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Laboratory Modeling of Erosion Potential of Seepage Barrier Material

Nathan E. Braithwaite
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LABORATORY MODELING OF EROSION POTENTIAL
OF SEEPAGE BARRIER MATERIAL

by

Nathan E. Braithwaite

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

Approved:

Dr. John D. Rice  Dr. Joseph A. Caliendo
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UTAH STATE UNIVERSITY
Logan, Utah

2013
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ABSTRACT

Laboratory Modeling of Erosion Potential of Seepage Barrier Material

by

Nathan E. Braithwaite, Master of Science
Utah State University, 2013

Major Professor: Dr. John D. Rice
Department: Civil and Environmental Engineering

Seepage barriers have been used extensively to mitigate seepage problems in dams and levees. Although the designs of many of these dams and levees have been based on intact seepage barriers, seepage barriers have been shown to be susceptible to deformation and cracking when high differential hydraulic pressures act across the barrier. Cracking and deformation have also been observed due to thermal expansion and contraction during seepage barrier curing. Under certain conditions, a crack 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 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 itself. Previous studies have investigated the first mode of erosion and studies are underway to look into the second mode. The objective of this study is to investigate the third mode of erosion and to
identify the conditions under which serious seepage problems can develop. The question considered was whether the combination of highly permeable material adjacent to a crack in a seepage barrier and a large differential head across the barrier combine to develop a velocity within the crack that is erosive to the seepage barrier material. Laboratory tests have been performed on a variety of seepage barrier materials to assess the potential for cracks to develop a preferred seepage path leading to a serious seepage problem. The results of this study will be useful in risk assessment studies of dams and levees with existing seepage barriers as well as in the design of new seepage barriers. Having knowledge of the conditions under which problems may occur will aid in the selection of seepage barrier types for new barriers, placement of instrumentation to monitor new and existing barriers, and mitigation of existing barriers where problems have been identified. The data provided will assist engineers in quantitatively assessing the potential for the propagation of critical seepage problems from cracks in seepage barriers.

(106 pages)
PUBLIC ABSTRACT

Laboratory Modeling of Erosion Potential of Seepage Barrier Material

by

Nathan E. Braithwaite, Master of Science
Utah State University, 2013

Major Professor: Dr. John D. Rice
Department: Civil and Environmental Engineering

Earthen dams and levees often use seepage barriers to reduce the flow of water through their foundations and embankments. A seepage barrier is a wall of less porous material built inside an embankment or its foundation. High water pressures and stresses during the curing of seepage barriers have been observed to cause the seepage barriers to crack. These cracks decrease the barrier’s effectiveness in reducing seepage flow and may lead to serious seepage problems. The purpose of this study is to determine, given highly porous soils and high water pressures, whether or not cracks in seepage barriers will erode and enlarge, thus progressing into a potentially dangerous seepage problem. Tests have been performed on several seepage barrier materials to determine their susceptibility to erosion. The seepage barrier materials that were found to be erosive were then tested to observe the effects of the surrounding soil on the erosion of the barrier materials. Having knowledge of the conditions where problems may occur will aid not only in the selection of barrier types for new barriers and the placement of
instrumentation to monitor new and existing barriers, but also in repairing existing barriers where problems have been identified. The data provided will help engineers assess the possibility for problems to develop from cracks in seepage barriers.
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Nathan E. Braithwaite
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CHAPTER 1
INTRODUCTION

1.1 Seepage Barriers

Seepage barriers of various types have been used extensively over the past few decades to mitigate seepage problems in dams and levees. Common barrier types include: slurry walls, secant pile walls, jet-grouted walls and deep soil mixed walls. In most cases the seepage barriers have helped reduce the flow of water through the pervious portions of the embankment or foundation thus decreasing the probability of seepage related problems and increasing the reliability of dams. However, it is important to recognize that the addition of a seepage barrier radically changes the seepage paths, hydraulic gradients, and hydraulic pressures in a dam and its foundation. The result is increased water pressures and concentrated hydraulic gradients through and around the barrier. The differential pressure that develops across the barrier introduces forces on the barrier that may cause large bending moments resulting in barrier cracking at locations where there is a significant difference between the deformation characteristics of the adjacent soil or rock layers and the seepage barrier (see Figure 1, Rice and Duncan, 2010b). Seepage is

![Figure 1 - Deformation of seepage barriers (Rice and Duncan, 2010b).](image-url)
proportional to the hydraulic gradient so any crack or defect under, around, or through the seepage barrier experiences much higher flows than before the seepage barrier was built. The dam may therefore be more susceptible to internal erosion.

Cracks have been known to form due to thermal expansion and contraction during seepage barrier curing. Cracks due to thermal contraction in deep mixed barriers are shown in Figure 2. Direct evidence (observed cracks) and indirect evidence (poorer than expected barrier performance) have been observed in case studies of seepage barriers in existing dams (Rice and Duncan, 2010a). The direct result of such cracks is to decrease the effectiveness of the barrier in retarding seepage flow.

The high hydraulic gradients across the barrier and resulting high seepage velocities within the cracks in the barrier combined with high permeability in adjacent soils may lead to erosion of either the barrier material itself or the soil surrounding the crack. Such erosion may in turn lead to the development of low-resistance, preferred seepage paths through the embankment. Concentrated seepage along a preferred seepage path may produce additional erosion and the eventual development of a more serious seepage problem. This process may develop soon after construction of the barrier, upon

![Figure 2 – Observed seepage barrier cracks in soil-cement-bentonite.](image)
first hydraulic loading of the barrier, or over a long period of time. Until now, no methods existed to quantitatively assess the potential for degradation of barrier performance due to these processes.

1.2 Internal Erosion

Internal erosion occurs when soil particles within an embankment dam or its foundation are carried downstream by seepage flow. Internal erosion is a broad subject and often misunderstood. It is a difficult process to model; therefore, in the past not much effort has been given to internal erosion research. In reality, however, internal erosion is responsible for roughly half of all dam accidents and failures. This makes it a problem comparable in magnitude to overtopping of embankments (ICOLD, 2012).

In recent years internal erosion has gotten much more attention. Considerable time and money have gone into understanding it, but there is yet much to be learned. This study focuses on understanding the erosion of seepage barrier materials along cracks. Greater detail is given in the literature review in Chapter 3 about internal erosion and some of the processes by which soil particles are carried downstream.

1.3 Purpose of Research

Previous studies have identified several scenarios where there may be potential for erosion to occur within or adjacent to a crack in a seepage barrier (Rice and Duncan, 2010a, 2010b). The question considered in this study is whether the combination of highly permeable material adjacent to a crack in a seepage barrier and a large differential head across the barrier combine to develop a velocity within the crack that is erosive to
the seepage barrier material itself. Laboratory tests were performed on a variety of seepage barrier materials in order to assess the potential for cracks to develop into a preferred seepage path leading to a critical seepage problem. This study is divided into two phases. The first phase is entitled “The Erosion Test Cell.” It assesses the erodibility of seepage barrier materials. The second phase is entitled, “The Seepage Test Cell.” It deals with the interaction between a cracked seepage barrier and the adjacent soil and/or bedrock.

1.4 Report Organization

This thesis is comprised of six chapters. Chapter 1 states the purpose of the research. Chapter 2 summarizes construction and properties of the various types of seepage barrier materials in use and the mechanisms by which they may crack. Chapter 3 presents an overview of the literature reviewed relating to this study. It discusses existing methods for determining the erosive characteristics of materials. Chapter 4 is a presentation and discussion of the Erosion Test Cell used in Phase 1 of the testing and it presents the test results. Chapter 5 is a presentation and discussion of the Seepage Test Cell used in Phase 2 of the testing with a presentation of the results. Chapter 6 presents the conclusions.
CHAPTER 2
SEEPAGE BARRIER CONSTRUCTION

2.1 Seepage Barrier Construction

A seepage barrier in this study is defined as a low-permeability hydraulic barrier that is narrow with respect to its depth and lateral extent. A brief discussion of the types of seepage barriers and their methods of construction is presented to provide a better understanding of the physical characteristics of the various types of seepage barriers and their interactions with adjacent soil. Further discussion concerning seepage barrier cracking is presented, including discussions on the locations where barriers typically crack and the mechanisms that drive cracking.

2.2 Seepage Barrier Material Properties

Six main types of seepage barrier materials are commonly used: (1) cement-bentonite, (2) soil-cement, (3) soil-bentonite, (4) soil-cement-bentonite, (5) conventional concrete, and (6) plastic concrete. The type of soil present and the desired properties of the barrier affect the choice of which material to use. The barrier deformation properties often should be similar to those of the surrounding soil to prevent stress concentrations and potential of the seepage barrier or soil.

A cement-bentonite barrier is a self-hardening slurry wall where a trench is filled with cement-bentonite slurry that is left to harden in place. It is used where a highly impervious barrier is wanted but where strength is not needed. It is a very weak material
with compressive strengths generally ranging from 15 to 90 psi. It deforms readily and any cracks that form, if small, often reseal themselves.

Soil-cement and soil-cement-bentonite barriers are typically built using the deep mix method but they can also be built using the slurry wall method. In the slurry wall method, the excavated soil is mixed with cement, bentonite, or cement and bentonite beside the trench or sometimes in a pug mill. It is then placed back into the trench. The deep mix method mechanically mixes the soil without excavation while injecting a slurry of water and cement or bentonite or both. Compressive strengths range widely from less than 500 psi to more than 1500 psi. Strength depends on the soil properties and the amount of cement and bentonite added.

Conventional concrete barriers can be built using the slurry method where a tremie pipe fills the trench from the bottom up. The jet grouting and secant pile methods can also be used with conventional concrete to build the barrier. Conventional concrete is used if high strengths are desired. Compressive strengths can range from 3000 to over 4000 psi. Plastic concrete barriers are similar to conventional concrete barriers except that bentonite is added as well. Bentonite makes the barrier weaker (around 1000 psi) but less permeable.

2.2.1 Slurry Walls

The first step in building any type of slurry wall seepage barrier is to dig a trench. The typical trench is around 3 feet wide but can range from 1 foot to 8 feet wide. Trenches can be dug using a backhoe to depths of up to 90 feet as seen in Figure 3. For deeper trenches a clamshell digger or a hydromill is needed. Photos of each are shown in Figure 4. Hydromills are also used if the soil contains large rocks or if it is desired to set
the wall into bedrock. Seepage walls have been built to a depth of over 400 feet. With a clamshell digger and Hydromill a seepage wall can be built in panels instead of in one large trench. The trench can be left open if it is a shallow cut or if the soil conditions allow it. With deeper trenches or with soils susceptible to collapse, trenches may be filled with bentonite slurry to prevent collapsing and limit ground water flow into the trench. The slurry is prepared in an on-site batch plant and pumped to the trench via pipes and/or hoses.

Three types of slurry wall backfill mixtures are commonly used: self-hardening slurries, soil-based backfills, and conventional concrete or plastic concrete. Self-hardening slurries typically contain water, clay (bentonite or attapulgite) and cementitious materials (Portland cement or slag cement). The soil based backfill mix is primarily comprised of the excavated soil, a borrow soil, or a combination of both mixed with clay, cement, or a combination of both. Plastic concrete is conventional concrete mixed with bentonite.

