Abstract

CARUSO, CARY WAYNE. In *Situ* Measurement of the Scour Potential of non-cohesive Sediments (ISEP). (Under the direction of Dr. Mohammed A. Gabr.)

A vertical probe (VP) employing a water jet has been developed for assessing the scour potential and scour rates of sediments. The probe termed “In situ Scour Evaluation Probe,” or ISEP, is based on the idea by Dr. Gabr that analysis of the probe penetration rate into the soil may be correlated with scour rate and erosion potential. This method measures the potential scour rate *in situ* and as a function of depth. Work in this thesis describes the applicability and the laboratory-based experimental verification of the ISEP and provides insights into the effects of the water jet parameters (i.e. water volume flow rate and jet nozzle velocity) on maximum probe penetration depth, the probe insertion rate, and the estimated erosion rate. In addition, ISEP is applied in the field at two sites for validation of the testing approach.

Results on the test sand with mean particle diameter (D_{50}) ~ 0.3 mm suggest that the maximum depth of advancement of the probe can be empirically correlated to the vertical velocity of the water at the tip of the probe raised to a positive exponent. The rate of probe advancement also seems to vary with moisture content. Thus far, scour rates determined with this method are comparable with published scour rates for similar sand type. results are presented from field trials showing the ISEP can differentiate between eroded and replaced sands of the Isabel breach and the non-eroded sands of the outer banks

Erosion rate and velocity are also related to the depth of the probe as well as tip water velocity for the sand in the test pit. A relationship between erosion rate and depth is presented for the sand in the test pit. The relationship between erosion rate and depth

suggests that effective stress and friction are major factors controlling erosion rate with depth.

Application of this technique to different sands in the Isabel Breach on the outer banks of NC shows that this technique has the ability to differentiate between reclaimed sands and uneroded sands. More specifically, erodibility values for the reclaimed sands in the breach area average between 4 and 5 times larger than the erodibility values for the uneroded sands outside of the breach. In addition, the shape of the erosion rate versus depth curves in the breached zone is different from the shape of the curve in the unbreached zone. Results are also presented for sands in two other field trials and are consistent with previous results.

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In *Situ* Measurement of the Scour Potential of non-cohesive Sediments (ISEP)

by
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DEDICATION

The work presented here is dedicated to my beautiful wife, Sue, and my beautiful daughter, Maggie, for putting up with my absences and my moods. Without their love, help and support, I never would have been able to finish. This thesis is truly a team effort and I will be forever grateful to them for this opportunity to make a change.

BIOGRAPHY

The author was born in Memphis, Tennessee and as a youth lived in Tennessee, New York, Arkansas, and Ohio. He attended Southern Methodist University in Dallas, Tx. where he earned a BS in Physics and an MS in Geophysics. After 5 years working in the Oil Industry for companies that no longer exist, he returned to school and earned a PhD in Geophysics from Cornell University in Ithaca, NY. He moved to Durham, NC to pursue Post-Doctoral research at Duke University and, while there, fell in love with and married the girl next door. He taught for 12 years at local universities and during this time he and his wife had a daughter. After much soul-searching, he decided to switch fields. He started in Civil Engineering at NCSU in 2007 and was invited to work with Dr. Mohammad Gabr during 2009. The rest of the biography is yet to be written.

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DISCLAIMER

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CHAPTER 1 INTRODUCTION

1.1 Background

In an unaltered setting, a river exists in a state of transient equilibrium as the processes of deposition and erosion are in balance. Increased flow events such as storm surges or alteration of the river's channel cause increased stresses on the bottom of the rivers channel resulting in increased removal of material from the bottom of the channel. This increased loss of sediment is a type of erosion called scour. When the river channel is altered, such as by placing bridge piers in the middle of the channel, the water flow is separated around the pier. One of the consequences of this flow separation is that there is an induced downward water flow at the front of the pier causing a horseshoe vortex. This increased water velocity can fluidize the material in front of the pier allowing increased material removal and creation of large scour holes. This phenomenon is referred to as local scour.

The process of placing bridge abutments along the sides of a natural stream also changes the flow of the stream. During times of increased flow, the presence of the bridge abutments effectively decreases the width of the stream creating an increased flow velocity, which increases erosion. This phenomenon is referred to as contraction scour. These two processes, local scour and contraction scour, are the primary cause of failure of bridge foundations and other hydraulic structures (Richardson and Davis, 2001). There are approximately 600,000 bridges in the US (Richardson and Davis, 2001) and as many as 25000 may be considered as scour critical (Briaud, 2002).

1.2 Problem Statement

The Federal Highway Administration (FHWA) estimated that 60% of all bridge failures in the US are caused by excess scour under the bridge piers or abutments, leading to collapse or serious structural damage. This translates to about 900 bridge failures between 1995 and 2005 caused by scour. In addition, about 26,000 bridges are classified as "scour critical" meaning that the foundations can fail due to scour, Briaud, (2002).

The prime motivation for this research is that current state-of-the-art measurement techniques of scour potential requires removal of sediment samples for laboratory testing thus limiting it's utility to cohesive materials. This limitation is especially important for non-cohesive materials since they are widely acknowledged to erode at a more rapid rate than cohesive materials suggesting that they can only be meaningfully measured *in-situ*.

1.3 Objectives

It has been proposed that a vertical probe (VP) employing a water jet can be used to estimate scour potential and erosion rates of sediments. The premise is that analysis of the probe penetration rate into the soil correlates with scour rate and hence, erosion potential. This method aims to measure the scour rate *in situ* and as a function of depth. The assumption is made that there is a 1-to-1 replacement of sand by probe as the probe advances into the sand.

This method for assessment of scour potential with depth using a vertical water jet was proposed by Gabr (2007) and presented in Caruso and Gabr (2010). It draws from the approach described by Hanson et al (2002). The method, termed In Situ Evaluation of Scour

Potential (ISEP), employs a cone-tipped vertical probe that is attached to a digitally controlled, centrifugal pump that provides controllable and repeatable vertical water velocity at the tip. The water jet is induced through a cone tip and as the vertical water jet is deflected by the soil, the now horizontally moving water applies a shear stress to the soil grains causing erosion.

Current techniques for measuring scour potential require either removal of soil samples for laboratory testing such as the Erosion Function Apparatus (EFA) proposed by Briaud et al (2001), or limiting measurements to scour on the surface of the sediment using inverted flumes, e.g. Aberle et al. (2003), or the use of surface jets as was presented by Hanson et al (2002) and Hanson and Cook (2004).

1.4 Scope

Assessment of scour potential at critical civil infrastructure is a persistent challenge for the profession. The motivation for this research is that about 60% of bridge failures in the US are caused by excess scour under the bridge piers (Briaud, 2008). Therefore, scour evaluation is critical for assessing the stability of structures prior to, and after, storm events as was learned from failures during and after Katrina. This thesis discusses the development of the ISEP and details some of the problems and their solution that occurred along the journey. Results are presented from the early stages of testing in non-cohesive sediments.

CHAPTER 2 LITERATURE REVIEW

2.1 Sediment Characteristics

Particle size is a primary factor controlling the ability of a soil to resist erosion. Soil particles that are retained on the No. 200 (0.075 mm) sieve are classified as coarse. For the purposes of this paper, coarse particles will be the main focus. The erosion of these particles is governed by the body forces acting on the particles. The primary body forces are the weight of a particle, which scales with the cube of the particle diameter, and the drag force, which scales with the square of the particle diameter.

The descriptions of the processes controlling the erosion of granular, non-cohesive sediments may be generally classified by deterministic or probabilistic approaches (Annandale (2006) and references therein). The deterministic processes are those that describe an individual particle of soil and then describe the forces and subsequent motion of the particle. The deterministic processes are the most appropriate descriptions for scour associated with laminar flow. The probabilistic processes would be more representative of erosion caused by the pressure fluctuations associated with turbulent flow applying forces on a particle (Annandale, 2006, Asadollahi, 2009).

2.1.1 Deterministic description on scour of non-cohesive Sediments

Erosion and scour of granular materials have been discussed extensively in the literature. Albert Shields (1936) described the initiation of motion of granular particles on the

basis of flume testing during his PhD research, as presented in Kennedy (1995). One of Shields' most important conclusions was that a critical shear stress (τ_c) existed below which particles will not be dislodged and moved. At low velocities, this critical shear stress value represents the viscous drag imparted by the moving fluid to the bed particles, and is related to a critical velocity of the water. At higher velocities, the critical shear stress appears related to the pressure differential between the upstream and the downstream sides of the particle (Leopold, 1994). The critical velocity necessary to create a given critical shear stress is a function of the depth of flow and the particle diameter. The forces necessary to determine the critical shear stress are described in the paragraphs below.

Following Julien (1995), Choudhry (2008) and Briaud (2001), a summary of the forces governing the erosion of non-cohesive silts and sands is given below when a spherical particle shape is assumed. The force of weight (F_w) and buoyancy (F_b) acting on a soil grain are as given as follows:

$$F_w = 1/6 \pi G_s \gamma_w d^3 \quad 2.1$$

$$F_b = 1/6 \pi \gamma_w d^3 \quad 2.2$$

where: G_s as the specific gravity of the soil grain, γ_w the unit weight of water, and d the particle diameter. The difference between these two forces is known as the submerged weight W_f .

$$W_s = F_w - F_b \quad 2.3$$

Another force exerted on a submerged soil grain at rest is the viscous drag force caused by the fluid flowing around it:

$$F_D = 1/2 \pi (D/2)^2 \rho C_D u_f^2 \quad 2.4$$

In Equation 2.4 above, C_D represents the drag coefficient, ρ the density of water, D is the exposed frontal area of the particle, and u_f is the flow velocity of the imparting fluid. For non-cohesive sediments, erosion occurs when the drag force is larger than the submerged weight ($F_D > W_s$). For an open channel, the hydrodynamic force per unit area is known as the applied shear stress, τ . This is expressed as

$$\tau = \gamma_w RS \quad 2.5$$

where R is the hydraulic radius (flow area divided by wetted perimeter) and S is the slope of the channel.

Briaud (2001) made the observation from slow-motion video of erosion experiments in flumes that the dominant means of particle motion (erosion) are sliding, rolling, and plucking. Furthermore, Briaud (2001) determined that the applied shear stress at which sediment motion is initiated (the critical shear stress, τ_c) is related to the average particle diameter, d_{50} . Briaud's relationship is given by:

$$\tau_c \text{ (N/m}^2\text{)} \sim d_{50} \text{ (mm)} \quad 2.6$$

and is similar to Shields' (1936) equation, which stated that the critical shear stress (τ_c) is related to particle diameter:

$$\tau_c \text{ (N/m}^2\text{)} = 0.63d_{50} \text{ (mm)} \quad 2.7$$

where : d_{50} = particle diameter (mm) corresponding to 50% finer fraction. Briaud et al (1999) suggested that Equation 2.6 is more generally appropriate for sands since Shields included different materials in his analysis.

Shields (1936) determined the incipient motion and the subsequent bed-load transport of non-cohesive sediments. The sediments he used were uniform-grained amber cuttings, brown coal, crushed granite and crushed barite. The densities of these sediments ranges from 1060 to 4250 kg/m³ allowing studies of a wide range of critical shear stresses while minimizing the need for more than the 2 flumes and 2 slopes he had available (Buffington, 1999). From these experiments, he was able to determine that the critical shear stress varies as a function of the boundary Reynolds number (Re^*). Re^* is also referred to as the grain shear Reynolds number. Shields plotted his results on a graph of dimensionless shear stress (τ^* , shown in equation 2.8) vs Re^* . The data plot in a wide band and define 3 transitions based on values of Re^* ; smooth ($Re^* < 2$), transitional ($2 < Re^* < 500$) and rough ($Re^* > 500$). In particular, he posited a value of 0.060 for τ_c^* for Reynolds numbers greater than 1000.

