Assessing the Potential for Seepage Barrier Defects to Propagate into Seepage Erosion Mechanisms

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ASSESSING THE POTENTIAL FOR SEEPAGE BARRIER DEFECTS TO PROPagate INTO SEEPAGE EROSION MECHANISMS

by

Ryan G. Van Leuven

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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2011
ABSTRACT

Assessing the Potential for Seepage Barrier Defects to Propagate into Seepage Erosion Mechanisms

by

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Utah State University, 2010

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Department: Civil and Environmental Engineering

Seepage barriers have been used extensively to mitigate seepage problems in dams and levees. Although the design of many of these dams and levees has been based on intact barriers, seepage barriers have been shown to be susceptible to deformation and cracking when high differential hydraulic pressures act across the barrier. Under certain conditions, these cracks can lead to serious seepage problems, which could potentially lead to the development of a low-resistance seepage pathway. Three scenarios have been identified where there is the potential for erosion to occur adjacent to a crack in a barrier: 1) erosion at the interface between a fine-grained soil and a course-grained soil, 2) erosion of overlying soil due to flow along a joint in bedrock, and 3) erosion of the barrier material. The objective of this study is to investigate the first mode of erosion and identify the conditions at which more serious seepage problems can develop. The research has been performed using a laboratory model to simulate conditions near a seepage barrier crack under the scenarios described above. The results from the laboratory testing were compared to finite element seepage models for each scenario to
estimate the flow velocities near the crack. The flow velocities were compared to estimated critical velocities of the soil to assess where erosion is likely to occur. A comparison was made between the observed behavior in the model and the behavior predicted with the computer model. The results of the research will be used to develop a method to assess the potential for erosion to occur and develop into a failure mode based on conditions near seepage barrier cracks.

(81 pages)
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CHAPTER 1
INTRODUCTION

1.1 Seepage Barriers

Seepage barriers are commonly used to reduce the flow of water through the foundations and embankments of earth dams and levees. Common barrier types include: jet grouted walls, slurry walls, deep mixed walls, diaphragm walls and secant pile walls. The design of many dams and levees has been based on the assumption of intact barriers, even though seepage barriers are susceptible to deformation and cracking. Barriers may deform and crack due to high differential hydraulic pressures acting across the barrier and case studies of existing dams have observed cracks and degrading barrier performance (Rice and Duncan 2010a). When a seepage barrier cracks, the barrier’s effectiveness in reducing seepage flow is decreased and more serious problems may result. It is conceivable that a crack in a seepage barrier may initiate erosion of the barrier material or under certain conditions erosion of soils adjacent to the barrier. After erosion begins, it may propagate into a low-resistance seepage pathway and eventually lead to a serious erosion problem. The purpose of this study is to investigate one of the scenarios that could lead to these erosion problems.

1.2 Piping and Internal Erosion

Piping generally refers to a subsurface erosion process where an erosive gradient is able to remove subsurface materials to a free exit. Dams and levees can be susceptible to piping erosion if an erosive gradient develops, an unprotected pathway for soil removal is present, and the pipe is able to remain open. Piping may develop when a dam or levee
that is constructed of impervious clays is placed on granular subsoil. In this case, excess water pressures may cause soil erosion to begin near the downstream toe of the dam or levee. Subsequently, erosion of the granular soil along its interface with the clay develops by progressing backwards toward the high head side of the embankment forming an open eroded channel or “pipe.” The non-homogeneity of the soil causes the channels to be irregularly shaped. If left to freely develop, the pipe could eventually allow water to flow freely from the upstream to the downstream side of the dam, which may cause further erosion of the dam and subsoil. Piping poses a serious threat to dams and levees and may even lead to the total collapse of the structure.

Internal erosion may occur when water flows through a cracks or defects in a foundation, compacted fill, or the contact between a foundation and a fill of a dam or levee (McCook 2004). When the water flows through the defect with enough velocity to erode material from the walls of the crack or defect, the defect will enlarge. Erosion of the defect may continue until the integrity of the dam or levee and may lead to failure of the structure. The process modeled in this study is internal erosion rather than piping.

1.3 Purpose of Research

The purpose of this research is to investigate internal erosion of soil near seepage barrier cracks at the interface of two different materials. A seepage test cell developed at Utah State University has been used to model conditions needed to induce erosion near a seepage barrier crack for several soil types. Results from each laboratory test were compared to a finite-element analysis and used to develop a method to predict the occurrence of erosion adjacent to a seepage barrier crack. The results of the study were
used to evaluate existing seepage barriers in dams and levees as well as in the design of new seepage barriers.

1.4 Report Organization

This thesis includes six chapters. Chapter 1 is an introduction to the purpose of the research. Chapter 2 presents an overview of the literature reviewed which relates to this study. It discusses seepage barrier cracking, the seepage test cell, suffusion, and soil erodibility. Chapter 3 is a discussion of the seepage test cell used for the laboratory testing in this research. First it presents an overview of the functions of the seepage test cell, then it explains in detail how data is collected and the test set up procedure used for this study. Chapter 4 discusses the uses of computer models in this study. Chapter 5 presents an analysis of results and is organized in sections based on soil types used in this study. Each section discusses erosive behavior during each test, presents features formed by erosion and sedimentation during each test, presents the results of the finite element analysis performed for each test, and discusses the theories behind the results. Chapter 6 presents the conclusions.
CHAPTER 2

LITERATURE REVIEWED

2.1 Introduction

A literary review of prior research relating to this project was performed in the following areas:

- Seepage barrier cracking
- Seepage test cell
- Piping
- Suffusion
- Soil erodibility

2.2 Seepage Barrier Cracking

Deformation of a seepage barrier may occur due to large differential pore pressures acting across the barrier (Rice and Duncan 2010a). At locations where material interfaces occur between materials with significantly different deformation characteristics, differential deformation of the seepage barrier and large bending moments may result in cracking of the seepage barrier. Rice and Duncan (2008) found that bending moments on barriers are greatest between layers of soft soils (silts, clays, etc) and dense gravel or between bedrock and soil as depicted in Figure 2.1.

Erosion of the barrier material or surrounding soil may result when these conditions are combined with high hydraulic gradients in the adjacent soil. Three scenarios were identified by Rice and Duncan (2008) where potential exists for erosion of
soil adjacent to a seepage barrier or for erosion of the barrier material to occur. The three scenarios identified are: 1) erosion of the barrier material along the defect, 2) erosion of soil along a soil boundary, and 3) erosion along bedrock joints.

A good example of cracking at preferential locations is at Navajo Dam in New Mexico where the locations of cracks in the barrier were mapped using sonic testing and core holes (Rice and Duncan 2008). The mapping showed that while many of the cracks were located in random locations, others in the shale bedrock and layered sandstone are aligned parallel to the bending in horizontal layers. Figure 2.2 highlights the horizontally aligned cracks with a dashed line which are likely the result of stress concentrations caused by the difference in deformation characteristics of the shale and sandstone layers (Rice and Duncan 2010a).