Figure 3 – View of a fully extended CAT 375L “long boom” excavator (USACE, 2013).
Backfill materials are introduced into the trench by one of three methods. The first method is to use trenching slurry that is self-hardening thereby serving as both the trench support and the backfill. Soil based backfills often mechanically mix bentonite and cement with the soil at the surface using a bulldozer, and then push it back into the trench. Considerable room is required next to the trench when using this method. Conventional concrete or plastic concrete can be used as backfill by inserting a tremie pipe at the bottom of the trench and filling it from the bottom to the top. The bentonite slurry is pumped out as the trench is filled with the seepage barrier material (Andromalos and Fisher, 2001).

There are many factors to consider when planning which slurry wall and method to use: the desired permeability, cost, constructability, strength, density, trench stability, compatibility with the surrounding soil, whether the on-site soil can be used or not,
availability of off-site backfill material, the available work area space, and whether to construct the slurry wall in panels or in one continuous trench.

2.2.2 Deep Mix Method

The Deep Mix Method mixes reagents with soils at depth to improve soil properties in-situ without excavation or removal. With granular soil, the soil is the aggregate and the cement-grout is the binder and hardening agent. Bentonite may also be used for lower permeability. The result is a continuous wall of mixed material.

Using the Deep Mix Method, treatment is possible to depths of up to 130 feet. The most common construction method is performed with a powerful drill based on a tracked crane advancing hollow stem augers with radial mixing paddles attached near the bottom. Figure 5 shows a triple auger system. Grout is pumped from the mixing plant through the hollow stems of the mixing tools and injected into the soil at the tip of the tool. The mixing tool flights and mixing blades on the stem blend the soil with the grout in continuous fashion. The mixing process is repeated during the withdrawal stage. Tool designs are frequently tailored to meet specific project conditions. A single or multi-auger configuration may be used, usually up to four augers (US DOT FHWA, 2006).

The deep mixing method works best in clean sands and gravels. And it works fine in silt or silty sand. It also used often in clayey soil but has some concerns due to the difficulty of obtaining a uniform clay-grout mixture. Undesirable lumps of soil are bigger and more frequent in a seepage barrier when working with finer and more plastic soil. Deep Mixing is also not as effective in coarse gravel, cobbles, or boulders due to destruction of the augers but it is still possible. Stiff soils and obstructions are sometimes predrilled ahead of the soil mixing process.
Wet soil mixing is best suited for soils with moisture contents up to 60 percent. If the moisture content is greater than 60 percent, dry soil mixing may be more economical (Hayward Baker, 2012). Dry soil mixing uses air pressure to inject powdered cement-grout instead of wet grout.

Soils vary widely in their ability to be mixed depending on the soil type, strength, water content, plasticity, stratigraphy, and texture. Almost any soil type, including organics, can be treated with wet soil mixing although some soils may require significant binder and/or pretreatment. Other factors controlling the properties of the soil cement: rate of penetration and withdrawal; mixing energy (RPM); the auger and mixing paddle configurations; the method of injection; and the properties, quality and amounts of cement.

Figure 5 – Deep Mix Method augers (USACE, 2013).
An automated batching system measures the water, cement, and other additives by weight which makes for consistent slurry. However, a batch plant includes a lot of equipment which drives up the mobilization cost. Consideration must also be given to handling the excess soilcrete known as “spoils.” Depending on the soil type, spoils generated may range from 10 to 40 percent of the treated volume. If possible, the spoils can be used elsewhere on the site.

Figure 6 shows a fairly new modified method of deep mixing called the TRD method. TRD stands for ‘Trench cutting and Remixing slurry Diaphragm wall.’ TRD soil mix walls use a specialized vertical cutter post mounted on a base crawler machine. The vertical cutter post resembles a large chain saw and is inserted vertically in segments by the crawler machine until the design depth of the wall is reached. The crawler machine then advances along the wall alignment while the cutter post cuts and mixes the in-situ soil with cement-based and bentonite-based binder slurry injected from ports on the post. The vertical mixing action blends the entire soil profile eliminating any stratification and

Figure 6 - TRD soil mix wall (USACE, 2013).
creating a homogenous soil mix wall with extremely low permeability (Hayward Baker, 2012).

The only real negative of the TRD method is the high cost of mobilization making this option possible only for very large projects. Again, consideration must be given for the handling and disposal of spoils.

2.2.3 Jet Grouting

Jet Grouting uses high-pressure, high-velocity jets of water, air, and grout to break down, mix and partially replace the in-situ soil or weak rock with a cement-based grout in order to create an engineered soil-cement product of high strength and low permeability. Figure 7 is a picture of water shooting out the jets. Soil not removed becomes mixed with the grout in-situ to form a treated mass. The result, when set, is usually termed soilcrete. Jet grouting can be performed above or below the water table and in most subsurface stratigraphies from cohesionless sands and gravels to highly plastic clays. Caution is needed in cohesive soils since, with the high pressures involved, heaving of the soil is possible. Interconnected overlapping columns are constructed in

![Figure 7 – Jet grouting jets spraying water (USACE, 2013).](image)
continuous rows in a primary/secondary sequence to create an impervious barrier.

The three basic systems of jet grouting in general use are: single-fluid, double-fluid, and triple-fluid. In the single-fluid system the fluid is the grout, and the high-pressure jet (up to 7200 psi) simultaneously erodes the soil and injects the grout. It involves only partial replacement of the soil.

In the double-fluid system the high-pressure cement jet is shrouded inside a compressed air cone. This system produces a larger column diameter than the one-fluid system and gives a higher degree of soil replacement, although often lower strength due to air entrainment (USACE, 2013).

The triple-fluid system uses upper ejection of high-pressure water (4400-7200 psi) inside a compressed air envelope for excavation with a lower jet that is usually set at a lower pressure emitting grout to replace the slurried soil. This system gives the largest diameter and replaces most of the soil thus giving the highest strength. It is also the most expensive of the three jet grouting techniques.

Jet grouting requires a lot of equipment, much of which is purpose built or specially adapted. Figure 7 shows the equipment used. The main components include: a silo; water supply; high-speed/high-shear colloidal mixer; agitator tank; a pump with precise pressure and volume control; grout parameter recording equipment; and grout lines with headers, packers, gauges, and valves.

The large amount of specialized equipment along with having to replace the entire wall with grout makes jet grouting much more expensive compared to other methods, yet it produces a more uniform and better product. As usual, consideration must be given to handling properly the large amount of spoils (up to 100%).
2.2.4 Secant Pile Walls

Secant piles are drilled shafts backfilled with concrete or plastic concrete constructed so that there is an intersection of one pile with another. The usual practice is to construct alternate piles along the line of the wall leaving a clear space of a little under the diameter of the required intermediate piles. The normal diameter of the secant piles is 36 inches with a center to center distance between the piles of 30 inches. Therefore, each pile overlaps with the adjacent piles as shown in Figure 8.

Concrete backfill is added and, before it has fully set, the secondary holes are drilled. The secondary piles are positioned between the primary piles and secant with, or overlapping, the primary piles. This allows the secondary holes to cut into the first piles forming an interlocking joint. Sequenced drilling and concreting of the individual cylinders allows the concrete to cure ensuring a tight seal between the cylinders for complete water cutoff. Secant piles can be constructed to depths that exceed 150 feet.

Secant piles can be constructed either with conventional drilling methods or through the use of continuous flight auger techniques. Secant pile walls can include reinforcement which generally consists of rebar cages or steel beams.

![Figure 8 - Secant pile drilling configuration to provide overlap.](image-url)
2.3 Seepage Barrier Cracking

Seepage barriers cause a large change in the seepage regime which introduces forces that can deform the seepage barrier. Cracks are formed in rigid seepage barriers very differently than in the softer soil-bentonite barriers. Previous studies have shown that rigid seepage barriers are most likely to deform and crack at interfaces between rigid soils or bedrock and the overlying alluvium or embankment fills. Soil-bentonite barriers, however, readily deform without cracking (Rice and Duncan, 2010a). Also, cracks may form in rigid seepage barriers due to thermal expansion or shrinkage whereas soil-bentonite barriers are not affected by these mechanisms. Defects may be present if the barrier was built in panels or individual piles. If not constructed properly, cold joints may form open cracks between panel sections or individual piles.

The soil-bentonite barriers may crack due to post construction settlement. The barrier material may separate from the overlying soil or barrier cap due to settlement of the infill, thus creating a crack. Also, settlement of the infill may drag on the trench walls as it settles thus reducing the stress in the backfill making the seepage wall susceptible to hydraulic fracture.

Cracks in seepage barriers result in performance different from that predicted by the design analysis or performance that significantly changes over time. The observed permeability of cracked seepage barriers can be up to three orders of magnitude greater than the predicted permeability depending on crack aperture and surrounding soil. The construction of seepage barriers causes the water pressure to increase upstream of the barrier and decrease downstream. The increased hydraulic gradients tend to concentrate
the seepage forces: (1) in the soil surrounding the seepage barrier, (2) through defects in the foundation, and (3) through any cracks or defects in the seepage barrier itself.

Under certain conditions, erosion of the seepage barrier material or the surrounding soil may be possible through cracks in the seepage barrier. Such conditions include: erodible seepage barrier material or surrounding soil, a gradient large enough to produce the flow velocities required to erode the barrier material or adjacent soil, and the existence of an unfiltered exit through which the eroded soil can leave. Figure 9 shows some highly permeable soils that may allow high flow velocities and provide an unfiltered exit. They include karstic or solutioned limestone, fractured bedrock, and sandy or gravelly soil. Further discussion of the erodibility of seepage barrier material through cracks in the barrier is given in the literature review in the following chapter.

Figure 9 - Unfiltered exit points: karst, extremely pervious soil, fractured bedrock.
CHAPTER 3
LITERATURE REVIEWED

3.1 Introduction

Literature about the performance of cracked seepage barriers is very limited. A literary review of relevant prior research was performed on topics related to this research. The subheadings in this chapter are citations for the literature reviewed. This chapter is divided into the following areas:

- Seepage barrier cracking
- Internal erosion
- Erosion test cell
- Seepage test cell

3.2 Seepage Barrier Cracking

These papers are based on the dissertation of Dr. Rice (Rice, 2007) and are published in the Journal of Geotechnical and Geoenvironmental Engineering. They summarize the study of more than 30 dam case histories of dams that had seepage barriers in place for over 10 years. Finite-element seepage and deformation analyses were also performed to better understand seepage barrier performance.

The case histories show that in most cases seepage barriers increase the reliability of dams. However, consideration must be given to the possibility of undiscovered mechanisms introduced by a seepage barrier that can affect the long term performance of the seepage barrier.
As stated in Chapter 1, seepage barriers increase the pore pressures, hydraulic gradients, and flow velocities in defects in and around the seepage barrier. Several dams showed a change in piezometric head and an increase in seepage flow. Many of the case studies showed the existence of post construction cracking. In some cases the hydraulic conductivities were as much as 3 orders of magnitude different than anticipated.

Finite-element analysis shows the relationship between crack aperture, permeability of the seepage barrier, permeability of the surrounding soil, and flow volume. As seen in Figure 10, the flow at small crack apertures is controlled by the permeability of the crack. With larger crack apertures, flow is controlled by the permeability of the surrounding soil to the point that crack aperture becomes insignificant. Figure 11 also shows that as the permeability of the surrounding soil increases the crack aperture has a greater effect on the flow in the crack.

