LABORATORY MODELING OF PIPING INITIATION BEHAVIOR THROUGH
CONSTRUCTED OUTLETS

by

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of

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ABSTRACT

Laboratory Modeling of Piping Initiation Behavior Through Constricted Outlets

by

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Internal erosion often occurs when seepage flow is concentrated into a small, unprotected opening. One such example is where sandy soil is eroded through a defect in an overlying clay layer, resulting in a sand boil in the process. The erosion initiates through the heave and backward piping mechanisms and continues beneath the clay layer through the piping process, forming a pipe that progresses toward the source of the seepage. The initiation of erosion at the seepage flow concentration is a complex mechanism involving a number of hydraulic and soil mechanics principles, including: flow concentration, soil arching, heave, detachment of soil grains, and transportation of soil grains.

A laboratory testing program is being performed to investigate the mechanisms of erosion into a concentrated, unprotected exit. The study builds upon previous research on the mechanisms of piping initiation performed at Utah State University and uses a
similar apparatus. A number of different soils representing a range of grain size, grain shape, and gradations are being forced to erode into a range of constricted seepage exits. The exit is fixed with a riser pipe to model the upward transport of eroding soils. The results are compared with axisymmetric finite element analyses in order to develop a better understanding of the initiation process for backward erosion piping.

(152 Pages)
Laboratory Modeling of Piping Initiation Behavior Through
Constricted Outlets

Ibrahim A. Ibrahim

Backward erosion piping is a type of internal erosion. It occurs beneath water structures as a result of seepage force. It needs two conditions to take place; 1) sufficient hydraulic head to drive the seepage force, and 2) cohesive layer overlies the sand layer and this will lead to concentration of flow. Many researches were done in the past in an attempt to predict the critical gradient. But it was just taking into account the buoyant unit weight which found in recent research that piping can occur at much lower values than was predicted. And other researches were too conservative for predicting critical hydraulic gradient.

The objectives of the research are to help better understanding the mechanism of failure during backward erosion piping and to investigate the factors affecting the average critical gradient. A physical model was developed at Utah State University to simulate this problem. Different types of soils with a wide range of gradation, unit weight, and specific gravity were used with different diameter of outlets to provide a good results of backward erosion piping process. This can help to improve more practical ways for predicting and preventing this type of erosion. This research can lead to further studies with different
scales of models and soil types for better predicting of backward erosion piping and trying
to develop more efficient methods. This will increase the public safety in terms of improve
the design of earth structure such as levees and dams and helps the efforts to avoid such
failures similar to which happened in the past.
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I’m very thankful to USAID for their financial support and for giving me the opportunity to study in USA.

A special thanks to my family. Words cannot express how grateful I am to my mother, brother, sisters, and brothers in law for all of the sacrifices that you’ve made on my behalf. Your prayer for me was what sustained me thus far.

Finally, I want to thank my Father (may Allah be merciful to him) for making this all possible.

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CHAPTER 1

1. INTRODUCTION

1.1 Backward Erosion Piping

Backward erosion piping is a type of internal erosion. Voids are formed beneath or inside water retaining structures due to removal of soil by water flow. Pipes then are formed due to continuous removal of soil and voids being connected.

Fig 1-1. shows the different phases of failure in levees or dams triggered by backward erosion piping. Two conditions are needed to form backward erosion piping; 1) sufficient differential head required to drive the force and 2) roof of low permeable cohesive soil or other rigid layer is required to prevent the pipes from collapse. This process is taking place in uniform and cohesionless soils. It starts after the cracking of the top layer as a result of high water pressure. Then seepage erosion occurring and this cause the soil particles to erode upward. The process progresses beneath the levee or dam is small erosion channels in which the flow velocity can transport the eroded particles downstream.

Backward erosion piping conditions are met in different areas in the world. In United States, along the Mississippi river. It had been documented that sand boils were formed due to excessive under seepage during the floods of 1973, 1993 and 1995. Excessive seepage and sand boils still can be observed nowadays. It is found that forming a sand boil was a result of subsequent floods. Also, the failure of New Orleans levee system was most likely caused by backward erosion. In Netherland, during the high waters of 1993 and 1995, sand boils had been observed along rivers Rhine, Waal, Ijssel and Maas. Several
failures were also attributed to backward erosion piping, for instance the failures near Zalk, Nieuwkuijk and Tholen. In China, 90% of total number of failures were because of backward erosion piping.

Research was performed by Sellmeijer and co-workers Delft Hydraulics and Delft Geotechnical laboratories in the Netherlands. This program was first started in the early 1970. It concluded that the backward erosion piping is mostly taking place in the foundation of levees or dams where the eroding soil is fine to medium sand with a uniformity of coefficient \( Cu < 3 \).

Research was done by Fleshman and Rice at Utah State University (Fleshman 2012, Fleshman and Rice 2013, 2014). They conducted vertical seepage tests to measure critical hydraulic conditions for the initiation of piping in sandy soils and to observe the
mechanisms associated with the initiation of piping. They found that the critical gradients depend on grain size, gradation, grain shape, and specific gravity.

This study builds upon previous research on the mechanisms of piping initiation performed at Utah State University and uses a similar apparatus. A laboratory testing program is being performed to investigate the mechanisms of erosion into a concentrated, unprotected exit. Tests were performed on four different types of granular materials: (1) Graded Ottawa according to ASTM C778-03, (2) Ottawa 20-30 according to ASTM C778-03, (3) Garnet Sand, and (4) Graded Zirconium beads. These soils with different gradation, specific gravity, angularity, and shear strength were used to provide a range of results for comparison. These soils are being forced to erode into a range of sizes of constricted seepage exits. The exit is fixed with a riser pipe to model the upward transport of eroding soils. The results are divided into two groups according to the outlet diameters. The larger diameters showed different mechanism than the smaller diameters. Results compared with a finite element analyses in order to develop a better understanding of the initiation process for backward erosion piping.

1.2 Purpose of Research

The purpose of this research is to investigate the mechanism of failure of sandy soils forced to erode through concentrated outlets. A seepage test cell was developed at Utah State University has been used in this research and previous research to measure the differential heads at specific location through the soil sample. This can lead to better understanding of the behavior of the soils during the process of backward erosion piping.
1.3 Report Organization

This thesis includes 6 chapters and two appendices. Chapter 1 is an introduction to the problem and a brief overview of the purpose of the research. Chapter 2 is the literature review related to the topic. Chapter 3 is describing the apparatus were designed to conduct the tests. Chapter 4 summarizing the testing procedure, set-up, and data collection. Chapter 5 presents the analysis of results. Chapter 6 presents the conclusion and findings. Appendix A shows detailed instructions to run the test. Appendix B presents graphs showing the comparison between laboratory and finite element results.
CHAPTER 2

2. LITERATURE REVIEW

2.1 Introduction

The term “internal erosion” is used by the US Bureau of Reclamation (USBR) and US Army Corps of Engineering (USACE) as a generic term to describe erosion of soil particles by water passing through a body of soil. The most common cause of dam failure in the world is from internal erosion of embankments or their foundations (ICOLD 1974, 1983, 1995), (Foster et al, 1998, 2000a).

Failures caused by internal erosion can be categorized into four general failures modes:

1- Internal erosion due to penetrating structures, such as conduits inside the dam;

2- Internal erosion through the embankment;

3- Internal erosion through the Foundation;

4- Internal erosion of the embankment or foundation due to seepage through the embankment eroding the material into the foundation or seepage in the foundation at the surface between foundation and embankment resulting eroding embankment material.

The process of internal erosion can be classified into four phases as it shown Fig 2-1:

1- Initiation of erosion;

2- Continuation of erosion;

3- Progression to form pipe;

4- Initiation of a breach
Fig 2-1. Models for the development of failure by internal erosion (After FOS 99b)
Four mechanisms for initiation of erosion have been identified:

1- Concentrated leak erosion: is the removal of soil particles through the cracks or hydraulic fractures caused by differential settlement during construction or in operation of dam or levee. It also may be due to the low stress around the conduits or in the upper part if the dam. These cracks developed by seepage water through the soil eroding the sides of the pathway.

2- Backward erosion (or piping): is the removal of soil particles from the downstream face progressing backward forming channels. It occurs where a critically high hydraulic gradient at the downstream of the dam erodes particles upward and backward below the dam through the erosion channels and the flow velocity can transport the eroded particles downstream.

3- Contact erosion occurs where a coarse soil like gravel is in contact with a fine soil, and high velocity flow parallel to the contact in the coarse soil erodes the fine soil.

4- Suffusion occurs when water flows through widely graded or gap-graded a cohesionless soils. The finer soil particles are transported by the seepage flow through the pores of the coarser particles. This mainly occurs in internally unstable gap graded non plastic soils, glacial tills, and some fills and filters in dams (ICOLD, 2013).
2.2 Backward erosion

2.2.1 Types of Backward erosion

2.2.1.1 Backward erosion piping

Backward erosion piping begins at the free surface on the downstream side of a dam or levee as shown in Fig 2-2. This free surface may be in a ditch or other excavation penetrating into the eroding cohesionless soil or may form by heaving or cracking of the roof strata overlying the cohesionless soil.

Backward erosion piping occurs where critically hydraulic gradients at the toe of a dam erode particles upwards and backwards beneath the dam through small erosion channels in which the flow velocity can transport the eroded particles downstream.

![Fig 2-2. Geometry of Delft backward erosion piping model (Koenders and Sellmeijer, 1992)](image)

2.2.1.2 Global backward erosion

The most likely for this type to occur is in the cores of dams where there a sloping core is not properly protected by filters or transition zone.
Particles are detached at the downstream surface of the core which not protected by filter or transition zone. The progression of the erosion process is assisted by the gravity, and in this case there is no need for a cohesive soil layer (roof) to form the roof for the pipe.

2.2.2 Experimental modelling of backward erosion piping

Sellmeijer and his colleagues from Delft Geotechnics Laboratories and Delft Hydraulics in the Netherlands carried out more than 70 backward erosion piping tests. The first tests were in flumes and are reported in De Wit et al. (1981), Silvis (1991), Weijers and Sellmeijer (1993). The tests were mostly on fine to medium sands. The sands were uniform with uniformity coefficient Cu = 1.58 to 3.53.

These experiments showed that backward erosion initiates through cracks in the strata overlying the eroding soil. The progress is more than like small channels not just a pipe. These channel are relatively small with heights typically 4 to 10 times the \(d_{15}\) of the soil and often less than 2 mm. The development of channels stops when the head is less than the critical head and start to erode if the head increased.

Further experiments were carried out in 2009 and 2010 at Deltares (Van Beek et al. 2011, Sellmeijer et al. 2011). The tests were performed at three different scales and the process of backward erosion was concluded as:

*Phase 1: Seepage*

Seepage is a precondition for piping, and it starts as soon as the hydraulic head is applied.
**Phase 2: Backward erosion**

At the beginning of erosion, there is a small rearrangement of grains and creating small channels. The sand transport increases as the hydraulic head is increased and it ceases if the hydraulic head drop below a critical value. In full experiment, sand boils increased due to increasing hydraulic head and it was observed in pore pressure gauges that a small decrease in pressure occurred at the locations of boils. In small and medium tests, as the hydraulic head increase, a crater formed as a result of sand deposition. This description of the process before reaching critical head. At head greater than critical head, the flow is increases and erosion rate increasing and doesn’t cease. The rate of erosion can be on the order of cubic decimeters per hour.