Following the development presented in Julien (1995), Shields' critical stress, when put into dimensionless form, represents the ratio of the hydrodynamic forces (or applied shear stress) to the submerged weight of the particle at the moment of incipient motion. This dimensionless critical shear stress is also referred to as Shields' parameter which is given by τ^* in the following expression:

$$\tau^* = \tau_0 / ((\gamma_s - \gamma_m)d_s) = \rho_m u_*^2 / ((\gamma_s - \gamma_m)d_s) \quad 2.8$$

where : τ_0 is the boundary shear stress, u is the shear velocity of the water, γ_s and γ_m are the specific weights of the sediment particle and the fluid respectively and d_s is a representative particle size.

Shields (1936) plotted his data against a second parameter, which is now known as the grain shear Reynolds number. This parameter represents the ratio of particle size to laminar layer thickness and is given by:

$$Re^* = u_* D / \nu = (\tau_c^* (\rho_s - \rho) g D / \rho)^{1/2} (D / \nu) \quad 2.9$$

where: u is the shear velocity of the water, ν = kinematic viscosity, D = diameter of the soil particle and usually assumed to be d_{50} , τ_c^* = critical shear stress, ρ_s = sediment density and ρ = fluid density. When motion is just starting, $\tau_0 = \tau_c$. The value at which this occurs depends on whether the flow is laminar or turbulent.

Shields (1936) developed these two parameters to describe incipient sediment motion in the widely used Shields diagram. However, his original form is difficult to use since the critical shear stress and particle diameter are both included in the definitions of the above parameters. Moreover, the critical shear velocity for the beginning of motion also appears in both relationships and is also difficult to ascertain.

To get around this difficulty in applying Shield's diagram to real data, different approaches have been attempted. One of the first attempts was to eliminate the shear velocity from the grain shear Reynolds number by defining a third quantity representing a dimensionless particle diameter, d^* ,

$$d^* = d_{50} ((G-1)g/\nu^2)^{1/3} \quad 2.10$$

where: G = specific gravity, g = gravitational acceleration, ν = kinematic viscosity. In this case, the Shield's Diagram is plotted as the critical value of the Shields parameter, τ_c^* , on the ordinate and d^* on the abscissa. For turbulent flows the critical shear stress may be directly determined from the particle diameter (Julien, 1995). This relationship assumes a uniform particle diameter for the sediment. When working with natural materials, the median particle size (d_{50}) is often used. The Shields Diagram is shown in Figure 2.1, below.

Another strategy to make the Shields diagram more useful is to fit the data with an empirically determined curve. For the upper and lower portions of the curve, $Re < 50$ and $Re > 1000$, the fit is straightforward since these are approximately linear, Cao (2006). Brownlie (1981), Guo (1997) Yalin and da Silva (2001), Cao (2007) and others have

employed curve matching. Most of these authors have tried to find an empirical fit to the data over the entire range of Re with varying degrees of success. In much of the literature examined to date, the Brownlee (1981) curve appears to be the most commonly cited. This is

Figure 2.1 The Shields Diagram showing tractive force coefficient versus grain Reynolds number. This figure was copied from Figure 6 in Shields (1936); translated by Ott and van Uchlen, date not given.

because of its simplicity to use. In Cao's paper, for example, a double logarithmic matching is employed for fitting only the middle "saddle-shaped" region, Cao (2007).

The Brownlee curve is given in Equation 2.11. The advantage of this technique is that the critical stress is determined from soil particle properties and water properties

$$\tau^*_c = 0.22 \text{ Rp}^{-0.6} + 0.006 \exp(-17.7 \text{ Rp}^{-0.6}) \quad 2.11$$

where Rp = particle Reynolds number = $\text{Re}/(\tau^*_c)^{1/2}$ and Re is defined in equation 2.8 and τ^*_c is defined in equation 2.11.

Most importantly, the basic conclusion of a relationship between critical shear stress and d_{50} for sands, has been mostly supported by other researchers, including White (1940) and others. On the other hand, for gravels, with diameters ranging between $d_{50} = 4.89 - 31.75$ mm, Dey and Raju (2002) suggested an experimentally-derived non-dimensional shear stress (τ) of:

$$\tau(\text{N/m}^2) = 0.013 F_d^2 d^{0.48} h^{0.49} \quad 2.12$$

where: F_d is the particle Froude number, d is the effective particle diameter (mm), and h is the water depth to the top of the virtual bed (cm).

Regardless of the particular form of the equation describing the relation between shear stress and particle size, particle size is one of the controlling factors in all of the common models describing a non-cohesive soil's ability to resist scour or erosion since it plays an important role in both a particle's weight and the effective surface area exposed to the moving fluid.

2.1.2 Stochastic description of scour of Non-cohesive Sediments

Kalinske (1943) was the first to recognize that the incipient motion of a particle was stochastic. Subsequently, Einstein and El-Sami (1949) were the first to publish suggestions that turbulent stresses were the cause of initial particle movements. Since these papers were written, numerous other papers have been written on the stochastic nature of sediment entrainment including: Einstein (1950), Gessler(1965,1967), Mehta and Christensen (1983), Cheng and Chiew (1998) and others.

For example, Gessler (1965) showed that the probability that a particle will move is equal to the probability that a particle will not move when the average shear stress is equal to the critical stress. This appears true regardless of whether the bed consists of all particles of equal size or particles of all sizes. However, the consequence of this implicit assumption is that the forces and the resulting moments they apply to the particles are represented by normal distributions, Papanicolaou (2002).

More recently, however, experimental results of Wang and Larsen (1995) and others suggest that a gamma distribution is a better representation of the stress components leading Papanicolaou (2002) to conclude that deterministic theories of sediment erosion and transport must always underestimate the actual values. While this approach appears promising, it has not been widely applied to real data so conclusions as to its ultimate utility may not be drawn at this time.

2.2 Modes of Erosion

The identified major forms of erosion include surface erosion, mass erosion, and fluidization as presented by Partheniades (1965) and Mehta (1991). During bed erosion, all three modes may be present in some proportion, though one typically dominates (Mehta, 1991). For coarse, non-cohesive materials, surface erosion occurs when the applied shear stress is at or above the sediment's critical shear stress. For surface erosion, the dominant means of particle motion observed on the videos are sliding, rolling, and plucking.

Fluidization, as defined by Mehta (1991), results when the eroding fluid penetrates into the pores of the sediment, thereby relieving the load of the skeletal forces and destroying the sediment structure. The sediment is subsequently entrained by the eroding fluid and mixed as downstream transport occurs.

For cohesive soils, much of the erosion occurs in a process known as mass erosion. In mass erosion, material is transported away from the bed as either individual grains or small flocs (cohesive groups) of grains. The rate of surface erosion also increases with the applied shear stress. At what Partheniades (1965) identified as the macroscopic shear strength of the bed, the sediment can fail along an entire plane below the surface, allowing all of the material above the failure plane to be transported downstream.

2.3 Erosion Measurements and Relationships

Properties of soils and sediments that allow investigators to estimate their erosion characteristics can be measured in several ways both in the laboratory and the field. In either setting, erosion flumes and impinging jets are both common apparatus.

2.3.1 Submerged Impinging Jets

It has long been recognized that scour downstream of many hydraulic structures such as culverts and spillways may be treated as analogues to jet scour. Consequently, this has been a topic of a great deal of research over many years. Doodiah et al (1953), Sarma (1965), Beltaos, and Rajaratnam (1974), Aderibigbe & Rajaratnam (1997) and many others have all made contributions to this field. Most of these studies were concerned with the problem of describing the maximum scour depth, as the jet is held stationary. For these cases, the height of the jet above the surface is greater than the length of the "potential core" as defined by Albertson et al (1948). The "potential core" is that part of the jet where water retains its original velocity. At distances greater than the potential core, the velocity decreases linearly with increasing distance from the jet orifice as was presented by Albertson et al (1948).

The earliest published use of vertical water jets to measure erosion found by the author is by Dunn (1959). The study was related to the laboratory investigation of the cohesive strength of stream sediments. Moore and Masch (1962) also developed a laboratory device for measuring scour using vertical water jets.

Stein et al (1993) derived an expression for the shear stress applied to a surface within the potential core of the jet as:

$$\tau_e(\text{N/m}^2) = C_f \rho U_0^2 \quad 2.13$$

where: τ_e = applied shear stress to bed in Newtons, C_f is the friction coefficient determined by Robinson (1992) and is equal to $(0.0474/2)R_0^{-1/5}$, $R_0 = 2y_0U_0/\nu$, U_0 = vertical velocity of

water at the tip (m/s), ρ = density (kg/m³), y_0 = jet thickness (m) and ν is the kinematic viscosity. This equation was also derived by Aderibigbe & Rajaratnam (1997) and is used in this paper as the basis for conversion between vertical water velocity and bed shear stress

Hanson et al (2002) and Hanson and Cook (2004) have described an apparatus using a vertical jet for surface measurement of scour potential. This device and the resulting stress distribution as presented by Hanson et al (2002) are shown in Figure 2.2. J_p is the length of the potential core of the jet, J_i is the initial distance of the jet above the surface and J^* is the distance of the orifice above the erosional surface. One of the motivating factors for their work was that in their formulation of the problem, they avoided the difficulty of having to identify a mode of erosion Hanson (1990). While the Hanson device has often been suggested to apply only to cohesive sediments, it was regularly applied to non-cohesive sediments during early testing of the apparatus, but the rapidity with which erosional equilibrium was achieved in data collection lead to the authors recommending use of the Shields diagram for analyses of non-cohesive sediments.

The basic idea of the Hanson and Cook apparatus is that the excess shear model is an adequate description of the erosion rate. Mehta (1991) developed a simple equation, called the excess shear model, to describe erosion. His model involved the mass rate of erosion per unit area, or erosion rate (E), and the difference between the applied bed shear stress (τ) and critical shear stress (τ_c) or excess shear stress.

Figure 2.2. Schematic of vertical jet apparatus and the resulting stress distribution, from Hanson and Cook (2004).

$$E = k_d (\tau - \tau_c)^\alpha \quad 2.14$$

where k_d is a constant and α is usually assumed to be 1. It will be assumed to be 1 for the research presented here.. The only ambiguity exists in the definition of the critical shear stress. In the past, definitions have included the shear stress at which erosion ceases or at which a predetermined small value of erosion occurs.

Additionally, a linear regression can be performed on erosion rate and bed shear stress data. The shear stress corresponding to zero erosion on the best-fit line is yet another possible definition of τ_c . With any of the methods listed above, k_d and τ_c are uniquely

determined by the sediment and the fluid conditions present. Equation 2.13 is by far the most commonly used model, but other researchers have proposed relationships that include sediment properties as well. Mehta (1991, 1994) reviewed data collected from numerous previous investigations and also summarized erosion relationships developed by several previous investigators. For the purposes of this work, these more advanced models add additional complexity that the data will not yet support.

As described above, the mass rate of removal (mass/area/time), E , is proportional to the difference between the applied bed stress and the critical shear stress of the sediments. The constant of proportionality is often called the detachment rate coefficient is k_d . When the applied shear stress, τ , is less than the critical stress, τ_c , erosion is assumed to not occur.

Hanson and Cook's (2001) analysis proceeds from the observation that as a scour experiment is performed, eventually, the dimensions of the resulting scour hole stop changing significantly. Blaisdell et al (1981) determined that for some soils, scour continued even after 14 months. In their paper, Blaisdell and his coauthors (1981) tested 3 functions for fitting long period scour experiments. Their conclusion was that the logarithmic, hyperbolic model provided the best fit; both since it provided a marginally better correlation coefficient, but more importantly it did not require that a time of end to scour be assumed or determined.