If the surrounding soil is permeable enough to allow erosive velocities in the crack, the crack aperture will increase. Figure 11 shows that as a crack widens the
The velocity initially increases because the flow is controlled by what the crack can handle. Later, with greater crack apertures, the velocity decreases due to the surrounding soil controlling the flow. Therefore, even if a crack could erode under a certain gradient, the crack width would initially increase but, at some point in time, the width would stabilize as the water velocity decreased in the crack. Also, when a large enough crack width is reached, surrounding soils would migrate to the opening and plug the crack (Rice and Duncan, 2010a).

The question is how much erosion will occurred before equilibrium is reached? Several cases have been identified where significant erosion can occur and possibly lead to a more serious seepage problem. The first situation is where the surrounding soil erodes through the crack and is washed away as shown in Figure 12a. This allows the surrounding soil to have more flow thus increasing the flow velocity in the crack and allowing for further erosion of the barrier material. Figure 12b shows the case where the surrounding soil is already coarse enough to allow significant erosion of the barrier.

Figure 11 - Crack aperture vs. flow velocity in a crack (Rice and Duncan, 2010a).
material. Figure 12c is the situation where the crack aligns with bedrock joints which allow large flows. If the flow velocity that causes erosion is very low, enlargement of the seepage barrier crack is possible.

Phase I of this study seeks to determine the erodibility of several types of seepage barrier materials and the flow velocity at which erosion occurs.

3.3 Internal Erosion

3.3.1 ICOLD (2012)

This bulletin deals the mechanics of internal erosion. It recommends quantitative risk assessment to assist in decision making in response to the uncertainty of internal erosion. It was also intended to serve as an additional source to help in assessing high hazard dam cases and high cost remediation.

In the past, the method of erosion called piping has been separate from internal erosion. This bulletin has combined the terms piping and internal erosion into one. They will both be considered “internal erosion” from this point on. Internal erosion occurs when soil particles within an embankment dam or its foundation are carried downstream
by seepage flow. Internal erosion initiates when the erosive forces imposed by the hydraulic loads exceed the resistance of the materials to erosion. The erosive forces are directly related to reservoir water level. Internal erosion if left unchecked may lead to accidents, incidents, or even failures in embankment dams. There are similar numbers of accidents relating to internal erosion in the foundation as in the embankment.

The four mechanisms through which internal erosion is initiated are erosion in concentrated leaks, backward erosion (piping), contact erosion, and suffosion. These four terms may be simplified into two categories: (1) inter-granular seepage flow, and (2) preferential flow paths. Sometimes both inter-granular seepage flow and preferential flow paths are to blame for an internal erosion problem.

Inter-granular seepage flow deals with the progressive removal of soil particles from a mass by percolating water through large voids in the soil, leading to the development of channels. Fine sand particles and silts with low mass are most susceptible. Preferential flow paths involve water flowing through a crack or defect and eroding the soil from the walls of the crack or defect. If the eroding water has enough velocity to continue to erode the soil in contact with the crack, the crack will enlarge from the erosion.

Both erosion categories need: (1) a source of water, (2) an unprotected exit from which material can escape, and (3) erodible material within the flow path. The higher the reservoir level, the higher the gradients and flow velocities will be. Erodibility and flow velocity are the most important factors to consider. Dispersive clays, low plasticity silts, and sands with enough fine particles to support an open crack are especially susceptible to erosion. Unprotected exits may include: large solution openings in gypsum and
limestone, fractured bedrock, and coarse granular embankment fills as shown in Figure 10 of the previous chapter.

When constructing new dams, protection against internal erosion is achieved by zoning and by providing filters. Filters designed to intercept internal erosion pathways must form a complete curtain that cannot be circumvented if they are to be effective. Many existing dams are not adequately zoned and do not have filters and may therefore be vulnerable to internal erosion.

Older dams that may not have a filter or that have had seepage issues can be helped by the construction of a seepage barrier. Seepage barriers can help control internal erosion. However, as stated in previous sections, seepage barriers can crack under certain circumstances. The soil near the crack and the seepage barrier material itself may be susceptible to internal erosion.

Many internal erosion problems become apparent on first filling as weaknesses in the dam or its foundation are revealed by the rising water. However, most dam accidents and failures occur many years after first filling as internal erosion continues to be a threat. Aging causes deterioration which can initiate internal erosion. For example, cracking may occur as a result of repeated reservoir draw downs, differential settlement, or desiccation. Or the reservoir may see greater gradients and seepage flow velocities than it has experienced before due to increased reservoir levels. Some dams were not designed to resist extreme loads such as those presented by very high water levels and earthquakes. And they may not be protected against internal erosion by filters or, if filters or transition zones are present, they may not be designed to modern standards and may therefore be ineffective.
Statistics of dam failures for large dams constructed between 1800 and 1986 show that internal erosion has been responsible for about 50% of the embankment dam failures where the mode of failure is known. By comparison, embankment slides account for only 4%, and failures due to earthquakes only 1.7% of embankment dam failures. The incidence of dam failures due to internal erosion is approximately equal to failures caused by overtopping in floods due to inadequate spillway capacity or the malfunction of gates and other outlets.

To avoid dam and levee accidents and failures associated with internal erosion, more attention must be focused on it. This study improves our understanding of mechanisms that initiate internal erosion making it possible to better assess the ability of dams and levees to resist such erosion. It also helps identify where problems may occur that need monitoring and/or remediation, and suggests methods of remediation.

3.3.2 Fell et al. (2003)

This paper deals with the time of progression of internal erosion based on the dam and foundation characteristics. Four stages are mentioned in this progression: initiation, continuation, progression and breach. Initiation of internal erosion is difficult to detect. Well placed and well maintained monitoring equipment is needed if any signs of internal erosion are to be detected (Fell, Wan, Cyganiewicz, and Foster, 2003).

A table is presented that summarizes the means by which erosion may initiate. Proper filters and transition zones are discussed. The criteria needed for the continuation and progression phases are discussed as well as ways in which breaching may occur.
3.3.3 McCook (2004)

This study combines the various mechanisms of soil erosion into two terms: “piping” and “internal erosion.”

Piping is defined as erosion of soil particles due to percolation of water through a soil body with no preferential seepage path. The most important condition affecting a soil’s susceptibility to piping is the hydraulic gradient at the point where soil can be eroded. Fine-grained, poorly graded sands are most susceptible to piping whereas clays and large-grained soils have a high resistance to piping.

Internal erosion is described as the removal of soil particles due to flow in concentrated pathways. The velocity of the water flowing through the pathway determines if internal erosion will occur. Soils that are able to support an open crack without collapsing are most susceptible to internal erosion. Such soils include dispersive clay and low plasticity silt.

McCook (2004) also describes several scenarios where piping and internal erosion can develop and progress to failure as well as measures that can be taken to reduce the risk of piping and internal erosion.

3.3.4 Wan and Fell (2008)

This paper focuses on the internal erosion topic of suffusion, which is: fine soil particles being moved by seepage forces through the voids in larger soil particles. Included are the criteria needed for suffusion to occur. Consequences of suffusion are mentioned including: ending up with a coarser soil that is more permeable, settlement of the embankment, and slope instability.
The Wan and Fell’s paper proposes more lab testing methods for predicting internal instability than previously existed. Testing was done using old and new methods.

The old method conclusions of Sherard (1979), Kenney and Lau (1985, 1986), and Burkenkova (1993) were found to be conservative.

3.4 Erosion Test Cell (Phase 1)

3.4.1 Wan and Fell (2004)

Many apparatuses exist for determining the erodibility of soils. Not all are comparable because of the several different mechanisms that cause erosion. For example, the Jet Erosion Test (JET) simulates back-cutting erosion with water plunging over a vertical face. The pinhole erosion test (Sherard, 1979) identifies the dispersivity of soils. Other tests such as the Hole Erosion Test (HET) and the Slot Erosion Test (SET) assess the critical hydraulic shear stress at which erosion initiates and the rate of erosion. This study seeks to identify erosion by this same mechanism.

The HET is conducted in a laboratory using an undisturbed tube sample or a soil specimen compacted into a Standard Proctor mold as shown in Figure 13. A 6mm diameter hole is pre-drilled through the center-line axis to simulate a concentrated leak, and the specimen is installed into a test apparatus in which water flows through the hole under a constant hydraulic head that is increased incrementally until progressive erosion is produced. The downstream water head is set at 100mm. Once erosion is observed, the test is continued at a constant hydraulic head for up to 45 minutes, or as long as flow can be maintained. Measurements of the increasing flow rate during the test and the initial and final diameter of the erosion hole are used to compute hydraulic stress and the rate of
erosion. Significant post-test work is needed to obtain the measurement of the final diameter of the hole (oven-drying, casting of a plaster mold of the eroded hole, caliper measurement of the mold diameter at several locations).

The rate of erosion per unit surface area ($\epsilon_t$) at time $t$ is plotted against the hydraulic shear stress ($\tau_t$) on the surface of the hole at time $t$. The slope of the best fitting line of the data is the coefficient of soil erosion ($C_e$). $C_e$ is used to calculate the Erosion Rate Index ($I$) as shown in Equation 1. Typical values of this index range from 1 to just above 6. Larger values indicate increasing erosion resistance. Soils with a number less than 2 are usually so erodible that they cannot be effectively tested in the HET device. The relationship is given by the equation:

$$I = -\log (C_e)$$  \hspace{1cm} \text{Equation 1}

The SET is a laboratory test with a setup similar to that of the HET except a much larger soil specimen is used. As shown in Figure 14, the soil is compacted inside a 0.15 m wide by 0.1 m deep by 1 m long aluminum sample box. The specimen is tested at a hydraulic gradient of 2.2. For the test a 2.2 mm wide by 10 mm deep by 1 m long slot is formed along one surface of the soil sample. The preformed slot is in contact with a transparent Perspex cover plate of the sample box through which erosion of the slot can be observed during the test. Water is passed through the slot in the soil sample to initiate erosion. The width of the slot is measured at chosen time intervals as it is widened by erosion. Equations similar to those for the HET enable estimation of $C_e$.

There is a strong correlation between test results from the HET and the SET. The HET is much easier and less expensive to perform and requires less soil. However, in the
SET a larger sample can be used which gives more accurate results. Also in the SET the erosion process can be observed through the Perspex face.

Critical shear stress can be obtained from the HET and the SET by extrapolation of the data but there is a large degree of variation among different specimens of the same soil. Results of HETs show the tendency of the coarse-grained soils to have lower critical shear stress values than the fine-grained soils, and that the critical shear stress values of the fine-grained soils increase as their Erosion Rate Index increases.

The Erosion Rate Index is influenced strongly by the degree of compaction and the water content of a soil. In most of the soil samples tested, a specimen compacted to a higher dry density and on the wet-side of the optimum water content has a higher erosion resistance than another specimen of the same soil compacted to a lower dry density and on the dry-side of the optimum water content. Some coarse-grained, non-plastic soil samples show the highest Erosion Rate Index when compacted to a high dry density and on the dry side of optimum. Predictive equations are proposed for estimating the Erosion
Rate Index values for coarse-grained soils and fine-grained soils. These equations can be used to assess existing dams, where it may be difficult to get soil samples.