**Phase 3: widening of the channels**

This phase is observed in all experiments. In full scale experiments it can be tracked through the water pressures measurements, while in small and medium scale experiments, it is observed through transparent cover.

As soon as the pipe reaches upstream side, a pressure surge occurs in the pipe which leads to sand erosion in case of small scale experiments. In medium and full scale experiments, blockage took place because the widening process took a long time. However, in small scale experiments, it doesn’t lead to significant blockage of the pipe. This may be due to the small seepage length. The widening pipe develops from the downstream to upstream. This process taking days in full scale experiment. The flow and sand transport to the exit is very small and, as soon as the widening reaches downstream,
the flow and sand transport increased suddenly. The change in situation can be observed by pore pressure gauges. But in field, there is a little warning before sand boils change to mud flow and failure.

**Phase 4: failure and breakthrough**

Failure occurs shortly after the increasing of sand transportation (widening phase complete), but can be delayed due to collapse of the levee causing the first pipes to close.

Richards and Reddy (2012) conducted a broad laboratory experimental programme using a newly developed True Triaxial Piping Test Apparatus (TTPTA). Non-cohesive soil and cohesive soil were tested to assess the factors affection the initiation of piping. The study showed that the presence of non-plastic fines considerably reduced the required seepage velocity that can initiate piping. Three modes of failure were recognized: concentrated leak erosion, backward erosion, and suffusion. It was found that for non-cohesive soils, backward erosion was the primary mode of failure with seepage velocity of 0.026 - 0.036 ft/sec required to initiate piping in uniform sand. Also this study concluded that seepage velocity is a better predictor for piping initiation in non-cohesive soil than the hydraulic gradient.

Richards and Reddy (2014) extended their research to study initiation of internal erosion (backward erosion and suffusion) in non-cohesive soil. They studied the initiation of backward erosion using kinetic energy. Critical kinetic energy ($E_{krit}$) occurs where the seepage overcomes the interparticle resisting force and soil particles are mobilized. Wide range of soil were tested using True Triaxial Piping Test Apparatus (TTPTA). It found that
effective stress and angle of seepage path are influencing $E_{\text{crit}}$. Also percentage of non-plastic fines increased the factor of safety against backward erosion.

Fleshman and Rice (2014) performed a laboratory tests to assess the mechanics of initiation the piping erosion process in non-cohesive soils. A laboratory apparatus was design to measure critical hydraulic conditions for the initiation of piping in sandy soil. the tests results showed that the higher specific gravity, the greater piping resistance. Also angular and graded soils showed greater piping resistance. From tests and observations, four stages of piping initiation development were detected (1) first visible, (2) heave progression, (3) boil formation, and (4) total heave. Also they found that hydraulic gradient deeded to initiate piping measured were higher than those calculated by Terzaghi and Peck 1948.

Rebecca Allan et al. (2015) performed an experimentally based study on backward erosion piping. They designed a flume as same as the one used at University of Florida (Schmertmann, 2000) to neglect scale effect. Observations on the backward erosion process showed good agreement with similar researches particularly those done by van Beek et al. (2011). Five stages were observed; fluidization, tip progression, equilibrium, forward deepening, and failure.
2.2.3 Methods for prediction of initiation of backward erosion piping

2.2.3.1 *Empirical methods*

In the past a lot of research on piping has been done. The methods of Bligh (1910), Lane (1935) are used to calculate piping. They involve calculation the length of the seepage path beneath a concrete structure, including cut-off walls. The ‘line of creep’ method developed by Bligh’s rule (1910) was applied in the Netherlands design practice till early 1990s. The critical line of creep, which is the total horizontal and vertical seepage line, is required to design safe dike dimensions. Lane (1935) improved Bligh’s empirical approach and extended his research for more soil types, but argued that vertical seepage length weighs three times more than horizontal length and adjusted the empirical rule. Based on a total of 200 cases in the United States, He developed an empirical formula called ‘weighted line of creep’. Lane’s empirical method took into account the erosion resistances of different types of soils. The weighted creep ratio is then expressed as:

\[
C_W = \frac{0.33B + \sum t}{h_{cr}}
\]

Where \( C_W \): weighted creep ratio

- \( B \): horizontal seepage path length
- \( t \): vertical seepage path length
- \( h_{cr} \): critical head difference between the upstream and downstream
Terzaghi and Peck (1948) discussed the problem in this method that it is entirely empirical and it gave designs which may be conservative, reasonable, or may lead to failure.

2.2.3.2 Terzaghi and peck

Terzaghi and Peck (1948) showed that backward erosion piping will occur when a heave or zero effective stress condition occurs in cohesionless soils at the downstream toe of a levee or dam. They consider for forming piping that it is necessary for a roof of cohesive or cemented soil to be overlying the erodible strata.

This method was dominating the design of levee and dams, particularly in the USA (e.g. USACE). However, it has commonly been assumed, at least implicitly, that if backward erosion initiates to form a sand boil it will progress to form a pipe, at least under repeated loading from successive floods. However, this is not a fact because there are many sand boils occurs in the foundation of levees on the Mississippi in any flood but there are few failures and that also supported by the experiments performed by the Delft and University of Florida (Schmertmann, 2000). The latest experiments by Deltares indicate that when the sand boil form the backward erosion will progress unless the gradient is reduced by dropping the water level or building sand bags around the sand boil.
2.2.3.3 USACE simplified method

This method may be used to localize area where backward erosion piping could initiate. The zero effective stress condition is calculated with the assumption that the horizontal seepage flow discharge in the aquifer layer under the impervious blanket is equal to the vertical flow through the silty or clayey soils that form the impervious blanket. It assumed one layer for the top stratum or pervious foundation with an effective thickness and permeability the effective length of the upstream ($L_1$) and downstream ($L_3$) blankets is

$$L_1 = \frac{k_f}{\sqrt{k_{br}Z_{br}d}} , \quad L_3 = \frac{k_f}{\sqrt{k_{bl}Z_{bl}d}}$$

where

$L_1$ = effective length of upstream blanket

$k_f$ = horizontal permeability of pervious foundation

$k_{br}$ = vertical permeability of upstream blanket

$Z_{br}$ = thickness of upstream blanket

$d$ = thickness of pervious foundation

$L_3$ = effective length of downstream blanket

$k_{bl}$ = vertical permeability of downstream blanket

$Z_{bl}$ = thickness of downstream blanket
Upstream blankets should be designed so that under maximum reservoir conditions the pressure head under the blanket at the downstream toe of the dam and the rate of discharge through the pervious foundation are acceptable. The pressure head under the blanket at the downstream toe of the dam (Fig 2-3) is

$$ h_0 = \frac{hL_3}{L_1 + L_2 + L_3} $$

$$ h_c = \frac{Z_{bL} Y_{sub}}{Y_w} $$

Heave occurs when the factor of safety (F) against uplift or heaving reaches one at the downstream toe.

$$ F = \frac{h_c}{h_0} $$

Fig 2-3. Seepage in earth foundations from USACE (1993)
2.2.3.4 Sellmeijer and co-worker at Deltares method

Sellmeijer (1988), Sellmeijer and Koenders (1991) simulated the progression of a pipe with a mathematical model for backward erosion piping based on experiments carried out at Delft laboratories.

This model was refined using the results of the Deltares testing and is presented in Van Beek et al. (2012) and Sellmeijer et al. (2011).

The critical gradient is determined as a product of three contributions: resistance factor, scale factor and geometrical shape factor. The refined equations for the critical gradient at which backward erosion will progress are:

\[
\frac{H}{L} = \frac{1}{C} = F_RF_SF_G
\]

\[
F_R = \eta \frac{\gamma p}{\gamma w} \tan \theta \left( \frac{RD}{RD_m} \right)^{0.35} \left( \frac{U}{U_m} \right)^{0.13} \left( \frac{KAS}{KAS_m} \right)^{-0.02}
\]

\[
F_S = \frac{d_{70}}{K} \left( \frac{d_{70m}}{d_{70}} \right)^{0.6}
\]

\[
F_G = 0.91 \left( \frac{D}{L} \right)^\alpha \text{ where } \alpha = \frac{0.28}{\left( \frac{D}{L} \right)^{2.7}} - 0.04
\]

where

\( H [m] \): hydraulic head across structure

\( L [m] \): seepage length (= base length of the embankment)

\( D [m] \): thickness of sand layer under the embankment

\( C [-] \): erosion coefficient

\( F_R [-] \): resistance factor
$F_s \ [-]:$ scale factor

$F_G \ [-]:$ geometrical shape factor

$RD \ [%]:$ relative density

$U \ [-]:$ uniformity coefficient $Cu = d_{60}/d_{10}$

$KAS \ [%]:$ roundness

$d_{70} \ [m]:$ soil particle diameter for which 70% by weight of the soil is finer

$\gamma_p \ [kN/m^3]:$ submerged unit weight of soil particles 9.8(G-1)

$G \ [t/m^3]:$ soil particle density

$\gamma_w \ [kN/m^3]:$ unit weight of water

$\eta \ [-]:$ Whites drag coefficient

$\vartheta \ [\text{deg}]:$ bedding angle (angle of repose) of sand

$K \ [m^2]:$ intrinsic permeability

Where

$$K = \frac{u}{\mu} k$$

$\mu \ [m^2/s]:$ kinematic viscosity

$G \ [m/s^2]:$ gravity

$k \ [m/s]:$ hydraulic permeability

$H, D \text{ and } L$ are defined in Figure 2.3. In these equations, the variables are normalized by the mean values in the data set. For the data set used by those authors, the mean values.
They indicate that the refinement has been determined from small scale tests and it is not completely clear if there is scale effect, so the outcome for large scale structures may not be properly modeled. They also indicate that the equations should only apply within the limits of the parameters during testing. These limits are given in Table 2-1.

Table 2-1. Limits of Sellmeijer et al. (2012) method

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Minimum</th>
<th>Maximum</th>
<th>Mean</th>
</tr>
</thead>
<tbody>
<tr>
<td>RD</td>
<td>34%</td>
<td>100%</td>
<td>72.5%</td>
</tr>
<tr>
<td>U</td>
<td>1.3</td>
<td>2.6</td>
<td>1.81</td>
</tr>
<tr>
<td>KSA</td>
<td>35%</td>
<td>70%</td>
<td>49.8%</td>
</tr>
<tr>
<td>d70</td>
<td>1.5E-04 m</td>
<td>4.3E-04 m</td>
<td>2.07E-04 m</td>
</tr>
</tbody>
</table>

They found that the equations predicted the large scale experiments behavior quite well for the fined grained soil, but it was not so accurate for the soil with d70 = 260 μm.