Hanson and Cook (2001) used estimates of the asymptotic state of depth of scour to derive an expression for τ_c . Their ability to accomplish this task was based on the realization by Stein et al (1993) that, for a fixed jet, as the depth of a scour hole increased, it would reach a depth where the horizontal shear stress became equal to τ_c . When the soil-water interface is within the potential core region of the jet, the depth increases in a linear fashion

since the velocity within the core is constant. As the erosion increases the distance between the jet orifice and the soil surface, the rate of change decreases. Rewriting 2.14 in the form of a non-linear, ordinary differential equation gives an expression for the time rate of change of scour depth when the depth is greater than the potential core length. This equation first appeared in Stein et al (1993) as their equation 13, but is given in equation 2.15 using Hanson and Cook's notation.

$$dJ/dt = k_d [\tau_0 J_p/J - \tau_c] \quad 2.15$$

where: J = scour depth at time, t = time, τ_0 = maximum applied bed shear stress (stress applied within core), k_d = detachment rate coefficient, J_p = erosional depth equal to length of potential core, and τ_c = critical shear stress.

The first step is to estimate the asymptotic depth of scour. In Hanson and Cook's formulation, they minimize a parameter x , which is a dimensionless distance equal to the square root of the squared difference between the dimensionless depth of the scour hole and the ultimate depth of scour. Once an estimate of the ultimate depth of scour is reached, the critical shear stress (τ_c) is obtained by solving equation 2.16.

$$\tau_c = \tau_0 (J_p/J_e)^2 \quad 2.16$$

where: τ_c = critical shear stress, J_p = erosional depth equal to length of potential core, J_e = ultimate depth of scour, and τ_0 = Maximum shear stress at nozzle.

Once the critical shear stress has been determined, the erodibility coefficient (k_d) may be determined by minimizing the dimensionless time as a function of the dimensionless scour terms and this is shown in equation 2.17.

$$t_m = T_r [0.5 \ln(1 + J^*/1 - J^*) - J^* - 0.5 \ln(1 + J_i^*/1 - J_i^*) + J_i^*] \quad 2.17$$

where: T^* = dimensionless time, t_m/T_r , t_m = measured time, T_r = a reference time, $J_e/(k_d \tau_c)$, J^* = dimensionless scour term, J/J_e , J_i^* = dimensionless scour term at J_i/J_e , J = the distance from the nozzle to the centerline depth of scour, and J_i = the initial distance from the nozzle to soil surface.

The Hanson and Cook approach is probably the most commonly used technique for determination of erodibility parameters and has been codified as ASTM standard D5852. However, this technique is not without its critics. Annandale (2006) points out that in the hands of a skilled operator, it gives reasonably consistent results. In a series of laboratory investigations, Mazurek (2010) points to an order of magnitude difference in values determined for k_d for clay samples and 41 Pa difference in determinations of the critical shear stress. The differences in k_d are attributed to not running the test for long enough times. Both Annandale and Mazurek point to potential problems in the assumption that there is "... a linear relationship between the erosion rate and the erosive capacity of water" (Annandale, (2006))

Following an idea presented in Patterson (1989), Tolhurst et al (1999) described, built and calibrated a "Cohesive Strength Meter" (CSM) using pulsating water jets for measuring the *in situ* critical shear stress of intertidal and estuarine sediments. The device consisted of two concentric cylinders with a brass nozzle above the center. The CSM was carefully placed

on the estuarine sediment to be tested and the area open to the sediment (6.6 cm^2) was filled with the local estuarine water. The submerged jet discharged in pulses applying up to 200 Pa of shear to the sediments. Optical sensors measured the light transmission through the water as the test progressed to determine when the sediment began to erode. The whole process was fully automated by an on-board computer. Even though this device was called a cohesive strength meter, the device was initially calibrated using sieved quartz sands and reproduced values obtainable from the Shields diagram. Most significantly, the measured critical erosion pressure measured by the CSM exhibited a strong exponential relationship to the median grain size (d_{50}) of the sediment.

Mazurek et al. (2001) and Ansari et al. (2003) both used submerged circular jets to study the erodibility of soils in a laboratory. Both apparatus involve filling a large cylindrical tank with water and lowering a nozzle to a specified height above the sediment bed surface. Mazurek et al. (2001) measured the erosion characteristics of clays with nozzle diameters ranging from 4 mm to 8 mm, and jet velocities between 4.97 m/s and 25.98 m/s. Maximum scour depth, scour depth at the jet centerline, and scour hole volume were measured at doubling time intervals until an equilibrium was reached. The values determined in this study were primarily related to characteristics of the jet, rather than characteristics of the soil.

Ansari et al. (2003) used artificial mixtures of clay ($d_{50} = 0.0053 \text{ mm}$) and sand ($d_{50} = 0.27 \text{ mm}$) with the sand portion ranging from 10% to 60% by mass. The nozzle diameter and height above the sediment varied between, 8.0 mm to 12.5 mm and 0.15 m to 0.3 m, respectively, with jet velocities ranging from 1.3 m/s to 5.75 m/s. The depth and side slopes

of the hole and the rate of scour were measured and empirically related to the clay content, dry density, and water content of the soils prior to testing.

2.3.2 Internal Jets

While scour produced by impinging jets has been extensively studied in the literature, there is little found in the literature concerning the behavior of a material in which the jet is embedded internally to the material. Most of what is found concerns the use of horizontal, perforated pipes for clearing shipping channels, (Hagyard et al (1969), Weisman et al, (1982), Weisman et al (1988), Lennon et al, (1990)). Niven and Khalili (1998) took a different approach and investigated internal jets or jets submerged within the material. While their analysis was concerned with in situ fluidization of sand beds, one of their conclusions was that the penetration depth for an embedded jet was related to scour beneath the jet, and may be described by a Shields-type of scour. Niven and Khalili discussed the mechanics and application of vertical, internal jets in a series of papers (Niven and Khalili, 1998, 2002a, 2002b).

While the goal of the experimental work by Niven and Khalili was primarily fluidization of soil, which is different from the goal of the research here, their results and conclusions may be used as a guide for the current research. Niven and Khalili (1998) presented results from a series of laboratory tests with internal jets in different sands. In particular to the research here, their conclusions included the following:

1. As an internal jet progresses into the soil, the shape of the fluidized zone remains constant for a given soil, and flow rate.
- 2 The fluidized zone transitions from an open, ellipsoidal form to a closed fluidized cavity at some critical depth.
3. The jet penetration depth (fluidization zone depth) is proportional to the jet velocity and may be explained by a Shields-type of relationship for the minimum particle Froude number but tends, in their work, to be limited to about 35 cm depth.
4. As the jet is lowered into the sand, the stability of the fluidized zone is governed by the ability of the flow to maintain a fluidized state.

In the field trials of their internal jet discussed by Niven and Khalili (2002a), depths of 15 m were obtained. Their jet was 75 mm ID and was operated at 12.5 l/s (198 gpm). The method by which these depths were reached was starting the water jet at the surface, letting it create and evacuate a fluidized zone, and then lowering the jet and repeating the sequence. An interesting note mentioned by Niven and Khalili was that the central zone of each trial remained fluidized for between 15 and 30 minutes after the termination of water flow.

2.3.2 Laboratory Flumes

Laboratory flumes are also commonly used to study the erosion characteristic of soils and sediments. McNeil et al. (1996) eroded rectangular sediment cores from rivers in a straight rectangular flume. The cores were 100 mm wide and 150 mm long and were placed at the end of the 1.35 m long flume. Applied shear stresses ranged from 0.2 Pa to 10 Pa and the vertical erosion rate (dz/dt) was measured visually with a meter ruler. The critical shear

stress was defined as the applied shear stress where $10^{-3} \text{ mm/s} < dz/dt < 10^{-2} \text{ mm/s}$. Dry and wet bulk densities, water content, grain size distributions, and total organic carbon of the samples were measured, but could not be related to the critical shear stress. Fluid characteristics, such as pH however, could be related to the critical shear stress in a manner similar to Ravisangar et al. (2001).

The Erosion Function Apparatus (EFA) was developed by Briaud et al. (1999) and later refined by Briaud et al. (2001). The EFA consists of a rectangular acrylic duct with a bottom port for extruding standard Shelby tube (76.2 mm diameter) samples into the flow for erosion. Bed shear stresses ranging from 0.1 Pa to 100 Pa are measured by two pressure ports immediately upstream and downstream of the sample as it is continuously extruded to maintain a steady height of 1 mm above the flume bed. Initial tests showed that samples extrusion heights of approximately 2 mm would produce significantly larger erosion rates than what would occur at heights between 0 mm and 1 mm; thus the 1 mm height was chosen. The EFA procedure also specifies that the sample be submerged one hour prior to testing and dictates an increasing sequence of flow velocities to maintain until 1 mm of sample erodes or one hour passes, whichever is first. The EFA has allowed the investigators to measure erosion rates as low as 1 mm/day and as high as several meters per hour.

Barry et al. (2006) measured the effect of adding small amount of clay to sands by eroding well mixed beds in a rectangular flume 4.3 m long, 0.15 m wide, and 0.19 m tall. The last 0.6 m of the flume contained the sediment. Adding 0% to 15% (by mass) clay to the sediment caused strong lubrication influences and significantly reduced the critical shear stress.

Ganaoui et al. (2007) tested two cores of surface river sediments and a core of coastal sediment from 160 m below the seabed. In a recirculating PVC flume 3.6 m long, the resuspension of the sediment was measured via turbidity, which was in turn related to the suspended sediment load measured by filtering samples of the water taken every 3 minutes. The samples were classified based on the magnitudes of the critical shear stresses. The first class of samples was identified as recently deposited “fluff” and were easily eroded. The second class of samples came from the deeper, more consolidated sediments. Class 2 sediments agreed well with past research, though the coarser samples deviate slightly from the Shields Diagram.

2.3.3 Benthic Flumes

Benthic flumes are upside down channels placed on the bottom of a river, lake, harbor, or any other body of water where the erosion of sediments is of concern. Water is pumped through the flume so that the erosion and sediment transport can be measured in numerous ways. The major advantage of benthic flumes is that the flow conditions (shear stress) of importance can be created on completely undisturbed sediments in situ. Ravens and Gschwend (1999) used a laser Doppler anemometer to measure the turbidity of the water pulled through a rectangular acrylic flume measuring 2.5 m long, 0.12 m wide, and 0.06 m tall. The flume was gently placed on the sediment bed where legs extended away from the flume and into the bed to stabilize the flume. Using a pump, water was pulled through the flume up to the boat where the turbidity was measured and later related to the erosion of the

sediment. The entrance of the flume contained a 2.5 cm tall bar to trip a turbulent boundary layer as the applied shear stress ramped up in 10-minute intervals up to 30 Pa. The measured critical shear stress values agreed with the Shields diagram and could best be related to the depth of the sediment.

Aberle et al. (2003, 2004, 2006) used the National Institute of Water and Atmospheric Research in situ flume (NIWA I) to measure the erosion characteristics of natural sediment beds. The straight, rectangular flume measured 1.2 m long, 0.2 m wide, and 0.1 m tall with the last 0.9 m of the flume's length leaving the sediment bed exposed to the flume. A propeller, driven by an electric motor, pulled water through the flume, eroding the bed. Turbidity was monitored at 1 Hz and averaged over a span of 30 seconds. Similar to Ravens and Gschwend (1999), data at each applied shear stress showed an initial spike in erosion that decayed exponentially. This behavior was attributed to depth limiting properties of the sediments. Before the start of the tests, ambient conditions were recorded and five sediment samples from around the flume were collected to measure the wet and dry bulk densities, water content, loss on ignition (LOI) and grain size distribution of the sediments. Debnath et al. (2007a,b) used a second flume (NIWA II) that included modifications proposed in Aberle et al. (2004). The flume measured 0.74 m long, 0.16 m wide, and 0.08 m tall with a 0.6 m test section at the end. It contained turbidity meters near the entrance and exit of the flume and well as a current meter. It also employed an air pump to replace the electrical pump, reducing the electrical noise of the system. The NIWA II flume provided results very much comparable to NIWA I.