The shear stress corresponding to initiation of erosion in an HET can be used in qualitative terms to assess the likelihood of initiation of piping erosion in an embankment dam. The Erosion Rate Index values obtained from an HET will give a guide as to how quickly a pipe will develop in a dam.

3.4.2 Garand et al. (2006)

This paper discusses the need for a better understanding of plastic concrete’s resistance to erosion in high velocity water flow. Some defects in seepage barriers may be caused by open joints between panels or tensile and shear cracks. Erosion might gradually enlarge these openings to dimensions such that the efficiency of the cutoff would be diminished substantially.
The Pin-Hole Test and Water Jet Test are qualitative tests. They can only express a trend and the measured parameters cannot be applied to full scale problems with a plastic concrete cutoff. Also the presence of gravelly aggregates in the plastic concrete can bias small scale tests designed to investigate the behavior of the material.

The writers of this paper designed a Controlled Water Velocity (CWV) test to test the erosion resistance of a representative area of plastic concrete in high velocity water. The sample tested was 150 mm (5.9 in) in diameter and 300 mm (11.8 in) long. The test allowed for flow velocities up to 14 m/s (46 ft/s) and lasted up to one month per test. The hydraulic gradients were up to 133. An opening of 30 mm (1.2 in) was selected to permit the largest aggregate to be plucked out of the concrete mass and easily washed away. The specimens were removed from the test bench at regular intervals to assess the progress of the erosion. Erosion merely flattened bulges and eroded the cement-bentonite binder paste which is similar to our results.

This study yielded an empirical mathematical relationship between: the gradient applied on the cutoff, the permeability of the surrounding soil, the crack width, and the water velocity in the crack. This relationship enables the definition of a safety factor expressed as the ratio of the critical water velocity that produces sample degradation divided by the actual water velocity in the crack. The plastic concrete showed good resistance to water erosion and the tests proved that it can sustain large gradients and high water velocities in a crack or discontinuity. Even if the surrounding soils are very permeable the water velocity in cracks of 1 cm to 10 cm width could not reach a critical velocity for plastic concrete.
3.5 Seepage Test Cell (Phase 2)

Laboratory tests were performed to assess in turn the potential for soil erosion in the scenarios of a crack in a seepage barrier occurring at the interface between a highly permeable soil and three different erodible soils. The study also included finite element analysis of a crack in seepage barriers to estimate the flow velocities in and around the crack. Both the lab model and the finite element analysis show the development of small troughs adjacent to the cracks. The test results indicate that for the soil configurations tested, the soil erosion is limited to a very small zone adjacent to the entrance and exit points of the seepage barrier crack. It should be noted, however, that there are other soil configurations where more extensive erosion may be possible.

Phase 2 of this study used the same seepage test cell that Van Leuven (2011) used in his study, but there are several differences between the two studies. Van Leuven studied only horizontal cracks, and he did not consider possible erosion and enlargement of the crack in the barrier material.
CHAPTER 4
PHASE 1 OVERVIEW AND RESULTS

4.1 Phase 1 Erosion Test Cell

Seepage barriers change the seepage regime in dam embankments and in their foundations. They cause high gradients and hydraulic pressures to act across the seepage barrier which can crack them at interfaces of dense soils or rock to softer soils. Cracks may also form due to expansion and contraction during curing. High hydraulic gradients across the barrier combined with highly permeable adjacent soils may create seepage velocities within the seepage barrier that are capable of eroding either the barrier material or the surrounding soil. If the seepage barrier has a crack, in order for erosion to occur: (1) the barrier material or adjacent soil must be erodible, (2) the adjacent soil or rock must be permeable enough to allow a great enough flow velocity in the crack for erosion of the barrier material or the adjacent soil, and (3) the adjacent soil or rock must provide an exit pathway for eroded material to go.

This study looks at conditions needed for erosion of seepage barriers to occur. Furthermore, it investigates the potential for erosion of fine-grained soils in contact with course-grained soils or defects in bedrock that are adjacent to a seepage barrier crack. Also noted are additional conditions contributing to the initiation of erosion of the seepage barrier or the surrounding soil.

The first task of the study was to develop a device to assess the erosion potential of various seepage barrier materials from water flowing through a crack in the seepage barrier. Such a device would enable us to identify the types of barrier materials that are...
erosive under typical gradients and hydraulic pressures and are, therefore, at risk of internal erosion through this mechanism. The Erosion Test Cell shown in Figure 15 answers the need for more accurate measurement of the amount of material that erodes by this mechanism and calculation of critical velocities than previous studies attained.

Building the Erosion Test Cell began by building the sample holder. It consists of a 1/4 inch thick cast iron pipe 20 inches long with an inside diameter of 6 inches. The pipe was cut in half lengthwise and flanges welded to the four cut edges as shown in Figure 16. Four 1/2-inch diameter holes were drilled in flange allowing the sample holder to be tightly bolted together around the seepage barrier sample. The inside of the sample holder is lined with 1/16-inch thick rubber sheets to seal around the seepage barrier sample so that water can only flow through the seepage barrier crack. The rubber sheets also serve as seals preventing leakage from the sample holder.

![Figure 15 - Phase 1 Erosion Test Cell and its components and sensors.](image-url)
The sample holder end pieces are made of 6 inch outer diameter Plexiglas tubes, 1/2 inch thick by 4 inches long. They are glued into recesses in the center of 8-inch square plates of 1-inch Plexiglas. The Plexiglas tubes fit snugly inside the split metal pipe and are sealed, along with the test sample, by the rubber sheets when the flange bolts are tightened (see Figure 17). The seepage barrier samples are 12 inches long and the two Plexiglas tubes take up the extra eight inches. This provides a water tight seal on the ends as well as allowing enough room to eliminate water jetting effects on the barrier material from the incoming and outgoing flows, ensuring that the erosion is due only to flow velocity in the crack and not some other mechanism.

Holes of 1/2-inch diameter holes were drilled in the four corners of the Plexiglas plates for threaded steel rods to bolt the end plates together. In the center of each end plate are female threads for a 1-inch pipe fitting to connect to the incoming and outgoing water. Two connectors for plastic tubes were fitted in each plate. Top quick connects serve as air releases to remove bubbles at the beginning of each test. After the air is
removed, they are connected to a differential pressure gauge to measure the difference in pressure across the seepage barrier sample. For a redundant system, two other quick connects in the Plexiglas plates are connected to pressure sensors also used to measure the differential pressure across the sample. On the inflow and outflow pipes and throughout the piping system, unions were added to facilitate disassembly of the sample holder and other components of the test setup as necessary.

The incoming water pressure is controlled using a pressure regulator as shown in Figure 15. Maximum pressure is limited by the pressure available in the lab (about 60 psi). Flow is measured upstream of the sample using an ultrasonic flow meter which is accurate to a tenth of a gallon per minute. In the downstream pipe is a turbidity meter measuring the outflow particles in parts per million.

Samples tested are presented in Table 1. Six mix designs were tested with a minimum of two crack apertures each. The tests were performed on four different materials that represent mix designs of real seepage barriers in dams that have
experienced seepage problems. The first barrier type was a weak cement-bentonite (CB) material. The A.V. Watkins Dam (Utah) CB barrier mix design was used to prepare the samples. The second type of barrier was a soil-cement-bentonite (SCB) barrier material which models the Deep Mixing Method barrier. These samples were collected directly from the seepage barrier as it was being constructed at Herbert Hoover Dike (Florida) from depths of 20 ft., 40 ft., and 65 ft. They were allowed two weeks to cure at the site and then were shipped to the USU lab. The last two barrier types are plastic concrete and conventional concrete mix designs fabricated at the USU lab using the mix designs of seepage barriers constructed at the New Waddell Dam (Arizona) and the Wolf Creek Dam (Kentucky), respectively. As testing progressed additional tests were performed to obtain more data or better understanding of the erosion process.

The mix designs used to fabricate the samples can be found in Appendix C. However, the mix designs for the Herbert Hoover SCB seepage barrier are proprietary so mix designs with similar properties are provided instead.

Table 1 - Phase 1 samples tested.

<table>
<thead>
<tr>
<th>Barrier Material Type</th>
<th>Mix Design Source</th>
<th>Compressive Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement-Bentonite</td>
<td>A.V. Watkins Dam</td>
<td>20-30</td>
</tr>
<tr>
<td>Soil Cement (Deep Mix Method)</td>
<td>Herbert Hoover Dike</td>
<td>300</td>
</tr>
<tr>
<td>20 ft. Depth</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soil Cement (Deep Mix Method)</td>
<td>Herbert Hoover Dike</td>
<td>350</td>
</tr>
<tr>
<td>40 ft. Depth</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soil Cement (Deep Mix Method)</td>
<td>Herbert Hoover Dike</td>
<td>500</td>
</tr>
<tr>
<td>65 ft. Depth</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Plastic Concrete</td>
<td>New Waddell Dam</td>
<td>1000</td>
</tr>
<tr>
<td>Conventional Concrete</td>
<td>Wolf Creek Dam</td>
<td>3000</td>
</tr>
</tbody>
</table>
The barrier samples that were mixed in the lab were made from raw materials as prescribed by the mix designs. They were cured in standard 12-inch tall by 6-inch diameter concrete molds. Extra samples were made to test the unconfined compressive strength of our mixtures at 7, 14, and 28 days cure time to verify that they were close to the design values given. In the cases of the conventional concrete and the plastic concrete cylinders, metal shims were included in the cylinder castings as Figure 18 shows to facilitate cracking them, to prevent damage during cracking, and to provide a flat surface on which to shim the crack open.

4.2 Data Collection

Monitored throughout the test were: (1) the flow through the crack, (2) the pore pressures upstream and downstream, and (3) the differential pressure across the crack. Data from the sensors was automatically recorded by a datalogger and could be viewed at

Figure 18 - Splitting of the various types of cylinders.
any time throughout the test on a computer running LoggerNet communications, collection and display software. The datalogger recorded data every second and average values were calculated and recorded every 3 minutes. Every day or two the data was downloaded to the computer. The data tables were imported into an Excel spreadsheet where flow and pressure were graphed to determine when erosion was complete. Figure 15 shows the placement of the sensors used in this study. A more complete description of the sensors used can be found in Appendix B. The differential pressure transducer and piezometers measure the pressure difference across the sample from which the gradient can be calculated. The pressure difference changes as the test progresses. As erosion occurs the crack widens and the pressure difference decreases. If the sample should collapse on itself or large eroded particles clog the crack the pressure difference would increase. The pressure difference will eventually stabilize as erosion stops and equilibrium is reached.

The flow meter is upstream from the sample holder and ultrasonically measures the flow of water though the system. Flow is limited by the crack in the sample material. As the crack erodes the flow increases until erosion has stopped or the maximum pressure allowed by the pressure regulator is reached. If the sample collapses or is clogged by large particles the flow can decrease.

Down-stream of the sample is a turbidity meter which measures the particulates in the outflow of the test in parts per million (ppm). The higher the turbidity the more erosion is occurring. The turbidity meter was used successfully in the conventional concrete and the plastic concrete tests, but with cement-bentonite the turbidity meter
gives erroneous data due to cement paste and bentonite from the seepage barrier sample coating the meter lenses.

The clear Plexiglas ends of the testing apparatus allow visual inspection of erosion and crack widening during the tests as seen in Figure 19.