Very significant scale effects are though to affect this method. The smaller the structure, the higher the critical gradient. In the tests, the value of the scale factor FS ranged from 0.134 for the large scale tests to 0.192 for the medium-scale tests and 0.421 for the small-scale tests.

It should be noted that Sellmeijer et al. (2012) emphasize that the above equations do not include any margin of safety and that for design rules they are likely to apply factors of safety.

2.2.3.5 Schmertmann method

Schmertmann (2000) developed a method based on backward erosion piping tests in flumes at the University of Florida. The tests were carried out in a number of groups, including those done by Townsend et al, (1988) who carried out 15 tests. Those tests were
carried out on a range of soils from fine to medium sands, up to coarse sand/fine gravel mixes.

There are differences in geometry between tests in university of Florida and those in Delft. So Schmertmann applied a correction factors to be able to plot all results together. He found that there a sharp increase of the critical average gradient related to the uniformly coefficient $C_u (d_{60}/d_{10})$ of the soils tested.

$$i_c = 0.05 + 0.183 (C_u - 1)$$

It is important to note that most of the tests done by Schmertmann (32 out of 39) carried out on sand with uniformity coefficient less than 3.2. The other 7 tests were carried out on gap-graded sand and well graded soil with uniformity coefficient around 6. So it is uncertain on how to use this method for soils with uniformity coefficient more than 3.
CHAPTER 3

3. TESTING APPARATUS

3.1 Introduction

The testing apparatus used to perform the experiments is designed to measure pore pressures and critical gradients in soil under vertical flow. It has been used in research by Fleshman and Rice (Fleshman 2012, Fleshman and Rice 2013, 2014), Jacobson (2013), and Keizer (2014). As it shown in Fig 3-1, the water flows between the two reservoirs through the soil specimen. The high head reservoir (at the left) connected to the bottom cell, while the low head reservoir (at the right) connected to the top cell and the flow goes perpendicular to the exit face of the soil specimen to ensure the symmetricity and uniform flow to determine the magnitude of the exit gradient more easily.

Fig 3-1. Schematic of testing apparatus
The soil sampler used in this research is a 5-inch long by 4-inch diameter cylinder shaped Plexiglas mold with four different outlet diameter risers (0.125, 0.25, 0.75, and 1-inch). The soil sampler provided with two retaining screens, one to support the soil and the other for retention of finer soil particles. Both retaining screens allow free flow of water through the soil sample. The sample holder is sealed between two enclosed cells, the low-head pressure cell and the high-head pressure cell. The high-head pressure is connected to the bottom cell and the low-head pressure is connected with the top cell. The differential pressure head can be controlled by the high head reservoir via the Mariotte tube.

The pore pressures are measured at four different locations within the soil sample. Three of them are at the center of the soil samples located at 3.75 inch, 2 inches and 0.75 inches from the top of the sample holder. The fourth port is located at 0.375 inches but shifted 0.5-inch outside from the center of the sample holder. A fifth measurement of the total differential head is also recorded across the entire sample. Each pore pressure measurement is made by using a Validyne DP15-26 differential pressure transducer installed between the port and the low head pressure cell. The pore pressure pressures are made relative to the top enclosed cell. A flowmeter is installed to measure the flow rate between the high head reservoir and the low head reservoir.

Campbell Scientific CR 1000 data logger is used to collect data every 0.1 seconds during a test. The data logger is connected to a computer so the data can be viewed in
real time on the computer screen and saved for later analysis. Each test is videoed from the side and can be correlated to video time.

3.2 Design of sample holder

The soil sampler is a 5-inch long by 4-inch diameter cylinder shaped Plexiglas mold as it shown in Fig 3-2. Two retaining screens (one made of sieve mesh for the retention of finer soil particles, and one less flow restrictive but stronger to support the soil sample) placed at the base to allow water to flow freely through the soil sample. Silicone gel coated the inside and top of the sample holders to model soil to soil contact through the sample and this mechanism was used by earlier research done by Fleshman and Rice (Fleshman 2012, Fleshman and Rice 2013, 2014).

Four different risers were used with the soil sample to represent the clay layer above the cohesionless soil to model the case of backward erosion piping. The diameters of the risers are 1, 0.75, 0.25, 0.125-inch and they attached at the top of the soil sampler. Silicone layer were used between the soil sampler and the riser to: 1) ensure the full
contact between the soil sampler and the riser, and 2) model the soil-soil contact. Fig 3-3 shows different diameters of risers.

The soil sample holder as it shown in Fig 3-2 contains four tubes to measure pore pressure within the soil sample. Three tubes extend to the center of the cross sectional area at 0.75, 2, and 3.75-inch from the top of the holder and labeled as PPA, PPB, and PPC respectively. The forth tube is shifted 0.5-inch outside the center and located at 0.375-inch from the top of the holder and labeled as PPD. The four tubes are connected to pressure transducers by way of quick connectors, and pore pressure measurements are recorded by differential pressure transducers installed between the port and the low head pressure cell. The total differential head between the reservoirs was also measured using a differential pressure transducer installed between the top and bottom cells.
3.3 Design of Differential Pressure Cells

The pressure cells are made of two cylindrical Plexiglas sections separated by sheet of Plexiglas with an opening in the middle for the placement of the soil sample holder as it shown Fig 3-4. Two 1-inch thick, 13-inch diameter cylindrical, Plexiglas plates are bolted to the top and the bottom of the pressure cells. They are sealed with O-rings and vacuum grease. The plates are bolted to each other by eight steel threaded rods to create an airtight pressure chamber.

![Fig 3-4. Differential pressure cell](image)

Ports for vacuum and CO₂ lines are located near the top of the lower cell and near the bottom in the upper cell. Also, there are ports at the top of the lower cell and in the upper plate to release air pressure during filling the cell with water. These ports are fitted with quick connectors that allow easy attachment and removal. Four pore pressure ports were installed through the top plate in addition to one through the wall of the bottom pressure cell to allow the pore pressure measurements to be made in the sample. The two quick
connect ports used for vacuum and CO2 also used to measure the total differential pressure between the top and bottom pressure cells. A bypass valve is used between the top and bottom pressure cells to control the pressure during the setup of the test.

3.4 Head Reservoirs

The head reservoirs consist of the high head reservoir and the low head reservoir. The two reservoirs are fixed and made of cylindrical Plexiglas with an inside diameter of 11.25-inch. The high head pressure connects to the bottom cell and it can hold up about 20 gallon of water, while the low head pressure connects to the top cell and it can hold about 10 gallon of water. These conditions fulfill all tests requirements.

As it shown in Fig 3-5, the high head reservoir is at a higher elevation than the low head reservoir. The differential pressure between the two tanks can be controlled by

![Fig 3-5. High-head and low-head reservoirs](image)
adjusting Mariotte tube inside the high head reservoir. Zero differential pressure occurs when the bottom edge of Mariotte tube is at the same elevation as the outlet tube in the low head reservoir. Back pressure of 15 psi is applying to both tanks resulting the bubble in the high head reservoir descends to the bottom of the Mariotte tube while the water level at the top of outlet tube. To increase the differential pressure, a hand crank is turned counter clock wise at 13 rotations per 1-inch. Also there is a scale attached to the high head tank to adjust the differential pressure in inches.

3.5 Instrumentation

The instruments used in the test are:

1. Magnetic-flux flow meter.


3. Validyne multi-channel carrier demodulator.

4. Campbell scientific CR 1000 Data Logger.

5. Desktop computer with Campbell Scientific Logger Net 3.4.1.

6. Video camera

   The flow meter installed between high head reservoir and low head reservoir and used to measure the flow rate during the test. Two flow meters were used but the low range one (2.4-7.8 gallons per hour) is used in most of the tests to be able to measure the flow rate through low permeable soils.

   The differential pressures during the tests are measured by Five Validyne transducers, as shown in Fig 3-6, connected to the pressure cells, soil sample, and demodulator. One
of the pressure transducers is connected between top and bottom cells to measure the total differential head across the sample. The other four are connected between the top cell and one of the four different ports labeled PPD, PPA, PPB, and PPC. The maximum differential head that can be endure by Validyne pressure transducer used in the tests is 16-inches.

The Validyne demodulator used to convert the signals sent by differential pressure transducers to (0-20 mA) signal that can be read by the data logger. The demodulator has four different channel and attached to another one channel Validyne demodulator model CD23 to acquire the five channels needed for the tests. The demodulators have zero and span screws that are used to zero and calibrate the pressure transducer reading so that the data collected is displayed in inches of water. Fig 3-7 and Fig 3-8 shows the two demodulators used in the tests. The pressure transducers were calibrated once and are checked regularly. The pressure transducers readings are zeroed at the beginning of each test.

![Validyne differential pressure transducers](image_url)

Fig 3-6. Validyne differential pressure transducers
The data logger used is a CR 1000 from Campbell Scientific as seen below in Fig 3-9. A program was written that controls how the data logger samples the flow meter and pressure transducers every tenth of a second and averages the readings over one second before storing them in a data file. The data logger is connected to a computer so that the data can be viewed and plotted in real time on the computer screen. Each test is videoed from the side. The timing of the video is exactly the timing of data logger so the data can be linked in the test analysis.
Fig 3-9. Data Logger CR1000
CHAPTER 4

4. TESTING PROCEDURE

4.1 Introduction

This chapter summarizes the testing procedure. Detailed step by step instruction to setup, and run the tests are included in Appendix A.

4.2 Sample Preparation

The soil sample is prepared by dry-raining and vibration techniques. The sand is placed in the soil sampler in small lifts (approximately 0.5-inch thick) and the sample holder tapped on the side to densify the sand by vibration. The soil holder is filled over capacity and leveled to the top of the sample holder. Silicon sheet shaped as a circle is put above the soil, then a riser is tied to the sample holder via cable ties. The sampler weighed before and after placing the soil. The weights are used to calculate dry unit weight which used to calculate the void ratio and theoretical critical gradient.

The sample holder is then put inside the pressure cell apparatus. Pressure ports PPC is connected first by the quick connect, and the sample holder is tightened down on the diving plate and gasket and sealed between the pressure cells. PPD, PPA, and PPB are then connected to the top plate by quick connects. After these connections are made, the top and bottom plates are bolted together to seal the pressure cells.
4.3 Carbon Dioxide (CO₂) Saturation

All valves except the bypass valve are closed to completely seal off the pressure cells before CO₂ saturation. The CO₂ line then connected to the top pressure cell port via quick connect and applied to the soil for 15 minutes. The density of CO₂ is heavier than the air, so CO₂ replaces the air in the voids between soil particles. CO₂ is soluble in water which speed up the saturation process.

The pressure cell then filled with water from the high head pressure cell (lower cell). The pressure cell is filled from bottom to top with a very slow rate to avoid distortion of soil sample when the water falls off the riser. Monitoring of the soil is needed when the water starts to rise up through the soil to avoid having any movement of the soil. The valve in the top plate is opened very slowly if needed to release the pressure and allow the water to fill the cell.