Recently, Ravens (2007) compared laboratory (McNeil et al., 1996) and in situ erosion measurements (Ravens and Gschwend, 1999) from the Lower Fox River in Wisconsin. Both samples were collected at the same site and similar depths. Quadratic models in the form of $E = M\tau^2$ were fit to the data to facilitate the comparison. It was shown that the samples tested in the laboratory were 5 times more erodible than the sediments tested in situ. It was proposed that the reason behind this discrepancy is that the benthic flume has a 110 cm test section, which is significantly longer than the 15 cm test section of the laboratory flume. This exaggerates the effects of the transition from hard to soft beds present in both flumes. Additionally, some reviewers proposed that the 5 year time span between testing events allowed more resistant sediments to migrate to the site. However, this claim is not supported by the associated geotechnical data and no biological mats were present during the in situ test.

3.1 Idea

Since most of the current methods for measuring erodibility are laboratory-based methods thus requiring extra time for collection and analysis of data, the idea was to use a vertical water jet and allow the probe to penetrate into the soil under its own weight. In this way, a device could be constructed that was easily portable, required minimal infrastructure to run, and was quick to set up and take down.

3.1.1 Plumbing

During the earliest stages of the research, much effort was expended in optimizing the plumbing of the ISEP. Early flow rates were below 13 gallons per minute (gpm) regardless of the pump's rpm setting. Examination of the manufacturer's flow rate data compared to the data collected in the lab suggested that something was obstructing the flow of water in the system since the manufacturer's calibration curves suggested flow rates in excess of 30 gpm should be easily obtainable. Replacement of the supply hoses and fittings allowed flow rates near 37 GPM. More about the measurement of flow rates and the relationship between flow rates and water velocities is found in section 3.1.3.

3.1.2 Tips

At the beginning of testing, there were 3 available tips; A 1/8 inch orifice in a 60° conical tip, a flat-topped tip with 6-1/4 inch orifices and a 1/2 inch orifice in a 60° conical

tip. Since then, a 3/4 inch orifice in a 60° conical tip has been constructed by reworking the 1/8 inch tip and has been used for most of the testing. Taken together, these tips covered the velocity range from 2 to 10 m/s, which also corresponds to jet Reynolds numbers between 5000 and 155,000. All of these tips are shown in Figures 3.1 a-c except for the 1/8 inch tip.

Figure 3.1a Left - The 1/2 inch conical tip.

Figure 3.1b Right - The Flat Top Tip.

Figure 3.1c The 3/4 inch tip

There do not appear to be any surviving pictures of this tip.

The probe with the 1/8 inch tip (full cone or FC) was never found to move into the sand unless the sand was completely saturated and then the sand fluidized and the probe

plunged into the pit. In flow rate testing, it was found that the tip constricted the water flow to such an extreme that there was never a flow rate greater than 4.44 gpm at maximum pump rpm. As the pump was slowed, flow decreased as well, dropping to about 2 gpm for a pump rpm of 1300. Consequently, the tip was sacrificed for construction of the 3/4 inch tip and no further testing was performed with the 1/8 inch tip. Charts of flow rates versus velocities are shown and discussed in more detail in section 3.1.3.

The probe with the flat-topped tip gave some of the best penetration results during early testing. In particular, this tip gave better penetration to the probe when the soil was dry at the surface than did the 1/2 inch orifice conical cone. It definitely gave better penetration results than the 1/8 inch tip although most of this better penetration is due to the increased water flow allowed by this tip. Flow rates for this tip ranged from 5 to 10 gpm over the useful rpm range of the pump. With the introduction of larger probe supply hose feeding the tip, the connector attached to tip no longer allowed use of this tip and further research using this tip was halted. If funding and time allow, this might be a productive avenue for further research especially in light of the increased penetration of the shrouded probe.

The 1/2 inch conical tip was used for most of the early testing since it gave the most consistent penetration results in both wet and dry sand conditions at the surface. Water velocities at the tip ranged from 5.8 to 13.2 m/s corresponding to flow rates between 11.4 and 26.6 gpm. These were the highest water velocities obtained with the experimental setup. However, penetration was still not considered to be adequate so the 3/4 inch orifice tip was constructed. Since this required replacement of the supply hose to obtain higher flow rates, testing ceased with this tip. Results from the testing with the tips are shown in Figure 3.2.

3.1.3 Flow Rate and Water Velocity

Much of the early part of the testing was devoted to understanding and controlling the water velocity available at the tip of the probe. At first, the goal was to maximize the water velocity with the expectation that this would allow greater penetration of the probe. As outlined above, much of the increase in water velocity was achieved by optimizing the plumbing. The pump controller controls the rotational speed of the pump and is calibrated in RPM. One of the earliest research goals was determination of the relationship between pump rpm and the water velocity at the tip of the pump. The flow rate at the probe's tip was measured first and then converted into water velocity.

The flow rate was determined by marking the locations of known volumes on the container. Markings were accomplished by taking the nominal volume of a smaller container, and determining the weight of water corresponding to that volume. The container was placed on a scale and tared. Then, an amount of water was added to the container corresponding to the nominal volume was added. The container was marked at the upper water surface as a

Figure 3.3 Water velocity versus pump rpm curves are plotted for each of the original tips using the 3/4 inch supply hose. Also shown is the curve for the 3/4 inch orifice tip.

known volume. This known volume was then used to fill larger containers and the known volumes marked on the larger containers.

To determine an average flow rate at a given rpm of the pump, a stopwatch was used to determine the amount of time necessary to pass a known volume of water. To convert this

volume time measurement pair into a water velocity at the tip, the volume time data was first converted from gallons per minute into cubic feet per second. From this, the velocity was calculated as:

$$v = Q/A$$

where: v = velocity (ft/sec), Q is discharge in cubic feet per second and A is the area of the orifice.

Finally, the velocity was converted into meters per second. For the 0.75 inch diameter tip used in much of the testing in this study, water velocities calculated using the pump ranged

3.1.4 Valve tree

Once the water velocities were in an appropriate range, data collection started in earnest. Experiments were performed collecting penetration rate data from various velocity ranges. As the data set expanded and analysis began, the ISEP collected data was compared with collected by other methods; in particular, Briaud's EFA. Early comparison of data collected with these two methods showed an apparent relationship between the two data sets.

The obvious question from this observation became "Does the ISEP data represent an upper portion of a broad based sigmoidal curve?" or "Are there separate curves that happen to intersect?" The answer as to which of these questions is true could only be determined by decreasing the water flow. In particular, it was felt that the question could only be answered

if water velocities below 1.0 m/s could be obtained. This became the goal for this segment of the research.

The pump itself has a lower limit on the allowable water flow; it has to maintain a flow of at least 6 gpm through the pump to keep the impellers cooled. With the plumbing available, this corresponds to about 1400 rpm on the pump. This would imply that the lowest

Figure 3.4 Valve tree used to decrease and control water velocity

safe water velocity is 2 m/s at the pump discharge. To maintain the flow while lowering the velocity, the decision was made to use a valve tree system to reduce the flow on the discharge side of the pump while maintaining the flow through the pump.

Ball valves and hose segments for returning water to the supply tank were obtained. Several different ball valve configurations were tried until water velocities below 1 m/s were obtained. The valve configuration chosen was to first divide the flow in half using a 2-way valve and then divide the flow by 4 using a 4-way valve. In all cases, unused water was returned to the tank. The valves are shown in Figure 3.4. Calibration runs were performed relating pump rpm and valve configuration to water velocity at the nozzle. Results from the calibration show that velocities as low as 0.5 m/s may be obtained with the setup. Using the valve tree decreases the available upper velocity to about 6 m/s. The decrease in the upper velocity occurs because the valves do not have the same internal diameter as the rest of the discharge system of the pump. For the purpose of the research here, this is felt to be an acceptable compromise.

3.1.5 Shrouding

Once the water velocity and flow rate problems were solved, data collection began. It became obvious through experiments that the probe still refused to penetrate much below 1 m unless the pit was completely saturated. In the fully saturated case, the probe tended to free-fall into the sand of the pit as the sand became fluidized by the water jet. As research examined and addressed some of the problems discussed previously, gradually, a consistent set of behaviors was detected when the sand was unsaturated. At the end of the run, when the probe had penetrated to its maximum depth and stopped progressing, the water pump would be turned off and the reservoir valve closed. After a brief time, less than a second to over 1 minute in duration, the probe would drop into the sand and stop abruptly. Further

examination of this behavior suggested that an over-pressurized zone was built up within the sand and that when water flow ceased, the decreasing pressure allowed the probe to settle into the formerly pressured region.

The reason that these over-pressurized regions formed became apparent with further testing. After the probe had entered the sand, entrained sand in the water, removed from the front of the probe, was being flushed up and was resettling between the probe and the side of the hole forming a sand pack around the probe. In this fashion, the process of erosion was interrupted. In a geological sense, the process of erosion consists of 2 parts, erosion and transportation. Erosion, in the sense of material removal, was still occurring, but there was no longer a means of transport of the entrained sediments away from the eroded surface leading to re-deposition of the sediments when the water flow ceased.

Once this phenomenon was understood, several methods of dealing with it were proposed and examined. Among the ideas examined were: attaching extra tubes to the external surface of the probe to help drain the pressure, using a wet-dry vacuum cleaner to help exhaust the material through the probe itself and adding an external shroud. The problem with the first idea is workability. How would one attach two or more external tubes to the ISEP. At this time, the ISEP was being fed vertically through a tripod and there was not enough room on the head on the tripod to machine a new head with extra holes. The problem with the second idea was that there was not enough room to move a significant amount of sand between the water supply hose and the inside of the ISEP. The third idea of adding an external shroud appeared to be the most easily implemented, but appearances can be deceiving.

Early experimentation with a shroud was to test the concept. The first shrouds were constructed out of Shelby tubes and were mounted concentrically to the ISEP with machine screws in threaded holes in the Shelby tube providing a friction grip to the ISEP. Early testing of this concept was frustrating since the shrouded ISEP no longer fit into the tripod used for guidance. However, experimentation gradually showed that the idea was viable and would solve several of the problems discussed in this chapter. Consequently, work was started in the machine shop on a full set of shrouding for the ISEP. Material was procured during the first week in September and the shrouding was finished in November.

Since the shrouding was ASTM standard sized 3.0 inch OD with a wall thickness of 0.065 inches and the tripod was designed for the first ISEP with a 1.9 inch OD, a new method of supporting the new shrouded ISEP (SISEP) was needed. A new, larger tripod was constructed of pressure treated wood and angle steel. The goals of the design were that it could easily be disassembled, transported, and quickly reassembled. In addition, it had to support a probe that could be up to 20 feet tall and weighing in excess of 200 pounds. The steel uprights of the first tripod did not have a sufficiently high resistance to applied bending moments because the steel was 16 gauge and was quickly disassembled. The uprights were recycled into the base where the gauge was sufficient since it was mostly in tension and 12-gauge angle steel proved to be sufficient for uprights of the tripod.

The tripod has an upper and a middle shelf and are primarily used to guide the SISEP during setup and as it enters the soil. In the first incarnation of the tripod, these guides were made of 3/4 inch thickness Scandinavian birch plywood. Constant exposure the wetting and drying cycles is causing the plywood to gradually delaminate, but 3/4 inch polyethylene

sheets have been procured and installed as a replacement. There is also an unaddressed issue of protruding bolts from the side of the SISEP. In the plywood guides, a large, linear taper router bit was used to create grooves in the side of the guide holes on the 2 guide shelves through which the protruding bolts could pass. This works most of the time as long as the operator remembers to rotate the probe after setup but before the run. This has delayed the construction of new guides since alternate ways of accommodating the protruding bolts are being sought before construction of the replacement guides. The tripod is shown in Figure 3.6 with the wooden platforms.