4.3 Setup Procedure

The sample cylinders were split in half a few days after casting to form a rough, natural crack 6 inches wide and 12 inches deep. The SCB sample cylinders, which did not have precast metal shims, were precut along both sides with a cement saw to both give the cracking vice starting lines on which to crack and to provide a flat surface on which to place shims. Figure 20 shows the special vice made to split the cylinders. The cement-bentonite samples were too soft to crack so they were cut using a thin-bladed
handsaw. Cylinders of like types were cracked at the same time and set in a water bath to continue curing until they were tested.

Next, the test cylinders were either oven dried (conventional concrete and plastic concrete samples) or towel dried (SCB and CB samples) and then carefully weighed. The seepage barrier samples containing bentonite, if oven dried, developed severe desiccation cracks and readily fell apart, so those samples were towel dried.

Before loading the seepage barrier test cylinders into the sample holder, the edges of the cracks were shimmed to create a crack of known aperture from 1 to 2 mm. Multiple layers of 0.05 inch thick metal shims were stacked to achieve the desired aperture. Calibrated rods were used to measure the crack and confirm the aperture.

The tests were started with a pressure difference of 2 psi across the sample. Then the pressure was increased in 2 psi increments following the same procedure until the maximum pressure available was reached. The data was monitored at each increment until no more erosion was detected. After the first few tests, no benefit was seen from
incrementally increasing the pressure verses starting at full pressure, therefore, the maximum available pressure was applied until the test was in equilibrium.

As the barrier material erodes, the crack aperture increases. As the crack aperture increases, the flow volume increases but the flow velocity and differential pressure across the sample decrease. Equilibrium is reached when the flow and the differential pressure stabilize. At that point it is assumed that the flow velocity has dropped to the critical velocity for the sample materials that are erodible and that erosion has essentially stopped. With the sample materials that are considered non-erosive under the conditions of this test, the flow velocity is not the critical velocity.

Following each test the sample holder was disassembled and the barrier sample inspected and photographed for erosion patterns. The test cylinders were oven dried or towel dried as before and weighed again to quantify the total amount of erosion that occurred.

While performing these tests, special attention had to be given to sufficiently control the flow. The seepage barrier samples needed to be sealed so that all of the flow was through the crack and not between the sample and the walls of the sample holder. Care had to be taken to accurately weigh the samples. The towel dried samples needed to be fully submerged under water before and after the test so that the water content is the same before and after testing for accurate weight measurement.

4.4 Phase 1 Results

This section presents the results from Phase 1 testing in the Erosion Test Cell. A brief discussion of the overall results is given along with the individual results by
material type. A summary of the results is found in Table 2. Appendix A contains a more detailed table of the results. Figure 21 shows the results graphically. Graph (a) shows the percent erosion types. Graph (b) shows the critical or final velocity divided into the different barrier types.

Figure 21 – Graphs of (a) Percent Erosion by material type and, (b) Critical/Final Velocity by material type.
Table 2 – Phase 1 simplified results.

<table>
<thead>
<tr>
<th>Barrier Type</th>
<th>28 Day Strength psi</th>
<th>Test Duration Days</th>
<th>Initial Crack Aperture mm</th>
<th>% Erosion by Weight</th>
<th>Final Crack Aperture mm</th>
<th>Final Flow gpm</th>
<th>Critical Velocity ft/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>New Wad. 1 Plastic Concrete</td>
<td>607</td>
<td>8</td>
<td>1</td>
<td>0.20%</td>
<td>1.27</td>
<td>17.00</td>
<td>19.91</td>
</tr>
<tr>
<td>New Wad. 2 Plastic Concrete</td>
<td>607</td>
<td>7</td>
<td>2</td>
<td>0.03%</td>
<td>2.05</td>
<td>26.50</td>
<td>19.18</td>
</tr>
<tr>
<td>New Wad. 3 Plastic Concrete</td>
<td>607</td>
<td>18</td>
<td>1</td>
<td>0.01%</td>
<td>1.02</td>
<td>21.00</td>
<td>30.63</td>
</tr>
<tr>
<td>New Wad. 4 Plastic Concrete</td>
<td>607</td>
<td>18</td>
<td>3</td>
<td>0.99%</td>
<td>4.32</td>
<td>22.30</td>
<td>7.65</td>
</tr>
<tr>
<td>New Wad. 5 Plastic Concrete</td>
<td>607</td>
<td>21</td>
<td>2</td>
<td>0.17%</td>
<td>2.23</td>
<td>22.00</td>
<td>14.59</td>
</tr>
<tr>
<td>AV Watkins 1 Cement-Bentonite</td>
<td>22</td>
<td>6</td>
<td>1</td>
<td>19.14%</td>
<td>26.58</td>
<td>26.80</td>
<td>1.49</td>
</tr>
<tr>
<td>AV Watkins 2 Cement-Bentonite</td>
<td>22</td>
<td>9</td>
<td>2</td>
<td>15.80%</td>
<td>23.11</td>
<td>26.80</td>
<td>1.72</td>
</tr>
<tr>
<td>AV Watkins 3 Cement-Bentonite</td>
<td>22</td>
<td>6</td>
<td>2</td>
<td>11.76%</td>
<td>17.72</td>
<td>27.00</td>
<td>2.26</td>
</tr>
<tr>
<td>AV Watkins 4 Cement-Bentonite</td>
<td>22</td>
<td>9</td>
<td>2</td>
<td>15.76%</td>
<td>23.06</td>
<td>15.40</td>
<td>0.99</td>
</tr>
<tr>
<td>Wolf Creek 1 Conventional Concrete</td>
<td>2775</td>
<td>8</td>
<td>1</td>
<td>0.01%</td>
<td>1.01</td>
<td>24.25</td>
<td>35.60</td>
</tr>
<tr>
<td>Wolf Creek 2 Conventional Concrete</td>
<td>2775</td>
<td>15</td>
<td>2</td>
<td>1.41%</td>
<td>3.89</td>
<td>24.10</td>
<td>9.24</td>
</tr>
<tr>
<td>HH Dike 1-1 Soil-Cement-Bentonite</td>
<td>513</td>
<td>8</td>
<td>2</td>
<td>9.78%</td>
<td>15.07</td>
<td>24.70</td>
<td>2.43</td>
</tr>
<tr>
<td>HH Dike 1-2 Soil-Cement-Bentonite</td>
<td>513</td>
<td>8</td>
<td>1</td>
<td>14.30%</td>
<td>20.11</td>
<td>17.00</td>
<td>1.25</td>
</tr>
<tr>
<td>HH Dike 1-3 Soil-Cement-Bentonite</td>
<td>513</td>
<td>12</td>
<td>1</td>
<td>9.38%</td>
<td>13.53</td>
<td>18.00</td>
<td>1.97</td>
</tr>
<tr>
<td>HH Dike 1-4 Soil-Cement-Bentonite</td>
<td>513</td>
<td>16</td>
<td>1</td>
<td>0.01%</td>
<td>1.02</td>
<td>21.00</td>
<td>30.52</td>
</tr>
<tr>
<td>HH Dike 1-5 Soil-Cement-Bentonite</td>
<td>513</td>
<td>12</td>
<td>1</td>
<td>0.05%</td>
<td>1.06</td>
<td>17.00</td>
<td>23.70</td>
</tr>
<tr>
<td>HH Dike 2-1 Soil-Cement-Bentonite</td>
<td>353</td>
<td>12</td>
<td>1</td>
<td>15.37%</td>
<td>21.54</td>
<td>17.90</td>
<td>1.23</td>
</tr>
<tr>
<td>HH Dike 2-2 Soil-Cement-Bentonite</td>
<td>353</td>
<td>10</td>
<td>2</td>
<td>3.50%</td>
<td>6.68</td>
<td>28.85</td>
<td>6.40</td>
</tr>
<tr>
<td>HH Dike 2-3 Soil-Cement-Bentonite</td>
<td>353</td>
<td>10</td>
<td>1</td>
<td>6.21%</td>
<td>9.30</td>
<td>27.75</td>
<td>4.42</td>
</tr>
<tr>
<td>HH Dike 2-4 Soil-Cement-Bentonite</td>
<td>353</td>
<td>11</td>
<td>1</td>
<td>0.02%</td>
<td>1.03</td>
<td>20.80</td>
<td>29.90</td>
</tr>
<tr>
<td>HH Dike 2-5 Soil-Cement-Bentonite</td>
<td>353</td>
<td>15</td>
<td>1</td>
<td>0.09%</td>
<td>1.12</td>
<td>22.05</td>
<td>29.11</td>
</tr>
<tr>
<td>HH Dike 3-1 Soil-Cement-Bentonite</td>
<td>319</td>
<td>8</td>
<td>1</td>
<td>5.02%</td>
<td>7.71</td>
<td>24.00</td>
<td>4.61</td>
</tr>
<tr>
<td>HH Dike 3-2 Soil-Cement-Bentonite</td>
<td>319</td>
<td>11</td>
<td>1</td>
<td>0.25%</td>
<td>1.33</td>
<td>20.00</td>
<td>22.23</td>
</tr>
<tr>
<td>HH Dike 3-3 Soil-Cement-Bentonite</td>
<td>319</td>
<td>7</td>
<td>2</td>
<td>0.05%</td>
<td>2.07</td>
<td>22.75</td>
<td>16.29</td>
</tr>
<tr>
<td>HH Dike 3-4 Soil-Cement-Bentonite</td>
<td>319</td>
<td>18</td>
<td>1</td>
<td>0.00%</td>
<td>1.00</td>
<td>19.90</td>
<td>29.49</td>
</tr>
<tr>
<td>HH Dike 3-5 Soil-Cement-Bentonite</td>
<td>319</td>
<td>12</td>
<td>1</td>
<td>0.05%</td>
<td>1.07</td>
<td>18.50</td>
<td>25.70</td>
</tr>
</tbody>
</table>

* Velocity is either the critical velocity where erosion ceases or the maximum velocity applied during the test.
4.4.1 Cement-Bentonite (A.V. Watkins Dam)

The cement-bentonite material proved to be very erosive. Figure 22 shows photos taken before, during, and after testing. What started as a 1 to 2 mm smooth crack became a 17 to 26 mm wide crack.

Figure 23 shows a typical plot of the data from the cement-bentonite testing. It shows that the flow increased and the pressure across the sample decreased during the test. The last 2 days of testing showed minimal change in flow and pressure which indicated that erosion had essentially ceased, equilibrium had been reached, and that flow had dropped to the critical velocity for this material. Graphs of all the tests performed may be found in Appendix D.

Figure 22 - Cement-bentonite before and after photos.
4.4.2 Conventional Concrete (Wolf Creek Dam)

Before and after photos of the conventional concrete sample are shown in Figure 24. There was no notable change in physical appearance. Figure 25 shows that all the changes in flow and pressure occurred within the first 2 days. There was only a 0.5 gpm increase in flow, and only a 1psi decrease in pressure. And these were assessed to be due to the loss of small loose particles which occurred during cracking of the cylinder. Conventional concrete is not erosive under the pressures and flows used in this test.
4.4.3 Plastic Concrete (New Waddell Dam)

Figure 26 shows photos of plastic concrete with black dye placed on the surface of the crack. The black dye was used to allow the observation of any preferential flow paths that might have developed due to erosion of the plastic concrete. On the average, slightly more weight loss was observed in plastic cement than conventional concrete. The

Figure 25 - Flow and differential pressure plots for a test on conventional concrete.

