make it more functional. The test was slightly enhanced by the Bureau with a higher ceiling so that the inlet tank could be raised more than Wan and Fell (2004) were able to raise their inlet tank. In a study performed at Utah State University, a municipal water source was used for the inlet in order to provide even more flow capacity (Erickson 2013).

The setup of a submerged JET has an in situ soil sample or a remolded sample in a mold that is exposed to a jet of water on the surface. There are no holes in the soil like the HET. A nozzle is used to create the stream of water for the test. This nozzle is placed at a desired distance from the surface of the soil and then the water pressure is increased to a desired pressure. The amount of scour of the material is observed during the test to be compared with other soil types and other types of tests.

The results that were found by Wahl and his associates (2008) for the HET very closely reflected those found by Wan and Fell (2004). Minor adjustments were suggested to improve the testing. An example of an adjustment made was to keep the head constant throughout a test performed. This allows for an exponentially increasing rate of erosion to occur. This is the
best method for finding the critical shear stress value at the initiation of erosion. This is the method that was used by Erickson (2013) in his study at Utah State University.

The conclusion to this study indicates that the JET and HET should be used for different erosion conditions that are being tested. The JET test is a test that can be used to determine scour effects in a dam and overtopping effects. The HET is better suited to crack erosion and internal erosion applications. The HET is more applicable to the study provided in this thesis than the JET.

Therefore, the Hole Erosion Test is determined to be just as effective as the Slot Erosion Test and the submerged Jet Erosion Test. A Hole Erosion Test will be most effective in comparing results with the data that will be found using the Internal Erosion Apparatus. For this purpose, a Hole Erosion Test was setup and performed on the same materials as those that will be tested in the Internal Erosion Apparatus. The Hole Erosion Test was performed at Utah State University (Erickson 2013).

2.8 Relevant Case Histories

**Von Thun et al. 1989**

This report is a review of the investigation to find the cause of failure of the Quail Creek Dike. The Quail Creek Dike was located in southern Utah just northwest of Hurricane, Utah. The 78 feet tall by 1980 feet long dike was constructed in 1984, but first filling did not occur until 1985. The primary purpose of this report is to learn from mistakes made in the construction and mitigation of this dike.

As with all dam or dike failures, many factors contributed to the actual breach of the dike. The primary mechanism that caused the ultimate failure of the structure was the erosion of the embankment soils down into openings in the bedrock. A few initial problems existed with
the foundation of the dike. The foundation of the dike was set on sloping bedrock, which
dipped into the left abutment, as shown in Fig. 15. Silty to clayey sand was used to fill in the
gaps formed by the dips in the bedrock of the foundation. This design caused this erodible
material to be along the entire width of the dike in some places. The other problem with the
foundation was seams of water-soluble gypsum contained in the bedrock foundation.

A triple grout curtain was installed in the buff sandstone bedrock foundation material
underlying the left abutment. However, no further grouting was performed along the rest of
the dam foundation. A cutoff wall was installed approximately 10 feet down into the bedrock
foundation along the centerline of the dam.

Seepage problems started occurring in the dam on first filling. Grout programs were
instituted on three different occasions to try to mitigate seepage problems. As grout was
pumped into the dam foundation, flow channels were cut off. The grout limited the amount of
flow paths through the foundation, which increased the pore pressures and differential head in
certain locations of the dam foundation (specifically in the breach location). The grout also
forced the flow to find new flow channels up along the embankment and foundation interface.
The higher gradients through the upper joints in the foundation of the dam caused the gypsum
in the rock to dissolve and increase the size of the natural forming conduits (Fig. 16). This
allowed high flows to come in contact with the silty to clayey sand material. This material began
to erode and sustain a pipe, due to sufficient cohesion and an unprotected exit point. The high
flows through the joints caused the embankment materials to become saturated, which
weakened them to the point of erosion and eventual breach of the dike (Fig. 17).

Eckerlin 1992

This repost is a case history of the installation of a cutoff wall into the existing Mud
Mountain Dam. The portion that will be focused on is the observations of the deterioration of
Fig. 15. Dips of Bedrock into the Left Abutment (Von Thun et al. 1989)

Fig. 16. Close-up of Upstream Channels Formed by Erosion (Von Thun et al. 1989)
the core of the dam before the seepage barrier was implemented.

Mud Mountain Dam is located about 24 miles east-southeast of Tacoma, Washington. The dam was constructed in a canyon with steep walls in 1941. The dam was 425 feet high by a crest length of 700 feet.

Open bedrock joints existed in the canyon walls prior to construction of the dam. Grout treatment of these open joints was performed before the construction of the dam. Unfortunately, the joints also contained varying amounts of soil that was not cleared out of the joints before the grouting was performed. Therefore, a possibility of seepage erosion occurring through the soil filled joints was introduced to the system.

A pre-investigation of the state of the dam embankment was conducted before a seepage barrier was installed. Results from the investigation include: large settlements along the left canyon wall, piezometers showed significant increases with fluctuations in the reservoir

Fig. 17. Breach of Quail Creek Dike from Downstream (Von Thun et al. 1989)
level at lower elevations and inflow tests demonstrated rapid flow through core defects in various locations. These observations demonstrate that there was likely erosion of the core material through the open joints in the abutments of the dam. This is also confirmed by the fact that no filter material was present between the canyon walls and the abutments of the dam.

This report does not give any specific conclusions of the reason for the deterioration of the core of the embankment. The results from the investigation are presented and then the mitigation process of installing a seepage barrier is described.

Davidson et al. 1992

This paper also summarizes the construction of the seepage barrier that was implemented in the Mud Mountain Dam. The specifics of the dam covered in Eckerlin’s (1992) report were also covered in this paper. This paper is being reviewed to give more evidence of the type of erosion occurring in the Mud Mountain Dam, which was the cause for a mitigation of the original design of the dam.

Davidson et al. (1992) describe the dam embankment as a graded mixture of sands and gravels with 15% to 20% fines. It is also verified in this paper that transition zones, which were placed between the core of the dam and the shell, are not made up of proper filtering material. The core itself is exposed to the canyon walls at both abutments.

The investigation showed that near the bottom of the core of the dam, clean gravels with all fines washed out were present. Davidson et al. (1992) believe this phenomenon to be linked to piping of the fines through the open bedrock joints of the canyon walls. The authors describe the process grouting that was performed during the dam construction and note that infilling soil material was not removed from the joints prior to grouting. Davidson et al. (1992) conclude that the in-filled soil eroded away to form a seepage path. This possibility describes how the fines in the core material were able to be washed out through the open exit of the
joints. This theory was never proven, since the dam is still in place, but it is extremely likely that
contact erosion was a primary erosive behavior transpiring in the Mud Mountain Dam before it
was mitigated.
CHAPTER 3
DEVELOPMENT OF TESTING APPARATUS

3.1 Design Introduction

The Internal Erosion Apparatus (IEA) will be presented in this manner. First, will be an explanation of the mechanisms this apparatus is designed to test. Then a description of the function of the apparatus is presented. Following that, is a presentation of the design and components of the apparatus. Then, a more detailed description of the most important components is given. Last, the instrumentation and devices used will be described.

3.2 Contact Erosion Mechanism and Function of the Apparatus

The mechanism that the apparatus is designed to simulate is contact erosion. As summarized in the introduction and literature review, contact erosion occurs when fine-grained soils are exposed to a coarse soil layer or other structure having a high flow capacity. For this study, the testing will consider a fine-grained soil over an open bedrock joint. In order for erosion to occur, the width of the joint must be larger than the smallest grain size of the embankment soil. Contact erosion will not occur until the soil is exposed to flow velocities that exceed the critical shear stress of the soils. The critical shear stress is the stress from viscous shear of the flowing water needed to detach soil particles.

The function of the apparatus is to create a direct flow path through a simulated open bedrock joint. The embankment material is to be placed on top of the open bedrock joint. This setup is in place to create the scenario similar to the base of the embankment material exposed to flow through the joint. The apparatus was also designed to have an open exit point so that if materials erode, they can be washed out of the system.
The goal is to provide a “piping” scenario along the bedrock joint and embankment interface. Part of this goal is fulfilled with an open exit, through which soil can be removed. The other part of the goal is fulfilled by using soil that is erodible but can still hold shape. This is accomplished with the right amount of cohesion in the soil. The apparatus is designed to have high flow velocities pass through it in order to provide erosive velocities to create contact erosion.

3.3 Design of Apparatus

The IEA design was modified from the Seepage Test Cell that was designed and constructed at Utah State University. Figs. 18, 19 and 20, show the side internal view, cut section of the top view and cut section of the end view. The locations that the top view and end views depict are included as cuts of the side internal view (see AA and BB on side Fig. 18). Dimensions of the features of the apparatus are included in the figures.

Many measures are taken to make the apparatus as close to a portion of a real dam as possible. The apparatus is capable of providing overburden pressure to simulate the overburden from a dam embankment. The significant test size area is intended to minimize end effects at the inlet and outlet and provide a more natural flow path to simulate the flow at the interface between a dam and its foundation.

The apparatus itself is made out of steel plates that are welded together. All pieces but the lid were welded together to help make the system water tight. The box’s outer dimensions are 14 inches wide by 45 inches long by 16 inches deep. Steel U-channels are welded on the top and the bottom of the box to provide addition reinforcement. Twelve half-inch diameter all-thread rods are used to secure the lid to the apparatus. The apparatus is set up on a wood base
and then on top of 4 pieces of 4 inches x 4 inches wood. This setup allows for access of the bottom of the apparatus. Refer to Fig. 21 for an actual picture of the apparatus.

The interior of the apparatus is set up, as described earlier, to allow water to pass through the joint in the bedrock material. As shown in the design pictures of the apparatus, the jointed bedrock is in the base of the apparatus. The bedrock material is modeled by high strength concrete (min 5000 psi strength). Two plexiglass boxes with holes in the bottom for flow to pass through were placed upstream and downstream of the concrete blocks. The downstream plexiglass box allows the flow to exit from the apparatus with no obstructions. Plexiglass fillers are placed above the plexiglass boxes at both ends to seal the flow off from the soil layer. Barriers are placed at each end of the joint to prevent erosion from initiating from the ends of the model, such as a knick point condition (soil eroded away because it sloughs off the endpoint) or jetting effects (water shooting into soil and eroding it because of a transition from large flow area to small flow area without a barrier to deter the water jet.) Two small sand bags are placed over the voids in the concrete at the ends of the joint to prevent jetting of the water into the soil. The test soil is placed on top of the concrete and compacted to a desired relative compaction (ranging from 85 to 95 percent relative compaction of the modified proctor test). Above the test soil, a coarse soil layer (washed mortar sand) is placed to fill in between the test soil layer and the air bladder that sits on top of all the soil. A plastic barrier is placed between the test soil and coarse sand layers to prevent contamination.

The main source of water for the system is the municipal water source in the laboratory. A pressure reducer has been inserted in the water line leading to the apparatus to allow the pressure to be regulated. A flow meter is installed in the flow line after the pressure regulator and before the apparatus to measure the flow going into the system. The outlet works of the setup is connected to a pipe that leads to the drain. A standpipe in the outlet pipe provides a
Fig. 18. Side View of Internal Erosion Apparatus (IEA)
Fig. 19. Top View of Internal Erosion Apparatus (IEA)

All Dimensions are in Inches
Scale 1:5
Fig. 20. Cross-Sectional View of Internal Erosion Apparatus (IEA)
Fig. 21. Side View of Internal Erosion Apparatus (IEA)
constant low-head backpressure at the downstream end of the model. The outlet pipe ends at
the drain where it flows into a containment bucket holding a #200 sieve. The sieve is there to
collect eroded particles. There is also a #60 sieve-sized filter fabric attached to the end of the
pipe to slow down the flow. This setup is shown in Figs. 22 and 23. Note that the project area is
outlined in black.

3.4 Components of Apparatus

The specific components of the apparatus to be discussed in this section include: the air
bladder, soils types used in testing, and the concrete blocks.

As described earlier, 6-inches tall concrete blocks were cast and placed in the bottom of
the apparatus to represent the bedrock foundation material. A picture of the mold used to cast
the concrete is depicted in Figs. 24 and 25. As shown, they were cast as two separate blocks.
Fig. 24 shows the concrete blocks are connected by a screw jack system which was cast into the
concrete. Each screw jack consists of 5/8-inches fine threaded all-thread that is connected to
the concrete blocks by placing a nut and washer on either side of both the blocks. There are
three screw jacks being used in the system. A hole for the washer and nut has been cast into
the block to make sure that the all-thread does not stick out past the concrete. The aperture of
the concrete is adjusted by adjusting the nuts in the screw jack system. The side view of the
design of the screw jack system is presented in Fig. 26. The design of the screw jack system for
the bottom layer is presented in Fig. 27.

Multiple anchors were cast into the concrete blocks. The anchors consisted of 5/8-
inches fine threaded couplers with a piece of all-thread propagating back into the concrete to a
small metal piece that was welded on. Two of these anchors were cast into the bottom of each
of the concrete blocks. These anchors are in place to secure the concrete to the bottom of the
apparatus (shown in Fig. 25). Corresponding holes in the bottom of the apparatus are in place to secure the anchors with all-thread, nuts, and washers. Threaded couplers (three on the outside of each block) that were cast into the side of the blocks provide side anchoring to the apparatus. These are no longer used because of sufficient anchoring of the block to the apparatus by the bottom anchors.

A flat, uniform-sided crack was cast into the concrete for the area that will be tested. The block’s surface is uniform (cast with a plexiglass divider depicted in Fig. 24) along its length, except for four inch (length) by one inch (width) voids at the front and back ends of the blocks to help more flow to enter and exit the system. An impermeable barrier material was placed at each end on top of the concrete to protect the soil from falling into the larger voids and to protect against erosion of the soil on the ends. Two small sand bags (custom made geotextile bags with highly permeable 20-30 Ottawa sand contained in them), were placed over the remaining portion of the void at the inlet and the void at the outlet to prevent jetting of the water into the soil. The highly permeable sand bags are also in place to encourage the flow to enter the soil layer. Fig. 28 depicts one of the sand bags being used in the experiment.

The air bladder is a custom-made rubber bladder with one threaded connector. This bladder is used to simulate overburden pressure in the soil. A pressure of 23 psi was used in all models. This pressure was used to provide a significant amount of overburden pressure and to prevent hydrofracturing in the soil and along the surface between the bottom of the air bladder and the top of the soil layer. A picture of the air bladder used is shown in Fig. 29.

3.5 Instrumentation and Devices

The purpose of our instrumentation is to measure key values that will provide indicators of erosion as it is occurring. The flow meter is used to measure the flow rate. LED lights and
Fig. 22. Inlet Portion of Setup (Project Area Outlined in Black)
Fig. 23. Outlet Portion of Setup (Project Area Outlined in Black)
Fig. 24. Main Portion of Concrete Form

Fig. 25. Bottom of Concrete Form
Fig. 26. Side View of Concrete Block Design
Fig. 27. Top View of Concrete Block Design
light sensors are used to indicate when the erosion void has reached certain widths along the top of the bedrock. This is done by setting the LED light across the joint from the light sensor on the surface of the bedrock at set distances away from the edge of the joint. The test soil is then compacted over the entire concrete surface, including in between the LED light and the light sensor. When the test soil is eroded from between the LED light and the light sensor, the light sensor detects the light. The light sensor works like an on/off switch giving an indication of when erosion occurred at a specific location.

Piezometers are being used to measure the pressure in specified locations along the top
of the joint. 1/8-inch diameter plastic tubes connect the input location for the piezometer to a Honeywell 26PC piezometer mounted to the outside of the device. The data collected from these instruments will be used to formulate results for the experimentation performed in this study.

A schematic drawing showing the location of the various instruments is presented in Fig. 30. Details of the numbering sequence of each type of sensor or LED light is indicated on the figure. The assigned values for each sensor or LED light will be referred to throughout the rest of the study.

The instrumentation and devices being used in the experiment will be presented from inlet to outlet. Figs. 22 and 23 show the specific location of each of the instruments or devices that are outside of the apparatus. For the instrumentation within the apparatus see Fig. 30. The function of each instrument or device will be discussed below.