![Figure 2.1 Deformation of seepage barrier due to differential water pressure (modified after Rice and Duncan 2010b)](image-url)
Figure 2.2  Cracks in Navajo Dam appear to follow geologic features (Rice and Duncan 2010a)

2.3 Seepage Test Cell

At Utah State University, a seepage test cell has been developed which can be used to model flow through a cracked seepage barrier. The test cell is constructed of an air-tight steel box which is separated by a fractured concrete seepage barrier. The aperture of the fracture in the seepage barrier can be adjusted and data such as pressure readings, flow rates and turbidity readings are recorded during testing. More detailed information on capabilities and functions of the seepage test cell is presented in Chapter 3 of this report.
In 2009, a study was conducted which compared measured values produced by the seepage test cell to theoretical values calculated by a finite element analysis. The study took into account the effect of variables such as crack aperture, crack type (fractured or smooth crack surface), hydraulic conductivity of the soil around the crack and the gradation of the sand used in the various tests (Whitmer 2009). Results from the study showed that the measured values agreed with the theoretical values until a certain point where the finite element analysis over predicted the measured flow rates. The study concluded that the difference occurred because the finite element analysis did not account for the infilling of the crack by sand once the crack aperture was large enough for soil particles to enter. The infilling resulted in a decrease in flow. It also showed that the type of crack (smooth or fractured) had a significant effect on the flow due to the fractured crack having a more turbulent flow.

Also in 2009, a study was performed which used the seepage test cell to test the validity of the Cubic Law to model flow through a fracture (Stephens 2009). The Cubic Law is commonly used to calculate flow through a crack as a function of the crack aperture cubed. The study concluded that the Cubic Law accurately predicts flows at apertures less than 0.012 in. (0.3 mm), but over-predicts flows at apertures larger than 0.016 in. (0.4 mm). As part of the study, experimental data were used to produce a modification to the Cubic Law which can be used to more accurately model the laboratory test in a finite element analysis. By following the process developed in the study, the head differential across the barrier and the crack aperture can be used to calculate transmission through the crack which more closely matches the values measured during the laboratory testing. The transmissivity values can be converted into an
equivalent hydraulic conductivity value to be used in finite element analysis to model the barrier in the test cell.

2.4 Piping

Because the Netherlands is located in a low area that is highly populated and susceptible to flooding, dikes or levees are required to provide a high degree of safety. Typically the structures are constructed of impervious clay and are placed on sandy subsoil. Due to their importance and vulnerability to piping, several studies have been conducted in the Netherlands to model and predict the initiation and propagation of piping under such structures.

Koenders and Sellmeijer (1991) developed a mathematical model which takes into account most erosion features observed under dams in the Netherlands. The model is based on a two-dimensional set up of a dike as shown in Figure 2.3. The model is a solution of a linear-water flow problem with assumed boundary conditions (Koenders and Sellmeijer 1991). The equilibrium state is calculated in the model without a time scale. Calculations showed that while a minute amount of material has eroded away, a stable equilibrium may still exist. Calculations also show that with a certain combination of parameters, the piping process will propagate. The study concluded that the presence of a piping channel matters much more than its length and that soil particle size is much more important than the soil’s permeability. The authors concluded that while their predictions were correct, the mathematical model should not be used for design as it makes several simplifying assumptions and it does not contain any safety. However, they do recommend the model for use when designing centrifuge and scale model tests to model piping.
Sellmeijer presented a method to design against piping in a dike made of impervious material with sloping sides over a granular material. The model requires that simple rules be used to specify the critical gradient under the dike and embodied into a conceptual model which is then incorporated into a numerical program (Sellmeijer 2006). Random computations are then performed to predict the critical head and the critical width of the dike. The model’s results proved accurate except for cases with very thin granular layers.

De Wit et al. (1981) performed laboratory testing to model the piping erosion under an impervious dike on pervious, granular subsoil. Three cases were modeled in two flumes with dimensions differing by a scale factor of three (De Wit et al. 1981). In all the tests, piping initiated at the downstream toe of the dike where the maximum hydraulic gradient occurs. As the head was raised, the erosion progressed through several distinct stages until the state of equilibrium was disturbed and meandering, backwards erosion began progressing between the clay and the sand. The study found that generally the coarser and denser the sand is, the higher the critical head will be. The models also
demonstrated that with a constant head, time or the weight of the dike do not have significant effects on the critical head. The results for each case from both flumes were compared to calculations using the Laplacean equations. Results from the model and the Laplacean equation agreed until piping initiated. The study demonstrated that by using results from tests on scale models, the occurrence of piping can be predicted.

2.5 Suffusion

A soil which is broadly graded and contains particles ranging from silt or clay to gravel size, are usually described as an internally unstable soil (Wan and Fell 2008). Internally soils can be susceptible to suffusion which is a process that occurs when seepage forces move finer soil particles through spaces between larger soil particles. This process can change the mechanical and hydraulic characteristics of the affected soil; for example altering the soil permeability (Bendahmane et al. 2008). The process of suffusion will produce a coarser soil structure and lead to increased seepage, possible settlement of embankments, and increased susceptibility to slope instability which could contribute to failure of an affected dam or levee (Wan and Fell 2008). Filters constructed of internally unstable materials may be susceptible to the erosion of the finer particles, causing the filter to be less effective and may result in piping failure.

Wan and Fell (2008) developed a laboratory test to assess current methods and develop improved methods for determining the internal instability of silt-sand-gravel and clay-silt-gravel soils. Results demonstrated that current methods for assessing internal instability are conservative (Wan and Fell 2008). They proposed an improved process for assessing internal instability based on grain size distribution. However, due to the complexity of the factors affecting internal stability of soil, they recommended that
laboratory tests be performed for important projects on soils which are in marginal areas of the criteria in order to confirm the predictions made by their proposed method.

Bendahmane et al. (2008) performed laboratory testing on a washed Loire sand to identify specific parameters leading to suffusion and backward erosion. Testing demonstrated that volume or particle size distribution of the samples was not significantly affected by the suffusion erosion occurring in the clay fraction of the sample (Bendahmane et al. 2008). The suffusion of the clay portion of the sample did decrease the sample’s permeability. The tests demonstrated that the hydraulic gradient controls the increase of the rate of suffusion and that initial clay content and porosity are important factors affecting suffusion. After suffusion is initiated, erosion of larger sand particles may begin and lead to backwards erosion if the hydraulic gradient reaches a threshold value.

2.6 Soil Erodibility

The information presented in this section refers to surface erosion, or erosion at the interface between the soil and a body of water. However, many of the same concepts are useful in the analysis of internal seepage erosion, which is the topic of this study.

Three main factors effecting soil erosion are: the erodibility of the soil being eroded, the velocity of the water that is eroding the soil and the geometry of any obstacles the water may encounter (Briaud 2010). Figure 2.4 shows a free body diagram of a submerged soil particle without water flowing over it. A normal stress acts around the particle due to hydrostatic pressure. Since the top of the particle is slightly higher than the bottom of the particle, the normal stress is slightly higher than at the top due to the increased hydrostatic pressure at the bottom. This difference in normal stress results in a
buoyancy force and a reduced effective weight of the soil particle. When water flows over the particle as in Figure 2.5, a drag force and shear stresses between the water and the particle develop, the normal stress acting on the top of the particle is reduced by water flow and the shear and normal stresses at the particle’s boundaries fluctuate due to turbid flow (Briaud 2010). Erosion occurs when the combination of the uplift force and the fluctuating drag force are able to cause the particle to move from its position and be carried away by the flowing water.

Figure 2.4 Free body diagram of a particle of soil for a no flow condition (Briaud 2010)
Briaud conducted a study of four case histories to show how erosion can be predicted by understanding the process of erosion and knowing the properties of the soil, water and obstacles involved in each case study. The erosion studied and predicted in the case studies was caused by bridge scour, meander migration, wave action and levee overtopping (Briaud 2010). Briaud demonstrated methods to accurately predict soil erosion in each of the four case studies. He also presented two methods of estimating a soil’s erodability; one based on the velocity of the water flowing across the soil and the soil type and the other based on the shear stress acting on the soil and the soil type. While it is easier to calculate the velocity of water than the shear stress acting on a particle, Briaud concluded that using the water velocity to estimate erodibility is less representative and more uncertain than using the shear stress. He also noted that the velocity and shear stress are not linked by a constant (Briaud 2010). Figure 2.6 can be used to estimate a soil’s critical velocity based on the mean grain size and Figure 2.7 estimates a soil’s critical shear stress.