4.4 Application of Back-Pressure

Once the apparatus is filled with water, the reservoirs are both pressurized to 15 psi back-pressure. The back-pressure dissolves any remaining air bubbles in the soil into solution and fully saturates the soil sample. Pore pressure lines are bleed to get rid of the air inside the lines and pressure transducers. The pressure transducers are zeroed and the system is ready to the test.
4.5 Data Collection

The laboratory instrumentation setup can be seen in Fig 4-1. Data logger is started after the reservoirs been pressurized. LoggerNet 3.4.1. is opened on the desktop computer. The data collection program is sent from the computer to the data logger and the camera is setup and started to film the top of the soil sample. Zeroing all pressure transducers again when the Mariotte tube and the tip of the outlet tube at the same elevation.

Fig 4-1. Test setup (minus the

4.6 Testing

Differential pressure across the sample is increased by rising up Mariotte tube. The Mariotte tube is slowly raised in 1-inch increments and held for at least 90 seconds until the sand starts to develop a failure mechanism. At this point, the Mariotte tube is raised in 0.5-inch lifts. The time for each lift at this point depends on the reaction of the soil and
the observation movement. The test is allowed to reach equilibrium before the differential pressure is increased. Increasing of differential pressure continues until the sample fails.

4.7 Sample Failure

After the soil sample has failed, the water valves left opened for one minute. The data logger continues to collect data until the pressure transducers reach zero. The camera is turned off and the video is backed up on a computer for later analysis. The data is collected from the data logger and saved as a data file and opened into an excel spreadsheet to be analyzed.
CHAPTER 5

5. DATA ANALYSIS AND RESULTS

5.1 Introduction

Tests were performed on four different types of granular materials: (1) Graded Ottawa according to ASTM C778-03, (2) Ottawa 20-30 according to ASTM C778-03, (3) Garnet Sand, and (4) Graded Zirconium. These soil with different gradation, specific gravity, angularity, and shear strength were used to provide a range of results for comparison. Table 5-1 shows the properties of soil tested. A 4-inch sample holder was used with four different riser diameters (1, 0.75, 0.25, and 0.125-inch) to understand the effect of the outlet size on the behavior of soil during erosion. A summary of the tests performed is presented in Table 5-2.

Table 5-1. Properties and characteristics of soil tested (Richard Keizer 2014)

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Specific Gravity</th>
<th>D50 mm</th>
<th>Coefficient of Uniformity</th>
<th>Internal friction Angle</th>
<th>Min &amp; Max Void Ratio</th>
<th>Sat. Unit Weight (pcf)</th>
<th>Void Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Graded Ottawa</td>
<td>2.65</td>
<td>0.37</td>
<td>1.76</td>
<td>38°</td>
<td>0.48 – 0.72</td>
<td>130</td>
<td>0.53</td>
</tr>
<tr>
<td>Garnet Sand</td>
<td>4.05</td>
<td>0.16</td>
<td>1.25</td>
<td>44°</td>
<td>0.96 – 1.54</td>
<td>154</td>
<td>1.08</td>
</tr>
<tr>
<td>Ottawa 20-30</td>
<td>2.66</td>
<td>0.70</td>
<td>1.51</td>
<td>38°</td>
<td>0.49 – 0.71</td>
<td>130</td>
<td>0.54</td>
</tr>
<tr>
<td>Graded Zirconium Beads</td>
<td>3.84</td>
<td>0.37</td>
<td>1.57</td>
<td>35°</td>
<td>0.46 – 0.60</td>
<td>180</td>
<td>0.51</td>
</tr>
</tbody>
</table>
Table 5-2. Summary of tests performed

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>1-inch riser</th>
<th>0.75-inch riser</th>
<th>0.25-inch riser</th>
<th>0.125-inch riser</th>
</tr>
</thead>
<tbody>
<tr>
<td>Graded Ottawa</td>
<td>16 tests</td>
<td>2 tests</td>
<td>3 tests</td>
<td>2 tests</td>
</tr>
<tr>
<td>Garnet Sand</td>
<td>6 tests</td>
<td>3 tests</td>
<td>2 tests</td>
<td>4 tests</td>
</tr>
<tr>
<td>Ottawa 20-30</td>
<td>1 test (didn’t fail)</td>
<td>1 test (didn’t fail)</td>
<td>1 test (didn’t fail)</td>
<td>2 tests (didn’t fail)</td>
</tr>
<tr>
<td>Graded Zirconium</td>
<td>1 test (didn’t fail)</td>
<td>1 test (didn’t fail)</td>
<td>2 tests</td>
<td>2 tests (didn’t fail)</td>
</tr>
</tbody>
</table>

The range of maximum differential pressure results for all tests of different soils with different outlet diameters are presented from Fig 5-1 to Fig 5-4.

Fig 5-1. Total differential pressure range in all tests with different diameters in Graded Ottawa
Fig 5-2. Total differential pressure range in all tests with different diameters in Garnet Sand

Fig 5-3. Total differential pressure range in all tests with different diameters in Ottawa 20-30
5.1.1 Overview of Analyses

This chapter contains results and discussion of measured pore pressures and flow rates. Normalized curves of the data were developed to illustrate the behavior of the soil during the erosion process. Results were then compared to videos recorded during the tests in an attempt to understand the stages of failure.

Four stages of failure were identified in tests performed with 1-inch and 0.75-inch outlet diameters: 1) first boiling, 2) boiling around perimeter, 3) boiling of the whole aperture, and 4) boiling reaching the top of riser. In the 0.25-inch and 0.125-inch outlets, two stages were identified: 1) first boiling, and 2) start of exiting at top of the riser similar
to step 4 for the larger diameter. A detail discussion of these stages is presented in the data analyses section.

Finite element seepage analyses were performed using analysis program SEEP/W. Results from laboratory tests were compared to FEM results. A reverse analysis was performed by modifying the hydraulic conductivities and boundary locations of the loosened soil zones to track the downward erosion progression.

Hydraulic exit gradients were calculated at the exit point using SEEP/W. Horizontal and vertical exit gradient were calculated in the different stages of failure to detect the mechanism of failure. Downward loosening of the soil and erosion channels the surface (pipes) were identified by the exit gradients and that was also observed in the recorded videos.

Reynolds number and terminal velocity (Stokes’ law) were calculated for the different types of soils with various outlet diameters. This can help to understand of the behavior of the soil through the risers. Results were compared to the velocity measured by flowmeter.

5.2 Data Analysis

Two different plots were created for each test to analyze the behavior of the soil during erosion. The first plot is the data collected through the test which includes: 1) the total differential pressure, 2) differential head between each of the pore pressure ports and the upper pressure cell (locations of pore pressure ports are presented in Fig 5-5), and 3) the flow rate. An example of this graph is presented in Fig 5-7. The second plot
The normalized curves were developed to detect the deviation in pore pressure due to the start of soil loosening. A linear relationship between the sensor readings and the total differential pressure was developed by plotting the data from PPA, PPA, PPB, and PPC versus the DP for the initial pre-erosion portion of the test. Then the data was divided by the expected differential head calculated using the linear relationship that would occur if no erosion or change in permeability occurred in the sample.

Steps used for creating the Normalized curves are: 1) Linear fitting of PPD, PPA, PPB, PPC, and FM (Flowmeter reading) before any movement in the soil to assure that there is no increasing in permeability as shown in Fig 5-6, 2) using Equation 5-1 to calculate Normalized Differential Head and Flow rate

\[
\text{Normalized Differential Head} = \frac{PP(A,B,C \text{ or } D) \text{or FM}}{m \text{ (Total differential head) + b}}
\]

Equation 5-1

Where \( m \) is the slope and \( b \) is the y-intercept. The normalized differential head and flow rate should be equal one until the permeability in the soil changes due to start of erosion.

This section will look at one set of plots for each soil with different outlet risers to analyze the progression of the tests and the stages which occurred during the test.
Fig 5-5. Soil sampler dimensions with locations of pore

Fig 5-6. Linear fit of pore pressure measurements and flow rate to total differential head
Tests were performed on Graded Ottawa and using a sampler holder with a 1-inch diameter riser. A plot of the pore pressure versus time for all four sensors is presented in Fig 5-7a and the normalized data is presented in Fig 5-7b.

The normalized curves for all four sensors fluctuate around 1.0 until the first boiling occurred at a total differential head about 2 inches and about 11 minutes into the test. After first boiling, the normalized differential head at PPD starts to deviate below a value of 1.0 apparently due to loosening of soil in the upper part of soil sample. PPA starts to follow PPD after about 4 additional minutes at about 3.5 inches of differential head (15 minutes into the test) due to the downward progression of soil loosening that cause drop in pore pressure at this location. The deviation in PPD and PPA continue increasing with increasing differential head until PPD reaches its maximum deviation at about 23 minutes into the tests. The maximum deviation of PPD corresponds with the time when the boiling was occurring around the entire perimeter of the riser outlet. This boiling stage was accompanied by formation of some channels along the surface radiating out from the outlet as it shown in Fig 5-8b.

Before the deviation of PPD reaches its maximum value a small deviation occurred in the normalized PPB and PPC data due to the continuous downward progression of the erosion. The deviation at PPD becomes constant for about 8 minutes while the boiling progressed from the boiling around the entire perimeter to boiling of the whole mass. The soil then starts to heave up into the riser tube in response to increasing differential head. At the beginning of heaving process, the normalized head at PPD starts to deviate
back upward while PPA continues to deviate downward; a response that is likely due to the soil starting to rise into the riser tube. The rising soil causes a relative buildup of pore pressure at PPD due to the resistance in the riser tube and a decreasing at PPA as the loosening of soil progresses downward. With increasing differential head, PPD and PPA begin to deviate upward together while PPB and PPC show small decreases. This behavior is likely due to the same process as just described for PPD and PPA expanding throughout the sample. This behavior progress until sample failure occurs.

The data from the test on graded Ottawa with 0.75-inch diameter, as shown in Fig 5-9. The normalized head hovers around 1.0 until around 2.5 inches differential head when the first boiling occurred. Deviation starts to occur in the normalized pressure transducer plots due to loosening of the soil in the upper part of the soil sample and this observation explains why the maximum deviation happened to PPD and the lowest to PPC. This behavior differs from the 1-inch diameter riser test, indicating more of a balance between the horizontal erosion mechanism and the vertical erosion mechanism. The deviations increase with increasing differential head due to downward progression of soil loosening until boiling occurred around the whole perimeter at about 19 minutes into the test and 4.5 inches differential head. Then the boiling progresses from the whole perimeter to the total mass. It took about 6 minutes for that progression with small deviation of all of them.

After the boiling of the whole mass, heaving process starts to occur. The soil is raising up relieving the pressure at the bottom transducers and it building up the pore pressure at the upper two transducers. That explains why at 37 minutes into the test the
normalized PPD and PPA are deviating up while the deviations in PPB and PPC are going down.

On the test of graded Ottawa with 0.25-inch diameter, as shown in Fig 5-10, the normalized head fluctuates around 1.0 until the first boiling. This occurred at a differential head of about 2.5 inches and 9 minutes into the test. The normalized data for PPD begins to deviate below a value of 1.0. Less deviation occurred in PPA due to the first boiling and almost nothing occurred in PPB and PPC. These reading appear to suggest the first boiling has allowed the upper portions of the soil to loosen.