The pressure reducer is a Type B Cash valve, which regulates the pressure from the municipal water source down to pressures ranging from 2 to 20 psi. A picture of the pressure regulator being used is presented in Fig. 31.

A 2-inch diameter Octave Ultrasonic Flow Meter manufactured by Master Meter is plumbed in upstream of the IEA to measure flow through the duration of the test. The flow meter is shown in Fig. 32. The meter uses ultrasonic sound waves that travel between two sensors located in the interior of the flow meter. These waves are sent with the direction of flow and then against the direction of flow to record an accurate velocity and volume of water. The output is a pulse current that ranges from 4-20 mA. The data logger is unable to produce enough power to operate the flow meter so an auxiliary power source has been connected in series with the flow meter. A 100 Ω current shunt terminal input module made be Campbell Scientific is also connected in-series with the flow meter and auxiliary power source to protect
the data collection device from surges in power. A digital reading on the flow meter itself was used to calibrate the value being read by the data logger.

As previously mentioned, several types of sensors are used in the apparatus to monitor the propagation of the erosion along the concrete and soil interface. Honeywell 26PC series piezometers with a +15.0 psi range are used at various locations to measure the water pressure. As depicted in Fig. 30, piezometers are located at the inlet and outlet of the IEA to measure the differential pressure across the entire system. These piezometers are installed on the inlet pipe right before it enters the apparatus and on the outlet pipe right after it exits the apparatus. Three piezometer sensors are placed along the length of the crack. These piezometers are

![Diagram of sensor locations](image)

**Fig. 30. Location of Sensors along Crack**
actually placed on the outside of the apparatus, but have 1/8-inch tubes running into the apparatus to the very edge of the crack. The locations of these three piezometers are presented in Fig. 30. Piezometers were calibrated using air pressure and a panel board. Fig. 33 is a photograph of a piezometer sensor used in the IEA.

The other type of sensor used in the apparatus is a light sensor system used to detect when erosion has progressed to prescribed distances from the crack. The light sensor system consists of two photo IC type high sensitive light sensors manufactured by NaPiCa Sensors and
an LED light to provide the light source. The LED light that is used in this experiment is a snap-in LED from the 558 Series (white color with straight terminals) manufactured by Dialight. LED lights are placed at an approximate distance of 1/4 inch from the crack, on top of the concrete, on one side of the joint, while light sensors are placed on the opposing side of the concrete. This can be seen in Fig. 30. The first sensor is placed approximately 1/4 inch from the edge of the crack in the concrete and the second sensor is placed approximately 1/2 inch away from the edge of the concrete. This set up is shown schematically in Fig. 34.

The LEDs and light sensors are placed in four evenly spaced locations along the length of the crack in the concrete (See Fig. 30) to help understand when, where and how much erosion is occurring during the duration of the test. Through the duration of the testing the LED lights are
illuminated by power received through a circuit board to a transformer. Two LED lights are set in series with a LM317LZX semiconductor, a 0.1 µF capacitor and a 62 Ω resistor. A schematic drawing of this setup is presented in Fig. 35. A photo of the complete circuit board is presented in Fig. 36. When the soil erodes to the point where both the LED and light sensor are exposed, the light sensor will detect the light and indicate the erosion as a change in voltage. This change is read by the data logger and is plotted in real time versus the elapsed time of the test. Thus, the four pairs of sensors give an indication as to the rate at which soil is eroding along the length of the crack. The light sensors act like an on/off switch (light/no light) and therefore were not calibrated to measure light intensity. Thus, although the peak values vary between the sensors, there is no correlation between these values and the intensity of the light. Pictures of a light sensor and a LED light are in Figs. 37 and 38, respectively.

![Schematic Drawing of Powering of LED Lights](image)

**Fig. 35.** Schematic Drawing of Powering of LED Lights

All sensors and LED lights were connected to the data logger using shielded, twisted-pair cables. Shrink tubes and epoxy or silicon sealants were used to waterproof the connections located inside the test area. Shrink tube was placed around each individual wire to prevent current from jumping between wires. Extra sealant measures were taken to seal LED lights and light sensors. Slow setting epoxy was set inside of shrink tube around the connections of the
wires. This potting effect worked fairly well in keeping water away from the wires. The
piezometer sensors were sealed using silicon and shrink tube. Figs. 39, 40 and 41 show a
piezometer with wire, a light sensor with wire and a LED light with wire, respectively.

![Photo of Circuit Board for LED Lights](image1)

Fig. 36. Photo of Circuit Board for LED Lights

![Photo of Light Sensor](image2)

Fig. 37. Photo of Light Sensor

All data from instrumentation is collected using a Campbell Scientific CR3000 data
Logger. The data logger is connected to the computer, which stores all data from the
experiments. Data collection has been programmed to occur every minute for this experiment.
Fig. 42 shows the CR 3000 with all the wires connected.
Fig. 38. Photo of LED Light

Fig. 39. Piezometer Soldered to 5 Wire

Fig. 40. Light Sensor Combination Soldered to 5 Wire
The outlet of the IEA consists of a stand pipe that rises about a foot above the top of the apparatus to ensure positive water pressure throughout the soil sample. Downstream of the stand pipe the water outlets into a filtering bucket depicted in Fig. 43. As shown in Fig. 43, the water flows through filter fabric on the end of the pipe (#60 mesh) and into a #200 sieve. The filter fabric is used to slow down the flow and catch the larger soil particles. The #200 sieve is used to retain finer soil particles that are eroded from the “embankment material” being tested. Because the fine (-200) sized portion of the soil is not retained on either the fabric or the sieve, the amount of soils collected is not a quantitative representation of the amount of erosion that has occurred. Rather the soils observed in the outlet provide a means for assessing when erosion may be occurring.
Fig. 42. Photo of Data Logger

Fig. 43. Photo of Drain Setup
CHAPTER 4

TESTING PROCEDURES

This chapter discusses the soils tested and the procedures used in performing the tests. First, the needed soil properties for an experiment will be outlined. The step by step process of how the tests were setup are presented. Next, the process for monitoring an experiment and when it should be terminated is given. Finally, the post-test documentation is discussed.

4.1 Pre-Experiment Soil Testing

Prior to using soils in the experiments, laboratory tests are performed to quantify the basic soil properties such as: maximum dry unit weight, optimum water content, grain size distribution and plasticity. The unit weight and water content values are used to establish target densities and water contents while constructing the laboratory models. The target range for soil density is 80 to 90 percent relative compaction. Other tests are used to provide good soil descriptions and give relevance to the results of the tests.

4.2 Laboratory Test Setup

1) The procedure for setting up the experiments is presented below. Set the width of the crack between the concrete blocks. Three sets of spacers are used to set the crack at 1mm, 2mm or 3mm. The crack width is set using the screw jack system described in Chapter 3. For all testing done for this study the crack width was set at 3 mm. Holes that were cast for the screw jack system are then covered with duct tape or window flashing to help prevent flow along the all-thread bolts of the screw jack system.
2) All holes along the side of the apparatus are sealed with duct tape or window flashing.

3) A piece of foam that is the width of the apparatus by the length of the concrete is glued (with contact cement) onto the base of the apparatus. Holes are drilled out of the foam to match the holes drilled in the base of the apparatus for the concrete block base anchors.

4) The concrete blocks are moved into their position in the apparatus. Threaded couplers cast into the surface of the concrete provide anchors for an overhead lift to assist moving the concrete blocks.

5) The bolt anchors on the bottom of the concrete blocks are tightened to compress the foam beneath the concrete blocks and seal the bottom of the apparatus. These anchors also secure the blocks to the apparatus.

6) The ends of the blocks are sealed off by placing window flashing over the outside edges of both ends of the concrete. Window flashing was also placed to cover the steel/foam interface and the foam/concrete interface. The flashing overlaps all joints by at least 1/2 inch to provide a seal.

7) Spray foam insulation is placed between the inside of the apparatus and the concrete blocks to prevent flow from flowing along this gap. Window flashing is applied liberally over the top of the gap now filled with spray foam.

8) Plexiglass boxes, with holes at their base to allow flow to enter and exit the apparatus unrestricted, are inserted into the gaps between the ends of the concrete blocks and the upstream and downstream ends of the apparatus. The downstream plexiglass box allows soil to be removed from the system, which is one of the crucial
points to allow contact erosion to occur. The top of the plexiglass is level with the
top of the concrete.

9) Plexiglass fillers that are placed above the plexiglass boxes can now be glued in place
on the inlet side and wedged into place on the downstream side. These plexiglass
fillers are placed to help deter flow from flowing along the edge of the apparatus
and into the upper soil layer. Silicon is used to seal around all edges as an extra
precaution.

10) The instrumentation is placed in designed locations along the length of the
apparatus as depicted in Fig. 44.

   a. The light sensors and the LED lights are placed 1/4 inch away from the edge
      of the concrete. The wires connecting the light sensors and LED lights are
      run along the top of the concrete and out through designated holes in the
      side of the apparatus.

   b. Tubing is placed from the piezometers on the outside of the apparatus to
      the locations depicted in Fig. 44.

   c. Portions of wires and tubes in the apparatus are glued or tapped down to
      the surface of the concrete to prevent them from moving while the soil is
      placed and the test is being run.

   d. The instrumentation is connected to the data logger.

11) Impermeable barriers are placed around the soil zone on the inlet and outlet sides
    of the apparatus. A rubber barrier is glued into place on the inlet side of the
    apparatus with at least 2 inches of overlap onto the steel and concrete. The outlet
    is covered with a piece of pvc sheeting that overlaps the concrete by 2 inches.
These are in place to limit flow along the interface of the sides of the apparatus and the soil.

12) Two small sand bags (See Fig. 28) are placed centered over the larger voids at the ends of the crack in the concrete block. These sand bags help protect against jetting effects and allow flow to come up into the soil layer when erosion occurs above the crack.

Fig. 44. Dimensions of Instrumentation
13) Plexiglass triangles are glued into the upper corners between the upstream plexiglass and the side of the apparatus to support the rounded corners of the air bladder. Two small metal plates are welded in place on the downstream side to perform the same function.

14) One-fourth inch thick pieces of foam matching the dimensions of plexiglass fillers are placed on top of the plexiglass fillers to prevent water from escaping between the top of the end plates and the lid of the apparatus.

15) Soil is placed on top of the concrete in 1/2 inch thick lifts. The water content for the soil is premixed and mass of soil in each lift is weighed, so that the density and relative compaction of the compacted soil can be calculated.

16) A plastic layer is placed on top of the test soil to prevent flow from entering the more permeable upper soil layer of washed mortar sand.

17) Dry washed mortar sand is placed and compacted on top of the layer of plastic to within about 1/2 inch of the apparatus top.

18) The air bladder is placed on top of the soil. Powdered bentonite is used to seal above and below the air bladder to prevent water from flowing around the air bladder.

19) The lid is placed on the apparatus. The groves along the underside of the lid are sealed using plumber’s putty. Plumber’s putty is also placed around the base of the air bladder connector.

20) Once the lid is in place, twelve 1/2 inch pieces of all-thread with nuts welded to the top of them are used to secure the lid to the apparatus.

21) The air bladder is connected to 23 psi air pressure (air source is a panel board).
22) A 1 hour long period, to allow the bentonite to further saturate, is waited before the test is initiated.

23) The data logger is started during the 1 hour long wait period. This is done to allow the light sensors to be zeroed to a consistent 0 mV reading before flow is introduced into the system. The power sources for the flow meter and LED lights are also plugged in at this point. A computer is connected to the data logger. This computer is equipped with the program Logger Net (created by Campbell Scientific) that is used to collect and display the data during the test.

24) Following the 1 hour waiting period, water flow though the device is initiated. The water is turned on slowly so as not to heave the soil sample. This is accomplished by using the pressure reducer to start at a low differential pressure and then slowly turning up the pressure over time. The rate of pressure increase should vary according to soil type and other factors.

4.3 Test Monitoring and Progression

Monitoring during the test consisted of observation of the collected data in real-time on a computer screen and checking the outlet filters for eroded soil. The purpose of monitoring is to determine the point at which a test is to be terminated. A discussion of the monitoring process is presented below along with a short discussion of warning signs from the data.

The data logger collected data from five piezometers (1 through 5), eight light sensors (1 through 8), and a flow meter. The locations of these sensors within the apparatus are presented in Fig. 30. The data is collected every minute. A real-time display of the data is presented on the computer screen allowing behaviors in the experiment to be monitored as they happen. Data is placed in a variety of different plots to see relationships between similar values. The
flow value versus elapsed time is placed on one graph, the piezometer values versus elapsed
time are placed on another graph and the sensor values versus elapsed time were placed on
another plot. All values were placed on a table which was viewable throughout testing.

Monitoring the data being collected on the computer helps to understand what is
happening in the experiment. The first change to be observed, in the test data, is the initiation
of erosion. The initiation of erosion is difficult to detect. An increase in flow without the inflow
pressure being adjusted is an indication that erosion could be initiating. Flow could also
decrease coupled with an increase in inlet pressure, which may be an indicator of eroding soils
blocking the flow path in the crack. There could also be local changes in the differential
pressure between adjacent piezometers, indicating a change in flow resistance between
piezometers. This is an indicator that erosion is happening at that specific location.

Most of the test results indicate erosion generally initiates in the location around
sensors 5 & 6. It is possible that this location represents a location where there is enhanced
turbulence or where flow is directed toward the soil. This area was specifically monitored to
watch for an indication of the initiation of erosion.

When erosion was not observed in the test, within a couple days, at a constant pressure
level, the differential pressure was increased. The pressure was increased until the inlet
pressure reached a maximum of 21 to 23 psi to ensure that the water pressure did not exceed
the bladder pressure.

The light sensors are able to provide evidence of erosion progression, but not indicate
the initiation of erosion. The light sensors are set back 1/4-inch and 1/2-inch from the crack and
therefore unable to detect when erosion initiates. However, as the erosion progresses, the
sensors give a direct indication of how large the erosion void has become. This data also helps
to assess the rate of erosion that is occurring in the soil. The sensor readings will generally rise
quickly and then level off at a value. This is not always the case though. Once “on,” the sensor readings might drop somewhat due to soil particles passing between the LED light and the light sensor. This could also be an indication of erosion propagating in other areas along the crack.

Other indications that erosion is progressing are provided by the flow and piezometer readings. Flow will, generally, continue to increase from the initiation of erosion until the erosion stops. The increases in flow in the tests run for this thesis were about 2% to 3%, so relatively small increases. As a general rule, inlet pressure (piezometer 1) will drop as erosion propagates and the outlet pressure (piezometer 5) will increase to counter the drop in the inlet pressure. The other piezometers may indicate local erosion between piezometers if the differential pressure between the piezometers drops, indicating a reduction in flow resistance between the piezometers locations.

The observation of soil in the outlet works, the bucket containing #200 sieve to catch soil particles, is used as a way to verify that erosion is happening within the system. However, since much of the fine grained soil washes through the sieves, this observation does not provide a quantitative assessment of the erosion progression.

There is no set time for experiments to run. The tests will be stopped when erosion has ceased, when it is decided that the soil being tested will not erode or if the soil being tested erodes very quickly. The test is to be stopped when progression of erosion has stopped or can no longer be monitored. This step is primarily contingent on the sensor readings. A test is also terminated, if the test has run for an extensive period (2 weeks +) with no indications of erosion.

The variables from the test data also help indicate when there is a problem in the experiment. If a test erodes very quickly, the sensors will reach the peak level of light from the LED and hold for an amount of time before dropping back down to 0 mV again. This is an indication that the test soil has eroded out and that the upper soil layer has caved in on top of
the LED light and light sensors. Indications of erosion may or may not be detectable in the flow and piezometers.

Other indicators that the soil has eroded out quickly will be shown in the outlet works and the air bladder pressure. The #200 sieve will be filled with a significant amount of soil if erosion has occurred rapidly (half or more than halfway full). The air bladder pressure could drop due to the expansion of the bladder or the rupture of the bladder.

4.4 Test Disassembly and Documentation

When a test was deemed complete, the following procedure was followed to document the final condition of the experiment.