Figure 2.5 Free body diagram of a particle of soil for condition with flowing water (Briaud 2010)
Figure 2.6 Critical velocity versus grain size (Briaud 2010)

Figure 2.7 Critical shear stress versus grain size (Briaud 2010)
Figure 2.8 is a diagram which can be used to estimate the critical velocity of a saturated, cohesive soil based on its grain size and void ratio (Axelsson 2002). There is some uncertainty in the critical velocities estimated by the chart due to other factors that affect erosion which are not accounted for such as sedimentary structures, and mineral composition. The diagram shows that a soil’s resistance to erosion increases as it is compacted which is due to water being expelled from voids in the saturated soil.

Presented in Figure 2.9 is a relationship relating velocity and soil particle size to soil’s erodability in air and water. The relationship can be used to estimate the velocity required to erode, transport or deposit a soil particle based on its particle size. The diagram demonstrates that fine sand particles are moved more easily than coarser or finer particles (Garrels 1951).

Figure 2.8  Relationships of critical erosion velocity to grain size and void ratio (Axelsson 2002)
Figure 2.9 Erosion and transport curves for air and water (Garrels 1951)
CHAPTER 3
SEEPAGE TEST CELL

3.1 Introduction

A seepage test cell was designed and constructed at Utah State University for the purpose of modeling flow though a cracked seepage barrier. A photo of the seepage test cell is shown in Figure 3.1. The cell is constructed of 0.5 in. (12.7 mm) plate steel and reinforced with steel channels to prevent deformation of the cell during testing. Figure 3.2 presents a schematic illustration of the seepage test cell showing the locations of the water inflow, outflow, concrete seepage barrier, pressure transducers, and the turbidity meter used for this test. The crack in the concrete block is able to be adjusted by turning four screw-jacks to increase or decrease the crack aperture. The inflow water pressure is controlled by a pressure regulator and a constant head is maintained by a raised outflow pipe. Outflow rates are measured by collecting outflow in a weigh tank. The screen from a No. 40 sieve was placed at the seepage cell’s outflow to prevent the coarse sand particles from filling the outflow pipe while allowing eroded particles to pass freely through the outflow and turbidity meter. The turbidity meter is a TF56 Sensor made by Optek-Sensors and is installed in line with the outflow pipe as shown in Figure 3.3.

3.2 Data Collection

A Cambell Scienific CR3000 data logger was used to collect data from each of the instruments every 4 seconds. The data logger averaged the collected data every 10 seconds and sent it to a computer in the laboratory. Measurements collected by the data
Figure 3.1 Seepage test cell

Figure 3.2 Schematic drawing of the seepage test cell
Figure 3.3 - TF56 Sensor attached to outflow for turbidity measurement

logger include pressure readings from 16 pressure transducers located throughout the cell, the weight of the water collection tank, and turbidity readings measured in parts per million (ppm) from the turbidity meter located on the cell’s outflow.

3.3 Test Construction

Figure 3.4 shows the test setup for the tests performed in this study. Four soil types were used in the construction of the various test scenarios. Physical properties of the four soils are presented in Table 3.1. Details of tests performed on the soils used are located in the Appendix. The coarse sand was used for the soil layer above the seepage barrier crack in all scenarios. The fine sand, the silt and the clay soils were alternated as the soil layer below the crack (the eroding material). Coarse sand was placed in the bottom 7 in. of the seepage test cell on the upstream and downstream sides of the concrete barrier to fill the space so that the eroding soils need not be compacted the full depth. The coarse sand was capped by 2 in. (50.8 mm) of concrete to provide a firm
compaction base for compacting soil in the cell for each test. On top of the concrete cap, 3 in. (76.2 mm) of the eroding soil was compacted so that the top of the soil layer was even with the bottom of the crack as shown in Figure 3.5.

<table>
<thead>
<tr>
<th>Soil</th>
<th>Optimum Water Content (%)</th>
<th>Dry Unit Weight (pcf)</th>
<th>Permeability (cm/s)</th>
<th>PL</th>
<th>LL</th>
<th>PI</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse Sand</td>
<td>NA</td>
<td>90*</td>
<td>7.78E-02</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Fine Sand</td>
<td>NA</td>
<td>109*</td>
<td>2.60E-03</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Silt</td>
<td>14.5</td>
<td>117**</td>
<td>1.56E-05</td>
<td>16</td>
<td>19</td>
<td>3</td>
</tr>
<tr>
<td>Clay</td>
<td>28.5</td>
<td>116**</td>
<td>3.11E-08</td>
<td>33</td>
<td>49</td>
<td>16</td>
</tr>
</tbody>
</table>

* 100% of maximum density  
** 95% of maximum density as determined by ASTM D1557-91

Figure 3.4 Test setup
Modified proctor tests (ASTM 1557) were performed on the clay and the silt to determine their maximum density and optimum water content. The results were used to determine the dry unit weight of the soil at 95% compaction. The soil in the test cell was compacted to 95% by determining the weight of the compacted soil that would be required to occupy a 1 in. (25.4 mm) thick layer in the seepage cell. After mixing the soil to the optimum water content and weighing out the calculated amount of soil needed, it was placed in the cell and compacted until it was reduced to a 1 in. (25.4 mm) thick layer. The soil was compacted in three 1 in. (25.4 mm) lifts to fill the area between the concrete cap and the bottom of the barrier crack as shown in Figure 3.5. The fine sand was compacted in three, 1 in. (25.4 mm) lifts by wetting the sand and compacting it until it was no longer able to be densified. The soil was compacted in this manner to avoid settlement during testing and provide a reference for test repeatability. A thin sheet of plastic was placed over the top of the soil near the outflow as shown in Figure 3.6 to prevent erosion from occurring along the downstream edge of the soil.
After compacting the soil below the barrier crack, the screw jacks were used to adjust the crack aperture as needed. The aperture was measured using a wire gauge. The coarse sand was placed on the compacted soil and compacted by tapping the sides of the seepage test cell to cause vibration. Carbon dioxide (CO$_2$) was used to displace oxygen in the soils since CO$_2$ more readily dissolves in water making saturation of the soil easier.

Next, the seepage test cell was flooded with de-aired water from the bottom up and the lid was placed on the test cell and sealed. An overburden pressure of 13 psi was applied over the upstream and downstream portions of the seepage test cell using air bladders as shown in Figure 3.7.

After the test was constructed, de-aired water was run through the test cell by applying a water pressure of about 2 psi on the upstream side of the seepage barrier by adjusting the pressure regulator. The outflow was measured and the turbidity levels were monitored. The pressure of 2 psi was maintained until turbidity levels stabilized, indicating that any loose particles had been flushed out of the system. After turbidity levels stabilized, the upstream pressure was increased to 4 psi. The process of monitoring...
the outflow and turbidity levels was repeated, increasing the upstream pressure by increments of 2 psi each time until an upstream pressure of 8 psi was achieved. De-aired water was used during the process until the de-airing tanks were no longer capable of producing sufficient de-aired water to maintain a constant flow. At this point (during the 4 or 6 psi level depending on crack aperture) tap water from the City of Logan, Utah was used for the remainder of the test. After the upstream pressure reached 8 psi, the pressure was maintained for the duration of the test. 8 psi was used for the upstream pressure to be used because the outflow pipe was not sized to handle a larger flow than that produced by the 8 psi upstream pressure with a 0.039 in. (1.0 mm) crack aperture in the seepage barrier. The flow rate was periodically monitored throughout the duration of the tests.