3.2 Difficulties and issues

Most of the major difficulties during the development of the ISEP were conceptual. As the equipment was debugged, the major problems encountered were unanticipated behaviors of the probe as it penetrated the sand. In many respects, understanding, anticipating, and correcting the behaviors discussed below occupied the majority of the time spent on this research to date.

3.2.1 Fluidization

One of the early obstacles affecting the development of the sour probe was the realization that the sand was fine enough so that it would fluidize under as water flowed within the sand. This manifested itself by starting the pump and having the probe free-fall into the sand as the bottom plate was removed and the water jet entered the sand.

Figure 3.5 Tripod constructed at CFL for supporting the shrouded probe.

This is a consequence of the sand in the test pit having been chosen specifically so that it would fluidize during earlier research. (Hurt, 1998).

During the early phases of testing on the unshrouded probe, it was discovered that any water flow at the tip was sufficient to fluidize the sand when the sand in the pit was saturated. Fluidization behavior was observed using water velocities as low as 1.5 m/s. Further testing showed that the fluidization process often occurred whenever the jet interacted with the water table. During the early phase of testing on the unshrouded probe, it was discovered that the water table had to be at least 1.5 m deep before there was a good

chance that data could be collected. Even then, at higher flow rates, it was not uncommon for the probe to insert itself some depth into the soil and then fluidization occurred allowing free fall. Consequently, much of the early testing was performed with the water table below several meters and as will be discussed below, lack of control of the water table contributed to much of the scatter in the early data due to the effects of water in the unsaturated part of the soil.

This was only part of the problem with fluidization. As the probe was inserting itself into the ground, the penetration rate slowed to a zero as the probe stopped. When this happened, the probe would not progress any more. Tests at 1800 rpm, the highest rate that the water supply could sustain for an extended length of time, were continued for as long as 45 minutes which showed no further movement of the probe. Consequently, the cessation of motion was usually considered to be the end of a test and depending on the flow rate might occur from 0.5-15 minutes after the test began. When the pump was shut off and the reservoir valve closed, the pump would settle an additional distance in the sand. These fall depths were similar in magnitude to the depth of the actively eroded portion of the test. Further experimentation revealed a pattern indicating the origin of this behavior. If the pump was shut off and the water valve to the reservoir closed, the fall occurred. If the pump were shut off but the water valve left open, the fall did not occur.

The explanation of this phenomenon is that the water is not able to leave the vicinity of the tip rapidly enough to allow the excess pore pressure to dissipate. This observation was first made by and is discussed in Niven and Khalili (1999) based on their experiments in the use of internal jets to fluidize soil for removal of contaminants. In some cases, the probe

would sit there, not moving and then suddenly free fall. This behavior occurred when the water table in the pit was elevated to near the bottom of the fluidized zone beneath the tip.

3.2.2 Displacement of sand

To investigate the assumption of erosion volume equaling advancing probe volume, a series of tests were run in an attempt to capture the sand displaced by the probe. While none of these were successful, they provided insight into the probe's interaction with the sand and ultimately suggested a solution of a different problem even though it was not recognized as such at first.

The first set of tests designed to capture the displaced sand began with procurement of a roll of landscaping cloth. This cloth was tested for its ability to contain the sand and allow water to pass. In anticipation, several frames were constructed for use in this experiment. To use the frames, cloth squares were prepared to size, weighed, and a 1.9 inch diameter hole cut into the middle of the square for the probe to pass through.

Most of these tests were run in pit where the water table was below the surface and as the tests were run, the defining observation taken from these tests was that there were no up-washed sediments from the hole around the probe. During the initial drop of the probe, there was a minor amount of splashed sand, but once the erosion phase started, up-washed sand reached up to near the ground surface, packed itself around the probe, and changed appearance as the moisture of the sand changed as the freshly-packed sand drained. As the flow rate was changed, this behavior was observed to remain constant.

Next, experiments were performed to see if the sand could be intercepted before it reached the near surface. The thought occurred that a thin-walled sampling tube implanted into the soil before the probe was allowed to progress down the central region of the tube might allow the scoured sand to be captured by providing a conduit to the surface. This idea was tried multiple times and results were found to be unpredictable. Probably, the most common cause of failure was that the sampling tube was unable to be placed in the sand in a perfectly vertical orientation causing the probe, which was held vertical with a tripod, to rub on the sampling tube and to slow itself to a stop. Several times, the probe caught the edge of the fabric and either tore holes in the fabric or carried the fabric into the hole. A small amount of sand was captured in 1 test but, the probe ripped the fabric and most of the sand was washed away. However, in several tests, an appreciable amount of sand was up-washed to the surface even if it was not caught.

The next attempt to capture sand started with the idea that sand could possibly be caught by attaching the tube to the probe itself. A thin walled sampling tube was drilled at the top and bottom and the drill holes were threaded. This was held in place on the scour probe via the friction of the machine screws reaching from outside the sampling tube to the surface of the probe. While this contraption never captured the sand it was supposed to capture, it did penetrate deeper into the sand for a given pump rpm than that of the probe by itself. Since the increased diameter of the contraption precluded the use of the tripod, numerous experiments were performed guiding the probe into the sand by hand and by the use of devices attached to the tripod. Most of the time the probe just fell over, but as noted earlier, the probe with this

shroud penetrated deeper when it penetrated. Even though the original idea of capturing the up-washed sand was never successful, it did provide the idea for shrouding the entire probe.

CHAPTER 4. ASSESSMENT

4.1. Scour Rate.

Since different erosional mechanisms affect materials of different particle diameters, the decision was made to measure penetration rate using the excess shear model. This model is used because it does not assume a particular model for the erosional process at the particle level (Hanson and Simon, 2010). While this is perhaps more important in describing the erosion of cohesive sediments where multiple erosional mechanisms are apparent, it is also applicable to non-cohesive sediments as well.

Penetration rate can be represented in several different fashions using the excess shear model. Using this model, If erodibility can be assumed proportional to penetration rate, erodibility can be represented as volume eroded versus time or mass eroded versus time. The choice of which representation to use depends on the ultimate application of the results and since the two different values are related through the density of the material, they may be considered to be the same. In this paper, we are concerned with rate of penetration or velocity of penetration, which will be shown also to be proportional to these values.

4.2. Theory.

The basic idea of the excess shear model is that hydrodynamic-induced shear stress in excess of a critical shear stress separates individual soil particles from the greater soil mass and removes them. This analysis is similar to the geological model of erosion, which consists

of two separate processes; the erosion process by which soil particles are removed from a soil mass and the transport process which removes the removed particles from the soil. In the excess shear model, the critical shear stress represents the stress required to initiate motion of a soil particle. This relationship is described by the following equation; first introduced as equation 2.14:

$$E = k_d (\tau - \tau_c)^\alpha \quad 4.1$$

where: k_d is a constant and α is assumed here to be 1. While the meaning of a critical shear stress is open to some debate, c.f. Annandale (2006), the existence of a threshold below which erosion does not occur is not open to debate. This observation is based on numerous empirical observations by Shields (1936), Temple and Moore (1994), Annandale (1995), Kirsten et al, (1995), and others. While much of the data presented in these studies are from jointed rocks, there are also data presented from particles of soil. Annandale (2006) makes a good case for casting relationships in the form of critical stream power rather than critical shear stress, but the basic concept of a threshold value beyond which scour occurs is the same; the only difference is the units.

4.3. Assessment of Penetration Rate from ISEP measurements.

Penetration rate is estimated from the ISEP measurements of the time rate of insertion of the ISEP into the sand. In other words, the penetration rate (E) may be calculated as:

$$E = \Delta x / \Delta t$$

4.2

where: E is the penetration rate, Δx is the change in amount of the probe's embedment, and Δt = change in time. Note that in the case the penetration rate has units of velocity. Since the cross-sectional area of the probe remains constant after the tip is embedded, the volume of sand removed from beneath the probe is simply proportional to the velocity of the probe's insertion.

To relate penetration rate to erosion rate, the assumption must be made explicit that, in this analysis, there is a one-to-one replacement of sand with probe. The lack of success at capturing up-washed sand during the early stages of experimentation was attributed to the observation that there was no sand washed up around the probe. An additional observation made during these experiments was that as the probe entered into unsaturated soil, the water and sand eroded from the tip were observed to come up almost to ground level and for the sand to form a solid plug around the probe as the probe penetrated deeper into the soil. The solidification of the sand around the probe also corresponded to the slowing of the probe into the sand.

The failure to obtain unwashed sand with a simple frame and fabric technique led to further attempts to capture the unwashed sand using other methods. The second set of experiments involved inserting a short, metal ring into the sand, attaching the filter fabric to the ring and allowing the probe to penetrate the sand through the ring. It was thought that perhaps water was entering the sand in the unsaturated upper part of the sand allowing the

sand to deposit as the water was removed. This idea also didn't work and sand was not successfully captured.

Since the idea at the time was that water was prematurely escaping, the idea of emplacing a full-length sampling tube into the sand and allowing the probe to penetrate through the tube was tried. Again, the filter fabric was attached to the sampling tube after it was emplaced in the sand. The probe was allowed to penetrate the sand through the sampling tube. No sand was captured was due to the ease with which the probe and the sampling tube would jam together due to misalignment since it was almost impossible to align the sampling tube and the probe and they would be tightly jammed together after each run. Numerous attempts were made to maintain alignment including construction of several jigs, but it was concluded that maintenance of alignment did not appear feasible without a significant increase in probe support infrastructure. While no meaningful sand capture measurements were made using this technology, several observations of increased probe penetration were made leading to the current configuration of the shrouded probe. The first observation is that when the probe had penetrated a small amount, sand and water would boil up and out the top of the emplaced sampling tube with vigor. The second observation is that the probe would penetrate much deeper and more regularly than normal when it was penetrating through the sampling tube. Since the maximum depth of penetration ranged between about 50-70 cm, the observation of increased depth of penetration suggested a new line of inquiry. A new 3-foot sampling tube was obtained and was attached to the probe by means of threaded bolts passing through tapped holes and holding the tube to the probe via friction. Immediately, the probe's penetration almost doubled for a given water velocity.

4.4. Assessment of Surface Scour Rate - The Hanson and Cook Approach

The assessment of scour rate at the surface was done using the method outlined in Hanson and Cook (1997). Assuming the excess shear model described earlier in section 4.2, Hanson and Cook present a method based on curve-fitting of experimentally determined scour data and extracting the critical shear stress and the erodibility coefficient.

Hanson and Cook (1997) limit their analysis to the case of a single, vertical circular jet at a height of H above the soil surface. The jet is further assumed to be fully turbulent with a diameter of d and the water is assumed to have a velocity of U_0 at the tip of the jet. Immediately after leaving the tip, the jet is in the free jet region where the potential core exists. Within the potential core region, the velocity of the water has a constant value of U_0 along the centerline of the jet. The length of the potential core is 6.3 times the diameter of the orifice for a circular jet, although a value of 6 was found to be hardcoded in the STRMJET analysis software obtained from Dr. Hanson. Since there were no comments in the code, this value was changed back to 6.3 to be compatible with both Hanson and Cook (2004) and Rajaratnam (1976). Next, Hanson and Cook (1997), calculates the critical shear stress based on the equilibrium scour depth. Since the time to achieve equilibrium scour depth can be in excess of a year for clays, Blaidsell et al, 1981, the assumption of a hyperbolic relationship between time and depth of scour is used to estimate the depth of scour. Minimizing the difference between the time-depth data collected and a hyperbolic curve allows determination of the critical stress. Once the critical shear stress is determined, the erodibility coefficient is determined using the critical shear stress, the measured scour depth, the time, and a dimensionless time function. Again, the problem is solved in a least squares fashion by

fitting a representation of the data in a dimensionless, time dimensionless depth form to a series of trial scour functions. This process continues until an acceptable fit has been determined.