1) Before the test is shut down all the data is to be downloaded onto the computer from the data logger. This file is saved as a text file that will be used for data analysis.

2) The water is then slowly turned off to try to preserve the shape of the pipe formed in the soil.

3) The air bladder is then deflated and disconnected from the air source.

4) The lid and air bladder are removed.

5) The washed mortar sand and plastic separator are removed.

6) The test soil is removed in vertical sections. The soil is first excavated from the downstream end to the locations of light sensors 7 and 8 and LED light 4. A picture is taken of the cross section of the soil layer at this point to document the extent of erosion. The shape of the pipe at this location is sketched for further documentation.
7) The test soil is now excavated back to piezometer 4 and similar documentation (photo and sketch) is performed. The excavation and documentation procedure is continued for the locations of light sensors 5 & 6 and LED light 3, piezometer 3, light sensors 3 & 4 and LED light 2, piezometer 2, and light sensors 1 & 2 and LED light 1.

8) The remainder of the soil is now excavated. The test soil is spread out to dry.

9) The last step in completing the test, is recording the density on the template. This value is calculated from the amount of test soil used when filling the apparatus with soil divided by the volume that the soil filled in the apparatus. This density value can then be compared with the maximum dry density from a modified proctor test to calculate the relative density. The relative density will be presented in the results portion of this study.
CHAPTER 5

ANALYSIS AND RESULTS

This chapter presents the result of the tests and discusses their significance to the overall study. Procedures for data processing and analysis are presented, followed by the processed results from the various tests performed. The results of the testing is then compared to published correlations for erosivity and the results of Hole Erosion Tests performed in the USU Earth Structures Laboratory as part of this study.

5.1 Data Analysis

This section covers the processing and evaluation of the data. The first step is to plot the data so that changes in behavior, that are indicative of erosion, can be observed. Secondly, the data is used to calculate values (including viscous shear stress and expected radius of the erosion channel) that will assist in the comparison of the test results to published erodibility correlations and other laboratory tests. While this section presents the equations and relationships used in the data analysis, the reader may find the application of the data analysis easier to follow in the actual descriptions of the test results presented in Section 5.2.

The data from each test was converted into an excel spreadsheet file. Initially, three plots were made from each data set: flow versus elapsed time, piezometer readings versus elapsed time, and light sensor readings versus elapsed time. These plots allowed for the observation of changes in data over time that were indicative of the initiation and continuation of erosion throughout the duration of the test. In some instances these plots were combined to compare the behavior of the two sets of data.
The data was further manipulated to compute values of: Reynolds number, velocity, transmissivity, shear stress through the crack, average friction ($f_{avg}$) and shear stress on the soil. These key values are used to assess the behavior of the flow in the crack and to obtain values that could be correlated with the results of other studies.

The Reynolds number is a unit less value used to assess whether the flow is turbulent or laminar. The Reynolds number was calculated using the following equation which applies to flow between parallel plates:

$$Re = \frac{QD_H}{\nu A}$$  \hspace{1cm} (5.1)

where, $Q$ (m$^3$/s) is the flow rate, $D_H$ (m) is the equivalent hydraulic diameter, $\nu$ (m$^2$/s) is the kinematic viscosity, and $A$ (m$^2$) is the cross sectional area of the void in the crack. The kinematic viscosity was found through dividing the dynamic viscosity ($\mu$) of water @ 5$^\circ$ C (0.001519 Pa·s or kg/(m·s)) by the density ($\rho$) of water @ 5$^\circ$ C (1000 kg/m$^3$). The equivalent hydraulic diameter was calculated as 4 times $A$ (m$^2$), the cross sectional area of the void in the crack, divided by $P$ (m) the wetted perimeter (Benson 2009). The Reynolds number was evaluated by the following relationships: if $Re > 4000$, then the flow is turbulent or if $Re < 4000$ but $Re > 2300$, then the flow is laminar (Holman 2002). By calculating the Reynolds number for each test, we are able to assess when the flow becomes turbulent and thus are able to select the appropriate correlations for erosivity.

The average velocity ($V_A$, m/s) at any time during the duration of a test is calculated using the following equation:

$$V_A = \frac{Q}{A}$$  \hspace{1cm} (5.2)
where, $Q$ (m$^3$/s) is the flow rate and $A$ (m$^2$) is the cross sectional area of the crack. The critical velocity for each test was determined by averaging the $V_A$ values over an hour long period near the time where the initiation of erosion was interpreted to have taken place. The critical velocity is used to compare the test results to published values of similar soils.

Transmissivity ($T$, m$^2$/s) is a measure of flow resistance in the entire test apparatus (crack and erosion void) and is calculated using the following equation:

$$ T = \frac{Q}{\Delta h} \quad (5.3) $$

where, $Q$ (m$^3$/s) is the flow and $\Delta h$ (m) is the differential head. As erosion occurs in the system, the transmissivity is expected to increase. Thus, by plotting transmissivity versus time an indication of the start and progression of erosion may be observed.

A theoretical value of shear stress applied by flowing water on the crack walls ($\tau_c$, Pa) was calculated using the following equation:

$$ \tau_c = \frac{\rho_w g H_f^2 W}{2(H_f + W)L} \quad (5.4) $$

where, $\rho_w$ (kg/m$^3$) is the density of water (1000 kg/m$^3$), $g$ (m/s$^2$) is the acceleration due to gravity (used 9.8 m/s$^2$), $H_f$ (m) is the head loss in the crack due to friction, $L$ (m) is the length of the crack and $W$ (m) is the width of the crack (ICOLD 2012).

The shear stress was calculated by averaging an hour’s worth of flow and head loss data points around the estimated initiation of erosion. The critical shear stress is the stress needed to initiate erosion and is obtained by calculating the shear stress using the data from the time when erosion is observed to initiate. It should be noted that Equation 5.4, is derived for a planar
crack and the validity of the calculation decreases as the size of the erosion void increases. The shear stress through the crack is graphed versus the measured flow for tests that held a pipe. The critical shear stress is used in comparison to other published data for similar soils.

Another measure of the shear stress is obtained by distributing the force required to develop the observed head loss along the walls of the length of crack over which the head loss occurs. The average friction \( f_{avg} \) (or frictional shear stress) was calculated using the following equation:

\[
f_{avg} = \frac{h}{L} \cdot \gamma_w \cdot \frac{A}{P}
\]

where, \( h \) (m) is the differential head between inlet and outlet or a set portion of the crack, \( L \) (m) is the length of the crack or a set portion of the crack, \( \gamma_w \) (kg/m\(^3\)) is the unit weight of water (used 9810 kg/m\(^3\)), \( A \) (m\(^2\)) is the area of the crack and \( P \) (m) is the wetted perimeter of the crack.

This equation was derived at Utah State University (James Bay, personal communication, May 10, 2013). This equation is used as a check to verify Equation 5.4 and to simplify the calculation process that was shown to yield approximately the same value as Equation 5.4. This equation was used to in flow versus shear stress plots for all tests performed for this research.

As erosion occurs in the soil above the crack, the resistance to flow through the device decreases. This results in more flow through the device under the same differential head. The increase in flow due to erosion, \( \Delta Q \), is related to the size of the erosion void described by an equivalent radius, \( r \). An equation by Longwall (1966) relating the frictional loss in pipe to its diameter was modified to calculate the effective radius of the erosion void responsible for the observed \( \Delta Q \):
\[
 r = \left( \frac{\Delta Q \ast (8 \ast n)}{\pi \ast \frac{dP}{dx}} \right)^{1/4}
\]

where, \(\Delta Q\) (ft\(^3\)/s) is the change in flow due to erosion, \(n\) (lb*s/ft\(^2\)) is the viscosity of water (used 6.3 * 10\(^{-7}\) lb*s/ft\(^2\)) and \(dP/dx\) (m) is the differential head between inlet and outlet or a set portion of the crack. This equation is used later in this chapter to correlate between the piezometer data and the erosion device.

5.2 Results of Testing

This section presents the results of the tests and the corresponding data for each test. Each test is presented separately and the results are summarized at the end of the section.

5.3 Sand Mixed with Clay

Test 1: Sand/Clay (80/20)

Test 1 was performed on a mixture of 80% (by weight) Garnet Sand mixed with 20% Helmer Clay (Kaolinite). The aperture of the crack was set to 3 mm. The compaction density (total unit weight) of the soil was about 143 lb/ft\(^3\) at a water content of 13%. The relative density of the test, based on a modified proctor, was about 89% (modified proctor curve in Appendix A).

The test ran for just under two days and the soil completely eroded out of the system very quickly. Fig. 45 shows the amount of soil that remained in the apparatus from the 3.25 in. deep soil layer of erodible soil (The top of Fig. 45 is the upstream side and the bottom of the figure is the downstream side). As can be seen, the sand mixed with clay layer almost completely eroded out of the apparatus in less than 2 days.
Fig. 46 is a plot of the light sensors and flow volume versus elapsed time through the duration of the test. As shown on the plot sensors 3, 4, 5 and 7 were triggered first, indicating the contact erosion started in the center of the apparatus; slightly towards the inlet side. The erosion propagated rapidly towards the outlet and then eventually began to erode along the entire length of the crack. The light sensors began to receive less light as the test went on. This is believed to be due to coarse material above the plastic layer collapsing down on top of the crack as the eroding soil was removed. The flow rate increased slightly at the beginning of the test, as highlighted in Fig. 46, and then showed a slight but constant decrease over the duration of the test. It is believed the erosion initiated very soon after the flow was started.
Fig. 46. Light Sensor Output and Flow Volume Versus Elapsed Time for Sand/Clay (80/20)
During the first day of the test, the sensors that were triggered held a relatively constant value. After the first day, all of the sensors triggered and all of them (including the first four) behaved erratically. It is believed this behavior is due to a pipe forming and holding open during the first day of the test. The erratic behavior in the later part of the test is thought to be due to successive collapsing of the pipe and sloughing of the sides of the eroded void; intermittently covering the light sensors or LEDs either partially or completely.

Fig. 47 presents a plot of the piezometers versus elapsed time. Piezometer 1 shows a drop in pressure, while piezometer 5 demonstrates a slight increase in pressure, as erosion initiates and continues to propagate. A significant increase in pressure in piezometer 2, slight increase of the pressure in piezometer 3 and the initiation of a decrease of the pressure in piezometer 4 all occurred at the same time as the erosion detected by the light sensors can be seen in Fig. 46. Also, the piezometer readings indicate slow and steady changes during the first day of the test and more erratic behavior in the latter days of the test. This also correlates well with the light sensor data. Thus, both sets of sensors indicate rapid formation of an erosion pipe at the start of the test followed by collapse and sloughing of the pipe in the later portion of the test.

A plot of the flow volume versus the calculated shear stress is presented in Fig. 48, in which two different curves are plotted. The darker colored plot represents the shear stress calculated using Equation 5.4 ($\tau_c$). The other curve represents the shear stress calculated using Equation 5.5 ($f_{avg}$). The two plots are very close to each other indicating close agreement between the two equations. Therefore, Equation 5.5, or the $f_{avg}$ shear stress equation, will be used for this plot throughout the remainder of the tests presented herein.
Fig. 47. Piezometers Versus Elapsed Time for Sand/Clay (80/20
In Fig. 48, the darker colors (black or gray) represent the flow versus shear stress values leading up to erosion (essentially as the differential pressure is applied). The lighter colors (blue to red) represent the flow versus shear stress values after the initiation of erosion. The purpose of the colors is to show how the erosion propagates. For this particular test, the erosion began very soon after the flow was started and continued until day 2 of the test. The propagation of erosion causes the blue-colored portion of the plot to migrate to the left (lower shear stress). This migration is indicative of a decrease in flow resistance resulting from the erosion void. As the test progresses, the green and red colored portions of the plot migrate back to the right, indicating an increase in flow resistance. This increase in flow resistance is likely due to the collapse of the erosion pipe. The increase in flow, $\Delta Q$, shown in Fig 48 represents the flow capacity of the erosion void at that point of the test. Because no erosion void was observed in this test, no comparison with a calculated effective radius (Equation 5.6) was performed for this test.

For this particular test, the primary erosion happened within the first day. This can be detected by the blue line being the farthest to the left in Fig. 48. The green and red points show that the overburden soil slowly began to collapse down over the crack to decrease the area that the water was flowing through, resulting in an increase in shear stress. As seen on the plot, when erosion began the flow increased slightly (vertical increase) and the shear stress decreased significantly (horizontal shift to the left). This is a trend that will be observed in the other erosive tests analyzed in this thesis. Flow volume versus shear stress, will be plotted for each test performed for the research performed for this paper. This will be used as a base comparison of all the tests run for this research.
Fig. 48. Flow Volume Versus Shear Stress for Sand/Clay (80/20
Test 2: Sand/Clay (75/25)(a)

Test 2 was performed on a sample consisting of 75% (by weight) Garnet Sand mixed with 25% Helmer Clay (Kaolinite). The aperture of the crack was set to 3 mm for this test. The compaction density of the soil was approximately 148 lb/ft$^3$ (total unit weight) at a water content of 12% (close to optimum, slightly dry). The relative density of the test, based on a modified proctor, was about 92% (modified proctor curve in Appendix A). This test ran for just over 22 days.

Figs. 50, 51, 52 and 53, are pictures of piping at each sensor location moving from downstream to upstream. The soil had enough cohesion to hold a pipe for the duration of the test. As seen in Fig. 49, it is believed that a portion of the pipe collapsed when the flow/water pressure was turned off. The collapsed portion was located toward the downstream side of the apparatus, where a greater amount of erosion appears to have occurred (Left side of Fig. 49 is the upstream side and the right side of the figure is the downstream side). The photos in Figs 50-53 indicate a significant amount of softening occurred in the test soil surrounding the pipe. This behavior fits with theoretical examples found in Fig. 10 from ICOLD 2012.

![Fig. 49. Picture of Top View of Contact Erosion for Sand/Clay (75/25)(a)]
Fig. 54 presents a plot of light sensor output and flow volume versus elapsed time. Fig. 55 presents a plot of the piezometer readings versus elapsed time. While erosion is not evident in Fig. 54 until after day 12 when the flow is turned up to 29 gpm, indications of erosion are evident in Fig. 55 in day 7 after the flow is turned up to about 28 gpm. In Fig. 55, the differential pressures across the entire model and between adjacent piezometers start decreasing as soon as the flow is turned up to 28 gpm. Piezometers 1 (the inlet), 2 and 3 experience a decrease in pressure and piezometer 4 shows an increase in pressure while piezometer 5 (the outlet) remains relatively constant. This behavior indicates that the greatest amount of erosion is occurring between piezometers 3 and 4 where the differential pressure decreases the most. The changes in pressure occur over about 2 days; after which the pressures stabilize until the flow is again increased on day 12. This stabilization is likely indicative of a state of relative equilibrium being achieved between the flow and the size of the crack and erosion void.

When the flow is turned up to 29 gpm, 12 days into the test, the renewed propagation of erosion is evident in both Figs. 54 and 55. In Fig. 55, the piezometers behave similarly after
Fig. 51. Picture of Contact Erosion at Sensors 5 & 6 and LED 3 for Sand/Clay (75/25)(a)

Fig. 52. Picture of Contact Erosion at Sensors 3 & 4 and LED 2 for Sand/Clay (75/25)(a)
the flow increase to 29 gpm as they did when the flow was increased to 28 gpm with the
greatest amount of erosion again occurring between Piezometers 3 and 4. In Fig. 54, the flow
becomes somewhat erratic, a condition thought to be associated with the dislodging and
transportation of soil through the crack. Around day 14 the light sensors start to indicate
widening of the erosion channel. Light sensors 5 and 6 are the first to indicate erosion; a fact
that supports our earlier assessment of erosion since these sensors are between piezometers 3
and 4. Later, other light sensors indicate widening of the erosion channel in other locations.