Figure 3.7 Air bladders on the upstream and downstream sides of the seepage barrier apply overburden pressure to the soil
CHAPTER 4
COMPUTER MODEL

Tests performed using the seepage test cell were modeled using the finite element analysis program, Slide (Slide 5.0, Rocscience 2008). In order to model the seepage test cell, constant head boundaries were used to model the upstream and downstream boundaries as presented in Figure 4.1. The top and bottom boundaries were modeled using no flow boundaries. The hydraulic conductivity of the soils used in testing were input into the computer model. Soil properties were determined by laboratory testing and are presented in the Appendix. A low hydraulic conductivity of 1E-10 ft/s (3.05E-11 m/s) was used to model the seepage barrier. The seepage barrier crack was modeled using a 0.125 in. (3.175 mm) high row of elements to represent the crack. An equivalent hydraulic conductivity was calculated for each crack aperture that resulted in equal transmissivity values for the row of elements and the actual crack. The equivalent hydraulic conductivity was calculated following a process developed in earlier studies using the seepage test cell (Whitmer 2009). The equivalent hydraulic conductivity is calculated by obtaining a flow rate (Q) from Figure 4.2 using the seepage barrier crack aperture and pressure head difference acting across the seepage barrier. The data in Figure 4.2 were obtained experimentally in a previous study using the seepage test cell (Stephens 2009). The pressure head difference can be determined by subtracting the readings of the pressure transducers downstream of the seepage barrier from those upstream.
Figure 4.1 Boundary conditions used to model laboratory tests in Slide 5.0

Figure 4.2 Planer crack power trend lines (Stephens 2009)

After the flow rate $Q$ is obtained, it is used in a modified version of the equation for transmisivity:

$$T = k \cdot b$$  \hspace{1cm} (4.1)

Where $k$ represents hydraulic conductivity, $T$ is transitivity and $b$ is the width of the crack as modeled in Slide. Rearranging Equation 4.1 we get:
The Flow volume, \( Q \), is defined by Darcy’s Law:

\[
Q = k \cdot i \cdot A
\]  

(4.3)

Where \( Q \) is the flow through the crack as determined from Figure 18 for the given crack aperture, \( i \) is the hydraulic gradient, and \( A \) is the cross section area of the flow region.

Substituting Equation 4.2 into 4.3 we get:

\[
Q = \frac{T}{b} \cdot A \cdot i
\]  

(4.4)

Because the area \( A \) is one foot (the width of the seepage barrier) by the width of the crack \( b \), the equation simplifies to:

\[
Q = T \cdot i \rightarrow T = \frac{Q}{i}
\]  

(4.5)

By using these equations, an equivalent hydraulic conductivity was calculated for each test based on crack aperture and the pressure difference acting across the barrier.
CHAPTER 5
ANALYSIS OF RESULTS

For each laboratory test, analyses were performed to compare the test results to the theoretical finite element analysis. This was done by using data collected by the turbidity meter to analyze erosion behavior and by examining features formed during the test by erosion and deposition. Measured flow rates were used to calculate flow velocities in the seepage barrier crack and were compared to velocities calculated in the finite element analysis. Calculated velocities were then compared to the material’s estimated erosive velocity to determine if features formed during testing were consistent with the calculated velocities.

5.1 Clay Results

Based on Figures 2.6, 2.8, and 2.9, the erosive velocity for the clay soil is expected to be approximately 1.34 to 1.97 ft/s (0.4 to 0.6 m/s). When velocities in the seepage test cell reach values near or greater than the erosive velocity, erosion is expected to initiate and propagate in the clay material.

Two tests were performed on the clay soil, one (Test C1) with a 0.020 in. (0.5 mm) crack in the seepage barrier and the other (Test C2) with a 0.039 in. (1.0 mm) crack. Test C1 was run for almost fourteen days and the recorded turbidity verses time data for the test is presented in Figure 5.1. Test C2 was run for sixteen days and Figure 5.2 shows turbidity verses time data for the test. The data in Figures 5.1 and 5.2 shows that the clay material responded similarly in the seepage test cell during both tests. After an initial high spike in turbidity, the remainder of the test follows a pattern of a lower baseline
turbidity with intermittent spikes throughout the tests. The initial turbidity spikes are likely due to a flushing of the system of any loose particles that were easily carried away by the initial surges of water.

After the initial flush, the low baseline turbidity was likely a result of the cohesiveness of the clay which makes the clay resistant to eroding quickly. It has been reported (Morgan 1979) that erosion of clay occurs when clumps of clay detach from the eroding surface rather than individual clay particles eroding from the surface. It is theorized that the intermittent turbidity spikes observed in the tests are due to sporadic erosion of clumps of clay. Following the detachment of a clump or clumps of clay from the surface, the flowing water was able to break apart the detached particles as they were filtered through the sand and carry the clay particles through the outflow and turbidity meter. As more material was able to be eroded away, a more turbid and erosive flow caused further erosion until an equilibrium was achieved, causing the turbidity measurements to again drop to the baseline level until the process repeated when another mass of material was able to break free.

The Test C2 results show a more constant and generally lower turbidity level compared to Test C1, indicating that the erosion rates between the spikes in turbidity levels were significantly less. This is thought to be due to a higher water velocity exiting the crack in Test C1. Although the crack aperture in the seepage barrier for Test C2 was twice as large as that of Test C1, the flow rate in Test C1 was similar to Test C2 (0.0033 cfs (9.3E-05 m$^3$/s) versus 0.0055 cfs (1.6E-04 m$^3$/s) for Tests C1 and C2, respectively) and thus the water velocity exiting the crack was significantly higher in Test C1. The
Figure 5.1 Turbidity versus time results for Test C1 (0.5 mm crack)

Figure 5.2 Turbidity versus time results for Test C2 (1.0 mm crack)
higher water velocity near the crack in Test C1 helps to explain the higher baseline erosion rates throughout the test.

After the tests were completed, they were carefully deconstructed and documented. Neither clay test exhibited any significant erosion of the clay near the upstream side of the crack. However, both tests had measurable eroded areas adjacent to the downstream side of the seepage barrier. The eroded area on the downstream side of the seepage barrier and the lack of significant erosion on the upstream side suggest that the primary cause of the erosion was the jetting of water out of the barrier crack. Dimensions of the erosion and depositional features formed during Tests C1 and C2 are presented in Table 5.1.

<table>
<thead>
<tr>
<th>Test</th>
<th>Erosion Trough depth/width</th>
<th>Depositional Berm height/width</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Upstream</td>
<td>Downstream</td>
</tr>
<tr>
<td>C1</td>
<td>NA</td>
<td>0.4 in. / 1.0 in.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(10.16 mm/25.4 mm)</td>
</tr>
<tr>
<td>C2</td>
<td>NA</td>
<td>0.8 in. / 1.0 in.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(20.32 mm/25.4 mm)</td>
</tr>
</tbody>
</table>

The erosion and deposition features from Test C1 are shown in Figure 5.3. The erosion process formed a trough-like feature immediately adjacent to the barrier. Immediately downstream from the trough-like feature was a small berm in which eroded clumps of clay material had been trapped in the voids between the coarse sand particles. Further downstream from the berm was an area where clay particles have been deposited by falling out of suspension. A photograph of a cross section cut through the berm is presented in Figure 5.4. It can be seen that some of the interstitial voids of the coarse
sand in the berm are filled with clay while others remain open. A sieve analysis performed on the berm material found that 85.5% of the berm by dry weight was made up of coarse sand and the remaining 14.5% was clay deposits by weight.