The deviation of PPD and PPA do not occur as the test progressed for an additional 11 minutes (21 minutes from the start of the test) and up to a differential head equal to 6.5 inches. At this point the soil begins to rise up and fall off the top of the riser and the normalized head for the four pressure sensors drops. This drop in the normalized data is because more soil eroded causing drop in pore pressure. The dropping in pore pressure is maximum at PPD and lowest at PPC because the erosion is taking place first at the higher portion of the soil sample. At about 29 minutes into the test and at about 8.5 inches of differential head, the pressure at PPD and PPA is increasing while the pressure is decreasing at PPB and PPC. This is due to the erosion now increasing at the lower portion of the sample thus decreasing the resistance to flow and transferring some of the head drop back up to the top of the sample. As more erosion occurs at the end of the test the resistance to flow becomes more uniform throughout the sample, albeit at a much higher hydraulic conductivity, thus the normalized plots all approach a value of 1.0.
On the test of graded Ottawa with 0.125-inch diameter, Fig 5-11 normalized test data shows the values are moving around 1.0 until the soil start falling off the riser. This occurred at a differential head of about 3 inches and about 11 minutes into the test. The normalized data for PPD begins to deviate below a value of 1.0. The normalized data for the PPD continues to drop with every associated increase of total differential head. This is due to the continued loosening of the top soil. The normalized data for PPB also starts to deviate like PPD with very similar values. That shows that in this case the soil failure is more loosening the soil as a column and less by creation of radial channels.

Also PPB and PPC start to deviate like other but with a small value because they are located in the lower portion of the soil sample. As the test progressed, the deviation of PPB and PPC increased due to the downward erosion progress. The flow meter normalized data also indicates the erosion process by deviation above the 1.0 values and increasing as the differential head increased.
Fig 5-7. Test data (a) and Normalized test data (b) for Graded Ottawa with 1-inch diameter

PPA, PPB, PPC, and PPD: Differential head between pressures at (A, B, C, and D) and upper part of the cell – DP: Total differential head – FM: Flow meter readings
Fig 5-8. Progression of boiling in Graded Ottawa with 1-inch diameter
PPA, PPB, PPC, and PPD: Differential head between pressures at (A, B, C, and D) and upper part of the cell – DP: Total differential head – FM: Flow meter readings

Fig 5-9. Test data (a) and Normalized test data (a) for Graded Ottawa with 0.75-inch diameter
Fig 5-10. Test data (a) and Normalized test data (b) for Graded Ottawa with 0.25-inch diameter
PPA, PPB, PPC, and PPD: Differential head between pressures at (A, B, C, and D) and upper part of the cell – DP: Total differential head – FM: Flow meter readings

Fig 5-11. Test data (a) and Normalized test data (b) for Graded Ottawa with 0.125-inch diameter

Plots for the Garnet Sand, Ottawa 20-30, and Graded Zirconium beads are presented below from Fig 5-12 to Fig 5-23. Very similar behavior was observed in all types of soil. It should be noted that the soil didn’t fail in the case of Ottawa 20-30 and Graded Zirconium
except for diameter 0.25-inch for Zirconium Beads. For Ottawa 20-30, the soil didn’t fail due to the high permeability of the soil. And for Zirconium, because it’s high specific gravity (3.84) with $D_{50} = 0.373$ mm which means that the soil particles are very heavy so our laboratory apparatus reached its maximum differential head without causing the soil to fail.

PPA, PPB, PPC, and PPD: Differential head between pressures at (A, B, C, and D) and upper part of the cell – DP: Total differential head – FM: Flow meter readings

Fig 5-12. Test data (a) and Normalized test data (b) for Garnet Sand with 1-inch diameter
It can be noticed from Fig 5-12b, that the first portion of loosening is close to the case of 0.75-inch with Graded Ottawa. While at the end, it is more than like 1-inch Graded Ottawa.

<table>
<thead>
<tr>
<th>PPA, PPB, PPC, and PPD: Differential head between pressures at (A, B, C, and D) and upper part of the cell – DP: Total differential head – FM: Flow meter readings</th>
</tr>
</thead>
</table>

Fig 5-13. Test data (a) and Normalized test data (b) for Garnet Sand with 0.75-inch diameter
Behavior of Garnet Sand as shown in Fig 5-13b is very close to Graded Ottawa with 0.75-inch outlet.

PPA, PPB, PPC, and PPD: Differential head between pressures at (A, B, C, and D) and upper part of the cell – DP: Total differential head – FM: Flow meter readings

Fig 5-14. Test data (a) and Normalized test data (b) for Garnet Sand with 0.25-inch diameter
Behavior of Garnet Sand as shown in Fig 5-14b is very close to Graded Ottawa with 0.25-inch outlet.

PPA, PPB, PPC, and PPD: Differential head between pressures at (A, B, C, and D) and upper part of the cell – DP: Total differential head – FM: Flow meter readings

![Graph showing differential head and flow over time.](image)

**Fig 5-15.** Test data (a) and Normalized test data (b) for Garnet Sand with 0.125-inch diameter
Behavior of Garnet Sand as shown in Fig 5-15b is very close to Graded Ottawa with 0.125-inch outlet.

PPA, PPB, PPC, and PPD: Differential head between pressures at (A, B, C, and D) and upper part of the cell – DP: Total differential head – FM: Flow meter readings

Fig 5-16. Test data (a) and Normalized test data (b) for Ottawa 20-30 with 1-inch diameter
PPA, PPB, PPC, and PPD: Differential head between pressures at (A, B, C, and D) and upper part of the cell – DP: Total differential head – FM: Flow meter readings

Fig 5-17. Test data (a) and Normalized test data (b) for Ottawa 20-30 with 0.75-inch diameter
PPA, PPB, PPC, and PPD: Differential head between pressures at (A, B, C, and D) and upper part of the cell – DP: Total differential head – FM: Flow meter readings

Fig 5-18. Test data (a) and Normalized test data (b) for Ottawa 20-30 with 0.25-inch diameter
PPA, PPB, PPC, and PPD: Differential head between pressures at (A, B, C, and D) and upper part of the cell – DP: Total differential head – FM: Flow meter readings

Fig 5-19. Test data (a) and Normalized test data (b) for Ottawa 20-30 with 0.125-inch diameter
Behavior of Graded Zirconium as shown in Fig 5-20b is very close Graded Ottawa with 0.75-inch outlet.
It can be noticed from Fig 5-21b, that the first portion of loosening is close to the case of 1.00-inch with Graded Ottawa. While at the end, it is more than like 0.75-inch with
Graded Ottawa.

Fig 5-22. Test data (a) and Normalized test data (b) for Graded Zirconium with 0.25-inch diameter

Behavior of Graded Zirconium as shown in Fig 5-22b is very close Graded Ottawa with 0.25-inch outlet.
Fig 5-23. Test data (a) and Normalized test data (b) for Graded Zirconium with 0.125-inch diameter

Behavior of Graded Zirconium as shown in Fig 5-23b is very close Graded Ottawa with 0.25-inch outlet.
5.3 FEM analysis and results

Finite element models were developed for all tests and types of soil with different apertures used in the research. The computer program SEEP/W was used to simulate the tests using an axisymmetric model presented in Fig 5-24. Fig 5-24 shows boundary conditions used to model the problem with axisymmetric analysis to simulate the 3-D effect. The solid line represents the wall of soil specimen and it indicates that there is no flow through this line (no-flow boundary condition). On the lower boundary the solid circles indicate the constant head boundary where the higher head was applied to the sample. The semi-solid circles along the upper boundary represent a constant head boundary with the lower head applied over the aperture diameter. A zero head was applied to aperture line because in the experiment the total differential head was measured by the difference between the upper and lower parts of the cell. Finally, the center line represents a no-flow boundary caused by symmetry and the radial 3-D effect.

Two models were developed for each soil type with all different apertures. The purpose of the models was (1) to provide a baseline of the hydraulic regime prior to erosion occurring and (2) to provide a means for simulating the effects of the progressing erosion. The first models are the soil as a one region with the original permeability constant throughout the sample. These models were performed to provide a baseline of the hydraulic pressure regime used to compare the laboratory results with SEEP/W results as shown in Fig 5-26 to Fig 5-27. The second model divides the soil region into three zones with different permeabilities as shown in Fig 5-29 to simulate the loosened and/or eroded
zones of soil that develop at different differential pressures. The size and shape of the soil zones is adjusted on a trial-and-error basis to match the observed pressures with FEM output. Also the video recording was used to observe the surface movement of soil to assist in defining the extent of the zones in the models.

It can be noticed from Fig 5-26 to Fig 5-27 that the difference between the finite element results and lab result are high at the upper part of the soil cell while it decreases at the lower part of the soil cell. This is because the soil erosion mainly occurs at the upper part.

![Diagram](image_url)
Fig 5-25. Comparison between Experiment and SEEP/W in Graded Ottawa with 1-inch riser and 1 inch DP

Fig 5-26. Comparison between Experiment and SEEP/W in Graded Ottawa with 1-inch riser and 2 inch DP
Fig 5-27. Comparison between Experiment and SEEP/W in Graded Ottawa with 1-inch riser and 4 inch DP

Fig 5-28. Comparison between Experiment and SEEP/W in Graded Ottawa with 1-inch riser and 3 inch DP
Regions of increased hydraulic conductivity (k) were used in the FE model to simulate the progression of erosion in the soil. These regions are summarized in Table 5-3. By changing the shape of these regions until the calculated heads match the measured head results at the sensor locations (PPD, PPA, PPB, and PPC) at various stages of the experiment. Thus the erosion progress can be monitored through subsequent increasing of differential pressure and noticing the changing in shape of regions. Region 2 represents soils that have loosened in response to the applied hydraulic gradient. Previous tests performed by Fleshman and Rice (Fleshman 2013, Fleshman and Rice 2014) indicate an increase in hydraulic conductivity of about 5 times occurs due to this loosening. Region 3 represents soils where horizontal flow channels have formed. A 50-fold hydraulic conductivity increase is estimated for this region and validated through trial and error and comparison with observed radial erosion during the tests.
Table 5-3. Hydraulic Conductivity used in FEM in Graded Ottawa sand

<table>
<thead>
<tr>
<th>Region</th>
<th>$H_C$ (ft/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$K_0 = 0.00165$</td>
</tr>
<tr>
<td>2</td>
<td>$5K_0 = 0.00825$</td>
</tr>
<tr>
<td>3</td>
<td>$50K_0 = 0.08225$</td>
</tr>
</tbody>
</table>

On the test of graded Ottawa with 1-inch diameter riser, finite element (FE) seepage analyses were performed to model the effects of the loosening soil. Fig 5-30 shows contours of pressure for cases of differential pressure (DP) varying from 1 inch to 4 inches. The results show the downward progression of regions 2 and 3 due to increasing differential pressure. It also can be noticed forming channel around the surface and that was observed through the video recording. FEM analysis was performed with 1-inch differential pressure increment to a value of 4.0. At DP greater than 4-inches boiling start forming, thus changing the SEEP/W model making no longer applicable to simulate the problem.