During application of the Hanson and Cook method, the tip shroud is removed allowing access to the probe tip and scour zone. The distance between the probe tip and the sand is measured and the pump is energized for a measured amount of time. The pump is stopped and the distance from the tip to the bottom of the scour hole is measured using a tape measure. From these pairs of time and depth pairs, the erodibility coefficient and the critical stress can be determined. A typical scour hole resulting from a trial is shown in Figure 4.1.

Figure 4.1 Scour hole from a Hanson and Cook style test to establish surface values of critical stress and erodibility. coefficient.

In a separate series of tests, the depth from the tip to the point of refusal was also measured, but thus far none of these experiments have provided a solution that converged. While values for the erodibility and critical stress may be determined for individual time-depth pairs, the results are still somewhat suspect due to non-uniqueness. However, the results do indicate a certain consistency that warrants further investigation.

Since most of these Hanson and Cook experiments were of near-surface shape measurements, the analysis was performed using the STRMJET EXCEL spread sheet supplied by Dr. Hanson from the USDA in Stillwater, OK. Since the spreadsheet was designed to use data collected with Hanson's JET apparatus, an EXCEL spreadsheet was developed to convert the data we collected into a form usable by the STRMJET spreadsheet. The locally developed spreadsheet was necessary since the JET apparatus contains a calibrated rod for measuring the depth of a scour hole and Hanson's spreadsheet expects decreasing values as the depth of the scour hole increases. In addition, the elevation of the jet orifice above the surface and the zero point of the rod must be included in the modifications made to the raw input data for compatibility. Examples of the output from a test in of the sand in the pit at CFL is shown in Figure 4.2. The value determined for the critical shear stress is small, but consistent with other methods of estimation. An estimate based on Briaud's 1999 relationship for the critical shear stress of this sand is 0.3 Pa. The program for the Blaisdell solution assumes cohesive soils, it is worth noting that the extrapolation used in the program assumes a longer time for equilibrium scour than is observed in non-cohesive soils. This assumption is what produces the data clustering to the left on the figure.

Figure 4.2 Results from Hanson's STRMJET spreadsheet for the sand in the CFL test pit.

4.5 Assessment of Scour Rate from ISEP measurements.

The estimation of an erosion rate based on the penetration rate is a process that yet requires significant development. One approach is empirical where the erosion rate for the soil is estimated from existing literature and then compared with measured penetration rate in the field. The estimated and measured data can be fit together for the development of an empirical relationship. A second approach is similar to that established by Hanson and Cook (2004) where the critical shear stress is estimated based on the progression of the eroded hole under the jetting stream. The rate of erosion with time is also estimated and normalized with the magnitude of applied shear stresses, as estimated from the velocity of the jet stream.

For preliminary analysis, and, a one-to-one volume replacement was assumed to estimate the rate of particle removal from the penetration process. This assumption was made in the absence of documentation or approach in the literature that can provide information on

such ratios and clearly this process needs a further study through modeling the penetration behavior of the probe.

One of the earliest observations made was that as the ISEP probe progressed into the soil, the rate at which it progressed decreased. It was assumed early in the research that an increase in skin friction with depth was causing the decrease in the penetration rate. In addition, the increase in the overburden stress, leading to the increase in the critical stress of the sand to increase implied that the erosion rate must decrease with depth. The observation that pressure build-up around the tip as well as the drive to achieve deeper penetration of the probe caused this aspect of the probe's behavior to be somewhat neglected. A typical example of the decrease in erosion rate may be seen in [Figure 4.3](#). In general, a decrease in the rate of penetration is observed whether the soil is saturated or unsaturated or whether the probe is shrouded or not.

Figure 4.3 Penetration rate decreasing with depth.

There was also a subset of the runs in unsaturated conditions that displayed a somewhat different behavior. The penetration rate of the probe would suddenly decrease over a small depth range and further probe movement would become slow, sometimes even stopping. As experience with the probe's behavior increased, along with increased understanding of the behavior of unsaturated soils, it seems that both interface friction and change with shear strength with depth should be considered in the data reduction process. In addition, the realization gradually occurred that closed zones of circulation were forming that caused the erosion beneath the tip to cease and the attempts to overcome this phenomenon decreased the relative importance of the effective stress during the middle part of the research. The development of the shrouded probe and the repeatability of the results suggested that the concept of effective stress and its effects needed to be revisited since the problem of circulating zones was fixed. Experiments were run trying to pin down the behavior of the probe in the unsaturated sand, but there were too many other effects to isolate this. However, as better control over the location of the water table was achieved, understanding of the probe's behavior progressed.

One key to understanding the behavior of the probe was the realization that there were numerous runs where the probe would enter the sand and progress in a normal fashion, showing the expected decrease in erosion rate as the probe went deeper. The normal decrease in the probe's penetration rate would become a more rapid decrease and the probe would slow to almost a stop. As water flow continued, the probe would progress very slowly and then suddenly increase its speed of penetration and would bury itself, as the sand would fluidize. Since this behavior was only observed to begin when the probe was between 20 and

40 cm above the water table, the obvious conclusion was that the probe's water jet and the water table were interacting.

4.7 Physical Measurement

Due to the speed with which depth changes occurred during experimental measurements of the erosion rate, it was necessary to videotape the experiments and analyze the videotapes. There were advantages and disadvantages to this decision. During the course of the research, several different video cameras and analysis techniques were used. At first, a Sony digital 8 camera belonging to the CFL was used exclusively. Images of the experiments were recorded onto tapes, the images transferred to the computer and time was recorded with a stopwatch while distance was recorded from marks made on the probe. However, importing and video conversion onto the computer could take up to 45 minutes for an experiment lasting only 5 minutes. It was discovered that the video camera could be controlled directly from the program 'IMOVIE' so that conversion to the computer format was not necessary. Subsequent to this discovery, all of the analysis used the camera as the tape player connected to a computer and timing was done with a handheld stopwatch.

Timing with the stopwatch was often difficult. Timing intervals of less than 1 second were avoided since errors expected in the time measurements were desired to be less than 20 % assuming a reaction time of 0.1 second. The vast majority of time measurements were made of probe movements of 1 cm. However, during the early part of a run, the velocity with which the probe buried itself was too fast for a meaningful time measurement of a 1 cm change to be made. In many of these cases, movements of 2 to 3 cm were measured. In some

cases, use of different technology allowed these intervals to be subdivided into 1 cm measurements.

Later on during the research program, additional cameras were tested. In particular, CFL acquired 2 Flip video cameras. These cameras worked very well for recording the experiments in this research since they recorded in a video format that was native to the computer and the files were transferred directly to the computer. The negatives to the FLIP technology was that there was only a 5 power zoom on the camera meaning that the camera was often in the way of the experiment or at risk of inundation during the experiment. In addition, care had to be taken since a number of different people were using the camera, there was often video of many experiments left on the memory of the camera and invariably, the batteries in the Flip cameras were dead or dying.

After slowing the movement to a measureable speed, the next biggest advantage of taping the experiment was that the experiment could be revisited if additional measurements were required. The regular playback speed was always used for making measurements, but the fast play/slow-play capabilities were also used extensively since it was not uncommon for a single measurement to be made multiple times. Lastly, the fast forward could be used during analysis to skip over those times when the tape was inadvertently left running while set-up or take-down was occurring.

There are also a number of disadvantages to the video techniques that were used. The first was that camera placement was critical for interpretable data to be obtained from an experiment. It was common for the probe to rotate once the bottom shield was removed often making it impossible to read the markings on the probe. Care also had to be taken with the

water hose since it was possible for the hose to move in front of the markings as the probe advanced into the sand. In addition, the last major disadvantage of the technique was that there was no way to slow the video down even further.

Toward the end of the research project, the penetration rate runs that were collected were input in the LOGGER PRO software from Vernier software. This software allowed the timing interval to decrease from 0.1 second (assumed for the reaction time) to 1/30 second time representing the time interval between the neighboring frames of the digital video signal. Moreover, as the rate analysis was underway, the data was tabulated as it was collected and output could be exported as an EXCEL file. The acquisition of a copy of this software, courtesy of Professor Martin Kamela of the Elon University Physics Department allowed a dramatic decrease in the amount of time necessary to analyze the results of the experiments.

The consequence of changing the cameras was the realization that a bias was inadvertently introduced into data collected in the analysis of the early data. Since the Sony camera was incapable of resolving time into increments as fine as the Flip camera and since the markings were blurred, high penetration rates are under-represented in the data presented here.

4.8 The role of side friction on the probe

Friction is one of the controlling forces on the kinetic behavior of the ISEP. One view of the motion of the probe is that the water jet excavates a hole into which the probe advances. In this view, the driving force of the probe is its self-weight. Once motion starts

and the probe starts to embed itself into the soil, the force of friction will increase as more of probe becomes embedded. As the probe continues to advance, the forces of friction and water pressure will gradually balance the probe's weight causing the probe's motion to cease. This section attempts to quantify the effects of skin friction on the behavior of the probe.

The model for studying the frictional force on the probe in the test pit is similar to that used in calculation of skin friction of a pile in sand. Since the sand is often saturated, an effective stress approach similar to the Beta Method will be used here for a first approximation of the frictional force per unit area. As the probe is buried in the test pit, the unit frictional resistance will increase approximately linearly with depth to a depth between 10 and 20 times the diameter of the probe. For depths greater than 10 and 20 times the diameter of the probe, the unit value of frictional force will remain constant per classical approaches (Kézdi, 1975). The diameter of the shrouded probe is about 7.8 cm suggesting a linear increase in the frictional force to a depths ranging between 78 cm and 156 cm. Since this is representative of the typical range of penetration of the probe, the assumption will be made for this study that the increase in the unit frictional force is linear with depth. There are several different cases that need to be investigated based on the location of the water table. The end-member cases are where the soil is dry and where the soil is saturated. These cases will be addressed in paragraphs below.

Determination of the unit frictional resistance requires is the definition of the state of stress in the sand through which the probe is advancing. For data collected with the ISEP, the assumption is made that the sand around the probe will be in the active condition; that is, the sand is being displaced toward the probe. The reason for this is that as the probe enters the

sand, the size of the hole excavated by the water jet is slightly larger than the size of the shroud. In their analysis of internal jets, Niven and Khalali (2002) presented an experimentally determined expression for the fluidized zone width versus water velocity. That expression is:

$$\text{diaRatio} = (0.006 + 2.514 [Fr_p(d_p/d_j)^{1.5}]^{0.820}) / (d_p/d_j) \quad 4.3$$

where: d_p = diameter of the surface scour hole, d_j = diameter of the jet at the orifice, Fr_p = densimetric particle Froude number, diaRatio = ratio of the diameter of the fluidized zone to the diameter of the surface scour hole.

A water velocity of 2 m/s produces a surface scour hole with a diameter near 0.3 m in the test pit. With a quartz sand having a $d_{50} \sim 0.3$ mm, the densimetric Froude number is about 28. The jet diameter is 0.19 m diameter giving a calculated width of the fluidized zone at the tip of the probe of 9.2 cm across. This calculated fluidization width is 1.4 cm larger than the diameter of the shroud, which is why the shroud diameter was chosen as it was. The author believed that keeping the probe size less than the width of the fluidized zone would reduce friction and that the probe would go deeper as a consequence. This was based, in part, on the observations of Niven and Khalili (2002), in their experimental work, that the shape of the fluidized zone is approximately a constant with depth, although their laboratory data did not penetrate as deep as the ISEP probe. Their observation was supported by several observations made during this research. In several runs made in moist sand both in the pit and at the beach, it was possible to observe a hollowed-out zone at the bottom of the hole left

by the probe when the probe was carefully removed after a penetration run. The height of the hollowed-out zone was 9 or 10 cm above the bottom of the hole, but the width was not measurable with the equipment available at the time, but was at least several centimeters larger than the probe on all sides. Above this wider part of the hole, the remainder of the hole was approximately the diameter of the probe to near the surface.