Test C2 produced features similar to those of C1, though they were less defined. Test C2 did not produce a continuous trough-like feature, but instead two individual half-circle eroded areas were formed as shown in Figure 5.5. Since two separate eroded features formed, it is possible that eroded material from upstream became lodged in the seepage barrier crack impeding flow in the crack and reducing erosion downstream from the impediment. Another possible explanation of this formation is that particles were initially broken off at the two eroded areas and propagated from those locations. If the latter explanation is correct, and had the test been left to run for an extended amount of time, the two formations may have joined to form one continuous trough feature similar to that in Test C1. The eroded areas formed berms downstream as in Test C1 and deposited materials can also be noticed further downstream. A sieve analysis was performed on the material forming the berm in Test C2, the results show that 92.0% of the material by weight was coarse sand and 8.0% was made of deposited clay material. The void ratio was calculated by assuming the specific gravity of the clay material to be 2.65 and using the following equation:

$$e = ((G_s \cdot \gamma_w)/(\gamma_{dry})) - 1$$  \hfill (5.1)

where $e$ is the void ratio, $\gamma_w$ is the unit weight of water (62.4 pcf), and $\gamma_{dry}$ is the dry unit weight of the coarse sand (90 pcf). Using Equation 5.1, the void ratio of the coarse sand was found to be approximately 0.84.
The results from the sieve analysis performed on the berm material for both tests can be used with the calculated void ratio to estimate the percentage of the voids filled in the coarse sand for each berm. The volume of the berm can be calculated from the measurements taken after the test which was found to be 0.001323 ft$^3$ for Test C1. From the sieve analysis the total weight of the berm material in C1 was found to be approximately 0.49 lbs with 15% of the material by weight being clay. The weight of the clay material can then be determined by taking 15% of the total berm weight, which is 0.074 lbs. Assuming the clay is packed in the berm at the unit weight it was compacted in the seepage test cell (116 pcf), the volume of the clay in the berm by dividing the weight of the clay in the berm by the compacted unit weight. From this calculation, the volume of clay in the berm was found to be 0.000638 ft$^3$. Knowing the volume of the berm (0.001323 ft$^3$) and the void ratio of the coarse sand (0.84), the volume of voids can be calculated from the following equation:

$$e = V_v/(V_t+V_v)$$

where $e$ is the void ratio, $V_t$ is the total volume of the berm, and $V_v$ is the volume of voids in the berm. Solving for $V_v$, the total volume of voids in the berm was found to be 0.0069 ft$^3$. The percentage of voids filled by clay in the berm can be determined by dividing the estimated volume of clay in the berm from the calculated volume of voids in the berm. The percentage of voids filled in the berm was found to be 9.3% for Test C1. Test C2 did not form a continuous berm, making the volume calculations more complex. However, the percent by weight of the clay in the berm from Test C2 is similar to that of Test C1 so it can be assumed that the percent of voids in test C2 would be similar to that of Test C1.
Figure 5.3 Erosion formations and deposits from Test C1

Figure 5.4 Cross-section view of berm formed during Test C1
Figure 5.5 Erosion formations and deposits formed during Test C2

Measurements of the erosion features for each test were used to build a model of the features in Slide to show how they affect flow velocities and how deposits formed. Slide models for both pre-erosion and post-erosion conditions were constructed. Analysis results of the pre-erosion conditions for Test C1 are shown in Figures 5.6 through 5.8 and results for the post-erosion are shown in Figures 5.9 through 5.11. Figures 5.6 through 5.11 show the discharge velocity contours as calculated by Slide which are equivalent to velocities where the water is flowing in the full area of the flow region and in a straight flow path. In reality, the water only travels in the pore spaces and must travel about twice the distance that is assumed by Slide because it must travel a tortuous path through the voids in the coarse sand. The assumption of a straight flowpath in the Slide models caused the calculated discharge velocities to be much lower than the actual flow.
velocities of the water on the surface of the clay. For this reason, it is necessary to adjust the estimated erosive velocities of the clay to make them comparable with the results of the Slide analyses. This was done by multiplying the erosive velocity by the porosity of the coarse sand and dividing it by 2 to account for the longer flow path of the water. The porosity can be calculated from the equation:

\[ n = \frac{e}{1+e} \]  

(5.3)

where \( n \) is the porosity and \( e \) is the void ratio (0.84). Using these equations the porosity of the coarse sand was found to be approximately 0.46. The adjusted erosive velocities then become 0.31 to 0.45 ft/s (0.09 to 0.14 m/s). The corrected critical velocities are within or near the range of the maximum water velocities calculated by Slide where the water exits the crack as shown in Figures 5.8 and 5.11. However, the corrected critical velocities are still slightly higher than those calculated in Slide in the areas downstream from the seepage barrier crack where erosion took place.

Discharge velocities calculated in the Slide models presented in Figures 5.6 to 5.11 indicate flow velocities lower than the erosive velocity calculated using Figures 2.6, 2.8, and 2.9 (1.31 to 1.97 ft/s or 0.4 to 0.6 m/s). Since erosion did occur, it can be assumed the erosive velocity for the clay material was exceeded. One reason for this discrepancy may be non-Darcian flow. The velocities calculated in Slide are based on Darcy’s Law and assume that the flow in the analyses is Darcian flow. The areas where the high water velocities in the crack jet into the coarse sand are non-Darcian flow due to the momentum and turbulence associated with the flow before it is dissipated and becomes Darcian further downstream. Therefore, the velocities in that area were not accurately calculated by Slide. Also, the crack widths in the Slide models are wider than
Figure 5.6  Pre-erosion Slide results showing discharge velocity contours for Test C1

Figure 5.7  Pre-erosion Slide results showing discharge velocity contours for Test C1 at downstream side of seepage barrier
Figure 5.8 Post-erosion Slide analysis for Test C1 showing discharge velocity contours

Max Velocity = 0.364 ft/s

Figure 5.9 Pre-erosion Slide Analysis for Test C2 showing discharge velocity contours
Figure 5.10  Pre-erosion Slide results showing discharge velocity contours for Test C2 at downstream side of seepage barrier

Figure 5.11  Post-erosion Slide analysis for Test C2 showing discharge velocity contours
those in the laboratory tests resulting in the crack velocities being lower in the Slide models. Further away from the crack where the flow becomes Darcian the Slide model is able to model the velocities more accurately.

In order to determine the water velocities in the crack, the flow rate measured during testing can be divided by the cross-sectional area of the seepage barrier crack in each test. The resulting value will give the velocity of the water flowing through the seepage barrier crack before its velocity is reduced by dispersion and being impeded by soil particles. Table 5.2 presents the flow rates for both tests and the calculated velocities for each test.