Very similar to 1-inch diameter, Fig 5-31 show how the loosened parts progressed in 0.75-inch diameter riser due to continuous increasing of differential pressure. It starts to progress downward with forming radial channels along the surface which were also observed through the recording video of this test.
In the test with 0.25 and 0.125-inch diameter risers with Graded Ottawa sand, Fig 5-32 and Fig 5-33 show the downward progression due to increasing the differential pressure. It can be noticed from these figures the difference between the smaller diameters and the larger diameters. In the small diameter, the loosened zone progresses downward because the hydraulic gradient increased around the outlet due to the narrow diameters. That increased the velocity of the flow beneath the outlet causing erosion of soil. The comparison between experimental data and FEM are shown in Fig 5-35 to Fig 5-44 for Graded Ottawa for the different diameters. The results from SEEP/W of the loosening soil model for all four pore pressure measurements showed very good agreement with laboratory data.
Fig 5-31. FE results of progression of loosening soil in Graded Ottawa with 0.75-inch diameter riser
Fig 5-32. FE results of progression of loosening soil in Graded Ottawa with 0.25-inch diameter

Fig 5-33. FE results of progression of loosening soil in Graded Ottawa with 0.125-inch diameter
Fig 5-34. Comparison between Experiment and SEEP/W in Graded Ottawa with 1-inch riser and 1 inch DP

Fig 5-35. Comparison between Experiment and SEEP/W in Graded Ottawa with 1-inch riser and 2 inch DP
Fig 5-36. Comparison between Experiment and SEEP/W in Graded Ottawa with 1-inch riser and 3 inch DP

Fig 5-37. Comparison between Experiment and SEEP/W in Graded Ottawa with 1-inch riser and 4 inch DP
Fig 5-38. Comparison between Experiment and SEEP/W in Graded Ottawa with 0.75-inch riser and 1 inch DP

Fig 5-39. Comparison between Experiment and SEEP/W in Graded Ottawa with 0.75-inch riser and 2 inch DP
Fig 5-40. Comparison between Experiment and SEEP/W in Graded Ottawa with 0.75-inch riser and 3 inch DP

Fig 5-41. Comparison between Experiment and SEEP/W in Graded Ottawa with 0.25-inch riser and 2 inch DP
Fig 5-42. Comparison between Experiment and SEEP/W in Graded Ottawa with 0.25-inch riser and 1 inch DP

Fig 5-43. Comparison between Experiment and SEEP/W in Graded Ottawa with 0.25-inch riser and 3 inch DP
Fig 5-44. Comparison between Experiment and SEEP/W in Graded Ottawa with 0.125-inch riser and 2 inch DP

Fig 5-45. Comparison between Experiment and SEEP/W in Graded Ottawa with 0.125-inch riser and 3 inch DP
Like Graded Ottawa, FEM were performed to the rest of the soil types and it showed similar results. Fig 5-47 to Fig 5-57 are showing the FEM results for Garnet Sand, Ottawa 20-30, and Graded Zirconium.
Fig 5-47. FE results of progression of loosening soil in Garnet Sand with 1-inch diameter riser

Fig 5-48. FE results of progression of loosening soil in Garnet Sand with 0.75-inch diameter riser
Fig 5-49. FE results of progression of loosening soil in Garnet Sand with 0.25-inch diameter riser

Fig 5-50. FE results of progression of loosening soil in Garnet Sand with 0.125-inch diameter riser
Fig 5-51. FE results of progression of loosening soil in Ottawa 20-30 with 1-inch diameter riser

Fig 5-52. FE results of progression of loosening soil in Ottawa 20-30 with 0.75-inch diameter riser
Fig 5-53. FE results of progression of loosening soil in Ottawa 20-30 with 0.25-inch diameter riser

Fig 5-54. FE results of progression of loosening soil in Ottawa 20-30 with 0.125-inch diameter riser
Fig 5-55. FE results of progression of loosening soil in Graded Zirconium with 1-inch diameter riser

Fig 5-56. FE results of progression of loosening soil in Graded Zirconium with 0.75-inch diameter riser
Fig 5-57. FE results of progression of loosening soil in Graded Zirconium with 0.25-inch diameter riser

Fig 5-58. FE results of progression of loosening soil in Graded Zirconium with 0.125-inch diameter riser
5.4 Comparisons of soils

5.4.1 Critical Gradient from FEM

Fig 5-59 shows the maximum differential head and average critical gradient for the soil used in the research with the different diameters of outlet. The points in circles shows that the soil didn’t failed at this specific diameter and that the value presented is the maximum differential head applied. It can be noted from this figure that the maximum differential pressure at failure increases as the unit weight increases as in Table 5-4. This is because the higher unit weight of the soil, the greater piping resistance the soil will have.

Table 5-4. Unit weight for the different types of soil

<table>
<thead>
<tr>
<th>Soil</th>
<th>Saturated unit weight (pcf)</th>
<th>Specific Gravity</th>
<th>Void ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Graded Ottawa</td>
<td>130</td>
<td>2.65</td>
<td>0.53</td>
</tr>
<tr>
<td>Garnet Sand</td>
<td>154</td>
<td>4.05</td>
<td>1.08</td>
</tr>
<tr>
<td>Ottawa 20-30</td>
<td>130</td>
<td>2.66</td>
<td>0.54</td>
</tr>
<tr>
<td>Graded Zirconium</td>
<td>180</td>
<td>3.84</td>
<td>0.51</td>
</tr>
</tbody>
</table>

The outlet diameters were divided into two groups to explain how the critical gradient changed in the same soil with different diameters. The first group is the larger diameters which are 1 and 0.75 inch. The second group is the lower diameters of 0.25 and 0.125 inch.
From Fig 5-59, it can be noticed that all the soils are resisting more in 1-inch diameter more than they resist in 0.75-inch diameter. Lower diameter in case of 0.75-inch causing the flow to concentrate more than 1-inch diameter. This is due to concentration of the flow increasing the velocity of the water at the outlet causing the soil to erode more rapidly.

The second group is the smaller diameters 0.25 and 0.125-inch. These diameters are too small therefore the soil arches over the opening, thus increasing the resistance to erosion. The soil with diameter 0.125-inch resist more than with 0.25 inch because in both small diameters the velocity of the flow is very high. But in case of 0.25 inch, more soil erodes due to high velocity with bigger diameter. Also in case of 0.125-inch diameter, the soil will probably be able to form a bridge between particles due to the small outlet. This is increasing the ability of the soil to resist more against erosion.

Fig 5-59. Max Differential pressure for the four types of soil (Note: circle means the soil didn’t fail at this diameter)
Fig 5-60 to Fig 5-63 shows differential pressure comparison between different soils at the four stages of loosening progression.

Fig 5-60. Differential pressure at first boiling (Stage 1) for the four types of soil

Fig 5-61. Differential pressure at boiling around perimeter (Stage 2) for the four types of soil
Fig 5-62. Differential pressure at boiling at whole aperture (Stage 3) for the four types of soil

Fig 5-63. Differential pressure at top of riser (Stage 4) for the four types of soil
SEEP/W was also used to calculate the maximum hydraulic exit gradients ($i_{xy}$, $i_x$, and $i_y$) at the exit point for the four types of soil during the stages of failure. Horizontal hydraulic gradient ($i_x$) is the change in head over the horizontal distance while the vertical hydraulic ($i_y$) gradient is the change in head over the vertical distance as it shown in Fig 5-64. The vector sum of the horizontal and vertical gradient is $i_{xy}$. The vertical gradient is responsible for the downward progression and the horizontal gradient causes the forming of the radial channel around the surface. Fig 5-66 to Fig 5-76 show that the gradients ($i_{xy}$, $i_x$, and $i_y$) increase as the soil progresses to failure due to the continuous erosion of soil.

Fig 5-64. Horizontal and vertical exit hydraulic
Fig 5-65. Maximum Hydraulic Exit Gradient in Graded Ottawa in different stages for 0.25 and 0.125-inch diameters

Fig 5-66. Maximum Hydraulic Exit Gradient in Graded Ottawa in different stages for 1 and 0.75-inch diameters
Fig 5-67. Maximum Hydraulic Exit Gradient in Garnet Sand in different stages for 0.25 and 0.125-inch diameters

Fig 5-68. Maximum Hydraulic Exit Gradient in Garnet Sand in different stages for 1 and 0.75-inch diameters
Fig 5-69. Maximum Hydraulic Exit Gradient in Ottawa 20-30 in different stages for 0.25 and 0.125-inch diameters

Fig 5-70. Maximum Hydraulic Exit Gradient in Ottawa 20-30 in different stages for 1 and 0.75-inch diameters
Fig 5-71. Maximum Hydraulic Exit Gradient in Graded Zirconium in different stages for 0.25 and 0.125-inch diameters

Fig 5-72. Maximum Hydraulic Exit Gradient in Graded Zirconium in different stages for 1 and 0.75-inch diameters
Fig 5-73. Comparison of Gradient ($i_{xy}$) measured form FEM for all types of soils in first and last stages at 1-inch riser.

Fig 5-74. Comparison of Gradient ($i_{xy}$) measured form FEM for all types of soils in first and last stages at 0.75-inch riser.
Fig 5-75. Comparison of Gradient ($i_{xy}$) measured from FEM for all types of soils in first and last stages at 0.25-inch riser.

Fig 5-76. Comparison of Gradient ($i_{xy}$) measured from FEM for all types of soils in first and last stages at 0.125-inch riser.
5.4.2 Flow Velocity

Stokes’ law was used to calculate the terminal velocity which occurs at the point of balance between gravitational force and viscous dragging force of upward flowing water. It was calculated using both maximum and minimum particle diameter for each type of soil as shown in Table 5-5 then a range of upper and lower limit of terminal velocity was developed.

\[ v = \frac{m g - \rho_f V g}{6\pi \mu r} \]  

Equation 5-2

Where \( m \) is mass of the particle (kg), \( g \) is gravitational acceleration (m/sec\(^2\)), \( V \) is the volume of the sphere (m\(^3\)), \( \rho_f \) is the density of the fluid (kg/m\(^3\)), \( \mu \) is the viscosity of the fluid (kg/m.sec), and \( r \) is the diameter of the sphere (m).