Fig. 56 presents details of the light sensor output and flow volume versus elapsed time
during the last 5 days of the test and Fig. 57 presents details of the flow data over most of the
same period. In Fig. 56, it is evident that increases in flow are often preceded by a temporary
drop in flow and often correspond with the triggering of light sensors. This may be evidence of
the episodic nature of the erosion where masses of soil erode at once and tend to temporarily
clog the crack or erosion void. Also noticeable in Fig. 56 is that not all of the sensors, specifically
sensors 2, 3 and 4, were an indication of erosion at the conclusion of the test. As seen in Figs.
50 and 51, it is possible that the softened soil could have still been blocking the light from

**Fig. 53.** Picture of Contact Erosion at Sensors 1 & 2 and LED 1 for Sand/Clay (75/25)(a)
Fig. 54. Light Sensor Output and Flow Volume Versus Elapsed Time for Sand/Clay (75/25)(a)
Fig. 55: Piezometers Versus Elapsed Time for Sand/Clay (75/25)
Fig. 56. Details of Light Sensor Output and Flow Volume Versus Elapsed Time for Sand/Clay (75/25)(a)
Fig. 57. Increasing Flow Volume Versus Elapsed Time over Higher Flows for Sand/Clay (75/25)(a)

reaching these light sensors.

The piezometers versus elapsed time plot, found in Fig. 55, gives further evidence to back up the data analysis performed using the flow and the light sensor output. As depicted in Fig. 55, the inlet pressure, represented by piezometer 1, begins to drop as erosion initiates (starting level just above 20 psi down to almost 18 psi). The outlet pressure, piezometer 5, increases slightly with the initiation of erosion. There are varying changes in the piezometers along the length of the crack. These changes can be correlated to continued erosion in specific locations. For example, piezometer 3 and piezometer 4 increased as more erosion occurs towards the downstream side of the apparatus on day 20 of the test. Piezometer 2 was at a higher initial pressure and dropped as the pressure at the downstream side began to increase. The effects of erosion are apparent in local piezometric pressures within the system. This also
helps verify that the light sensor output readings are correct.

Fig. 58 presents flow volume and differential head across the model (i.e. the difference in head between piezometers 1 and 5) plotted versus elapsed time. This plot shows that drops in differential head are often accompanied with increases in flow; a condition that would not occur if erosion were not occurring. Fig. 59 presents flow volume and transmissivity through the model plotted versus elapsed time. Transmissivity is inversely proportional to the differential head and thus has an opposite trend on the plot. Similar to Fig. 58, erosion is evident by rises in transmissivity corresponding with rises in flow (the opposite would be observed if erosion were not occurring).

Fig. 60 is a plot of flow volume versus shear stress, calculated using Equation 5.5, versus flow. The black portion of the plot maps the increases in flow through the non-erosive portion of the test and the varied color portion is the erosive portion of the test. The test was held at a non-erosive flow near 22 gpm where the data indicates that the shear stress held constant (see the cluster of data at 22 gpm). As previously noted, erosion initiated with the increase in flow to 28 gpm. At this point the data is plotted with a different color for every day of the test, from cooler colors (pink, blue, purple, etc.) to warmer colors (yellow, orange, red, etc.). As can be seen in the plot, the data at 28 gpm migrated with decreasing shear stress as the flow held relatively steady or slightly increased. Because the flow in the crack remains a constant function of the differential head and shear stress, the difference between the flow at the end of erosion and the flow on the non-erosion curve directly below the end of flow, ΔQ, is attributed to flow in the erosion void.

When the flow is increased to 29 gpm a similar progression of erosion is denoted with the color progression, albeit at a faster erosion rate, and a similar ΔQ can be measured for the larger erosion void.
Fig. 58. Flow Volume and Differential Head Versus Elapsed Time for Sand/Clay (75/25)(a)
Fig. 59. Flow Volume and Transmissivity Versus Elapsed Time for Sand/Clay (75/25)(a)
Fig. 60. Flow Volume Versus Shear Stress for Sand/Clay (75/25)(a)
The red vertical line on the plot is the critical shear stress, 1.3 psf (62 Pa), measured in a Hole Erosion Test performed at USU (Erickson 2013). The critical shear stress found from the data in Fig. 60 is approximately 2.9 psf (140 Pa).

**Test 3: Sand/Clay (75/25)(b)**

Test 3 was performed on a sample consisting of 75% Garnet Sand and 25% Helmer Clay (by weight). The aperture of the crack was set to 3 mm for this test. This test was run successfully but no light sensor information is available for this test due to instrument error. The soil was compacted to a density of approximately 146 lbs/ft^3 (total unit weight) at a water content of 14% (wet of optimum). The resulting relative density based on a modified proctor was about 91% (modified proctor curve in Appendix A). This test ran for 14 days and contact erosion was evident in this experiment as shown in the pictures taken when taking apart the laboratory test. Figs. 61, 62, 63, 64 and 65, show the pipe formed at each sensor or piezometer location moving downstream to upstream. It is interesting to note that while Tests 2 and 3 were run on samples with the exact same sand/clay compositions, the behaviors vary somewhat. The difference in behavior appears to be due to a slight difference in water content and test variability.

Figs. 61 to 65, show the pipe is largest in the middle of the apparatus and smallest near the inlet of the apparatus. The pipe appeared to be working from the outlet towards the inlet. As shown in Fig. 64, the pipe went from large to small in a short distance indicating the pipe may have been working like a progressive knick point as the enlarging process worked its way upstream. If the test had run for more time the larger sized pipe may have worked its way back to the inlet sandbag.

It is interesting that while the same sand/clay proportions were used in Tests 2 and 3, Test 2 had less removal of soil particles and more softening of the material. This is likely related
Fig. 61. Picture of Contact Erosion at Sensors 7 & 8 and LED 4 for Sand/Clay (75/25)(b)

Fig. 62. Picture of Contact Erosion at Sensors 5 & 6 and LED 3 for Sand/Clay (75/25)(b)
Fig. 63. Picture of Contact Erosion at Sensors 3 & 4 and LED 2 for Sand/Clay (75/25)(b)

Fig. 64. Picture of Contact Erosion at Piezometer 2 for Sand/Clay (75/25)(b)
Fig. 65. Picture of Contact Erosion at Sensors 1 & 2 and LED 1 for Sand/Clay (75/25)(b)

to the previous test material being drier. The compaction water content of the soil may have
affected the propensity of the soil to absorb water and soften upon wetting and may have
effected how the particle eroded in the pipe. Higher water content, in this case, created a
cleaner pipe as seen in the figures of this laboratory test. The velocities and pressures to initiate
erosion were similar in both tests.

There is no light sensor data for this test, but the piezometer and flow data will be used
to analyze the data. Additionally, piezometer 3 did not function throughout data collection and
is therefore not included in the plots. The flow and piezometers versus elapsed time graphs are
included in this section. Though not as clear as in previous tests, the data output shows signs of
erosion.

In Fig. 66, the piezometers show erratic movement during the first few days of the test.
However, because the piezometers stabilize with little change and because erosion is not noted
in the next flow increase, this erratic behavior is not thought to be indicative of contact erosion.
When the flow is increased to 26 gpm, the piezometers remain relatively constant and thus
significant erosion is not believed to have occurred at this flow. The first strong signs of erosion
occur at around day 6 when the flow is increased to about 32 gpm. It is believed that the erosion began causing rapidly, wild fluctuations of the piezometer readings. This is followed by a rapid increase in pressure in piezometer 4, bringing it essentially equal to the pressure in piezometer 2. This indicates that erosion occurred rapidly between piezometers 2 and 4 in the center of the apparatus. This is consistent with the observations at the end of the test as presented in Figs. 61 to 65. Following the rapid erosion, the piezometers stabilized, indicating the erosion had essentially ceased.

Flow volume versus shear stress, calculated using Equation 5.5, is plotted in Fig. 67. This figure is similar to Fig. 60, which was previously described. At first glance this figure would suggest that erosion is occurring when the flow is at 22 gpm. However, the scatter in the colored region of this plot is indicative of the erratic behavior previously discussed and the shear stress actually increases toward the end of this flow level. The behavior at 26 gpm is similar (without the erratic behavior), showing an increase in shear stress over the flow level. This increased stress may be due to clogging of the crack. Finally, the plot shows erosion occurring when the flow is increased to about 32 gpm. Most of this erosion occurred during the first day followed again by increasing shear stress. The critical shear stress found from the data in Fig. 67 is approximately 2.7 psf (129 Pa).

**Test 4: Sand/Clay (70/30)**

Test 4 is performed on soil consisting of 70% Garnet Sand combined with 30% Helmer Clay (Kaolinite) (by weight). The aperture of the crack was set to 3 mm for this test. The soil was compacted to a total density of about 142 lb/ft³ at a water content of 15% (wet of optimum). The relative density of the test, based on a modified proctor, was about 88% (modified proctor curve in Appendix A). This test ran for a total of 14 days with very little to no
Fig. 66. Piezometer and Flow Volume Versus Elapsed Time for Sand/Clay (75/25)(b)

*Note: There is no data for Piezometer (3) for this experiment.
Fig. 67. Flow Volume Versus Shear Stress for Sand/Clay (75/25)(b)
erosion of the test soil occurring.

Fig. 68 shows the amount of erosion at light sensors 7 and 8 and LED 4. From all documented pictures of this test, this was the area where the most erosion occurred. It is assumed that the test could have run for a significant amount of time at higher gradients to achieve recordable erosion rates for this soil type.

![Picture of Contact Erosion at Sensors 7 & 8 and LED 4 for Sand/Clay (70/30)](image)

**Fig. 68.** Picture of Contact Erosion at Sensors 7 & 8 and LED 4 for Sand/Clay (70/30)

Fig. 69 presents the light sensors and flow volume versus elapsed time graph for Test 4. The sensors stayed at zero, indicating that erosion did not erode to the point where light from the LED could reach the light sensors. The flow for the test only increased when the flow for the system was increased.

Fig. 70 presents the piezometer readings versus elapsed time. There are no significant fluctuations or indicators of erosion occurring from the piezometer readings. Piezometer 3 was not reading correctly for most of the test (likely due to being clogged). The final increase of flow marked the proper function of piezometer 3. Therefore, our data analysis confirms that no erosion occurred in the system.
Fig. 69. Light Sensor Output and Flow Volume Versus Elapsed Time for Sand/Clay (70/30)
Fig. 70. Piezometers Versus Elapsed Time for Sand/Clay (70/30)

Piezometer (3) likely blocked during these readings.
Fig. 71 presents flow volume versus shear stress and is shown primarily for comparison with plots form previous tests (Figs. 48, 60 and 67). Note that no changes in shear stress occurred at the various flow stages. This indicates the flow was largely consistent with flow through the crack reacting to changes in differential pressure and that no significant erosion occurred.

5.4 Sand Mixed with Bentonite Results

Test 5: Sand/Bentonite (87.5/12.5)(a)

Test 5 was performed on a soil consisting of a mixture of 87.5% Garnet Sand mixed with 12.5% Bentonite (by weight). The aperture of the crack was set to 3 mm for this test. The compacted total weight of the soil was about 137 lb/ft$^3$ at a water content of 12% (slightly wet of optimum). The relative density of the test, based on a modified proctor, was about 87% (modified proctor curve in Appendix A). This test ran for just under one day, in which the soil eroded out very quickly.

Fig. 72 shows the amount erosion that occurred in the 3.50 in. deep soil layer (The top of Fig. 73 is the upstream side and the bottom of the figure is the downstream side). As can be seen, the sand mixed with bentonite layer nearly completely eroded out of the apparatus in less than 1 day.

Fig. 73 is a plot of the light sensors and flow volume versus elapsed time. The plot indicates that the erosion progressed fastest near Sensors 7 and 8, which is near the outlet. The erosion then propagated back towards the inlet in sporadic fashion until it formed a pipe all the way along the crack length. While the initial readings from Sensors 7 and 8 were steady, they soon became erratic, likely indicating the collapse of the pipe walls and the start of sloughing. In the latter half of the test, the other sensors activated and also produced erratic behavior. With
Fig. 71. Flow Volume Versus Shear Stress for Sand/Clay (70/30)
time some sensors decreased in strength and shut off indicating the collapse of the coarse
grained layer as the soil had completely eroded.

Fig. 74 presents a plot of the piezometer readings versus elapsed time. The largest
change early in the test is in piezometer 4, which corresponds closely with sensors 7 and 8. The
pressure readings are relatively stable until the time, where Fig. 73 indicated, the erosion was
spreading more rapidly. At this point there is a large jump in piezometers 3 and 4, followed
Fig. 73. Light Sensor Output and Flow Volume Versus Elapsed Time for Sand/Bentonite (87.5/12.5)(a)
Fig. 74. Water Pressure Versus Elapsed Time for Sand/Bentonite (87.5/12.5)(a)
shortly after with erratic readings. These readings are consistent with the interpretation of Fig. 73, presented above. The correlation of the different types of sensors helps to support the interpretations being made in other tests.

Fig. 75 is a plot of flow volume versus the shear stress. Similar to previous plots, the black line represents the flow versus shear stress values as the pressure is being applied to the test. The green line represents the time when erosion is occurring. Note that the green line first moves to the left as the pipe forms (less flow resistance) and then moves to the right as the soils collapse (more flow resistance).

**Test 6: Sand/Bentonite (87.5/12.5)(b)**

Test 6 was run on a sample with the same soil combination as Test 5, consisting of 87.5% Garnet Sand mixed with 12.5% Bentonite (by weight). The aperture of the crack was set to 3 mm for this test. The soil was compacted to a total density of 133 lb/ft$^3$ at a water content of 15% (wet of optimum). The relative density of the test, based on a modified proctor, was about 85% (modified proctor curve in Appendix A). This test ran for just under 21 days and behaved considerably different from Test 5. Also of note was that the flow was shut off twice during the test, resulting in three cycles of flow loading.

The soil in this test had enough cohesion to hold a pipe for the duration of the test and only moderate amounts of erosion occurred. Figs. 76, 77, 78 and 79, are pictures of piping at each sensor location moving from downstream to upstream. These photos show that a small pipe (up to 3/4-inch diameter) formed along most of the crack with slightly more erosion occurring closer to the downstream portion of the apparatus.

Fig. 80 presents the light sensor output and flow volume versus elapsed time graph and shows the three separate cycles of hydraulic loading. Fig. 80 indicates that only one sensor (sensor 3) indicated the propagation of erosion. This is consistent with Fig. 78, where a wider
Fig. 75. Flow Volume Versus Shear Stress for Sand/Bentonite (87.5/12.5)(a)
Fig. 76. Picture of Contact Erosion at Sensors 7 & 8 and LED 4 for Sand/Bentonite (87.5/12.5)(b)

Fig. 77. Picture of Contact Erosion at Sensors 5 & 6 and LED 3 for Sand/Bentonite (87.5/12.5)(b)
Fig. 78. Picture of Contact Erosion at Sensors 3 & 4 and LED 2 for Sand/Bentonite (87.5/12.5)(b)

Fig. 79. Picture of Contact Erosion at Sensors 1 & 2 and LED 1 for Sand/Bentonite (87.5/12.5)(b)
Fig. 80. Light Sensor Output and Flow Volume Versus Elapsed Time for Sand/Bentonite (87.5/12.5)(b)
pipe developed. Erosion did not initiate until the highest level of flow was reached at around 34 gpm. At this point, the general trend of the flow starts to slowly but steadily increase as highlighted in Fig. 81. The increase in flow indicates that the area where the water is flowing through is increasing by eroding a pipe through the soil.

Fig. 82 presents the piezometers versus elapsed time plot and provides further evidence to the previous interpretation of the flow and the light sensor data. Fig. 82 shows a very slight decrease in the inlet pressure (piezometer 1) and piezometer 2, immediately after the initiation of erosion, while piezometers 3 and 4 slightly increase. The outlet pressure, piezometer 5, also increases slightly with the initiation of erosion. The behavior of the piezometers is consistent with the greatest amount of erosion occurring between piezometers 2 and 3, thus corroborating the highest rate of erosion occurring at sensor 3.

Fig. 83 presents flow volume and differential head plotted versus elapsed time. This plot illustrates how, once erosion has initiated, the flow begins to increase slightly as the differential pressure starts to drop as erosion propagates. Similarly, Fig. 84 correlates flow and transmissivity to elapsed time. The flow volume and transmissivity on Fig 84, diverge until the initiation of erosion after which they converge. Both of these plots corroborate the point of erosion initiation identified in the previous plots.