<table>
<thead>
<tr>
<th>Test</th>
<th>Crack aperture</th>
<th>Measured Flow Rate</th>
<th>Velocity (ft/s)</th>
<th>Velocity (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>0.020 in. (0.5 mm)</td>
<td>0.0033 ft³/s (9.3E-05 m³/s)</td>
<td>2.01</td>
<td>0.61</td>
</tr>
<tr>
<td>C2</td>
<td>0.039 in. (1.0 mm)</td>
<td>0.0055 ft³/s (1.6E-04 m³/s)</td>
<td>1.68</td>
<td>0.51</td>
</tr>
</tbody>
</table>

The velocities in the crack calculated from the flow rates are comparable with the range of critical velocities determined earlier. The erosive velocities in the crack suggest that as the water jetted out of the crack and into the adjacent material, erosion began taking place at the interface of the seepage barrier and the soil and propagated away from the seepage barrier until the velocities had dispersed enough to become non erosive.

5.2 Sand Results

Based on Figures 2.6, 2.8, and 2.9, the erosive velocity for the sand is expected to be approximately 0.66 to 0.98 ft/s (0.2 to 0.3 m/s). When velocities in the seepage test
cell reach values near or greater than the erosive velocity, erosion is expected to initiate and propagate in the sand.

Two tests were run on the fine sand material at 8 psi. Test S1 was run with a 0.020 in. (0.5 mm) crack aperture in the seepage barrier, and an aperture of 0.039 in. (1.0 mm) was used during Test S2. Test S1 was run for two days and Test S2 was run for three days. Figure 5.12 shows the turbidity levels verses time for Test S1 and Figure 5.13 shows the turbidity data for Test S2. Both sets of turbidity data show that the sand behaves similarly in both tests and differently than both the silt and clay materials. The turbidity levels for both sand tests goes from high to low and continue to fluctuate without settling to a base line level as observed in the clay tests (C1 and C2). This can be explained by the fine sand’s lack of cohesion which allowed the sand particles to be eroded individually rather than in clumps resulting in a more uniform erosion rate than that observed in the clay and silt.

The water flowing out from the seepage cell was run though a No. 200 sieve to collect the sand particles passing through the outflow. A relatively small amount of sand was collected on the sieve for each sand test. When compared with the turbidity levels from the silt and clay tests, the sand test turbidity levels are generally higher and might suggest that more material should have been collected on the sieve. However, the turbidity results are relative for each soil type due to their particle size. The sand particles are much larger compared to those of the silt and clay soils which means that a single sand particle passing though the turbidity meter will block more light and have a higher turbidity reading than many of the other soil particles.
The tests were carefully deconstructed and documented after each test run. Upon deconstruction both sand tests showed that the sand material was much easier to erode, but most of the eroded material was deposited soon after being eroded. In each test, a trough feature was formed parallel to the seepage barrier crack on the barrier’s downstream side and downstream of the trough a high and steep berm was formed due to the eroded sand being deposited in the interstitial voids of the coarse sand material. A much smaller trough feature was formed on the upstream side of the barrier. These formations may have been caused by the higher velocities near the crack easily eroding the non-cohesive sand but depositing them due to the sands larger grain size once the velocities dissipated and were unable to carry large amounts of sand thought the outflow. Figure 5.14 shows the features formed during Test S1 and Figure 5.15 shows a cross section view of the berm formed in Test S1. Figure 5.16 shows the features formed during Test S2 and the berm formed during Test S2 is shown in Figure 5.17. Dimensions of the erosion and depositional features formed during Tests S1 and S2 are presented in Table 5.3.

Table 5.3  Measured size and extent of erosional features for Tests S1 and S2

<table>
<thead>
<tr>
<th>Test</th>
<th>Erosion Trough depth/width</th>
<th>Depositional Berm height/width</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Upstream</td>
<td>Downstream</td>
</tr>
<tr>
<td>S1</td>
<td>0.4 in. / 1.3 in. (10.16 mm / 33.02)</td>
<td>0.5 in. / 1.5 in. (12.7 mm / 38.1 mm)</td>
</tr>
<tr>
<td>S2</td>
<td>1.0 in. / 2.0 in. (25.4 mm / 76.2 mm)</td>
<td>0.6 in. / 1.6 in. (15.24 mm/40.64 mm)</td>
</tr>
</tbody>
</table>
Figure 5.12  Turbidity versus time results for Test S1 (0.5 mm crack)

Figure 5.13  Turbidity versus time results for Test S2 (1.0 mm crack)
Measurements of the erosion formations from each test were used to construct a model in Slide to calculate velocities and flow volumes. Figures 5.18 and 5.19 present the analysis results for Test S1 before erosion occurred and Figures 5.20 and 5.21 show the analysis results after erosion occurred. Similarly, Figures 5.22 and 5.23 are the pre-erosion analysis result for Test S2 and post-erosion results are presented in Figures 5.24 and 5.25. The estimated erosive velocities for the sand material can be modified to account for the non-linear flow path as was done for the clay results. This is done similarly to that described for the clay tests, by multiplying the erosive velocity by the porosity of the coarse sand and dividing it by two. The porosity was calculated using Equation 5.3 and was found to be 0.46. The adjusted erosive velocity of the fine sand is 0.15 to 0.23 ft/s (0.046 to 0.069 m/s). The adjusted erosive velocities are within the range of the velocities calculated by Slide in Figures 5.18 through 5.25. However, the area

Figure 5.14  Erosion formations and deposits formed during Test S1
Figure 5.15  Cross-sectional view of berm formed during Test S1

Figure 5.16  Erosion formations and deposits formed during Test S2
Figure 5.17 Cross-sectional view of berm formed during Test S2

Figure 5.18 Pre-erosion Slide analysis results for Test S1 showing discharge velocity contours
Figure 5.19  Pre-erosion Slide results for Test S1 showing discharge velocity contours at downstream side of seepage barrier

Figure 5.20  Post-erosion Slide analysis results for Test S1 showing discharge velocity contours

Max Velocity = 0.351 ft/s
Figure 5.21  Post-erosion Slide results for Test S1 showing discharge velocity contours at downstream side of seepage barrier

Max Velocity = 0.353 ft/s

Figure 5.22  Pre-erosion Slide analysis for Test S2 showing discharge velocity contours
Figure 5.23 Pre-erosion Slide results for Test S2 showing discharge velocity contours at downstream side of seepage barrier.

Figure 5.24 Post-erosion Slide analysis for Test S2 showing discharge velocity contours.
where the minimum adjusted erosive velocity occurs is not large enough to account for the erosion that occurred during the test.

The discharge velocities calculated in the Slide models presented in Figures 5.18 to 5.25 are lower than the erosive velocities estimated using Figures 2.6, 2.8, and 2.9 (0.15 to 0.23 ft/s or 0.046 to 0.069 m/s). It can be assumed that the erosive velocity for the sand was exceeded during the tests since erosion did occur. Similarly to the other tests, the difference between the estimated erosive velocity and the velocities calculated by Slide can be attributed to the non-Darcian flow near the inlet and outlet of the seepage barrier crack due to the turbulence and momentum of the flow. The calculated water velocities in the barrier crack during Tests S1 and S2 from the test data are presented in Table 5.4.
The calculated seepage barrier crack velocities presented in Table 5.4 are significantly higher than the erosive velocities determined earlier. Velocities much higher than those required to cause erosion of the sand were present in the seepage barrier crack. On the upstream side of the barrier, small eroded troughs are theorized to have formed due to the flow increasing in velocity as it entered the restricted crack. The increased velocity in this location was sufficient to erode and transport sand particles through the crack and downstream until the flow was dispersed and the particles were deposited. The erosive velocities in the crack are thought to have jetted into the coarse sand and compacted sand with enough velocity, momentum and turbulent flow to erode the compacted sand immediately downstream from the seepage barrier. The eroded particles were then carried downstream until velocities dissipated to a level so that the flow was no longer sufficient to transport the sand and deposited them in the berm or further downstream from the berm. A very small portion of the eroded material was carried though the outflow and turbidity meter due to the high velocity required to transport the sand particles. The erosion that occurred adjacent to the crack on the upstream side of the barrier is theorized to have occurred due to the flow being constricted in the barrier crack. As the water flow approached the barrier crack, the water velocity increased to an erosive level. It is also possible that the water flow was turbulent immediately adjacent to the crack on the upstream side which would have increased erosion. However, the jetting action of water exiting the crack on the downstream side of the barrier was not present on the upstream side which helps to explain why the erosion on the downstream side was much more significant.
5.3 Silt Results

The erosive velocity for the silt soil is expected to be 0.33 to 0.98 ft/s (0.1 to 0.3 m/s) based on Figures 2.6, 2.8, and 2.9. Erosion is expected to begin and propagate in the silt material when velocities in the seepage test cell reach values near or greater than the erosive velocity.