The origin of the sand pack around the probe in the hole appears to be caused by a combination of sources depending on whether or not the annular region of the probe gets clogged. If the annular region becomes plugged, the sand and water mixture at the tip comes up along the side of the probe and will deposit itself as the water drains when the pump is switched off. If the annular region is not plugged, the water and sand from the tip exit at the top of the probe and some of it runs down the side of the probe into the hole. In both cases, the sand is deposited when the water drains as the pump is shut off.

The significance of the preceding several paragraphs is that the sand, at least for some depth below the surface, is possibly collapsing toward the probe suggesting that the coefficient of lateral earth pressure is at the active case. If the sand is just deposited from the water, it will not have been subjected to excess vertical pressures suggesting that that an appropriate upper limit is the at-rest coefficient of lateral earth pressure. There are then four cases for analysis: unsaturated-active, unsaturated-at-rest, saturated-active, and saturated-at-rest.

The friction between the soil and the probe is described here using a sliding friction model as given in equation 14.19 in Coduto (2001):

$$f_s = \sigma'_x \tan \phi_f \quad 4.4$$

where: f_s = unit side friction, σ'_x = horizontal effective stress, $\tan \phi_f$ = coefficient of friction between soil and probe. This model can be rewritten as shown in equation 14.21 of Coduto (2001) as:

$$f_s = K_0 \sigma'_z (K/K_0) (\phi_f/\phi') \quad 4.5$$

where: f_s = unit side friction, σ'_z = vertical effective stress, K_0 = coefficient of lateral earth pressure before probe is inserted, K = coefficient of lateral earth pressure after probe is inserted, ϕ_f = soil-probe interface angle, and ϕ' = effective friction angle of the soil. Coduto (2001) tabulated results for the ratios ϕ_f/ϕ' and K/K_0 in tables 14.4 and 14.5. For friction angle values between soil and the smooth steel of the probe, ϕ_f/ϕ' , values between 0.5 and 0.7 are suggested in Coduto (2001). However, Al-Mhaidib (2005) in his analysis of steel-on-sand interface angles finds results between steel and sand ranging between 0.4 and 0.47 depending on the shearing rate. In this paper, a value of 0.5 will be used. Similarly for K/K_0 , values ranging between 0.5 and 0.7 are suggested in Coduto (2001) for jetted piles and are the values which are used here due to the similarities in jetting piles and in emplacing the probe.

In chapter 5, typical values for the sand in the test pit are given. A representative value of the friction angle is 31° and the average dry density of the sand is 1442 kg/m^3 . For the sand in the pit, the coefficients of lateral earth pressure at rest and in the active case are determined to be (Das, (1990)):

$$K_0 = (1 - \sin\phi) = (1 - \sin(31^\circ)) = 0.515 \quad 4.6$$

$$K_a = \tan^2(45^\circ - \phi/2) = (1 - \sin(31^\circ))/(1 + \sin(31^\circ)) = 0.295 \quad 4.7$$

where all of the symbols have been previously defined.

As a test of the technique, assume a 1 m length of embedded probe into the dry sand of the test pit. Further assume that the horizontal stress is related to the vertical stress by an at-rest relationship. Solve for the total frictional force on the probe. The total frictional force will be determined by multiplying the unit frictional force times the total area. Since the unit frictional force as defined in equation 4.5 is observed to be linear with respect to the vertical pressure which is, itself, linear with depth, a representative value the unit friction for the 1 m section of probe will be calculated at a depth of 0.5 m. This will be done for 2 cases; one case where both K/K_0 and $\phi/\phi' = 0.7$ and one case where they both equal 0.5. First calculate the unit friction from equation 4.5 using the K_0 value from equation 4.6.

$$f_s(0.7) = (0.515)(0.5)(0.7)(1442 \text{ kg/m}^3)(0.5\text{m})(9.8 \text{ m/s}^2) = 1274 \text{ Pa} \quad 4.8$$

$$f_s(0.5) = (0.515)(0.5)(0.5)(1442 \text{ kg/m}^3)(0.5\text{m})(9.8 \text{ m/s}^2) = 910 \text{ Pa} \quad 4.9$$

The surface area of the buried probe can be calculated by multiplying the circumference of the probe times the depth buried. Only the outer surface area is included in

this calculation since in operation, the annular region is full of moving water and sand. The diameter of the outer shroud is 3.067 inches which is 7.8 cm. This implies that the 1 m buried section of the probe has an area of 0.2447 m^2 . From this area, the static friction force can be calculated from 4.8 and 4.9 to be:

$$F(0.7) = 0.2447 * 1274 = 312 \text{ N} \quad 4.10$$

$$F(0.5) = 0.2447 * 910 = 223 \text{ N} \quad 4.11$$

This is significant since the weight of 2 sections of the probe, the tip and the hose is about 300 N suggesting that approximately 1 m is the depth at which the static friction of the sand on the probe and the probe's weight balance in dry sand. If the dry sand is in the active condition, the depth of penetration would be expected to be above 2 m.

A similar analysis can be performed if the sand is saturated to the surface. In this case the unit weight of the soil is replaced by the buoyant unit weight of the soil. The saturated unit weight of the sand at a relative density of 22% is calculated to be 18.75 kN/m^3 using $e = 0.84$ and $G_s = 2.68$ from Cifaldi (1997). The buoyant unit weight is calculated as the difference between the saturated unit weight of the soil, above, and the unit weight of water (9800 kN/m^3) and is found to be 8948 kN/m^3 . Following the same calculations for the unit friction in 4.x6 and 4.x7 gives:

$$f_s(0.7) = (0.515)(0.5)(0.7)(8948 \text{ kN/m}^3)(0.5\text{m}) = 806 \text{ Pa} \quad 4.12$$

$$f_s(0.5) = (0.515)(0.5)(0.5)(8948 \text{ kN/m}^3)(0.5\text{m}) = 576 \text{ Pa} \quad 4.13$$

In a similar fashion as the calculations outlined above, the total friction may be calculated by multiplying the unit frictions in 10.12 and 10.13 by the surface area of the probe. From this, the total friction acting on the probe with 1 m of the probe buried in saturated soil is found to be:

$$F(0.7) = 0.2447 * 806 = 197 \text{ N} \quad 4.14$$

$$F(0.5) = 0.2447 * 576 = 141 \text{ N} \quad 4.15$$

Comparison of the results in 4.14 and 4.15 with those from 4.10 and 4.11 show that the total friction acting on the probe in saturated soil is only 64% of that acting on the probe in dry soil. This agrees with observations made both in the lab and in the field; when the soil is saturated, the probe goes deeper. Again, if the saturated sand is in the active condition as well, the expected depth of penetration is in excess of 2 m.

It must be noted that these are static friction values. If a common rule of thumb is used that says that the dynamic coefficient of friction is about 0.5 of the static coefficient of friction, then penetrations of 1.5 m to 2 m should be an upper limit if friction is the limiting factor in dry sand. Since penetrations greater than 3 meters have been achieved in saturated

sand, more analysis is needed. In particular, the role of friction on the probe passing through a heavy fluid of sand and water needs to be investigated.

CHAPTER 5. EXPERIMENTAL PROGRAM

5.1 Test Pit

The testing for this research was carried out in the geotechnical test pit in the large Geotechnical Laboratory at NCSU's Constructed Facilities Laboratory. Much of the information about the internal structure of the pit itself comes from Hurt (1998) as well as from personal inspection.

5.1.1 Dimensions

The test pit is constructed of concrete and is circular in cross-section with a diameter of 10 feet and a depth of 20 feet. An adjacent, auxiliary shaft allows access to the plumbing controls and in the wall of the shaft, there are 6 evenly-spaced 10"x10" portals allowing access to or instrument placement within the sand of the pit when the pit is drained. Located within the bottommost 18 inches of the pit are the water inlet and the drain outlet. These lie within a filter bed consisting of 9 inches of #57 stone overlain by 9 inches of 'pea' gravel. Figure 5.1, modified from Hurt, 1998, shows a vertical cross section of the pit, the location of the water inlet and outlet and the access portals. The access shaft is to the left of the pit. The water inlet system is shown in Figure 5.2 in a plan view of the pit and shaft.

Figure 5.1 A vertical cross section of the test pit showing location of water inlet and outlet, filter bed and access panels. Figure modified and redrawn from Hurt (1999).

5.1.2 Plumbing

The filling and draining controls are external to the main pit and are located in the auxiliary shaft. The water inlet valve is a ball valve connected to a 3 inch supply line operated at city water pressure. In Figure 5.2, the water inlet control is located on the left wall of the access shaft about 5 feet below floor level and about 3 feet from the far side of the shaft. In the bottom of the test pit, within the filter material described earlier, the water main branches into 12 feeder pipes so that water fills the pit as evenly as is reasonably possible. Depending on the level of remnant water in the pit, the fill time averages about 15 minutes.

The drain is a 6-inch pipe located at the bottom of the filter bed and is controlled by a gate valve operated by a wheel attached to a 20 foot long shaft extending to the top of the auxiliary shaft. The wheel is located about 2 feet above the floor level along the left side of

Figure 5.2 Plan view of pit and access shaft showing location of water inlet and outlet.

Figure modified and redrawn from Hurt (1999).

the shaft. The water drains out the gate valve into the bottom of the auxiliary shaft and flows out from the access shaft through 2 drains into a holding area.

When the pit filling process is operated at full flow, the sand within the pit is fluidized as the water percolates upwards through the sand. The sand in the pit was chosen to for this behavior so that the sand in the pit would be fluidized each time the pit was refilled. Fluidizing the sand allowed removal of grout bulbs injected into the sand during earlier

research projects. For the current research, the ability of the sand to be easily fluidized provides the starting point for each experimental sequence. Density testing with the Troxler gauge reveals that the sand is returned to a density near the ASTM measured minimum density for this sand after each reset. This will be discussed in more detail in the section below.

5.2. Test Soil and Properties

As mentioned above, the sand in the pit was chosen for its ability to fluidize when the pit is filled with water from below, Hurt (1998). Approximately 70 tons of sand was placed in the pit during earlier research projects and this sand is what was used during the current research. The sand was purchased from Southern Products and Silica and originates from a quarry at Drowning Creek; near Hoffman, NC. The sand is referred to as "Hoffman Sand" by both Hurt (1998) and Cifaldi (1997), although no reference to this as the proper geological name has been found to date. The sand was stated on the Southern Products website to have been mined from an "...underwater, alluvial deposit" and is stated to be "...free of clay, silt, iron and mica." Analysis of the sand when it was freshly placed in the pit verifies this, revealing a content of mica and magnetite of less than 1% (Cifaldi,(1997)). Recent microscopic inspection of the sand shows that it is clean, quartzite sand, although it has been contaminated during the decade it has been in the pit. The individual sand particles are sub-rounded to sub-angular and examples are shown in Figure 5.3. Properties of this sand were determined by Cifaldi (1997) and for a relative density of 22 %, the internal friction angle was determined to be 31°.

Figure 5.3 Photomicrograph of sand particles from the test pit. In the lower right corner are a pink potassium feldspar crystal and 2 black magnetite crystals.

Most of the observed contamination appears to have been from clay-type soil, which were stored on the cover over the test pit. Early in the testing, repeated refilling of the pit brought these fines to the surface where they were observed forming a crust on the sand. In the upper layer of sand in the pit, grain size analysis showed as much as about 3% fines while samples from deeper layers showed a grain size distribution with a fines content of less than 1%. To return the sand to a condition more similar to the original condition, the decision was made to back-wash the sand.

The backwash process entailed controlling the flow of water from the water inlet after the pit had been flooded. After the pit was flooded, the water flow was decreased until there was no entrained sand observed in the water filling the pit. Water flow was continued at this rate and the pit was allowed to overflow into the scupper drains around the pit. After this process was completed, the major part of the fines forming the crust had been removed.

Subsequent grain size distributions showed fines content in the upper portion of the sand were reduced to less than 1 %.