Fig. 85 plots flow volume versus shear stress, calculated using Equation 5.5. The black and gray portions of the graph are the non-erosive portions of the test (divided into the three cycles) and the varied color portion is the erosive portion of the test. The erosion works its way to the left on the graph and slightly upward. The movement to the left is influenced by the drop in differential pressure and the upward movement is due to slightly increasing flow. Contrary to what was observed in Tests 1, 3 and 5, the erosion progressed only to the right and not to the left, indicating the pipe was still enlarging at the end of the test and had not started to collapse.
The critical shear stress found from this graph is approximately 3.9 psf (188 Pa).

While Fig. 85 presents flow volume versus shear stress for the entire apparatus, Fig. 86 plots the same quantities for the just the portion of piezometers 2 and 3 of the test. Again, the erosion works its way to the left on the graph and slightly upward. However, the erosion in this plot represents a much larger portion of the shear stress, thus the erosion channel is accounting for a much larger percentage of the flow in this region.

The erosive behavior is much more pronounced in this area of the crack, than it was across the entire crack length. This supports the evidence of erosion occurring at light sensor 3 which falls into this segment of the crack. As shown, the erosion progresses slowly at first, but begins to progress more rapidly as time goes on. The critical shear stress found from this graph is approximately 1.1 psf (51 Pa), which is significantly less than the 3.9 psf (188 Pa) for the whole
Fig. 82. Water Pressure Versus Elapsed Time for Sand/Bentonite (87.5/12.5)(b)
Fig. 83. Flow Volume and Differential Head Versus Elapsed Time for Sand/Bentonite (87.5/12.5)(b)
Fig. 84. Flow Volume and Transmissivity Versus Elapsed Time for Sand/Bentonite (87.5/12.5)(b)
Fig. 85. Flow Volume Versus Shear Stress from Inlet to Outlet for Sand/Bentonite (87.5/12.5)(b)
Fig. 86. Flow Volume Versus Shear Stress from Sensors 2 to 3 for Sand/Bentonite (87.5/12.5)(b)
length of the crack.

**Test 7: Sand/Bentonite (85/15)**

Test 7 was performed on soil consisting of 85% Garnet Sand combine with 15% Bentonite (by weight). The aperture of the crack was set to 3 mm for this test. The total unit weight of the soil was about 145 lb/ft³ at a water content of 16% (wet of optimum). The relative density of the test, based on a modified proctor, was about 95% (modified proctor curve in Appendix A). This test ran for a total of 18 days and there was very little to no erosion of the test soil.

Fig. 87 shows the amount of erosion at light sensors 7 & 8 and LED 4. As can be seen, no significant erosion occurred. There was a little bit of softening of the material just above the crack, but almost no visual erosion was apparent all along the length of the crack.

![Contact Erosion at Sensors 7 & 8 and LED 4 for Sand/Bentonite (85/15)](image)

**Fig. 87. Picture of Contact Erosion at Sensors 7 & 8 and LED 4 for Sand/Bentonite (85/15)**

Fig. 88 is the light sensors and flow volume versus elapsed time graph for Test 7. The sensors stayed at zero and did not read indicating that little to no erosion occurred. The flow for the test only increased when the flow for the system was increased. This is different from the
Fig. 88. Light Sensor Output and Flow Volume Versus Elapsed Time for Sand/Bentonite (85/15)
tests that eroded. As shown on the light sensor and flow plot, the flow drops down to zero on
day six. This is due to planned cyclic loading.

Fig. 89 is the piezometers versus elapsed time graph. There are no significant
fluctuations or indicators of erosion occurring in the apparatus. Therefore, our data analysis
confirms that no erosion occurred in the system.

Fig. 90 presents the flow versus shear stress graph. The darker curve represented the
first cycle run in the test and the lighter curve represents the second cycle run in the test. This
plot confirms that no significant erosion occurred in this test. The values progress toward the
right throughout the two curves and there are no decreases in shear stress coupled with slight
increases in flow. This figure is presented primarily for comparison with similar figures for the
other erosive tests (Figs. 75, 85 and 86).

5.5 Teton Core Material Results

Test 8: Teton Core

Test 8 was performed on material from the core of Teton Dam, in Idaho, which failed
due to internal erosion in 1976. The aperture of the crack was set to 3 mm for this test. The
compaction total unit weight of the soil was about 117 lb/ft$^3$ at a water content of 14%
(optimum water content). The relative density of the test, based on a modified proctor, was
about 91% (modified proctor curve in Appendix A). This test ran for a total of 22 days with very
little erosion occurring.

Fig. 91 shows the amount of erosion at light sensors 7 & 8 and LED 4. The small pipe
shown is partly due to probing. From all documented pictures of this test, this was the area
where the most erosion occurred.
Fig. 89. Water Pressure Versus Elapsed Time Sand/Bentonite (85/15)
Fig. 90. Flow Versus Shear Stress for Sand/Bentonite (85/15)
Fig. 91 Picture of Contact Erosion at Sensors 7 & 8 and LED 4 for Teton Core

Fig. 92 presents a plot of the light sensors and flow volume versus elapsed time for Test 8. The sensors stayed at zero, indicating that erosion did not occur to the point where light from the LED could reach the light sensors. The flow for the test only increased when the flow for the system was increased. The drop in flow, just before day six, was incidental and did not correlate to any erosion. This drop in flow can also be seen as a drop in pressure in the piezometers in Fig. 93.

Fig. 93 presents the piezometers versus elapsed time. There are no significant fluctuations or indicators of erosion occurring in the apparatus. As mentioned before a slight decrease in pressure happened just before day six that was not linked to erosion.

Fig. 94 presents a plot of flow volume versus shear stress. Similar to Test 7, the curve progresses toward the right throughout the whole curve, indicating that no significant erosion occurred in this test.
5.6 East Branch Core Material Results

**Test 9: East Branch Core Material**

Test 9 was performed on material from the embankment of East Branch Dam in Pennsylvania. The soil is a sandy clay with gravel of low to moderate plasticity (LL= 22, PI=4). The aperture of the crack was set to 1.5 mm for this test. The soil was compacted to a total density of the soil was approximately 138 lb/ft$^3$ at a water content of 9.5% (optimum water content). The relative density of the test, based on a modified proctor, was about 95% (modified proctor curve in Appendix A). This test ran for just under 47 days.

This test had enough cohesion to hold a pipe for the duration of the test. Fig. 95 shows a sinkhole that formed towards the upstream end of the apparatus that is believed to have collapsed when the water pressure was turned off (left side of Fig. 95 is the upstream side and the right side of the figure is the downstream side). A slight amount of softening of the test soil was encountered near the upstream end of the apparatus (near piezometer 2), as depicted in Fig. 95. The erosion appeared to occur closer to the upstream portion of the apparatus. Figs. 96, 97, 98, 99 and 100 are pictures of erosion at each sensor location moving from downstream to upstream.

Fig. 101 plots the light sensor output and flow volume versus elapsed time and Fig. 102 presents the piezometer readings versus time plot. It should be noted that the LED for sensors 3 and 4 failed during the test and thus, there are no readings for these sensors through the duration of the test. While erosion likely initiated when the flow was increased to the highest level of about 30 gpm, the first real indicators that erosion is occurring show up at about day 8 of the test with an increase in pressure in piezometer 2 and sensor 3 detecting erosion. At day 3, there is a slight increase in the flow coupled with a rapid increase in pressure in piezometer 4.
Fig. 92. Light Sensor Output and Flow Volume Versus Elapsed Time for Teton Core
Fig. 93. Water Pressure Versus Elapsed Time for Teton Core
Fig. 94. Flow Volume Versus Shear Stress for Teton Core
**Fig. 95.** Picture of Top View of Contact Erosion for East Branch Core

**Fig. 96.** Picture of Contact Erosion at Sensors 7 & 8 and LED 4 for East Branch Core
Fig. 97. Picture of Contact Erosion at Sensors 5 & 6 and LED 3 for East Branch Core

Fig. 98. Picture of Contact Erosion at Sensors 3 & 4 and LED 2 for East Branch Core
**Fig. 99.** Picture of Contact Erosion at Piezometer 2 for East Branch Core

**Fig. 100.** Picture of Contact Erosion at Sensors 1 & 2 and LED 1 for East Branch Core
Fig. 101. Light Sensor Output and Flow Volume Versus Elapsed Time for East Branch Core
Fig. 102. Water Pressure Versus Elapsed Time East Branch Core
The flow does not increase with time as it does in the other tests. This could be due to the narrowing of the crack, which increases the water pressure. Fig. 103 presents details of the flow volume and light sensor data over the last 35 days of the test.

Fig. 104 is a graph of the flow over the last 35 days. As alluded to earlier, the flow decreases after the initial increase of flow. This is most likely due to the narrow aperture used in the test and the slight amount of erosion that occurred through the duration of the test. This test acted differently from other tests performed in this study.

Figs. 105 and 106 provide graphic illustrations of the effects of erosion on the pore pressure date. Fig. 105 presents flow volume and differential head plotted versus elapsed time. The flow and the differential head show opposite type behavior and the divergence of the two lines make it easy to see the point of erosion initiation. Fig. 106 correlates flow volume and transmissivity to elapsed time. Except for the beginning of the test, the transmissivity is close to the reverse trend of the flow. This correlates with other data and confirms the slight erosion that occurs.

Fig. 107 plots flow volume versus shear stress, calculated using Equation 5.5. The darker portion of the graph is the non-erosive portion of the test and the varied color portion is the erosive portion of the test. The plot shows a small amount of erosion occurring. The critical shear stress found from this graph is approximately 3.9 psf (188 Pa).

Fig. 108 is similar to Fig 107, but the calculated shear stress is based on the distance from piezometer 2 to piezometer 3. The erosive behavior is much more pronounced in this area of the crack than it was across the entire crack length of the crack. The erosion propagates rapidly at first and then slows considerably as time goes on. The critical shear stress found from this graph is approximately 3.2 psf (150 Pa), which is less than the 3.9 psf (188 Pa) calculated for the whole length of the crack.
Fig. 103. Details of Light Sensor Output and Flow Volume Versus Elapsed Time for East Branch Core
5.7 Summary of Testing Results

Table 1 presents a compilation of the results of the testing and analysis. The median grain size is the $D_{50}$ of the soil. The effective grain size is calculated using equation 5.7 that was originally presented by Guidoux et al. (2010) (ICOLD 2014). The effective grain size was thought, by Guidoux et al. (2010), to be more representative of the soil’s erosive behavior for fine grained soils. Reynolds numbers were calculated using Equation 5.1 and assessed by the criteria for a pipe, where flows are considered turbulent when the Reynolds number exceeds 4000. The critical velocity reported is calculated using Equation 5.2 and represents the average velocity in the crack at the time of erosion initiation. Note that because the flow in the tests is increased incrementally, the critical velocities may be overestimated or more than the minimum velocity at which erosion may occur. Finally, the critical shear stresses taken from the plots of shear
Fig. 105. Flow Volume and Differential Head Versus Elapsed Time for East Branch Core
Fig. 106. Flow Volume and Transmissivity Versus Elapsed Time for East Branch Core
Fig. 107. Flow Volume Versus Shear Stress from Inlet to Outlet for East Branch Core
Fig. 108. Flow Volume Versus Shear Stress from Sensors 2 to 3 for East Branch Core
stress (calculated using either Equation 5.3 or 5.4) versus flow volume are presented. The first column, at erosion initiation, represents the calculated shear stress when erosion is first noted. The second column, at end of erosion, represents the calculated shear stress at the end of erosion. The second value for critical shear stress is lower due to the reduction in shear stress that results from the larger flow area as a result of erosion. The equation for the second value for critical shear stress is presented below:

\[ d_H = \left( \sum_{j=1}^{m} \frac{F_j}{d_j} \right)^{-1} \]  \hspace{1cm} (5.7)

where, \( F_j \) (\%) is the percentage of the diameter and \( d_j \) (mm) is the diameter on the gradation curve of the soil.

The final column in Table 1 shows the results of Hole Erosion Tests (HETs) performed in conjunction with this study to provide a comparison of the internal erosion testing apparatus and another test directly measuring critical shear stress. A report on the HET testing program is included as Appendix B of this report.

Several observations should be noted from Table 1. First, the comparison with the HET results in a very loose correlation. This is not completely unanticipated in that there are a number of factors that could result in variation of the critical shear stress including: perturbations and variation in the flow path, minor local variation in the soil density and structure, and the length of test (HET tests are typically performed in 1 to 2 hours). Also of interest, is the variation between tests performed on samples of identical soil makeup (Tests 2 and 3, and Tests 5 and 6). These results suggest that the water content during compaction, rather than the density, plays a large role in the erodibility of the soil. Also worth noting is the variation in results in Test 6 when the shear stress is calculated between piezometers 2 and 3,
versus over the entire apparatus. This variation is believed to be the result of the erosion being concentrated between piezometers 2 and 3 and the flow being controlled by regions where little or no erosion has occurred.

The primary factor affecting the erosive behavior of soil was found to be the composition of the soil, that is, the type of fines in the soil and the percentage of fines. However, along with the composition of the soil, the compaction water content appears to affect the behavior. In Tests 2 and 3 the soil consisted of the same mixture of sand and kaolinite (75/25 ratio) and in Tests 5 and 6 the soil consisted of the same mixture of sand and bentonite (87.5/12.5 ratio). The only difference was in Tests 2 and 5 the soil was compacted near the optimum water content while in Tests 3 and 6 the soil was compacted 2 and 3 points above the optimum water content, respectively. For Tests 2 and 3, erosion was initiated at similar critical shear stresses yet the propagation of the erosion occurred differently. The soil in Test 2 (the drier soil) softened around the pipe causing collapse, where the soil in Test 3 formed a stable pipe through stiff soils. In Tests 5 and 6, the difference in behavior was even more extreme with erosion in the dryer soil of Test 5 initiating at a much lower shear stress and propagating much more rapidly than the sample in Test 6 that was compacted wet of optimum. While it cannot be proven with certainty why these samples behaved differently, it is likely that the soils compacted wet of optimum have a more stable structure of the fine grained portion of the sample and are less susceptible to softening upon wetting; thus the greater resistance to erosion.

Another interesting observation is that in nearly all the tests it appeared that the erosion initiated at one location along the crack and then propagated along the crack from the initiation point. This was evidenced by the piezometers, the light sensors, and the observations at the end of the tests. It is not known whether this initiation was caused by a point in the soil
## Table 1. Critical Values from Contact Erosion Tests

<table>
<thead>
<tr>
<th>Test Number and Material</th>
<th>Median Grain Size, $D_{50}$ (mm)</th>
<th>Effective Grain Size, $D_{H}$ (mm)</th>
<th>Crack Aperture (mm)</th>
<th>Reynolds Number*, Re (Unitless)</th>
<th>Critical Velocity, $V_c$ (m/s)</th>
<th>Critical Shear Stress, $\tau_c$ (Pa)</th>
<th>Critical Shear Stress from Hole Erosion Tests (Pa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 1 - Sand/Clay (80/20)</td>
<td>0.12</td>
<td>0.014</td>
<td>3</td>
<td>12000</td>
<td>3.1</td>
<td>65</td>
<td>54</td>
</tr>
<tr>
<td>Test 2 - Sand/Clay (75/25)(a)</td>
<td>0.12</td>
<td>0.011</td>
<td>3</td>
<td>15000</td>
<td>3.2</td>
<td>147</td>
<td>132</td>
</tr>
<tr>
<td>Test 3 - Sand/Clay (75/25)(b)</td>
<td>0.12</td>
<td>0.011</td>
<td>3</td>
<td>17000</td>
<td>4.4</td>
<td>142</td>
<td>128</td>
</tr>
<tr>
<td>Test 4 - Sand/Clay (70/30)</td>
<td>0.12</td>
<td>0.0098</td>
<td>3</td>
<td>15000</td>
<td>3.8**</td>
<td>113**</td>
<td>113**</td>
</tr>
<tr>
<td>Test 5 - Sand/Bentonite (87.5/12.5)(a)</td>
<td>0.12</td>
<td>0.0024</td>
<td>3</td>
<td>12000</td>
<td>2.8</td>
<td>92</td>
<td>70</td>
</tr>
<tr>
<td>Test 6 - Sand/Bentonite (87.5/12.5)(b) (full test)</td>
<td>0.12</td>
<td>0.0024</td>
<td>3</td>
<td>18000</td>
<td>4.6</td>
<td>210</td>
<td>190</td>
</tr>
<tr>
<td>Test 6 - Sand/Bentonite (87.5/12.5)(b) (between Piezometers 2 and 3)</td>
<td>0.12</td>
<td>0.0024</td>
<td>3</td>
<td>18000</td>
<td>4.6</td>
<td>174</td>
<td>52</td>
</tr>
<tr>
<td>Test 7 - Sand/Bentonite (85/15)</td>
<td>0.12</td>
<td>0.0020</td>
<td>3</td>
<td>15000</td>
<td>3.8**</td>
<td>160**</td>
<td>160**</td>
</tr>
<tr>
<td>Test 8 - Teton Core</td>
<td>0.033</td>
<td>0.033</td>
<td>3</td>
<td>19000</td>
<td>4.8**</td>
<td>225**</td>
<td>225**</td>
</tr>
<tr>
<td>Test 9 - East Branch Core</td>
<td>0.1</td>
<td>0.1</td>
<td>1.5</td>
<td>16000</td>
<td>8.2</td>
<td>205</td>
<td>190</td>
</tr>
</tbody>
</table>

* A Reynolds number above 4000 is considered turbulent flow in this configuration; therefore, all flows in the tests performed appear to be turbulent.