Figure 26 - Plastic concrete before and after photos.
small weight loss seen was due to the erosion of loose chunks created during cracking and the loss of some cement-bentonite paste. No preferential flow path was observed so the black dye method was discontinued.

It was found that, as with conventional concrete, plastic concrete is not erosive with the flow velocities and pressures used in this test. The cement paste in the sample is erosive but the sand aggregate on the surface is not. The cement paste on the surface was observed to erode and expose the sand and aggregate underneath. The cement paste is strong enough to hold on to the sand particles while the sand “armoring” protects the paste from further erosion. This armoring effect is thought to be the reason why the plastic concrete samples eroded less than expected.

The changes in flow and pressure shown in Figure 27 are due to manual flow increases. In subsequent tests the method of starting with low pressure and increasing it throughout the test was abandoned. Starting the tests with the maximum flow allowed better monitoring of the progression of erosion and more accurate determination of the initial and final flow velocities.

![Figure 27 - Flow and differential pressure plots for plastic concrete.](image-url)
4.4.4 Soil-Cement-Bentonite (Herbert Hoover Dike)

Figure 28 shows some photos of soil-cement-bentonite cylinders. They are samples collected directly from the deep mix wall during construction at Herbert Hoover Dike in Florida. The in-situ soil consists mainly of fine sand together with clay, a small amount of peat, and some 1/8 to 1/4 inch diameter shells. A large part of the erosion detected in these cylinders was due to small pockets of soil being cleaned out by the flow of water. Also, there were chunks of shell and sand loosened during cracking of the cylinders. After the initial cleaning of the crack, erosion of the SCB mixture did occur.

Some inconsistencies were noted with the erosion behavior of this material. Figures 29-31 present the flow and differential pressure data from three tests performed on samples from the Herbert Hoover Dike. The data in Figure 29 is from a sample taken at a depth of 65 feet with a sample age of 59 days. A decrease in flow and little variation in pressure were observed. This sample did partially collapse. The data in Figure 30

Figure 28 – Soil-cement-bentonite before and after photos.
Figure 29 - Flow and differential pressure plots for a test on a 65 foot soil-cement-bentonite sample aged 59 days.

Figure 30 - Flow and differential pressure plots for a test on a 40 foot soil-cement-bentonite sample aged 95 days.

Figure 31 - Flow and differential pressure plots for a test on a 20 foot soil-cement-bentonite sample aged 105 days.

is from a sample taken at a depth of 40 feet with a sample age of 95 days. A gradual increase in flow and a decrease in pressure were observed. The data in Figure 31 is from a sample taken at a depth of 20 feet with a sample age of 105 days. Very little variation in flow and pressure was observed.
Some unexpected variations in erodibility over time were observed with this barrier material. As seen in Figure 32 and Figure 33 both the percent erosion and the critical velocity vary with time. As the barrier material aged, the total amount of erosion decreased and the critical velocity increased. We believe this was due to cement slag included in the mix as a retarder to slow the curing of the concrete and thus, the observed changes in behavior were due to strength increases that occurred well after the 28-day breaks.

Figure 32 - Percent erosion vs. time for all SCB samples tested.

Figure 33 - Critical velocity vs. time for all SCB samples tested.
Figures 34 and 35 are plots that show the percent erosion and the maximum stable velocity versus the 28-day unconfined compression (UC) strength for all of the samples tested. These plots were created to determine if predictions of flow velocities and the amount of erosion could be made by only knowing the 28-day UC strength.

Due to the slow strength gain in the SCB material the UC strengths were adjusted for time considerations as shown in Figures 37 and 38. The strength significantly increases beyond the typical 28-day UC strength. The adjustments were made based on
data from long-term UC testing on similar samples from Herbert Hoover Dike. This data was obtained from the US Army Corps of Engineers. Two equations obtained by interpolation of this data are shown in Figure 36. Equation 2 was used for the SCB materials that had a 28 day UC strength of around 300 psi while Equation 3 was used for SCB materials with a 28 day UC strength around 200 psi. The age of the SCB samples were noted at the beginning of each test and the UC strengths were adjusted to the strength predicted at the time each test was started using either Equation 2 or 3 and then using Equation 4.

\[ Y(t) = 137.11\ln(t) - 156.93 \text{ for UC}(28) = 300\text{psi} \quad \text{Equation 2} \]

\[ Y(t) = 88.138\ln(t) - 94.528 \text{ for UC}(28) = 200\text{psi} \quad \text{Equation 3} \]

\[ \text{UC}(t) = \text{UC}(28)\frac{Y(t)}{Y(28)} \quad \text{Equation 4} \]

Figure 36 - Long-term strength of HH Dike samples.
The SCB samples that had lower percent erosion were shifted right as Figure 37 shows. Figure 38 shows the samples that had a higher maximum stable velocity were shifted right. The plots adjusted for the SCB UC strengths still do not accurately predict maximum stable velocities and percent erosion based on UC strengths. The data indicates that significant erosion ceases at a UC of about 700 to 800 psi but there is not much confidence in this assessment. Further testing and data is needed to make an accurate prediction. It is expected that different material types will have different UC values at which erosion ceases.

Figure 37 - Percent erosion vs. adjusted unconfined compression (UC) strength.

Figure 38 – Maximum stable velocity vs. adjusted unconfined compression (UC) strength.
CHAPTER 5

PHASE 2 OVERVIEW AND RESULTS

5.1 Phase 2 Seepage Test Cell

The purpose of Phase 2 was to qualitatively assess the performance of cracked erodible seepage barriers adjacent to various soil and bedrock configurations. The Seepage Test Cell used in Phase 2 is shown pictorially in Figure 39 and schematically in Figure 40.

A reinforced metal box was designed big enough to adequately model a cracked seepage barrier and the adjacent soil without interference from the sides of the apparatus. The Seepage Erosion Test Apparatus has room for a 2 foot tall by 1 foot deep by 1 foot wide seepage barrier block. Soil is placed in 18-inch long zones both upstream and downstream from the block. Zones of gravel are placed at the entrance and exit of the test.
cell so that the entire cross-section has the same incoming and outgoing hydraulic head. Air bladders are placed above the soil and pressurized to 23 psi to simulate a depth of 25 to 30 feet (depending on the unit weight of the modeled soil). The reinforcing on the outside of the test cell contains the pressure inside the cell without significant deformation. The exit pipe is 2 inches in diameter and raises a foot above the top of the Seepage Test Cell to provide a constant tail pressure as shown in Figure 40.

In Phase 1 the cement-bentonite (A.V. Watkins) and soil-cement-bentonite (H.H. Dike) seepage barrier materials were identified as erosive. In order for the seepage barrier to erode, certain criteria must be met: (1) the adjacent soil must allow high flows, (2) the barrier material or adjacent soil must be erodible, and (3) there needs to be a pathway through which the eroded material can be removed.

Figure 40 - Schematic drawing of the Seepage Test Cell.
Three soil, bedrock, and seepage barrier configurations were identified as having potential for internal erosion. Figure 41 part (a) shows the first configuration: fine sand sitting on top of coarse sand. The coarse sand used in the test is very pervious and allows for high flow velocities. The barrier material and fine sand are erodible and the coarse sand has large voids providing an exit for the eroded material. Sand found at Herbert Hoover Dike and graded Ottawa sand were used for the fine sand, and #8-sieve sand was used for the coarse sand.

Figure 41 part (b) shows the second configuration: fine sand above a rock layer with a karstic void. The fine sand allows high flows and the karstic void provides a pathway for material to be removed. The karstic void is simulated by a ½-inch pipe set in lean concrete. The entrance to the void is set 1 inch downstream from the seepage barrier sample crack.

![Figure 41 - Phase 2 soil configurations.](image-url)
Figure 41 part (c) shows the third configuration: fine sand over an open bedrock joint. The open bedrock joint allows high flows and an exit for the eroded material. A 3-mm crack is cast into lean concrete and is offset 1 inch from the crack in the seepage barrier. Properties of the soils used in Phase 2 may be found in Appendix B.

The soil configurations and the seepage barrier samples tested are shown in Table 3. Both of the seepage barriers will be tested in the three soil profiles described above.

Two outcomes were thought possible in these tests: (1) that fine sand fills the crack preventing erosion, or (2) that erosion would happen fast enough that the sand would wash through the crack and travel downstream with the eroded barrier material into the coarse sand, bedrock joint, or solutioned void.

Table 3 - Phase 2 summary of tests performed.

<table>
<thead>
<tr>
<th>Soil Profile</th>
<th>Barrier Material</th>
<th>Crack Aperture (mm)</th>
<th>Max Flow (gpm)</th>
<th>Crack Flow Velocity (ft/s)</th>
<th>Final Crack Filling/Erosion and Downstream Cementation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fine Sand Above Coarse Sand</td>
<td>Cement Bentonite</td>
<td>1</td>
<td>11.1</td>
<td>1.88</td>
<td>Downstream cementation. The crack filled completely with sand.</td>
</tr>
<tr>
<td>Fine Sand Above Coarse Sand</td>
<td>Cement Bentonite</td>
<td>2</td>
<td>7.5</td>
<td>2.55</td>
<td>Extensive downstream cementation. The crack filled completely with sand.</td>
</tr>
<tr>
<td>Karstic Bedrock</td>
<td>Cement Bentonite</td>
<td>2</td>
<td>1.0</td>
<td>0.68</td>
<td>Extensive downstream cementation. Crack filled completely with sand. Some cementation in crack.</td>
</tr>
<tr>
<td>Karstic Bedrock</td>
<td>Soil Cement Bentonite</td>
<td>1</td>
<td>2.0</td>
<td>0.68</td>
<td>Extensive downstream cementation. Crack filled partially with sand. Pipe formed in downstream sand.</td>
</tr>
<tr>
<td>Karstic Bedrock</td>
<td>Soil Cement Bentonite</td>
<td>2</td>
<td>1.2</td>
<td>0.81</td>
<td>Extensive downstream cementation. The crack filled completely with sand.</td>
</tr>
<tr>
<td>Jointed Bedrock</td>
<td>Soil Cement Bentonite</td>
<td>2</td>
<td>*</td>
<td>*</td>
<td>Failed too quickly to fill the crack or cement the soil.</td>
</tr>
</tbody>
</table>

* No data. Erosion occurred too rapidly.
5.2 Data Collection

Figure 42 shows the sensor setup for this test. A pressure regulator and an ultrasonic flow meter were placed upstream from the Seepage Test Cell so that the water pressure could be set as desired. Piezometers were employed to sense upstream and downstream pressures on the seepage cell.

Data from the sensors was automatically recorded by a CR1000 datalogger and could be viewed at any time during the test on a computer running LoggerNet data communications, collection and display software. The datalogger acquired data every second and an average was calculated and recorded every three minutes. Every day or two the data was collected to the computer. The data tables were imported into an Excel spreadsheet where flow and pressure were graphed to determine when the system reached equilibrium.

The crack aperture and differential head applied across the crack created conditions similar to those required to initiate erosion in the cylindrical samples of the Erosion Test Cell.

Figure 42 - Seepage Test Cell setup and sensors.
5.3 Setup Procedure

The first step in setting up this test was to cast a one foot wide by one foot deep by two feet tall sample block following the given mix designs. The two materials that were found to be erosive in Phase 1 were the A.V. Watkins CB and H.H. Dike SCB materials. Samples of these mixes were cast into blocks each with a two foot vertical crack which best represents the seepage barrier cracks observed in the field. The crack was formed by a thin sheet of Plexiglas placed in the mold.