<table>
<thead>
<tr>
<th>Test</th>
<th>Crack aperture</th>
<th>Measured Flow Rate</th>
<th>Velocity (ft/s)</th>
<th>Velocity (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>0.020 in. (0.5 mm)</td>
<td>0.0043 ft³/s (1.2E-04 m³/s)</td>
<td>2.62</td>
<td>0.80</td>
</tr>
<tr>
<td>S2</td>
<td>0.039 in. (1.0 mm)</td>
<td>0.0060 ft³/s (1.7E-04 m³/s)</td>
<td>1.83</td>
<td>0.56</td>
</tr>
</tbody>
</table>

Two tests were performed on the silt soil, one (Test M1) with a (0.020 in.) 0.5 mm crack in the seepage barrier and another (Test M2) with a 0.039 in. (1.0 mm) crack. Test M1 was run for just over six days and Test M2 was run for about twelve days. Turbidity verses time data for Tests M1 and M2 are presented in Figures 5.26 and 5.27, respectively. Both data sets in Figures 5.26 and 5.27 show that the silt material responds similarly in the seepage test cell during both tests. When the test first begins, there is a significant spike in the turbidity which is followed by a pattern of lower turbidity and intermittent spikes in turbidity. The initial spike in turbidity is theorized to be from loose soil particles being flushed out of the system. After the initial flushing of the system, a lower turbidity level is likely due to the cohesion of the silt which causes resistance to erosion. However, some material is still eroded and eventually a large clump of material
breaks free, is broken up and is transported through the outflow which causes the spikes in turbidity. After the large clump erodes, a lower baseline turbidity level is again achieved due to the soil’s cohesion. The lower turbidity baseline in the silt material is similar to that observed earlier during Tests C1 and C2. During the periods between the spikes in turbidity, the turbidity appears to behave similar to tests S1 and S2 were the erosion occurred much more sporadically and the turbidity fluctuated rapidly. As noted earlier, the rapid fluctuation of turbidity readings during the sand tests were likely caused by the sand particles being more easily eroded than smaller cohesive particles. The similar behavior observed in Tests M1 and M2 can likely be attributed to the sand-sized particles in the silt soil being eroded more easily than the smaller, cohesive particles. The silt tests exhibit behavior similar to both the clay and sand tests because the silt soil contains both sand and clay-sized particles. In both Tests M1 and M2, the spikes in turbidity eventually reduce in magnitude until turbidity levels become more constant.

Figure 5.26 Turbidity verses time results for Test M1 (0.5 mm crack)
Figure 5.27 Turbidity verses time results for Test M2 (1.0 mm crack)

After the tests were completed, they were deconstructed and documented. Test M1 produced a continuous trough-like feature which ran parallel to the seepage barrier on its downstream side. The feature was not as deep as those produced in the clay and sand tests and there was very little material deposited downstream to form a trough feature in contrast to the sand and clay tests. The formation is shown in Figure 5.28. Figure 5.29 shows some of the sand material that was included in the silt soil which was sorted and deposited near the downstream trough. Some material was removed from the upstream side of the barrier in places adjacent to the crack during Test M1, but it was not a measurable amount. A sieve analysis performed on the soil deposited immediately downstream from the trough feature in Test M1 found that 23% of the material was coarse sand and 77% was silt. A possible reason for the high percentage of silt material is that the berm feature was not of significant size or height as in the sand and clay tests. The material deposited in Test M1 was deposited in a shallow layer rather than a thick, high berm. Silt particles are theorized to have been deposited in a more consistent and
slower rate than in the previous testing, allowing the particles to fill voids in the coarse sand more efficiently. The thin deposit in Test M1 also made sampling the material for the sieve analysis more difficult. Any silt material scraped from the underlying compacted silt layer could greatly skew the results due to the small amount of deposited material available for analysis.

Test M2 produced features that were somewhat different than those produced during Test M1. Erosion occurred on the downstream side of the barrier where it appears that the erosion began at the barrier crack and began propagating towards the downstream end of the seepage test cell. Figure 5.30 shows the extent of the erosion that occurred during Test M2. During Test M2, enough material was eroded on the upstream side of the barrier to produce a trough like feature which ran continuously along the barrier crack. The feature is shown in Figure 5.31 and was much shallower than the one produced during Test M1 on the downstream side of the barrier. Dimensions of the erosion and depositional features formed during Tests M1 and M2 are presented in Table 5.5.

<table>
<thead>
<tr>
<th>Test</th>
<th>Erosion Trough depth/width</th>
<th>Depositional Berm height/width</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Upstream</td>
</tr>
<tr>
<td>M1</td>
<td>NA</td>
<td>0.3 in. / 1.0 in. (7.62 mm / 25.4 mm)</td>
</tr>
<tr>
<td>M2</td>
<td>0.4 in. / 0.6 in. (10.16 mm / 15.24 mm)</td>
<td>0.4 in. / 1.0 in. (10.16 mm / 25.4 mm)</td>
</tr>
</tbody>
</table>
Figure 5.28  Trough feature formed on the downstream side of the seepage barrier during Test M1

Figure 5.29  Sorting of coarser material in downstream trough occurred during Test M1
Figure 5.30  Downstream erosion features that propagated during Test M2

Figure 5.31  Shallow trough feature formed on the upstream side of the seepage barrier during Test M2
Measurements were taken of the erosion features formed during the tests were used to build models in Slide to analyze how flow velocities were affected by the formations and how they formed. Slide models were constructed for both pre-erosion and post-erosion conditions for both M1 and M2. Results from the Pre-erosion Slide analysis for Test M1 are shown in Figures 5.32 and 5.33 and the post-erosion results are presented in Figure 5.34. Pre-erosion Slide analysis results are presented in Figures 5.35 and 5.36 and post-erosion results in Figures 5.37 and 5.38 for Test M2. The velocities calculated by Slide near the seepage barrier crack for both tests are within the range of the estimated erosive velocity for the silt material (0.33 to 0.98 ft/s or 0.1 to 0.3 m/s), but are lower than the erosive velocities further downstream from the crack where erosion also occurred. The erosive velocity can be adjusted to be compatible with the Slide results as in the clay and sand tests by multiplying the erosive velocity by the porosity of the coarse sand (0.46) and dividing it by two. This is done because Slide assumes a straight-line flow path when in actuality, the flow path travels about double the distance because it must travel through the voids in the coarse sand and around the course sand particles. The adjusted erosive velocities for the silt are 0.08 to 0.23 ft/s (0.02 to 0.07 m/s). Adjusted erosive velocities for the silt material occur within the range of velocities calculated by Slide near the seepage barrier crack for both Tests M1 and M2 in Figures 5.32 through 5.38. However, the velocities do not occur in a large enough area to account for all of the features formed during each tests.
Figure 5.32 Pre-erosion Slide analysis results showing discharge velocity contours for Test M1