** These were tests that had minimal or no erosion occur. For these tests the velocity reported is the maximum velocity reached and the shear stress is the maximum hydraulic shear stress reached through the test. These values will not be plotted on graphs for comparison.
that had a greater susceptibility to erosion or if there was a concentration or deflection of flow in one location. In either case, this phenomenon represents an added difficulty in predicting when this type of erosion is expected to occur since minor perturbations in either the soil makeup or the flow regime can cause the initiation of erosion to occur at lower average flows than would be necessary given uniform flow conditions.

5.8 Comparison of Test Results to Other Research

This section compares the calculated results from Table 1 to published results from other research (Briaud 2008, ICOLD 2012).

Comparison to Briaud (2008)

The first comparison will be with charts relating critical velocity and critical shear stress developed by Briaud’s (2008). This work was previously discussed in Chapter 2 (the Literature Review). The critical velocities and critical shear stresses presented in these charts were measured for water flow across the top of a soil sample.

The comparisons with Briaud’s charts were made using both the median grain size \(D_{50}\) and the Effective Grain size (calculated using Equation 5.7) of each soil (see Table 1). In the case of the manufactured soils, the median grain size was equal to the grain size of the sand portion since the clay portions were all less than 50 percent of the mixtures. The median grained size for the natural soils was calculated from sieve analyses. The fact that the median grain size for all of the manufactured soils end up being the same shows the limitation of using the median grain size. Therefore, the median grain size is a poor predicting method for erosive behavior in that very different behaviors were observed in the tests run on these soils. A better indicator of erosive behavior is the effective diameter of the soil calculated by Equation 5.7. Both the mean and effective diameters for the soils tested are plotted on Figures 109 through 120.
Figs. 109, 110, 111, 112, 113 and 114, are Briaud’s plots of critical velocity versus mean grain size with the results of this study plotted with respect to both mean and effective grain size. Figs. 115, 116, 117, 118, 119 and 120, are Briaud’s plots of critical shear stress versus mean grain size with the results of this study plotted with respect to both mean and effective grain size.

The results from the tests performed in this study appear to correlate well with Briaud’s critical velocity versus mean grain size plots when they are plotted using the effective grain size calculated using Equation 5.7. When plotted using the median grain size, however, many of the test results plot outside of the range of values presented by Briaud. The same observation can be made for the critical shear stress versus median grain size comparison.

One exception to the observations from the comparison to Briaud’s plots is the behavior of the East Branch Dam material. In this soil, the median and effective grain sizes are very similar and both plot outside of the bounds of Briaud’s data. This is likely due to the oversized material in the soil skewing both the median and effective grain sizes in the coarse direction. Thus, it can be concluded, while the Briaud charts match well with our test results, there are soils that are exceptions and are not well represented by either the mean or effective grain size.
Fig. 109. Comparison of Sand/Clay (80/20) Results to Modified Briaud’s Critical Velocity Versus Grain Size (Briaud 2008)
Fig. 110. Comparison of Sand/Clay (75/25)(a) Results to Modified Briaud’s Critical Velocity Versus Grain Size (Briaud 2008)

- Test Results from Sand/Clay (75/25)(a) using mean diameter
- Test Results from Sand/Clay (75/25)(a) using effective diameter
Fig. 111. Comparison of Sand/Clay (75/25)(b) Results to Modified Briaud’s Critical Velocity Versus Grain Size (Briaud 2008)
Fig. 112. Comparison of Sand/Bentonite (87.5/12.5)(a) Results to Modified Briaud’s Critical Velocity Versus Grain Size (Briaud 2008)

- Test Results from Sand/Bentonite (87.5/12.5)(a) using mean diameter
- Test Results from Sand/Bentonite (87.5/12.5)(a) using effective diameter
Fig. 113. Comparison of Sand/Bentonite (87.5/12.5)(b) Results to Modified Briaud’s Critical Velocity Versus Grain Size (Briaud 2008)

- Test Results from Sand/Bentonite (87.5/12.5)(b) using mean diameter
- Test Results from Sand/Bentonite (87.5/12.5)(b) using effective diameter
Fig. 114. Comparison of East Branch Core Results to Modified Briaud’s Critical Velocity Versus Grain Size (Briaud 2008)

- Test Results from East Branch Core Material using mean diameter
- Test Results from East Branch Core Material using effective diameter
Fig. 115. Comparison of Sand/Clay (80/20) Results to Modified Briaud’s Critical Shear Stress Versus Grain Size (Briaud 2008)
Fig. 116. Comparison of Sand/Clay (75/25)(a) Results to Modified Briaud’s Critical Shear Stress Versus Grain Size (Briaud 2008)
Fig. 117. Comparison of Sand/Clay (75/25)(b) Results to Modified Briaud’s Critical Shear Stress Versus Grain Size (Briaud 2008)
Fig. 118. Comparison of Sand/Bentonite (87.5/12.5)(a) Results to Modified Briaud’s Critical Shear Stress Versus Grain Size (Briaud 2008)

- ☆ Test Results from Sand/Bentonite (87.5/12.5)(a) using mean diameter
- ◊ Test Results from Sand/Bentonite (87.5/12.5)(a) using effective diameter
- △ Test Results from HET using effective diameter (Erickson 2013)
Fig. 119. Comparison of Sand/Bentonite (87.5/12.5)(b) Results to Modified Briaud’s Critical Shear Stress Versus Grain Size (Briaud 2008)

Legend:
☆ Test Results from Sand/Bentonite (87.5/12.5)(b) using mean diameter
◇ Test Results from Sand/Bentonite (87.5/12.5)(b) using effective diameter
△ Test Results from HET using effective diameter (Erickson 2013)
Fig. 120. Comparison of East Branch Core Results to Modified Briaud’s Critical Shear Stress Versus Grain Size (Briaud 2008)

- Test Results from East Branch Core Material using mean diameter
- Test Results from East Branch Core Material using effective diameter
- Test Results from HET using effective diameter (Erickson 2013)
Comparison to Guidoux et al. 2010

Fig. 121 presents a plot of critical Darcy velocity verses effective grain size from experimental work performed by Guidoux et al. (Guidoux et al. 2010, ICOLD 2012). The test results presented in Fig. 121, are from tests performed with erodible soils in contact with coarse sands and gravels. The first thing to note on this plot is that the critical velocity results presented in Table 1 are well above the Guidoux results, and in most cases, do not even plot on the figure. One should also note that the critical velocity values for cohesive materials presented by Guidoux are about an order of magnitude lower than those presented by Briaud (2008).

Review of Guidoux et al. (2010), suggests two reasons for the discrepancy between the Guidoux plot and the Briaud plots and the data from this study. First, the soils tested by Guidoux and others are highly erosive, having very fine contents on the order of 10 percent. Secondly, the flow through gravel tends to deflect the flow into the eroding soil in some locations. The flow deflected at the soil will be much more erosive than flow that is parallel to the soil surface. The flow conditions in the tests from this study are more likely to be consistent with the Briaud flow conditions; a conclusion that is supported by the data.
Fig. 121. Comparison of IEA Results to Modified Guidoux’s Critical Darcy Velocity Versus Effective Diameter (ICOLD 2012)
A laboratory apparatus was designed and constructed to assess the erosion potential of various soils due to water flow through an adjacent crack. The device was designed to model water flow through joints in bedrock beneath a dam that is in contact with the dam embankment or core. Tests using the apparatus were performed on a variety of manufactured and native soils to assess the critical conditions needed to initiate erosion and rate at which erosion is expected to occur.

The testing performed in the Internal Erosion Apparatus (IEA) produced a variety of results depending on the soil properties and preparation techniques. A summary of calculated critical velocity and critical shear stress values were previously presented in Table 1 and a summary of the observed erosion behavior along with details of the test soil preparation is presented in Table 2. Test results varied from very rapid erosion, where complete collapse of the erosion void occurred in one or two days, to very slow erosion were the tests were run for as long as 47 days with minimal amounts of erosion.

The primary factor affecting the erosive behavior of soil was found to be the composition of the soil, that is, the type of fines in the soil and the percentage of fines. However, along with the composition of the soil, the compaction water content appears to affect the behavior. Tests performed on soils consisting of the same mixture of sand and clay appeared to be notably more resistant to erosion initiation when compacted at water contents greater than the optimum water content. An even greater difference was noticed with the propagation of erosion, where the soils compacted at lower water contents tended to erode faster and also experienced softening of the soil adjacent to the eroding void. Such softening
<table>
<thead>
<tr>
<th>Test Number and Material</th>
<th>Unit Weight (pcf)</th>
<th>Water Content (%)</th>
<th>Relative Compaction (%)*</th>
<th>Crack Aperture (mm)</th>
<th>Test Duration (days)</th>
<th>Observed Erosion Behavior</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 1 - Sand/Clay (80/20)</td>
<td>143</td>
<td>13</td>
<td>89</td>
<td>3</td>
<td>2</td>
<td>Completely eroded test soil</td>
</tr>
<tr>
<td>Test 2 - Sand/Clay (75/25)(a)</td>
<td>148</td>
<td>12</td>
<td>92</td>
<td>3</td>
<td>22</td>
<td>Eroded small pipe surrounded by collapsing softened zone</td>
</tr>
<tr>
<td>Test 3 - Sand/Clay (75/25)(b)</td>
<td>146</td>
<td>14</td>
<td>91</td>
<td>3</td>
<td>14</td>
<td>Stable, open pipe; 1-1/2” to 2” diameter</td>
</tr>
<tr>
<td>Test 4 - Sand/Clay (70/30)</td>
<td>142</td>
<td>15</td>
<td>88</td>
<td>3</td>
<td>15</td>
<td>Very small pipe; 1/4” diameter</td>
</tr>
<tr>
<td>Test 5 - Sand/Bentonite (87.5/12.5)(a)</td>
<td>137</td>
<td>12</td>
<td>87</td>
<td>3</td>
<td>1</td>
<td>Completely eroded test soil</td>
</tr>
<tr>
<td>Test 6 - Sand/Bentonite (87.5/12.5)(b)</td>
<td>133</td>
<td>15</td>
<td>85</td>
<td>3</td>
<td>21</td>
<td>Stable, open pipe; 1/2” to 3/4” diameter</td>
</tr>
<tr>
<td>Test 7 - Sand/Bentonite (85/15)</td>
<td>145</td>
<td>16</td>
<td>95</td>
<td>3</td>
<td>18</td>
<td>Very little erosion</td>
</tr>
<tr>
<td>Test 8 - Teton Core</td>
<td>117</td>
<td>14</td>
<td>91</td>
<td>3</td>
<td>22</td>
<td>Discontinuous pipe; maximum of 1/4” diameter</td>
</tr>
<tr>
<td>Test 9 - East Branch Core</td>
<td>138</td>
<td>9.5</td>
<td>95</td>
<td>1.5</td>
<td>47</td>
<td>Irregular shaped erosion void; up to 1” diameter</td>
</tr>
</tbody>
</table>
lead to the collapse of the piping erosion void.

Another interesting observation is that in nearly all the tests it appeared that the erosion initiated at one location along the crack and then propagated along the crack from the initiation point. This was evidenced by the piezometers, the light sensors, and the observations at the end of the tests. It is not known whether this initiation was caused by a point in the soil that had a greater susceptibility to erosion or if there was a concentration of flow or deflection of flow into the location of erosion initiation. In either case, this phenomenon represents an added difficulty in predicting when this type of erosion is expected to occur since minor perturbations in either the soil makeup or the flow regime can cause the initiation of erosion to occur at lower average flows than would be necessary given uniform flow conditions.

The results of the IEA testing were compared to results from Hole Erosion Tests (HET) performed in conjunction with this study, the results of soil erodibility tests published by Briaud (2008), and the results of contact erosion tests performed by Guidoux et al. (2010). The critical shear stress values from the IEA test results were similar to those from the HET tests albeit, in general, the IEA tests results were somewhat lower values of critical shear stress. This observation is thought to be due to the longer duration of the IEA tests that allows a slow rate of erosion to be monitored and assessed.

When compared to the Briaud (2008) data, the IEA tests were found to be within the range of critical shear stress and critical velocity values when the effective grain size was used in the assessment. When the mean grain size was used the IEA test results (critical velocity and shear stress) were higher than indicated by Briaud (2008). This observation is not surprising in that a small percentage of fines in a fine sandy soil is expected to control the behavior of the soil.
When compared to the internal erosion test results of Guidoux et al. (2010) the critical shear stress values from the IEA tests were found to be over an order of magnitude higher. This is believed to be due to two factors. First, the soils tested by Guidoux et al. generally had lower fines contents than those tested in the IEA. This factor illustrates the limitations of using the effective grain size for assessment of erodibility. The second factor is believed to be the deflection of the flow into the eroding soils by the gravels (the flow in the Guidoux tests was through coarse gravel rather than a crack). This directed flow is thought to be more erodible than the water flowing parallel to the erodible soil surface. Thus, even though all of the tests performed in the IEA appear to have achieved turbulent flow, the angle of incidence of the flow appears to have even a larger effect on the erodibility. Although not tested in this study, it is possible that bends or junctures in a joint system may be capable of deflecting the flow into the eroding soil.

The discussion above illustrates the difficulty in assessing the critical erosion properties of soils. First, it appears that a number of factors in addition to the gradation and index properties of the soil affect the erodibility of the soil. It was found during this testing that compaction water content and soil density has a significant effect on the erodibility of the soil. In addition, it was found that erosion appeared to initiate in one location along the joint and propagate outward from there. This means that even a small, isolated mass of soil that was placed drier or at a lower density can initiate the erosion which can then propagate through the remainder of the soil deposit. Furthermore, small perturbations in the flow field can change the angle of incidence of the flow with the soil, thus enhancing the erosion potential of the water.

Despite the difficulties in assessing the critical erosion properties that are discussed in the previous paragraph, it is believed that the IEA provides an improvement for assessing behavior of soils adjacent to water flowing in bedrock joints. First, it provides a longer length of
flow over which the soil can erode, thus increasing the ability to assess the effects of small soil perturbations on the erosion behavior. Secondly, the IEA performs tests over a longer period of time, than some of the other tests discussed, thus assessing the effects of flow velocities and critical shear stresses that cause a slow rate of erosion.
REFERENCES


APPENDICES
APPENDIX A: Soil Types and Properties of Soil Used

Garnet Sand

Source: “Fine Garnet Sand”

Blasting Products
820 N Warm Springs Rd
Salt Lake City, UT 84116

D_{50} of Soil: 0.125 mm or #120 sieve size.

Helmer Clay (Kaolinite)

Source: Kaolinite Clay that was mined near Lewiston, Utah in Cache County.