<table>
<thead>
<tr>
<th>Soil</th>
<th>( d_{\text{min}} ) (mm)</th>
<th>( d_{\text{max}} ) (mm)</th>
<th>( d_{50} ) (mm)</th>
<th>Specific Gravity</th>
<th>Max. Velocity (ft/sec)</th>
<th>Min. Velocity (ft/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Graded Ottawa</td>
<td>0.20</td>
<td>0.50</td>
<td>0.37</td>
<td>2.65</td>
<td>0.73</td>
<td>0.12</td>
</tr>
<tr>
<td>Garnet Sand</td>
<td>0.12</td>
<td>0.16</td>
<td>0.14</td>
<td>4.05</td>
<td>0.14</td>
<td>0.08</td>
</tr>
<tr>
<td>Ottawa 20-30</td>
<td>0.60</td>
<td>0.85</td>
<td>0.70</td>
<td>2.65</td>
<td>2.38</td>
<td>1.19</td>
</tr>
<tr>
<td>Graded Zirconium</td>
<td>0.20</td>
<td>0.50</td>
<td>0.37</td>
<td>3.84</td>
<td>1.27</td>
<td>0.20</td>
</tr>
</tbody>
</table>

Also terminal velocity was measured in the lab by throwing particles for each type of soil into a 50 cm tube filled with water. Time of settle then measured by stopwatch.
Maximum and minimum velocities were calculated by the range of time for the soil particle to settle through the tube.

Fig 5-77 show the average velocities of the flow through the risers in Graded Ottawa compared to the range of calculated and measured terminal velocities. In the cases of 1 and 0.75-inch diameter outlets, it shows a slight increasing of the velocity through the stages. But still less than the lower range of the terminal velocity. In case of 0.25 and 0.125-inch diameter outlets, the velocity of the flow through the stages in both risers was found to be within the range of the terminal velocity. Laboratory results showed consistency with our theory.

Fig 5-77. Flow velocity versus different stages in Graded Ottawa sand for all diameters

In Garnet Sand test as it shown in Fig 5-78, for 1 and 0.75 inch risers, the velocity in the risers are slightly increasing through the stages but less than the lower range of the terminal velocity. On the other hand, in case of 0.125 and 0.125 inch diameters, the first
stage starts with a velocity almost within the range of the terminal velocity and become more than the upper range when it reaches the second stage.

Fig 5-78. Flow velocity versus different stages in Garnet sand for all diameters

Fig 5-79. Flow velocity versus different stages in Graded Zirconium for all diameters
Very similar to graded Ottawa, Graded zirconium as shown in Fig 5-79, for 1 and 0.75-inch diameters, the velocity of the flow through the riser are less than the terminal velocity. While in 0.25 and 0.125-inch diameters, the velocity of the flow is within the range of the terminal velocity.

In the tests of Ottawa 20-30, as shown in Fig 5-80, the velocity of the flow in case of 1 and 0.75 inch diameters is less than the terminal velocity in both stages. However, in 0.25 the velocity of the flow in the riser in the first stage is less than the terminal velocity and increased to be in the range of terminal velocity in the second stage. And in 0.125-inch diameter, the flow velocity is in the range in the first stage and exceed the range in the second stage.

![Graph showing flow velocity versus different stages in Ottawa 20-30 for all diameters](image)

**Fig 5-80. Flow velocity versus different stages in Ottawa 20-30 for all diameters**
Reynolds number used to examine the flow in the risers to assess whether it is laminar or turbulent. The value of Re considered laminar if Re is less than 2000 and considered turbulent if it is higher than 4000. Equation 5-3 was used to assess Re.

\[ Re = \frac{\rho V D}{\mu} \]  

Equation 5-3

Where \( \rho \) is the density of the fluid (kg/m\( ^3 \)), \( V \) is the velocity of the flow in the riser (m/sec), \( D \) is the diameter of the riser (m), and \( \mu \) is the viscosity of the fluid (kg/m.sec).

In the tests of Graded Ottawa with the four outlet diameters as shown in Fig 5-81, the Re is varying from 100 to 900 which indicates that the flow is laminar. In Garnet sand with the four diameters as shown in Fig 5-82, the Re is ranging from 20 to 400 and that indicates the flow is laminar. In Graded Zirconium as it is shown in Fig 5-83, Re is ranging from 100 to 1400 which is laminar flow.

In Ottawa 20-30 tests as it shown in Fig 5-84, in case of 1 and 0.75 inch risers, Re is ranging from 300 to 700 which considered laminar flow. While in case of 0.25-inch outlet, Re ranging from 1000 to 2000 and in 0.125-inch outlet, Re is ranging from 2000 to 5000. That indicates the flow starts becomes transitional and turbulent. The cause of that is due to the high permeability of Ottawa 20-30 and the lower diameters of the risers and that increases the velocity of the flow very much.
Fig 5-81. Reynolds number versus different stages in Graded Ottawa for all diameters

Fig 5-82. Reynolds number versus different stages in Garnet Sand for all diameters
Fig 5-83. Reynolds number versus different stages in Graded Zirconium for all diameters

Fig 5-84. Reynolds number versus different stages in Ottawa 20-30 for all diameters
5.5 Interpretation of Soil Behavior in Riser

The following is an interpretation of the soil behavior in the riser based on the Reynolds number and Stokes’ law plots and the observed failure behavior, the outlet diameters were divided into groups. The first group includes 1 and 0.75-inch diameters and the second group includes 0.25 and 0.125-inch diameters. In the larger diameters as the soil starts to rise up in the tube the water and suspended soil particles begin to act as a combined fluid until its combined in one bulk. As the concentration of soil particles increases in some areas of the riser, the high density areas begin to fall down and the lower density areas start rise up, thus creating the boiling observed in these tests. The same process occurs again with increasing the differential head until the upper portion soil begins to fall off the riser. So, in this case, Stokes’ law doesn’t apply because of the interaction between the soil particles and water, where zones of varying densities are rising up and falling down. This behavior creates a condition that is against the assumption of Stokes’ law which is the particle is moving in one direction. Thus explaining why in 1 and 0.75-inch diameters the velocity is lower than the terminal velocity.

On the other hand, in case of 0.25 and 0.125-inch diameters, the soil particles during erosion are rising up very fast due to the high velocity of the flow in the smaller diameters. The particles begin falling off the riser at a point close to where the average velocity is equal to the terminal velocity. Also, the concentration of soil particles in the riser remains much lower than with the larger diameter risers; essentially remaining clear. Thus, it appears that the flow in the smaller diameter risers is staying close to the laminar flow
condition. With increasing differential head, the soil particles again start to rise up in the tube and fall off from the edges. So, in this case it is applicable to apply Stokes’ law because there is no interaction between zones of varying density and the particles are moving in one direction. This is observed in the plots of Stokes’ law in lower diameter in almost all soil types. Because the first stage the velocity of the flow is within the range of the terminal velocity and in the final stage the flow velocity starts to exceed the upper limit of the terminal velocity which means the soil begins to fail.
CHAPTER 6

6. SUMMARY AND CONCLUSION

The thesis presents the results of laboratory testing to measure the differential head at four locations and the total differential head in an attempt to better understanding the behavior of different sandy soils during the initiation of backward erosion piping. Four different outlet diameters were attached to 1-inch riser were used to represent the cohesive roof which is necessary for initiation of backward erosion piping. Four different soils were used in the tests representing a wide range of grain size, grain shape, gradation, and specific gravity. The outlet diameters were 1, 0.75, 0.25, and 0.125-inch.

The results were divided into two groups according to the behavior through the failure process. The first group is the larger diameters including 1-inch and 0.75-inch. Four stages of failure were observed 1) first boiling, 2) boiling around perimeter, 3) boiling of the whole aperture, and 4) boiling reaching the top of riser. First boiling occurs when the soil around the outlet start to loosen due to the differential head applied. This boiling increases due to increasing the differential head until it develops around the perimeter of the outlet. With continuous increasing of differential head, the boiling become at the whole area of aperture and then the soil starts to heave up into the riser tube until it reaches the top of the riser and then falling down when the failure starts to occur.

The second group is the smaller diameters including 0.25-inch and 0.125-inch. Two stages were noticed 1) first boiling, and 2) boiling reach the top of riser. The first stage is due to the loosening of the upper portions of the soil. As the differential head increases,
particles of the soil from the upper portion start to rise up through the riser and reaches the top of the riser and then fall down. This process is repeating as the differential head increasing, but after each erosion the state of equilibrium occurs where the particles can rearrange and most likely be able to form bridging between particles to resist erosion. At the failure the soil can’t resist any more due to the high differential head and a huge amount of soil start to rise up and fall off the riser causing the sample to collapse.

Characteristics of soil such as gradation, size, and specific gravity affects the average critical gradients measured at the laboratory tests. It showed that the soils with higher specific gravity resisted more than the soils with lower specific gravity. And the angular soil showed greater piping resistance except for graded Zirconium beads it resists more because the high specific gravity and the larger $D_{50}$.

A finite element models were developed for all tests basically for simulation the loosening parts by increasing the permeability. And by playing with the shape of zones of higher permeabilities in an attempt to match the results with lab tests results. This showed understanding how the loosens parts progressed during the backward erosion piping and it compared to the videos recorded for the tests and it showed good agreement.

Stokes’ law can apply for smaller diameters (0.25 and 0.125-inch) because the mechanism of failure with these diameters is matching the assumption of Stokes’ law. However, in case of larger diameters (1 and 0.75-inch), the results were off the limits because the failure behavior includes interaction between particles and also random
motion of particles due to boiling. The mechanism of failure is against Stokes’ law, so Stokes’ law can’t be applied in this case.
REFERENCES


APPENDICES
APPENDIX A

Step by step instructions to run tests:

1. Make sure wall tank is full of water (between the top metal strap and the top of tank)
   a. If it is full, move on to **step 2**
   b. If wall tank needs to be filled:
      i. Turn on water vacuum in sink (**MAKE SURE THE BOTTOM RED VALVE IS CLOSED**)
      ii. Plug in RED #2 line into “VACUUM HOSE” on panel board, then plug into large wall tank (**WAIT FOR 1 MINUTE**)
      iii. OPEN BLUE needle valve on bottom of large wall tank (you should see flow coming through hose)
      iv. Plug in RED TANK #1 hose to panel board #1 and vent (or pressurize to 3.00 psi)
      v. Once tank is full:
         1. Close needle valve on bottom of large wall tank
         2. Unplug RED TANK #1 hose from panel board and turn off panel #1
         3. Continue to let vacuum run for 1 hour to de-air; **charge camera**
   c. **If TANK #1 (tall gray tank behind you) is empty and needs to be filled:**
      i. Connect RED TANK #1 hose to panel #1 and VENT
      ii. Connect BLUE TANK #1 hose to “FILL CELL” on panel board (**if the tank is completely empty, the tank will fill in 15 minutes, but no longer!!!; be careful to NOT LET THE CELL OVERFLOW**)
      iii. Turn off “FILL CELL” and UNPLUG BLUE TANK #1 and RED TANK #1 from board
2. **De-air Wall tank**
   a. Turn on water vacuum in sink (**MAKE SURE THE BOTTOM RED VALVE IS CLOSED**)
   b. Plug in RED #2 line into “VACUUM HOSE” on panel board, then plug into large wall tank
   c. Let tank de-air for 1 hour; **charge camera**
3. **Prepare Soil Sample**
   a. Weight the sample holder.
   b. Fill sample holder with the appropriate sand (use vibratory compaction by taping sides while filling)
   c. Weigh full sample holder
   d. Attach appropriate riser to the top of soil sample by cable ties.
4. **Assemble sample holder and Water cell**
   a. Connect the bottom pore pressure tubes
b. Washers and wing nuts – tighten
c. Connect other pore pressure tubes to the appropriate tubes through the lid
   i. Connect proper lid tube to the top pore pressure measurement (PPD)
   ii. Connect proper lid tube to the pore pressure measurement (PPA)
   iii. Connect proper lid tube to the middle pore pressure measurement (PPB)
   iv. Tuck tubes to the back side so there is a clear camera view of the sample
d. Put the lid on & washers and wing nuts – tighten

5. Vacuum and CO2 through soil sample
   a. CLOSE TOP and BOTTOM APPARATUS needle valves
   b. OPEN BYPASS VALVE
   c. Connect CO2 to top water cell (for ~10 min)
   d. Turn on CO2 tank
   e. Disconnect CO2 from top cell and turn off CO2 tank after ~10 min.