The CB and SCB materials were weak and it was difficult to shim the crack open, therefore, a 4-inch thick shell made of conventional concrete backing was constructed to support the barrier material as seen in Figure 43. The backing decreases the amount of barrier material used in each test and allowed nuts to be cast into the concrete backing to facilitate opening the crack to a known aperture. Figure 44 shows the turnbuckle system used to adjust the crack in the sample block.

After curing the barrier sample and removing the Plexiglas sheet, the block was

Figure 43 - Concrete shell used to support the seepage barrier material.
placed in the testing apparatus and sealed around the edges using tar tape and silicon rubber to prevent flow around the block. The block crack was opened to the desired aperture. The soils were then compacted into the apparatus in 2-inch lifts. In the fine sand, strings were placed horizontally on top of every lift near the crack. The strings were placed to monitor deformation in the sand due to internal erosion. Air bladders were placed on top of the fine sand to provide confining pressure and to simulate depth. The bladders were pressurized with air to about 23 psi to simulate a soil depth of 25-30 feet.

The Seepage Cell full of sand was then saturated with water. Then the lid was sealed with putty and bolted down. Water pressure was slowly added until significant flow was seen coming out of the exit pipe. Data was monitored throughout the test. Testing proceeded until the flow and pressure stabilized.

After completion of the test the surrounding soil and the seepage barrier block were carefully removed. Evidence of block erosion, soil deposition, and erosion were observed and documented.
5.4 Phase 2 Results

This section presents the results from Phase 2 testing in the Seepage Test Cell. A brief discussion of the results is given along with results by soil profile type. A summary of the results is found in Table 3. Appendix A contains a more detailed table.

It was anticipated that testing with the Seepage Erosion Test Cell would show one of three possible mechanisms associated with the continuation of seepage barrier erosion: (1) that the particles of eroding seepage barrier material would be filtered by the soil downstream from the crack resulting in the clogging of the crack and reduced flow velocity, (2) that the aperture of the crack would increase through erosion until sand grains entered the upstream side of the crack and reduce the flow velocity, or (3) that the eroding barrier particles would not be filtered and the erosion of the barrier would continue.

5.4.1 Fine Sand Overlying Coarse Sand

This scenario consists of fine Herbert Hoover Sand overlying uniform coarse sand retained between the #4 and #8 sieves and is designed to model the configuration shown in Figure 41 part (a). The test was run for 8 days at a maximum flow of 11 gpm in the first test and 21 days at a max of 7.5 gpm in the second test. At the conclusion of the tests it was observed that the cement paste from the CB seepage barrier material was washed downstream and solidified in the voids of the sand as shown in Figure 45 and Figure 46. The cement paste and fine sand penetrated into the coarse sand and formed a cemented zone. To a lesser degree a cemented zone formed in the fine sand as well. The Herbert
Hoover sand has high calcium content. This is believed to have added in the cementing of the sand.

Some sand washed into the upstream seepage barrier crack but it did not wash out of the crack as shown in Figure 47. The downstream cementing of the sand prevented any erosion of either sand or barrier material through the crack. Some of the crack filled completely with sand and cementitious material. Figure 48 is a graph of the data for this test. There was a gradual decrease in flow and a small increase in pressure.
5.4.2 Fine Sand Overlying Karstic Bedrock

This test was performed to model the configuration shown in Figure 41 part (b).

The bottom half of the Seepage Test Cell was filled, both upstream and downstream, with lean concrete. Downstream a ½ inch pipe was placed with its opening 1 inch downstream from the crack used to simulate karstic void. The downstream end of the pipe exited into
the downstream constant head zone. The top half of the Seepage Test Cell was filled with fine Herbert Hoover sand.

The CB material performed similar to the previous scenario. Cement paste was washed downstream into the fine sand. This created a hard cemented layer of sand just downstream of the crack as shown in Figure 49. Some erosion of the fine sand did occur before the cement paste hardened as shown by the small amount of piping shown in Figure 50. The piping erosion extended up about 4 inches. The 1/2-inch pipe, which simulates a solutioned void, was full of the fine sand and cementitious material at the end of the test. Minimal sand was carried out of the pipe.

Figure 51 shows a graph of the flow and pressures in the test cell. The decrease in flow and increase in pressure are thought to be due to the crack filling with sand and the cement paste hardening in the downstream sand and decreasing its permeability.
The SCB material performed similarly to the CB material. Figure 52 shows the cementing of the downstream sand as well as the filling of the seepage barrier crack with sand. Fingers of cemented Ottawa sand extended through the entire 18 inches of downstream soil. Figure 53 shows a graph of the data. Flow and pressure don’t change significantly throughout the test. It can be assumed that the fine sand just down stream of the crack cemented as soon as the test started thus inhibiting erosion.
5.4.3 Fine Sand Overlying Jointed Bedrock

This test was performed to model the configuration shown in Figure 41 part (c).

The bottom half of the Seepage Test Cell was filled with cement with a 3-mm vertical crack formed both upstream and down that is offset from the barrier crack by 1 inch. The upper half of the Seepage Test Cell was filled with graded Ottawa sand.
Two tests were performed using this profile. The first test had a barrier crack of 3 mm. A #200 sieve was placed below the exit pipe and was filled to overflowing with sand in about five minutes. Measures were taken to better seal potential problem leak areas but the second test, with a crack aperture of 2 mm, failed in about 45 minutes.

The jointed bedrock combined with the open crack made this configuration very pervious with a high gradient over a short distance. The water had an open entry and exit and readily eroded the surrounding sand. This configuration could allow erosive flow velocities in the seepage barrier crack thus enlarging the crack and progressing on to a serious seepage problem. Figure 54 shows the erosion of the upstream sand through the crack as well as the formation of a sinkhole downstream.

Figure 54 – Sinkholes upstream and downstream in the fine sand overlying jointed bedrock test.
CHAPTER 6
SUMMARY AND CONCLUSIONS

6.1 Erosion Test Cell (Phase 1)

The Erosion Test Cell showed cement-bentonite and soil-cement-bentonite to be highly erodible. It can be assumed that, if the surrounding soil allows high enough flows, the cracks in the CB and the SCB seepage barriers will enlarge. This may lead to more serious seepage related problems.

The SCB material tested exhibited time effects: over time it became more resistant to erosion. The mix design tested is proprietary but it can be assumed that cement slag was used which allowed for significant strength increase of the material beyond the typical 28 days when the samples were tested for strength.

Plastic concrete was found to be non-erosive due to an armoring effect with the aggregate. The paste connecting the aggregate together is erodible, however, as the paste is eroded and the sand aggregate is exposed, the paste holds on to the sand particles preventing them from being eroded while the sand particles protect the paste from further erosion.

Conventional concrete is considered to be non-erosive under the seepage velocities used in this study. It was able to resist erosion at flow velocities up to 35 feet per second.

The seepage barrier samples were tested for up to 2 weeks. This short time frame does not include possible degradation of the seepage barrier that can takes years to occur. Tests should be performed much longer for long-range effect to be noticed.
6.2 Seepage Test Cell (Phase 2)

In all of the CB cases with the Seepage Test Cell there was a cementing or calcification of the downstream soils. The Herbert Hoover sand tends to calcify and may have added to this effect. Small amounts of cement and bentonite paste that eroded from the barrier were deposited in the sand and led to the creation of a hardened, erosion-resistant layer which allowed little erosion of the seepage barrier crack or surrounding soil. The Herbert Hoover sand also had a large percentage of fines that tended to create its own filter. The crack was wide enough to allow sand into the crack but the hardened sand and fine-sand filter did not let it exit.

The SCB material response was similar to that of the CB material in the Seepage Test Cell. The amount of cementation was slightly less, still a hardened, erosion-resistant layer formed just downstream of the crack. This hardened layer inhibited erosion of the seepage barrier material and the crack simply filled up with sand.

The graded Ottawa sand had less cementing than the Herbert Hoover sand being silica based rather than calcium based sand. But enough cementing occurred, due to the cement paste from the seepage barrier washing into the downstream sand, to form a hardened, erosion-resistant layer.

6.3 Conclusions

The Erosion Test Cell and Seepage Test Cell combination introduces a new approach to understanding erosion due to cracks in seepage barriers. Figure 37 and Figure 38 show the relationship between maximum achieved velocity and percent erosion verses adjusted, unconfined compressive strengths. More testing is needed to be able to make
some solid correlations between the barrier type and the critical velocities needed to erode them. This is just a beginning. Further study is also needed on mix designs of each type of seepage barrier with various UC strengths and cement contents. The resulting data could be combined with the data in this study to determine a dependable trend.

The ability to determine critical flow velocity for a given seepage barrier will aid in quantitatively assessing the potential for critical seepage problems propagating from cracks in seepage barriers. This will aid in the selection of barrier types for new barriers as well as the placement of instrumentation to monitor new and existing barriers. This study can help mitigate existing barriers where problems have been identified.
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APPENDICES
### APPENDIX A

#### Table 4 - Phase 1 Test Results

<table>
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<tr>
<th></th>
<th>Start Date</th>
<th>End Date</th>
<th>Test Duration (days)</th>
<th>Crack Aperture (mm)</th>
<th>Initial Weight (lbs.)</th>
<th>Final Weight (lbs.)</th>
<th>% Erosion</th>
<th>7 Day Strength (psi)</th>
<th>14 Day Strength (psi)</th>
<th>28 Day Strength (psi)</th>
<th>Design Strength, 28 Day (psi)</th>
<th>Sample Depth (ft)</th>
<th>Critical Velocity (ft/s)</th>
<th>Notes</th>
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<td>11/10/2011</td>
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<td>1</td>
<td>25.190</td>
<td>25.170</td>
<td>0.20%</td>
<td>383</td>
<td>453</td>
<td>607</td>
<td>585</td>
<td></td>
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<td>25.350</td>
<td>0.01%</td>
<td>383</td>
<td>453</td>
<td>607</td>
<td>585</td>
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<td>New Waddell 4</td>
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<td>20.050</td>
<td>20.000</td>
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<td>0</td>
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<td>22.226</td>
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<td>16.293</td>
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<td>9/28/2012</td>
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<td>0.05%</td>
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<td>319</td>
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<td>20</td>
<td>25.704</td>
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</tbody>
</table>

- Towel dried.
- Problems with the Flow Meter calibration.
- Oven dried. Significant desiccation. Sample collapsed.
- Fun dried. Partial Collapse of upstream crack.
- Fun dried. Partial collapse of upstream crack.
- Towel dried.
- Tongue dried.
- Tongue dried.
- Tongue dried.
Table 5 - Phase 2 Test Results