Figure 5.33 Pre-erosion Slide results for Test M1 showing discharge velocity contours at downstream side of seepage barrier
Figure 5.34  Post-erosion Slide analysis results showing discharge velocity contours for Test M1 at downstream side of seepage barrier

Figure 3.35  Pre-erosion Slide analysis results showing discharge velocity contours for Test M2
Figure 3.36  Pre-erosion Slide results for Test M2 showing discharge velocity contours at downstream side of seepage barrier

Figure 3.37  Post-erosion Slide analysis results showing discharge velocity contours for Test M2
The velocities calculated by Slide in Figures 5.32 through 5.38 are generally lower than the adjusted erosive velocities calculated earlier. The difference in the size of the erosive features and extent that the adjusted erosive velocities extend into those features is due to Slide calculating the velocities based on Darcian flow. Flow near the barrier crack is non-Darcian due to the momentum and turbulence associated with the flow as it jets from the seepage barrier cracks. The velocity in the crack can be calculated by dividing the flow rate measured during testing by the cross-sectional area of the crack for each test. Velocities calculated from the measured flow rates for tests M1 and M2 are presented in Table 5.6.

The calculated velocities in the crack are much higher than the estimated erosive velocities of 0.33 to 0.98 ft/s (0.1 to 0.3 m/s). High velocities in the crack are theorized to have jetted into the coarse gravel material at a velocity which exceeded the velocity of...
the silt and began the propagation of erosion near the crack. After erosion was initiated, the particles were transported in a turbid flow which propagated further erosion. Eroded silt particles were transported through the outflow. Higher velocities in Test M1 than those of Test M2 are likely the reason for the more defined erosion features which occurred in Test M1. The trough formed on the upstream side of Test M2 may have been caused by flow becoming turbulent before entering the crack. Also, the difference between the calculated velocities and estimated erosive velocity for the silt are relatively small. This difference could be attributed to an error in the estimation of the erosive velocity or a small discrepancy between the laboratory model and the finite element analysis.

<table>
<thead>
<tr>
<th>Test</th>
<th>Crack aperture</th>
<th>Measured Flow Rate</th>
<th>Velocity (ft/s)</th>
<th>Velocity (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M1</td>
<td>0.020 in. (0.5 mm)</td>
<td>0.0031 ft³/s (8.8E-05 m³/s)</td>
<td>1.93</td>
<td>0.59</td>
</tr>
<tr>
<td>M2</td>
<td>0.039 in. (1.0 mm)</td>
<td>0.0050 ft³/s (1.4E-04 m³/s)</td>
<td>1.52</td>
<td>0.46</td>
</tr>
</tbody>
</table>
Laboratory tests were performed to assess the potential for soil erosion under the scenario of a crack in a seepage barrier occurring at the interface between a highly permeable soil (coarse sand) and three different erodible soils (fine sand, silt, and clay). Results of the tests showed erosion occurring downstream of the seepage barrier crack for all three soil types and upstream of the seepage barrier crack for the more erosive soils (silt and sand). The erosion formed shallow “troughs” adjacent to the seepage barrier crack extending about 1 in. (25.4 mm) to 3 in. (76.2 mm) away from the seepage barrier crack opening. The test results also indicated that deposition occurred directly downstream of the erosion troughs, forming depositional berms in the sand and clay tests. In the sand tests, the berms were theorized to form where the seepage velocity slowed to the point where the sand grains dropped out of suspension. In the clay tests, a similar phenomenon is theorized except the deposition is thought to occur when the velocity drops below the level required to push clumps of eroded clay through the interstitial voids of the coarse sand.

Finite element analyses were performed to model the laboratory tests and estimate the seepage velocities occurring near the entrance and exit points of the seepage barrier cracks. For the silt tests, the finite element analyses accurately estimated the extent of erosion by comparing the estimated erosive velocities with the calculated velocities. For the sand and clay tests, the finite element analyses underestimated the extent of erosion observed in the tests. It is postulated that the discrepancy between the calculated and
observed erosion is due to non-Darcian flow (jetting and turbulence) caused by the high hydraulic velocities in the seepage barrier cracks.

The test results indicate that for the soil configurations tested the soil erosion is expected to be limited to a very small zone adjacent to the entrance and exit points of the seepage barrier crack. However, it should be noted that there are other soil configurations where more extensive erosion may be possible. One possible configuration would be the tested scenario inverted (erodible soil on top with coarse sand below). In such a scenario, the erosion may form a void which may collapse bringing more erodible soil from above into the erosion zone. It is possible under such a scenario that continued cycles of erosion and collapse could lead to the upward propagation of erosion and eventually result in a sink hole propagating to the surface of a dam.
REFERENCES


Soil Properties

<table>
<thead>
<tr>
<th>Soil</th>
<th>Optimum Water Content (%)</th>
<th>Dry Unit Weight (pcf)</th>
<th>Permeability (cm/s)</th>
<th>PL</th>
<th>LL</th>
<th>PI</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse Sand</td>
<td>NA</td>
<td>90*</td>
<td>7.78E-02</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Fine Sand</td>
<td>NA</td>
<td>109*</td>
<td>2.60E-03</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Silt</td>
<td>14.5</td>
<td>117**</td>
<td>1.56E-05</td>
<td>16</td>
<td>19</td>
<td>3</td>
</tr>
<tr>
<td>Clay</td>
<td>28.5</td>
<td>116**</td>
<td>3.11E-08</td>
<td>33</td>
<td>49</td>
<td>16</td>
</tr>
</tbody>
</table>

* 100% of maximum density
** 95% of maximum density as determined by ASTM D1557-91

Coarse Sand

Source: Quarry near Cove, Utah in Cache County

Preparation: Before the coarse sand was used for testing, it was washed of all fine material

Dry Unit Weight: Determined by compacting soil in a modified proctor mold and dividing the weight of the soil by the volume of the mold.

Permeability: Determined by performing a constant head test (ASTM D2434 – 68)
Fine Sand

Source: “Emerald Creek Garnet” distributed by:

Heavy Minerals Incorporated
1875 N. Lakewood Dr., Suite 201
Coeur d’Alene, ID 83814

Dry Unit Weight: Determined by compacting soil in a modified proctor mold and dividing the weight of the soil by the volume of the mold.

Permeability: Determined by performing a constant head test (ASTM D2434 – 68)
**Silt**

**Source:** Southwest of Emmett, ID in Gem County. Approx. 43°52’40’’N 116°42’23’’W

**Preparation:** Before the coarse sand was used for testing, organics were removed and clumps were broken up with a sieve.

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

![Compaction curve for silt soil](image)

**Permeability:** Determined by performing a permeability analysis with Digi Flow Pumps following manufacturer’s (GEOTAC) process.

**GEOTAC**
6909 Ashcroft Dr. STE 104
Houston, TX 77081

**Atterburg Limits:** Determined following ASTM D4318-10
Clay

Source: Kaolinite Clay obtained from the Utah State University Art Department, mined near Lewiston, Utah in Cache County.

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

Compaction curve for clay soil

Permeability: Determined by performing a permeability analysis with Digi Flow Pumps following manufacturer’s (GEOTAC) process.

GEOTAC
6909 Ashcroft Dr. STE 104
Houston, TX 77081

Atterburg Limits: Determined following ASTM D4318-10