D_{50} of Soil: 0.0031 mm found from Hydrometer Test performed by CEE 6340, Class of 2012 (ASTM D422).

Bentonite

Source: “Pure Gold Grout: contains bentonite”

Intermountain Drilling Supply Corp
3412 West 2400 South
Salt Lake City, UT 84119

D_{50} of Soil: 0.0003 mm based on an estimate for montmorillonite clay.
80% Garnet Sand mixed with 20% Helmer Clay (Kaolinite)

**Dry Unit Weight:** Determined by performing a Modified Proctor Test (ASTM D1557).
75% Garnet Sand mixed with 25% Helmer Clay (Kaolinite)

**Dry Unit Weight:** Determined by performing a Modified Proctor Test (ASTM D1557).
70% Garnet Sand mixed with 30% Helmer Clay (Kaolinite)

**Dry Unit Weight:** Determined by performing a Modified Proctor Test (ASTM D1557).

---

**Modified Proctor:**
Garnet Sand w/ 30% Clay

---

Dry Unit Weight (pcf)

---

Water Content (%)
87.5% Garnet Sand mixed with 12.5% Bentonite

Dry Unit Weight: Determined by performing a Modified Proctor Test (ASTM D1557).
85% Garnet Sand mixed with 15% Bentonite

**Dry Unit Weight:** Determined by performing a Modified Proctor Test (ASTM D1557).

![Modified Proctor Test Graph](image)
**Teton Core Material**

**Source:** Core Material from failed Teton Dam embankment near Teton, Idaho.

**D$_{50}$ of soil:** 0.033 mm found from Hydrometer Test performed by Nate Braithwaite in 2011 (ASTM D422).

**Dry Unit Weight:** Determined by performing a Modified Proctor Test (ASTM D1557).

![Modified Proctor Test Graph](image-url)
East Branch Core Material

Source: Core Material from the existing East Branch Dam located in Elk County, Pennsylvania.

$D_{50}$ of soil: 0.1 mm found from a Sieve Analysis performed by Tyler Coy in 2012 (ASTM C136).

Dry Unit Weight: Determined by performing a Modified Proctor Test (ASTM D1557).
APPENDIX B: MODIFIED HOLE EROSION TEST

Modified Hole Erosion Test: Procedure and Analysis for Determining the Erodibility of Cohesive Soils

by

Jacob C. Erickson

A report submitted in partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE

in

Civil and Environmental Engineering

Approved:

Dr. John D. Rice
Major Professor

Dr. Gilberto E. Urroz
Committee Member

Dr. Joseph A. Caliendo
Committee Member

Utah State University
Logan, Utah
2012
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Introduction

The use of earth structures to contain and control the flow of water is practiced throughout the world. Most commonly known as dams and levees they serve the purpose of creating reservoirs for water storage and as protection against flooding. Unfortunately, these structures may be prone to different types of erosion if they are not designed and/or maintained properly. The two main types of erosion to cause failure in earthen embankments are overtopping and internal erosion (Foster 2000).

After construction even the best engineered earth dams will experience a degree of settlement, which may cause cracking within the structure. The resulting internal cracks can lead to internal erosion and failure of the structure. An improved understanding of erodibility parameters and the conditions that cause erosion would be useful to the mitigation of failures caused by erosion. The Hole Erosion Test (HET) was developed to study erosion rates and the critical shear stress required to initiate erosion of soil particles through a pipe of soil. The main type of internal erosion modeled by the HET is more specifically identified as concentrated leak erosion. This type of erosion occurs when a flow path is created within the soil which allows hydraulic shear stresses to carry away soil particles (ICOLD 2012). To model this behavior a soil sample is compacted into a standard mold with a hole drilled through the center so that flowing water can cause erosion through the pipe of soil.

This report shows the procedure and analysis for determining the erodibility of soils through the use of a physical testing method known as the Hole Erosion Test. The
soils tested for this report were prepared in a manner to simulate similar conditions (i.e. compaction density and water content) to other types of erosion testing performed by two Utah State University Graduate students, Tyler Coy and Joseph Zaleski. The tests performed by Joseph Zaleski and Tyler Coy are projects funded by and supervised by the United States Army Corps of Engineers (USACE). The results from this report are intended to provide data that can help correlate with the results of the Internal Erosion Testing apparatus (Zaleski) and the Foundation Void Erosion Testing apparatus (Coy) to mitigate erosion and contribute to the prioritizing of maintenance needs for current dams and levees owned by USACE. Ideally, the HET apparatus will be used on futures studies of seepage erosion at the Utah State University Earth Structures Lab and provide useful results on other soil types. The analysis and procedure found in this paper is based on the method created by Chi Fai Wan and Robin Fell (2004) and later performed by Tony Wahl (2008). Some modifications to the test performed by Tony Wahl and the United States Department of the Interior, Bureau of Reclamation have been made in this testing apparatus. The main reasons for the modifications are to help automate the data collection process and expand the capabilities of the original design. Further details and discussion on the design modifications will be presented later in this report.

**Background**

In order to gain a deeper understanding of the HET, two published articles were reviewed by the writer. In 2004 Chi Fai Wan and Robin Fell published a journal article
that compared the Hole Erosion Test to the Slot Erosion Test. For the purposes of this report, that article was mainly used to learn the basic procedure and principles needed to run a HET. According to Wan and Fell the erodibility of soil is defined from two behavioral aspects. The first aspect is the rate of erosion that occurs when a given hydraulic shear stress is acting on the soil. The second is the ease of initiating erosion of the soil. Although the concept of these two aspects may be simple, the understanding of two distinct aspects is pertinent to the analysis of the erosion process. Foster and Fell described the potential for erosion in embankments as a four-stage process which relates to the aforementioned aspects. The four stages are initiation, continuation, progression, and breach/failure (Foster 1999). Results from the HET show that both behavioral aspects of erosion can differ depending upon the type of soil, the preparation conditions, and the physical properties. Previous testing of erosion by other researchers has shown a linear relationship between the erosion rate and the hydraulic shear stress. The equation derived from earlier tests is also useful in the analysis of a HET and is expressed as:

$$\dot{\epsilon}_t = C_e (\tau_t - \tau_c)$$  \hspace{1cm} (1)

Where $\dot{\epsilon}_t$ is the rate of erosion per unit surface area of the hole at time $t$ (kg/s/m^2), $C_e$ is the proportionality constant also called the coefficient of soil erosion (s/m), $\tau_t$ is the hydraulic shear stress along the hole at time $t$ (N/m^2), and $\tau_c$ is the critical shear stress (N/m^2). The values of $C_e$ obtained for the HET range from $10^{-1}$ to $10^{-6}$ which allows
the introduction of a new term called the Hole Erosion Index ($I$). This term becomes a convenient way to classify each soil in terms of erodibility and is written as:

$$I = -\log(C_e)$$  

Since the range of soil erosion coefficients is from $10^{-1}$ to $10^{-6}$, the values obtained for $I$ will range from 0-6. For low values of the erosion index the soil is categorized as a more rapidly erodible soil. For larger values approaching 6 the erosion index suggests that the soil is much less erodible or highly resistant to erosion. A qualitative description for the six categories determined from the Erosion Rate Index can be found in Table 1.

<table>
<thead>
<tr>
<th>Group number</th>
<th>Erosion rate index</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>&lt;2</td>
<td>Extremely rapid</td>
</tr>
<tr>
<td>2</td>
<td>2-3</td>
<td>Very Rapid</td>
</tr>
<tr>
<td>3</td>
<td>3-4</td>
<td>Moderately rapid</td>
</tr>
<tr>
<td>4</td>
<td>4-5</td>
<td>Moderately slow</td>
</tr>
<tr>
<td>5</td>
<td>5-6</td>
<td>Very slow</td>
</tr>
<tr>
<td>6</td>
<td>&gt;6</td>
<td>Extremely slow</td>
</tr>
</tbody>
</table>

For the soils tested by Wan and Fell there was a limitation to their HET apparatus design, which only allowed for testing of soils up to a head difference of 1,200 mm. A schematic for the HET as used by Wan and Fell is presented in Figure 1. As seen in Figure 1 the differential head is limited to the allowable height of the tank providing the
inlet pressure. This limitation could possibly cause the exclusion of soils from groups 4 and 5 that may require high differential pressures to initiate erosion.

![Figure 1. Schematic of Wan and Fell HET Apparatus](image)

Also mentioned in the Wan and Fell article are more equations relating to the analysis of the HET, these equations will be presented later on in the analysis section. According to Wan and Fell, the results from a HET test should be used in a qualitative manner to determine the likelihood of initiation of internal erosion by looking at the critical shear stress. The erosion rate index \( I \) can be used to estimate how quickly the erosion may progress to failure/breach of the structure.

The other article that was useful in obtaining background information about the HET was a report for the U.S. Department of the Interior, Bureau of Reclamation (USBR)
by Tony Wahl, Pierre-Louis Regazzoni, and Zeynep Erdogan. Once again, the purpose of this second article was to compare the HET to another type of erosion test, the Jet Erosion Test (JET), but for the writer’s purpose the article was intended to supplement current understanding of the HET. While Wan and Fell did well at summarizing the HET test and explaining useful information about applying the data, there was a lack of detailed information that may be helpful to analyzing test results. The report from the Bureau of Reclamation was much more extensive about procedural details important to the HET. The analysis of data for the HET in the Bureau’s report followed the method developed by Wan and Fell and also compared it to another analysis method developed by Bonelli et al (2008). The Bonelli analysis is not included in this report due to an insufficient understanding, but could be useful to future research as it requires no post-test work to obtain an estimate of the final diameter of the eroded hole. Some important information found in the report by Tony Wahl included the mention of a need for progressive erosion during testing to obtain usable data. Plots of USBR data to refer to as an example were also found to be helpful. Without these reference items it is likely that no successful tests would have been obtained due to the modifications made to the apparatus design.

**Apparatus Design and Modifications**

Design details for the HET device in the Utah State University Earth Structures Lab (USUESL) were obtained from Tony Wahl of the USBR, thus the containment apparatus is virtually identical to the USBR’s device. Details for the soil sample
containment device are shown in Figure 2. Some minor changes to the design in Figure 2 include a different outer diameter on the downstream end of the compaction mold due to the mold having a uniform thickness at the bottom. Also changed was the location of the piezometer connection being placed at the midpoint of the inlet and outlet faces of the acrylic plates. Neither of these changes should affect the functionality of the device.

Another slight modification was the use of inlet and outlet piezometers along with a differential pressure transducer. This was done as a back-up check to ensure that measured pressures were acceptable for analysis.

The main modifications for the HET used in this report relate to the flow measurement and the mechanism used to increase the differential head. Both the USBR and the research by Wan and Fell had a setup that used an adjustable tank of water to create an upstream head by raising or lowering the tank. The tank used by Wan and Fell was limited to reach a height of 1,200 mm above the downstream head, whereas the USBR constructed a high-head tank that allowed a head differential of 5,400 mm. The setup at Utah State University is configured with a pressure regulating valve that is adjusted to change the inlet pressure head. The utilization of a pressure
Figure 2. Drawing Details of Soil Sample Container
regulator allows for pressure heads in excess of 5,400 mm without being constricted to the height of the ceiling. The other main modification to the design involves the use of an ultra-sonic flow meter to measure flow rates as opposed to a v-notch weir. The flow meter was calibrated to the range of flows for a typical test and then the data was collected by the data logger. A flow bypass was added after initial complications with flow measurement readings at the beginning of each test. This allowed for the data collection to begin before actually running water through the sample to ensure that flow rates displayed on the flow meter were consistent with the flow rates that were output by the data logger. The formation of air bubbles within the flow meter may have caused the discontinuities in the flow meter’s operation. The flow bypass also served as a good method for determining the initial inlet pressure by observing the flow rate through a partially closed valve so as to imitate the size of the hole drilled through the sample. The main modifications mentioned above can be seen in Figure 3.
Test Procedure

The testing procedure was performed as follows:

1. Following compaction and specimen preparation the sample is weighed to determine the density of the soil sample.

2. A ¼-inch diameter hole is drilled through the specimen using a drill press and wood auger bit to minimize compaction of the side walls of the hole. Drilling is performed at the slowest possible speed and the bit is advanced slowly and cleaned repeatedly during drilling.

3. If deemed necessary the hole is cleaned using a 0.22-inch diameter rifle brush.
4. Flow through the bypass is started to ensure that data logger values are consistent with those displayed on the flow meter (Sometimes up to 30 minutes was needed for the flow values to converge to a similar value).

5. Specimens are installed into the apparatus with the original top surface (last compacted layer) upstream. If the soil is expected to be highly erodible or susceptible to scour of the upstream and downstream faces, protective end plates are also installed. A plastic geofabric mesh filter is also installed in the upstream chamber to reduce turbulence when specimens are expected to be highly erodible.

6. The test facility is filled slowly with water and all air is bled from piezometer tubes connected to pressure sensors.

7. The pressure regulator is positioned to the desired starting pressure level. For specimens of unknown erodibility, tests are usually started at 50 mm of head.

8. The data acquisition system is started and the inlet valve upstream from the test specimen is opened.

9. The flow rate is monitored to determine whether it is increasing or becoming steady. If the flow rate stabilizes at a given head, then the pressure regulator valve is turned to increase the head.

10. When the flow rate begins to accelerate, the test head is maintained until at least several minutes of accelerating flow is observed. The operator should be aware of the approximate maximum flow increase that can occur if end plates have been installed. For example, if 10 mm end plates have been installed, the ratio of flow rates with a 10 mm hole diameter to the flow through the original 6 mm diameter hole is approximately (10/6)^2≈3. Thus, one should stop the test well before the flow rate has tripled from its value at the start of accelerating flow. If the test is allowed to continue too long, the orifice plate opening will begin to limit the flow rate, which will hinder the data analysis.

11. After the test is stopped, the upstream and downstream chambers are drained and the specimen is removed from the test facility. An initial visual estimate of the final hole diameter is made, and the specimen is weighed.
12. Specimens are oven-dried, weighed, and then a plaster casting is made of the erosion hole.

13. After the plaster has dried specimens are re-soaked to assist in obtaining the casting from the soil without damaging the plaster core.

14. Hole diameters are determined from the casting, typically at 3 positions (two cross measurements per location) spaced approximately equally along the length. The length of the portion of the casting that is of relatively uniform diameter is also recorded. (Large scour holes at the upstream or downstream end are considered to reduce the effective length of the hole, which is taken into account in the data analysis.)

Analysis Process

The analysis of the data obtained from the data acquisition system was one of the greatest obstacles in obtaining results that seemed reasonable compared to those obtained by Tony Wahl and the USBR. The method developed by Wan and Fell is a deterministic analysis that calculates the hole diameters at each time step during the test. These hole diameters are then used to estimate the erosion rate and the shear stress. The use of an excel spreadsheet allows for ease of adjusting data and for plotting results to identify critical values such as the critical shear stress and the critical velocity.