<table>
<thead>
<tr>
<th>Soil Configuration</th>
<th>Seepage Barrier Material</th>
<th>Type of Sand</th>
<th>7 Day Strength (psi)</th>
<th>14 Day Strength (psi)</th>
<th>28 Day Strength (psi)</th>
<th>Crack Aperture (mm)</th>
<th>Start Date</th>
<th>End Date</th>
<th>Duration (Days)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fine Sand Overlying Coarse Sand</td>
<td>Cement-Bentonite</td>
<td>Herbert Hoover Sand</td>
<td>-</td>
<td>-</td>
<td>22</td>
<td>1</td>
<td>8/9/2012</td>
<td>8/16/2012</td>
<td>8</td>
<td>There was a gradual decrease in flow and a gradual increase in pressure. Both the fine sand and coarse sand had cementing but was greater in the coarse sand. No visible erosion of the seepage barrier block was noted. The crack did fill with fine sand and was partially cemented.</td>
</tr>
<tr>
<td>Fine Sand Overlying Coarse Sand</td>
<td>Cement-Bentonite</td>
<td>Herbert Hoover Sand</td>
<td>-</td>
<td>-</td>
<td>22</td>
<td>2</td>
<td>8/21/2012</td>
<td>9/10/2012</td>
<td>21</td>
<td>There was a gradual decrease in flow and a gradual increase in pressure. Both the fine sand and coarse sand had cementing but was greater in the coarse sand. No visible erosion of the seepage barrier block was noted. The crack did fill with fine sand and was partially cemented. The test was run longer than the first time and cementation was slightly more extensive.</td>
</tr>
<tr>
<td>Fine Sand Overlying Karstic Bedrock</td>
<td>Cement-Bentonite</td>
<td>Herbert Hoover Sand</td>
<td>-</td>
<td>-</td>
<td>22</td>
<td>1</td>
<td>11/1/2012</td>
<td>11/12/2012</td>
<td>12</td>
<td>A small pipe formed downstream along the crack interface directly above the PVC tube. The 'piped' soil had soft fines lining it. There was significant hardening of the fine sand, extending about 12 inches downstream from the seepage barrier. The flow decreased and the pressure increased throughout the test. The crack had filled with fine sand and had cemented about half of the upper portion of the barrier material. The PVC pipe also filled with the fine sand and cementitious material.</td>
</tr>
<tr>
<td>Fine Sand Overlying Karstic Bedrock</td>
<td>Soil-Cement-Bentonite</td>
<td>Herbert Hoover Sand</td>
<td>549</td>
<td>673</td>
<td>900</td>
<td>1</td>
<td>12/5/2012</td>
<td>12/17/2012</td>
<td>13</td>
<td>A small pipe formed in the downstream soil. Not much cementing of the fine sand occurred. The crack filled with sand as did the PVC pipe. The flow did gradually increase as did the pressure across the barrier.</td>
</tr>
<tr>
<td>Fine Sand Overlying Karstic Bedrock</td>
<td>Soil-Cement-Bentonite</td>
<td>Ottawa Graded Silica Sand</td>
<td>394</td>
<td>500</td>
<td>-</td>
<td>2</td>
<td>12/18/2012</td>
<td>12/30/2012</td>
<td>13</td>
<td>Switched to Ottawa sand instead of Herbert Hoover sand to eliminate calcification. Fingers of cementitious material were seen in the downstream soil penetrating 18 inches and along the seepage barrier face. Data was missed due to faulty sensors. The entire crack and the PVC pipe filled with sand. No noticeable erosion of the barrier material occurred.</td>
</tr>
<tr>
<td>Fine Sand Overlying Fractured Bedrock</td>
<td>Soil-Cement-Bentonite</td>
<td>Ottawa Graded Silica Sand</td>
<td>-</td>
<td>365</td>
<td>-</td>
<td>2</td>
<td>1/12/2013</td>
<td>1/12/2013</td>
<td>5 min.</td>
<td>The graded Ottawa sand started pouring out as soon as the test started. A #200 sieve was placed below the exit pipe and was completely filled to overflowing in about 5 minutes. This profile is too pervious or has too much leakage.</td>
</tr>
<tr>
<td>Fine Sand Overlying Fractured Bedrock</td>
<td>Soil-Cement-Bentonite</td>
<td>Ottawa Graded Silica Sand</td>
<td>-</td>
<td>365</td>
<td>-</td>
<td>2</td>
<td>1/25/2013</td>
<td>1/25/2013</td>
<td>45 min.</td>
<td>Measures were taken to better seal potential problem leak areas. Despite the fixes the graded Ottawa sand still started pouring out as soon as the test started. The #200 sieve placed below the exit pipe was filled in about 45 minutes. This profile is too pervious.</td>
</tr>
</tbody>
</table>
APPENDIX B

Sensors and Equipment Used:

**Data Logger** – Campbell Scientific, Inc.

![Figure 55 – CR1000 Data Logger, AM 16/32 B Relay Multiplexer.](image)

**Flow Meter** – Master Meter.

![Figure 56 - 2-inch Octave ultrasonic meter.](image)

**Pressure Differential Meter** – Validyne Engineering Corp.

![Figure 57 - CD23 digital transducer indicator.](image)
Pressure Regulator – Black Mountain, NC.

Figure 58 - Cash-Valve, 10-45 psi range.

Pressure Sensors “Piezometers” – Honeywell.

Figure 59 - 26PC series pressure sensors.

Turbidity Meter – TF56 Optek sensor.
Optek inline control.

Figure 60 - TF56 Optek sensor.
APPENDIX C

Soils Used:

**Coarse Sand** – 
Source – Quarry near Cove, Utah in Cache County  
Preparation – Washed before using through a #8 sieve to remove all finer soil particles.  
Dry Unit Weight – 90 pcf. Determined by compacting the soil in a modified proctor mold and dividing the weight of the soil by the volume of the mold.  
Permeability – 7.78E-02 cm/s. Determined by performing a constant head test. (ASTM D2434-68)

![Figure 61 - #4 - #8 sieve coarse sand.](image1)

**Herbert Hoover Sand** – 
Source – Herbert Hoover Dike, Lake Okeechobee, Florida  
Preparation – Removed particles larger than 1 inch in diameter.  
Dry Unit Weight – 100 pcf. Determined by compacting the soil in a modified proctor mold and dividing the weight of the soil by the volume of the mold.  
Permeability – 3.027E-02 cm/s. Determined by performing a constant head test. (ASTM D2434-68)

![Figure 62 - Herbert Hoover sand.](image2)
**Graded Ottawa Sand** (conforms to ASTM C778) –

Source –
Preparation – NA
Dry Unit Weight – 94 pcf. Determined by compacting the soil in a modified proctor mold and dividing the weight of the soil by the volume of the mold.
Permeability – 2.61E-01 cm/s. Determined by performing a constant head test. (ASTM D2434-68)

![Graded Ottawa sand](image)

Figure 63 - Graded Ottawa sand.

**Seepage Barrier Specifications and Mix Designs:**

**A.V. Watkins Dam**

Location – Willard Bay, Willard, Utah
Owner – Bureau of Reclamation
Material – Self-Hardening Cement-Bentonite Slurry
Mix Design –
Water – 80.7%
Cement – 14.5%
Bentonite – 4.8%
Compressive Strengths –
7 Day – NA
14 Day – NA
28 Day – 22 psi

![A.V. Watkins Dam](image)

Figure 64 - C-B trench at A.V. Watkins Dam near Ogden, UT.
**Herbert Hoover Dike**

Location – Lake Okeechobee, Florida  
Owner – US Army Corps of Engineers  
Material – Soil Cement with Bentonite, Deep Mix Method  
Mix Design – (Estimate)  
  - Water – 14.8%  
  - Cement – 13.3%  
  - Bentonite – 1.5%  
  - Herbert Hoover Sand – 70.4%  
Compressive Strengths –  
  - 7 Day – NA  
  - 14 Day – NA  
  - 28 Day – about 395 psi

**New Waddell Dam**

Location – 30 miles north of Phoenix, AZ. Forms Lake Pleasant  
Owner – Bureau of Reclamation  
Material – Plastic Concrete  
Mix Design –  
  - Cement – 6.3%  
  - Fly Ash – 2.7%  
  - Water – 6.7%  
  - Sand – 42.7%  
  - Gravel – 40.2%  
  - Bentonite – 0.6%  
Compressive Strengths (psi) –  
  - 7 Day – 383  
  - 14 Day – 453  
  - 28 Day – 607
Wolf Creek Dam

Location – On the Cumberland River in Russell County, Kentucky
Owner – US Army Corps of Engineers
Material – Conventional Concrete, Secant Piles an Rectangular Panels
Mix Design –
  Cement – 5.9%
  Fly Ash – 10.0%
  Gravel – 42.4%
  Sand – 35.4%
  Water – 6.3%
Compressive Strengths (psi) –
  7 Day – 1836
  14 Day – NA
  28 Day – 2775

Figure 66 - New Waddell Dam, Arizona.

Figure 67 - Wolf Creek Dam, Kentucky.
APPENDIX D

AV Watkins

Figure 68 - AV Watkins cement-bentonite Trial 1.

Figure 69 - AV Watkins cement-bentonite trial 2.

Figure 70 - AV Watkins cement-bentonite trial 3.
Figure 71 - AV Watkins cement-bentonite trial 4.

Wolf Creek

Figure 72 - Wolf Creek conventional concrete trial 1.

Figure 73 - Wolf Creek conventional concrete trial 2.
New Waddell

Figure 74 - New Waddell plastic concrete trial 1.

Figure 75 - New Waddell plastic concrete trial 2.

Figure 76 - New Waddell plastic concrete trial 3.
Figure 77 - New Waddell plastic concrete trial 4.

Figure 78 - New Waddell plastic concrete trial 5.

HH Dike 1

Figure 79 - Herbert Hoover Dike soil-cement-bentonite 65 ft. trial 1.
Figure 80 - Herbert Hoover Dike soil-cement-bentonite 65 ft. trial 2.

Figure 81 - Herbert Hoover Dike soil-cement-bentonite 65 ft. trial 3.

Figure 82 - Herbert Hoover Dike soil-cement-bentonite 65 ft. trial 4.
Figure 83 - Herbert Hoover Dike soil-cement-bentonite 65 ft. trial 5.

HH Dike 2

Figure 84 - Herbert Hoover Dike soil-cement-bentonite 40 ft. trial 1.

Figure 85 - Herbert Hoover Dike soil-cement-bentonite 40 ft. trial 2.
Figure 86 - Herbert Hoover Dike soil-cement-bentonite 40 ft. trial 3.

Figure 87 - Herbert Hoover Dike soil-cement-bentonite 40 ft. trial 4.

Figure 88 - Herbert Hoover Dike soil-cement-bentonite 40 ft. trial 5.
HH Dike 3

Figure 89 - Herbert Hoover Dike soil-cement-bentonite 20 ft. trial 1.

Figure 90 - Herbert Hoover Dike soil-cement-bentonite 20 ft. trial 2.

Figure 91 - Herbert Hoover Dike soil-cement-bentonite 20 ft. trial 3.
Figure 92 - Herbert Hoover Dike soil-cement-bentonite 20 ft. trial 4.

Figure 93 - Herbert Hoover Dike soil-cement-bentonite 20 ft. trial 5.

Phase 2 Data:

Figure 94 - Phase 2 CB fine sand overlying coarse sand test 1.
Figure 95 - Phase 2 CB fine sand overlying coarse sand test2.

Figure 96 - Phase 2 CB fine sand overlying karstic bedrock.

Figure 97 - Phase2 SCB fine sand overlying karstic bedrock.

The second SCB test of fine sand overlying karstic bedrock and the two tests for the jointed bedrock do not have graphs due to data collection issues. The data was not good or erosion occurred too rapidly.