6. Fill water cell with water
   a. Fill water cells from the bottom cell very slowly with continuous monitoring
      (make sure it doesn’t disturb the soil).
   b. Open the upper needle very slowly to relieve the pressure and to allow the
      water to fill the cell.
   c. Let the pore pressure lines fill with water before bleeding out air to completely
      fill the top cell

7. Bleeding Pore Pressure Lines
   a. Leave the bottom cell connected to the large wall tank (make sure bypass valve
      is open)
   b. Bleed the pore pressure lines one at a time
   c. Bleed out remaining air in top of water cell

8. Finish setting up Test
   a. Connect wall tank lines to top and bottom water cells.
      i. LOOSELY connect SMALL WALL TANK hose to top
         1. Let hose fill up by opening valve from SMALL WALL TANK, but
            not the APPARATUS VALVE
         2. Keep SMALL WALL TANK VALVE open

9. Connecting Pressure Transducers – watch differential pressures
   a. Open bleed valves on pressure transducers
   b. Bypass valve on the outside of the water cells is OPEN
   c. Connect the negative side of the pressure transducers to the top water cell
   d. Connect the positive side of the pressure transducers to the bottom water cell
   e. Bleeding positive and negative lines
      i. Slightly open bottom valve to APPARATUS to provide flow
ii. **After done bleeding transducer lines, close bottom valve to APPARATUS and open top water cell bleed valve (needle valve) to relieve pressure (slowly)**

f. Disconnect the positive side of the pressure transducers (quick connects)

g. Connect the pore pressure lines to the positive side of the pressure transducers

i. Top pressure measurement (PPD) to Pressure transducer #5

ii. Second pore pressure measurement (PPA) to Pressure transducer #1

iii. Third pore pressure measurement (PPB) to Pressure Transducer #2

iv. Bottom pore pressure measurement (PPC) to Pressure Transducer #3

h. Slightly bottom valve to APPARATUS to let lines bleed air bubbles

i. **Tightly** Close bleed valves on pressure transducers

10. Zeroing the Pressure Transducers

a. Open Logger Net on computer (on the desktop)

b. Turn on the power to the data logger

c. **Resend** the piping program

d. Click the “**Connect**” button

e. Set the “**Table Start**” value to 1 to start collecting the data

f. Open **graph 1** to view the pressure transducers

g. **Zero** the pressure transducers using the Logger Net

h. Collect the data by clicking “**Collect Now**”

i. Click the “**Disconnect**” button to stop collecting data (where the “connect” button was)

j. Reset the “**Table Start**” and “**Zero LCDM**” value to zero

k. Open My computer → C drive → Campbell Scientific → Logger net → CR1000 data, right click and delete the data.

11. Pressurize the Wall Tanks

a. Make sure the bypass valve is open

b. Open all water valves (not bleed valves) to large wall tank, except the top valve

c. Remove little thingy, and close **RED** drain valve

d. Connect the “air wall tanks” line to Panel 2 and pressurize the wall tanks slowly up to 15 psi

12. Starting the Test

a. Set up camera side view

b. **Connect** the data logger by clicking the “connect” button on the connect screen

c. Start Video

d. Set “**Zero LCDM**” and “**Table Start**” values equal to 1 (in that order)

e. After zero’s are established open top cell valve first

f. Close the bypass valve

13. Proceed with test

a. Raise in one inch increments
b. Wait to raise head until the pressure transducer readings level out (~1.5 to 3 min)
c. After reaching a certain point (depending on the type of sand) start raising at 0.5” increments
d. If any sand boils form stop raising the head and wait for a little bit before raising again

14. At failure
   a. Stop camera recording
   b. Wait for about a minute before closing the valves after the failure
   c. Keep collecting data until the pressure transducers readings level off (~3 to 5 min)
   d. Collect the data by clicking the “Collect Now” button in the “Connect Screen”
   e. “Disconnect”
   f. Open My computer → C drive → Campbell Scientific → Logger net → CR1000 data, right click and open with Notepad
   g. Save as → Desktop → Ibrahim “date sand type”

15. Clean up
   a. Close logger net
   b. Turn off data logger power
   c. Close BOTH VALVES on APPARATUS
   d. Open Bypass valve
   e. Disconnect “air wall tanks” from panel board, turn off pressure
   f. VENT LARGE WALL TANK and RED VALVE to let water empty to the sink
   g. Put bottom hose into the sink
   h. SLOWLY open bottom valve of APPARATUS just to relieve pressure
   i. Open bleed valves on pressure transducers
   j. Disconnect the pressure lines on the back of the APPARATUS
   k. Disconnect the pore pressure lines and re-connect the positive pressure lines (the ones in parallel)
   l. Drain water from the water cell (open needle valves to allow water to drain)
   m. Take lid off and disconnect pore pressure lines
   n. Take water cell to sink after disconnecting the riser and wash out sand into a #200 sieve
   o. Remove the sample holder from water cell
   p. Copy Video from camera to external hard drive

16. DONE!
APPENDIX B

Fig B-1. Comparison between Experiment and SEEP/W in Garnet Sand with 1-inch riser and 1 inch DP

Fig B-2. Comparison between Experiment and SEEP/W in Garnet Sand with 1-inch riser and 2 inch DP
Fig B-3. Comparison between Experiment and SEEP/W in Garnet Sand with 1-inch riser and 3 inch DP

Fig B-4. Comparison between Experiment and SEEP/W in Garnet Sand with 0.75-inch riser and 1 inch DP
Fig B-5. Comparison between Experiment and SEEP/W in Garnet Sand with 0.75-inch riser and 2 inch DP

<table>
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<th>DP</th>
<th>PPD</th>
<th>PPA</th>
<th>PPB</th>
<th>PPC</th>
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<tbody>
<tr>
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<td>1</td>
<td>1.5</td>
<td>2</td>
</tr>
</tbody>
</table>

Fig B-6. Comparison between Experiment and SEEP/W in Garnet Sand with 0.75-inch riser and 3 inch DP

<table>
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<th>DP</th>
<th>PPD</th>
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<th>PPB</th>
<th>PPC</th>
</tr>
</thead>
<tbody>
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<td>0</td>
<td>0.5</td>
<td>1</td>
<td>1.5</td>
<td>2</td>
</tr>
</tbody>
</table>

<table>
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<tr>
<th>DP</th>
<th>PPD</th>
<th>PPA</th>
<th>PPB</th>
<th>PPC</th>
</tr>
</thead>
<tbody>
<tr>
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<td>2.5</td>
<td>3</td>
<td>3.5</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Fig B-7. Comparison between Experiment and SEEP/W in Garnet Sand with 0.25-inch riser and 3 inch DP.

Fig B-8. Comparison between Experiment and SEEP/W in Garnet Sand with 0.25-inch riser and 1 inch DP.
Fig B-9. Comparison between Experiment and SEEP/W in Garnet Sand with 0.25-inch riser and 2 inch DP

Fig B-10. Comparison between Experiment and SEEP/W in Garnet Sand with 0.125-inch riser and 1 inch DP
Fig B-11. Comparison between Experiment and SEEP/W in Garnet Sand with 0.125-inch riser and 2 inch DP

Fig B-12. Comparison between Experiment and SEEP/W in Garnet Sand with 0.125-inch riser and 3 inch DP
Fig B-13. Comparison between Experiment and SEEP/W in Ottawa 20-30 with 1-inch riser and 1 inch DP

Fig B-14. Comparison between Experiment and SEEP/W in Ottawa 20-30 with 1-inch riser and 2 inch DP
Fig B-15. Comparison between Experiment and SEEP/W in Ottawa 20-30 with 1-inch riser and 3 inch DP

Fig B-16. Comparison between Experiment and SEEP/W in Ottawa 20-30 with 0.75-inch riser and 1 inch DP
Fig B-17. Comparison between Experiment and SEEP/W in Ottawa 20-30 with 0.75-inch riser and 2 inch DP

Fig B-18. Comparison between Experiment and SEEP/W in Ottawa 20-30 with 0.75-inch riser and 3 inch DP
Fig B-19. Comparison between Experiment and SEEP/W in Ottawa 20-30 with 0.25-inch riser and 1 inch DP.

Fig B-20. Comparison between Experiment and SEEP/W in Ottawa 20-30 with 0.25-inch riser and 2 inch DP.
Fig B-21. Comparison between Experiment and SEEP/W in Ottawa 20-30 with 0.25-inch riser and 3 inch DP

Fig B-22. Comparison between Experiment and SEEP/W in Ottawa 20-30 with 0.125-inch riser and 1 inch DP
Fig B-23. Comparison between Experiment and SEEP/W in Ottawa 20-30 with 0.125-inch riser and 2 inch DP

Fig B-24. Comparison between Experiment and SEEP/W in Ottawa 20-30 with 0.125-inch riser and 3 inch DP
Fig B-25. Comparison between Experiment and SEEP/W in Graded Zirconium with 1-inch riser and 1 inch DP

Fig B-26. Comparison between Experiment and SEEP/W in Graded Zirconium with 1-inch riser and 2 inch DP
Fig B-27. Comparison between Experiment and SEEP/W in Graded Zirconium with 1-inch riser and 3 inch DP

Fig B-28. Comparison between Experiment and SEEP/W in Graded Zirconium with 0.75-inch riser and 1 inch DP
Fig B-29. Comparison between Experiment and SEEP/W in Graded Zirconium with 0.75-inch riser and 2 inch DP

Fig B-30. Comparison between Experiment and SEEP/W in Graded Zirconium with 0.75-inch riser and 3 inch DP
Fig B-31. Comparison between Experiment and SEEP/W in Graded Zirconium with 0.25-inch riser and 1 inch DP

Fig B-32. Comparison between Experiment and SEEP/W in Graded Zirconium with 0.25-inch riser and 2 inch DP
Fig B-33. Comparison between Experiment and SEEP/W in Graded Zirconium with 0.25-inch riser and 3 inch DP

Fig B-34. Comparison between Experiment and SEEP/W in Graded Zirconium with 0.125-inch riser and 1 inch DP
Fig B-35. Comparison between Experiment and SEEP/W in Graded Zirconium with 0.125-inch riser and 2 inch DP

Fig B-36. Comparison between Experiment and SEEP/W in Graded Zirconium with 0.125-inch riser and 3 inch DP