The terms used throughout the analysis process are defined below:

\[ \tau = \text{shear stress along the sides of the hole, (Pa)} \]
\[ L = \text{length of the hole, (m)} \]
\[ \rho_w = \text{fluid density, (kg/m}^3\text{)} \]
\[ \rho_d = \text{dry density, (kg/m}^3\text{)} \]
\[ g = \text{acceleration due to gravity, (m/s}^2\text{)} \]
\[ \Delta h = \text{head difference across the hole from upstream to downstream, (m)} \]
The first step of the analysis is to solve for the friction factor at the start and end of the test. Depending on whether laminar or turbulent flow conditions exist throughout the test will change some of the equations used in the analysis. Laminar and turbulent flow can be determined from the Reynold’s number Eq. 3.

\[ Re = \left( \frac{V \cdot d}{\nu} \right) \]  

(3)

Where \( V \) is the velocity, \( d \) is the diameter and \( \nu \) is the kinematic viscosity. Turbulent flow is considered to exist when \( Re > 2000 \). The subscripts \( L \) and \( T \) denote laminar or turbulent flow. The equations for calculating the friction factor for each case are:

\[ f_L = \frac{\rho \cdot w \cdot g \cdot \Delta h \cdot \pi \cdot d^3}{Q \cdot L \cdot 16} \]  

(4)

\[ f_T = \frac{\rho \cdot w \cdot g \cdot \Delta h \cdot \pi^2 \cdot d^5}{Q^2 \cdot L \cdot 64} \]  

(5)

The friction factors are assumed to vary linearly throughout the test so once they are calculated at the beginning and end they can be quickly interpolated for all the points in between. Next the diameters at each successive data point can be calculated using the following equations:
\[ d_L = \left[ \frac{f_{Li} Q_i + L_i \cdot 16}{\rho w g \Delta h_i \pi} \right]^{1/3} \]  
(6)

\[ d_T = \left[ \frac{f_{Li} Q_i^2 + 2 \cdot L_i \cdot 64}{\rho w g \Delta h_i \pi^2} \right]^{1/5} \]  
(7)

Once the diameters for each time step are calculated the shear stress that occurs at each time step can also be computed using Equation 8:

\[ \tau = \frac{\rho w g \Delta h_i \cdot d_i}{4 \cdot L_i} \]  
(8)

This leaves only one more erodibility parameter to be calculated, the erosion rate. To determine the erosion rate it is necessary to plot the hole diameter vs. time and fit a polynomial function (2\textsuperscript{nd} or 3\textsuperscript{rd} order) to the plot to determine the equation that best describes the diameters as time increases. The erosion rate then becomes the time derivative of that polynomial function all multiplied by one half of the dry density. Figures 6 (3\textsuperscript{rd} order), 11 (2\textsuperscript{nd} order), and 15 (3\textsuperscript{rd} order) show examples of equations used to calculate the erosion rate.

\[ \dot{\epsilon} = \frac{\rho d}{2} \cdot \frac{dd_i}{dt} \]  
(9)

The erosion rate can be plotted against the shear stress to determine the coefficient of soil erosion \((C_e)\) in order to classify the erodibility. The coefficient of soil erosion is taken as the slope of the erosion rate verses shear stress curve, but only during the progressive erosion phase. An example from a lab test result is shown below in Figure 4.
Referring back to Eq. (2) we would find that the Hole Erosion Index for the soil tested from Figure 4 is classified as \( I = 3 \) and could therefore be described as eroding moderately rapid. The critical hydraulic shear stress can also be found in Figure 4 by extrapolating the linear fit line and finding where it crosses the x-axis.

![Graph showing Erosion Rate vs. Shear Stress](image)

\[ y = 0.0003x - 0.0454 \]
\[ R^2 = 0.9169 \]

**Results**

The soil types and targeted density for the tests in this report were based off of soils and soil densities tested by Joseph Zaleski and Tyler Coy. Unfortunately due to the modifications to the design of the apparatus a majority of the tests performed didn’t produce any attainable results. Some reasons for the erroneous data stemmed from a misunderstanding of how the test procedure should have been altered as a result of altering the design. Although most of the earlier tests showed signs of erosion as the flow increased throughout the test and the final hole diameter was larger, the erosion never became progressive. In many cases the flow seemed to increase linearly with
time. According to Wahl (2008) when progressive erosion is reached and maintained the flow will start to increase at an accelerated rate.

A suggestion by the USBR is to plot hole diameter surrogates \((Q/\Delta h)^{1/3}\) for laminar flow and \((Q^2/\Delta h)^{1/5}\) to determine if the flow is progressive or not. Some of the tests performed at the USUESL appeared to have reached the progressive state judging by the data plotting of the surrogates, but yet an analysis afterwards proved to be indeterminate of any meaningful results. Even after the correct procedure was identified there were other problems that hindered the production of any useful data.

Seven soil types were to be tested; they included four manufactured soils and two soils retrieved from onsite. The four manufactured soils consisted of a high percentage of garnet sand with a small percentage of kaolinite in one and two differing mixtures of garnet sand and bentonite and a mixture of coarse sand with kaolinite. The garnet sand with 12.5\% bentonite was tested with two different water contents one with 12\% and the other with 15\% water. The material retrieved from each dam core was taken from the Teton Dam in Idaho and East Branch Dam in Pennsylvania. All of the tests had Reynold’s numbers above 2,000 therefore turbulent flow conditions were assumed for each analysis.

**Garnet Sand with 25\% Kaolinite**

This soil specimen was compacted with 12\% water content and to target a wet density of 145-150 pcf a standard proctor hammer was dropped with ten blows per lift with a total of three lifts. End-plates with 22 mm diameter holes were used to discourage
any spalling of soil from the upstream and downstream faces. Table 3 shows the initial conditions assumed for the start and end of the test along with the friction factors and shear stress values that were calculated for the analysis.

Table 2. Garnet Sand with 25% Kaolinite, Initial and Final Conditions

<table>
<thead>
<tr>
<th>Turbulent (start of test)</th>
<th>Turbulent (end of test)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( L_0 ) ( (m) )</td>
<td>( L_f ) ( (m) )</td>
</tr>
<tr>
<td>---------------------------</td>
<td>------------------------</td>
</tr>
<tr>
<td>0.115</td>
<td>0.106</td>
</tr>
</tbody>
</table>

The soil erosion index from this test resulted in a category 4 soil implying that it would experience moderately slow erosion. The data collected during the test, along with the other useful plots required for the analysis can be seen below in Figures 5-8.
Figure 6. Computed Diameter of Eroded Hole for Garnet Sand with 25% Kaolinite

Figure 7. Shear Stress and Erosion Rate vs. Time for Garnet Sand with 25% Kaolinite

\[ y = 7 \times 10^{-14} x^3 - 4 \times 10^{-10} x^2 + 3 \times 10^{-6} x + 0.007 \]

\[ R^2 = 0.9962 \]
Unfortunately, the results did not allow for the determining of the critical shear stress by extrapolation as the plot shows a negative value for the critical shear stress. Referring to Table 2 and Figure 5 it can be deduced that the critical shear stress occurs below 62.5 Pa. Although the critical shear stress isn’t directly calculated it is concluded that it is small in comparison to the other soils tested for this report.

**Garnet Sand with 15% Bentonite**

Similar specimen preparations were followed with this test except that a water content of 16% was used. The 22 mm metal end plates were also in place for this test. This test analysis was performed with some adjustments to the initial conditions based on data that was adjusted to conform better with the period of progressive erosion. At the start of the test there was no increase in flow so it was assumed that no erosion was occurring. After successive increases to the inlet pressure erosion occurred, but only cleanout/stabilizing erosion was apparent so another increase in pressure was applied.
to initiate progressive erosion (see Figure 9). In an attempt to complete the analysis a preliminary analysis was performed to estimate the hole diameter at the time of the last increase in the differential head. The results for this test may have been affected due to the analytical approach for determining the initial diameter at the start of progressive erosion. This explains why Table 3 contains a different initial diameter from the 6.35 mm drilled hole. It was assumed that the change in length of the hole was negligible.

Table 3. Garnet Sand with 15% Bentonite, Initial and Final Conditions

<table>
<thead>
<tr>
<th></th>
<th>Turbulent (start of test)</th>
<th></th>
<th>Turbulent (end of test)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$L_0$</td>
<td>$D_0$</td>
<td>$Q_0$</td>
</tr>
<tr>
<td>(m)</td>
<td>(m)</td>
<td>(m$^3$/s)</td>
<td>(m)</td>
</tr>
<tr>
<td>0.115</td>
<td>0.0097</td>
<td>0.00030</td>
<td>1.523</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>Turbulent (start of test)</th>
<th></th>
<th>Turbulent (end of test)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$L_f$</td>
<td>$D_f$</td>
<td>$Q_f$</td>
</tr>
<tr>
<td>(m)</td>
<td>(m)</td>
<td>(m$^3$/s)</td>
<td>(m)</td>
</tr>
<tr>
<td>0.106</td>
<td>0.0133</td>
<td>0.00055</td>
<td>1.478</td>
</tr>
</tbody>
</table>

Figure 9. Differential Pressure Increases for Progressive Erosion

The results from the adjusted data corresponding with the period of progressive erosion can be seen in Figures 10-13. A 2$^{nd}$ order polynomial was used to fit the computed hole
diameter plot Figure 11. The second order polynomial was applicable in this case (in lieu of third order) because the cleanout erosion had already occurred.

Figure 10. Test Head and Flow vs. Time for Garnet Sand with 15% Bentonite

Figure 11. Computed Diameter of Eroded Hole for Garnet Sand with 15% Bentonite
This soil is also classified with an index value of 4 so it is moderately slow at eroding, but the critical shear stress is much higher at 250 Pa. This shows how the soil may erode at a rate similar to the soil mixture with garnet sand and 25% Kaolinite, but may require a much higher stress to initiate the erosion process.

Garnet Sand with 12.5% Bentonite
This specimen was prepared similar to the other garnet sand with bentonite, with the exception of being compacted with a water content of 12%. No major concerns applied to the setup and analysis of this test the corresponding tables and figures are found below. The index for this soil was a category 3 so it is slightly more erodible than the 15% bentonite mixture and the critical hydraulic shear stress is 150 Pa. As expected the 12.5% bentonite mixture has a lower critical shear stress and is classified as be more erodible than the 15% bentonite mixture.

<table>
<thead>
<tr>
<th>Table 4. Garnet Sand with 12.5% Bentonite, Initial and Final Conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Turbulent (start of test)</strong></td>
</tr>
<tr>
<td>( L_0 )</td>
</tr>
<tr>
<td>(m)</td>
</tr>
<tr>
<td>0.115</td>
</tr>
<tr>
<td><strong>Turbulent (end of test)</strong></td>
</tr>
<tr>
<td>( L_f )</td>
</tr>
<tr>
<td>(m)</td>
</tr>
<tr>
<td>0.105</td>
</tr>
</tbody>
</table>
Figure 14. Test Head and Flow vs. Time for Garnet Sand with 12.5% Bentonite (12% water)

\[ y = 1E-10x^3 - 5E-08x^2 + 2E-05x + 0.0062 \]
\[ R^2 = 0.9966 \]

Figure 15. Computed Diameter of Eroded Hole for Garnet Sand with 12.5% Bentonite (12% water)
Test results for the other tests can be seen in Table 5 below. The corresponding plots can also be found in the Appendix A. The addition of 1/8 inch foam placed between the
soil and metal end plates was used on both the East Branch and Teton Core test specimens to discourage spalling of the soil at the front and back of the specimen in order to achieve test results. For two of the tests a critical velocity couldn’t be found because the critical shear stress was unknown due to misrepresentations from the extrapolation method (see Figure 8 and Figure A-4). The majority of the tested soils resulted in an HET index of 4 with only one in a class 3 and another in a class 6. However, the critical shear stresses range from less than 60 to 400 pascals among the tested soils, which can have a strong impact on the erodibility of the soil.

<table>
<thead>
<tr>
<th>Soil</th>
<th>Water Content</th>
<th>$I_{\text{HET}}$</th>
<th>$\tau_c$ (N/m²)</th>
<th>$v_c$ (m/s)</th>
<th>Test Density (pcf)</th>
<th>Standard Proctor (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Garnet Sand and 25% Kaolinite</td>
<td>12%</td>
<td>4</td>
<td>$\tau_c &lt; 60$</td>
<td>--</td>
<td>160.35</td>
<td>162.60</td>
</tr>
<tr>
<td>Garnet Sand and 15% Bentonite</td>
<td>16%</td>
<td>4</td>
<td>253</td>
<td>4.00</td>
<td>151.50</td>
<td>157.35</td>
</tr>
<tr>
<td>Garnet Sand and 12.5% Bentonite</td>
<td>12%</td>
<td>3</td>
<td>151</td>
<td>3.02</td>
<td>140.85</td>
<td>150.30</td>
</tr>
<tr>
<td>Garnet Sand and 12.5% Bentonite</td>
<td>15%</td>
<td>4</td>
<td>$\tau_c &lt; 170$</td>
<td>--</td>
<td>149.55</td>
<td>155.70</td>
</tr>
<tr>
<td>East Branch</td>
<td>9.5%</td>
<td>6</td>
<td>400</td>
<td>5.60</td>
<td>129.90</td>
<td>135.15</td>
</tr>
<tr>
<td>Teton Core</td>
<td>14%</td>
<td>4</td>
<td>322</td>
<td>5.20</td>
<td>122.85</td>
<td>124.95</td>
</tr>
<tr>
<td>Coarse Sand and 35% Kaolinite</td>
<td>14%</td>
<td>4</td>
<td>136</td>
<td>3.03</td>
<td>126.84</td>
<td>127.35</td>
</tr>
</tbody>
</table>

**Summary and Conclusions**

The intent of this project was to provide applicable data concerning the erodibility of soils that are being tested by Joseph Zaleski and Tyler Coy of Utah State University. An HET is a test that has been used by other researchers to assess and
quantify erodibility characteristics of cohesive soils. The type of internal erosion best modeled by a HET is specified as concentrated leak erosion.

Previous HET setup configurations have been subject to limitations in attainable head differentials between flow inlet and outlet of the device. The apparatus at Utah State University overcomes these limitations, but also experiences other issues due to some of the modifications. Adjustments to the current configuration could help overcome some of these problems. Referring back to figures 5, 10, and 14 it can be seen that the test head tends to decrease as progressive erosion proceeds. To maintain the pressure differential at a constant value the test operator must continually adjust the valve of the pressure regulator. The current configuration could be much improved by simply moving the location of the data output screen so as to view the pressure readings as the valve is being turned.

Some other issues of concern may have more to do with the types of soil and erosion preferences, but a solution is likely attainable through more research and testing. Both the Teton and East Branch materials experienced erosion at the inlet and outlet of the soil pipe much more rapidly than within the pipe itself. Installing end plates with a 22 mm diameter hole was done to discourage that behavior, but still no tests were successfully completed. In many cases test heads were increased without any acceleration of flow until the specimen collapsed through all at once. Foam rings 1/8 inches thick were placed between the plate and soil to provide a confining pressure to the faces of the specimen. Figure 18 shows a comparison between plaster cores
taken from an East Branch Test with foam between the plates and one without foam. The casting on the right was tested with just end plates and the casting on the left had end plates and foam.

Figure 18. East Branch Plaster Cores (Left Side: Foam and Plates, Right Side: Plates Only)

There seemed to be some improvement by adding foam, but it is likely that even better results could be attained through more research. Optimistically, test results from this report will combine with the testing results by Zaleski and Coy to enhance the understanding of internal erosion processes.
References


Appendices

Appendix A: Soil Analysis Figures

**Figure A-1. Test Head and Flow vs. Time for Garnet Sand with 12.5% Bentonite (15% water)**

**Figure A-2. Computed Diameter of Eroded Hole for Garnet Sand with 12.5% Bentonite (15% water)**
Figure A-3. Shear Stress and Erosion Rate vs. Time for Garnet Sand with 12.5% Bentonite (15% water)

Figure A-4. Erosion Rate vs. Shear Stress for Garnet Sand with 12.5% Bentonite (15% water)
Figure A-5. Test Head and Flow vs. Time for East Branch Core

Figure A-6. Computed Diameter of Eroded Hole for East Branch Core
Figure A-7. Shear Stress and Erosion Rate vs. Time for East Branch Core

Figure A-8. Erosion Rate vs. Shear Stress for East Branch Core
Figure A-9. Test Head and Flow vs. Time for Teton Core

Figure A-10. Computed Diameter of Eroded Hole for Teton Core
Figure A-11. Shear Stress and Erosion Rate vs. Time for Teton Core

Figure A-12. Erosion Rate vs. Shear Stress for Teton Core
Figure A-13. Test Head and Flow vs. Time for Coarse Sand with 35% Kaolinite

Figure A-14. Computed Diameter of Eroded Hole for Coarse Sand with 35% Kaolinite
Figure A-15. Shear Stress and Erosion Rate vs. Time for Coarse Sand with 35% Kaolinite

Figure A-16. Erosion Rate vs. Shear Stress for Coarse Sand with 35% Kaolinite
Appendix B: Additional Figures

Figure B-1. Example of Variations in Plaster Castings

Figure B-2. Different Types of End Plates Used to Discourage Spalling
Figure B-3. Example of Surface Spalling with Minimal Hole Erosion

Figure B-4. Different Drill Bits Tested to Minimize Disturbance of Soil