5.2.2 Grain Size Distribution

Following ASTM standards, grain size distributions were run on samples of sand from the pit. Results of this analysis are shown in Figure 5.4. The soil has a D_{50} slightly above 0.3 mm and about 97 % of the sand particles have diameters between 0.1 mm and 0.6 mm. More specifically, the sand has the following characteristics determined from the grain size distribution shown in Figure 5.4. The values chosen from this distribution are: $D_{10} = 0.17$ mm, $D_{30} = 0.25$ mm, and $D_{60} = 0.34$. From these parameters, the values for the coefficient of uniformity (C_u) and the coefficient of curvature (C_c) may be determined following the method outlined in Holtz and Kovacs (1981). C_u is found to be 1.7 and C_c is found to be 1.08; implying that the sand is classified as a uniform graded sand.

5.2.3 Minimum Density

During testing, densities of the sand were measured using several different techniques. The first method of measurement for minimum and maximum density were performed using the methods outlined in ASTM D4254 and D4253 respectively. The measurements for the minimum density using this technique averaged 1424 kg/m^3 and for maximum density averaged 1810 kg/m^3 .

Figure 5.4 Grain size distribution of sand samples from the test pit. Sand in test pit is a clean, sub-angular silica sand from a quarry near Sanford, NC. D_{50} is approximately 0.3 mm.

These measurements were used as the standards throughout the course of the research. The technique used for *in situ* measurement of the densities throughout testing was a Troxler 3430 neutron density gauge. Application of this technique, after the sand in the pit was reset by fluidization, gave densities ranging from 1373 kg/m^3 to 1475 kg/m^3 . The average of these measurements after resetting the sand was 1431 kg/m^3 .

5.2.4 Moisture Content

The technique used for *in situ* measurement of the moisture content throughout testing was a Troxler 3430 neutron density gauge. The results from the Troxler measurements were checked by standard moisture content testing using scales and an oven.

The results from the Troxler measurements showed that during the testing, the moisture content varied between 2.5% and 15%. When the moisture content was between 10% and 15%, the water table tended to be near the surface. Under water flow from the probe, the sand at this moisture content tended to fluidize which allowed the probe to bury itself. Consequently, for most of the data collected in this study, the moisture content of the sand was found to vary between 2.5 and 7% reflecting the water table ranging between 1.3 to 2 m below the surface.

5.2.5 Capillary rise

Since the interaction of the water jet and the water table fluidized the sand below the jet, the location of the water table became important in designing the experimental sequences. While the water table is well defined, water moves up into the unsaturated parts of the soil via capillarity. The size of the capillary rise can be obtained by the use of Hazen's relationship relating D_{10} and the height of capillary rise, h_{cr} , which is given in Terzaghi et al (1996):

$$h_{cr} \text{ (m)} = 0.15/D_{10}(\text{mm}) \quad 5.1$$

Since the D_{10} for this sand is found to be 0.17 mm (Figure 5.4), the capillary rise estimated with this relationship is 0.88 m. This estimation was based on assuming the capillary tube size equal to 0.034 mm (20% of D_{10} .) However, measurements of the capillary rise in the pit yielded value of 0.14 m.

5.3 Testing Program

During the course of testing, approximately 180 penetration rate tests were run which are outlined in Appendix 1. During the early stages of testing, many of these exploratory mainly trying to test the operation of the equipment. In all, 378 points were collected from tests on the conventional probe. An additional 150 flow rate tests were performed during calibration of the scour probe and its component systems.

5.4 Repeatability of experimental data

During the early course of testing, the effects of the unsaturated soil near the surface were not understood. This lack of understanding and the subsequent lack of control of the water table led to much of the scatter in the data collected during early experiments. A piezometer was installed in the pit early in the testing, but poor design lead to repeated clogging and consequently results were not trusted. This culminated in removal of the piezometer one day when the piezometer indicated that the water table was below the bottom of the piezometer. In reality, the water table was just below the surface as discovered when the author jumped into the pit and sank to knee level.

As the testing progressed, and the effects of the unsaturated portion of the sand became obvious, a new piezometer was designed and installed. Since effective control of the water table was now possible, the data became more repeatable. Results the repeatability testing are shown in Figure 5.5. These two tests were run to test the repeatability of the method when the water table was maintained at a depth of 1.35 meters. The tests were run on two different days and the pit was drained and refilled between the runs.

Figure 5.5 Comparison of data collected in runs at 1800 rpm.

CHAPTER 6. RESULTS

In this chapter, results of the laboratory portion of the ISEP research are presented. First, results relating penetration depth to water velocity are presented. Next, penetration rate is examined as a function flow rate, water velocity, shear stress and stream power. The penetration rates determined from ISEP measurements are compared to erosion rate results from similar sands collected with the EFA.

6.1. Penetration Depth vs. Water velocity

When the data collection phase of the experiment began, one of the first relationships observed was a relationship between depth of penetration and water velocity. Data showing this relationship is shown in Figure 6.1, which shows the penetration depth versus the water velocity. What is somewhat surprising about this result is that the depth of penetration was predicted to be related to water velocity raised to the 0.995 power by Niven and Khalili in their series of papers from 1999 to 2001. As observed on Figure 6.1, a coefficient of 1.32 was determined here. There are several possible reasons explaining the differences determined for the coefficients in the two studies. First, Niven and Khalili, based their results on data collected in a study where tests were run using 8 different sands. Each of the sands in their study had different D_{50} values ranging from larger to smaller than our test sand. In addition, all of their results were determined from analysis of fully saturated sands whereas results from the study presented here were collected from both fully saturated and unsaturated sands.

Figure 6.1 Penetration depth versus vertical water velocity. Data collected with ISEP.

6.2. Penetration rate dependency on Flow rate/ Water Velocity

In this section, results from ISEP penetration rate are presented as a function of water velocity at the tip and as a function of the flow rate through the probe. All of the results presented in this section were collected with the unshrouded probe and the results presented here also include data collected when the pit was both saturated and unsaturated. Combining the data in this way is a consequence of the original piezometer used to measure the water level in the pit. It tended to clog regularly and consequently the results were untrustworthy and often not recorded. However, some results were recorded so that they can be separately

displayed. For comparison purposes, results collected using the ISEP will be compared to results collected using the Erosion Function Apparatus on a 0.3 mm sand later in the chapter.

A plot showing erosion rate versus water velocity for all of the data collected with the unshrouded ISEP is shown in Figure 6.2. When first plotted in this fashion, examination of these results caused some confusion and the apparent scatter of 2 orders of magnitude called into question the whole experimental procedure. In particular, the question of what was being measured was called into question. As analysis continued, several different effects were identified explaining the apparent scatter.

Figure 6.2 Plot of raw penetration rate data collected with ISEP.

First, the data were re-plotted so that the erosion rate for a specific depth range was plotted as a function of water velocity. An example of this type of plot for the depth range of 20-25 cm may be observed in [Figure 6.3](#). The data in this figure are a subset of the data shown in Figure 6.2. Most significantly, the scatter has been greatly decreased. For comparison, erosion rate data from the depth interval of 25-35 cm is shown in [Figure 6.4](#). In this figure, the important observation is that the erosion rate versus water velocity points in this figure have a similar slope but are shifted to a lower erosion rate at a given water velocity when compared to points in Figure 6.2. If similar plots are prepared for other depth

Figure 6.3 Erosion rate versus water velocity for ISEP in upper 25 cm of sand.

These data were collected with the 0.5 tip.

intervals, plotted data show a similar slope, but shifted to a lower penetration rate for each deeper interval.

If the penetration rate versus depth data are sorted by depth intervals and the average value within each interval is plotted versus depth, a smooth decrease in penetration rate versus depth may be observed. This is shown in Figure 6.5 where the original data is grayed out to make the average value in each depth interval easier to observe.

Figure 6.4 Penetration rate versus water velocity for a depth range of 25-35 cm. These data were collected with the 0.5 tip.

Figure 6.5 Penetration rate averaged over 10 cm depth intervals plotted vs depth for sand in the test pit. Data is for all tips and all water tables.

As discussed in earlier sections, the shear stress applied by the water jet to the surface beneath the nozzle can be calculated using an expression determined by Stein et al 1993. Stein et al (1993) derived an expression for the shear stress applied to a surface within the potential core of the jet:

$$\tau_e = C_f \rho U_0^2 \quad 6.1$$

where: τ_e = applied shear stress to bed, C_f = friction coefficient = $(0.0474/2)R_0^{-1/5}$, R_0 = jet Reynold's number = $2y_0U_0/\nu$, U_0 = velocity of water at the tip, ρ = density, y_0 = jet thickness ν = kinematic viscosity. The penetration rate data collected in this study can be plotted against the shear stress calculated in this fashion and these results are shown in Figure 6.6. If this figure is compared to Figure 6.2, the first impression is that the penetration rates shown in each case are similar. A closer inspection does show differences. Transforming to shear stress spreads the data out. In terms of the water velocity data shown, the data are spread out

Figure 6.6 Penetration rate plotted as a function of shear stress applied to the surface by the water jet.

over a range of 4/5 of a decade while the same data transformed to shear stress is spread out to cover a full decade.

6.2.2 Comparison with EFA results

For comparison purposes, results collected with Briaud's EFA are plotted on Figure 6.7 along with the ISEP data. While it might be tempting to suggest the ISEP data presented here are directly comparable with the data collected with the EFA, careful examination of the data shown above in Figures 6.3-6.5, as well as others not shown here, suggests that this comparison is not valid. It was first thought that the ISEP data formed part of a continuous curve with the EFA data and extended Briaud's relationships to higher water velocities or higher shear stresses. As understanding has evolved, the results from the EFA were realized to represent an upper limit or upper bound on the erodibility data when compared to the ISEP-collected data. Since erodibility decreases as effective stress increases, erodibility data collected with the EFA corresponds to erodibility data collected with the ISEP at zero effective stress. In other words, EFA data corresponds to very near surface ISEP data. Another significant difference between the techniques is the role of turbulence, which is discussed in the next section.

Figure 6.7 Comparing erosion rate results collected with the EFA against penetration rate results collected with the ISEP.

6.2.3 Turbulent flow effects

One of the major differences between Briaud's EFA and the ISEP is that for all velocities and flow rates possible with the ISEP apparatus, the water flow is fully turbulent. This is in contrast to the EFA, where turbulence is minimized by the ad hoc requirement that the sample be maintained at a constant height of 1 mm. In this fashion, the water flowing adjacent to the specimen is assumed to be in the boundary region and to thus be laminar or transitional. Robinson (1992) determined the Reynold's number, R_0 , for a jet to be $R_0 = 2y_0U_0/\nu$; where: y_0 = jet diameter, U_0 = centerline velocity, and ν = kinematic viscosity.

Results from calculations of Reynold's number for the velocity range used in this study are shown in Figure 6.8 and for all the velocities available with the apparatus, the flow is fully turbulent. The jet has Reynold's numbers greater than 10,000 for all velocities greater than 0.4 m/s signifying fully turbulent flow. For the 0.75 mm tip, the Reynold's numbers are even higher. These results are significant since fully turbulent flow allows an additional mechanism

Figure 6.8 Reynold's number as a function of water velocity for a 0.5 " jet.

lifting it from the bed as was described by Annandale (2006). One source of this lift is the pressure difference caused by the steady flow of water over and under the particles of a sediment bed. This source of lift is applicable regardless of the flow condition. A second source of lift is caused by pressure fluctuations associated with vortices developed by the turbulence. Annandale (1995) showed that 'Stream Power' is a useful estimate of the magnitude of the pressure fluctuations associated with turbulent flow. Applied stream power, P , is given below:

$$P_e = \tau U_0 = \tau (\tau / C'_f \rho)^{0.5} \quad 6.2$$

where: P_e = applied stream power, τ = applied shear stress, ρ = water density, C'_f = friction coefficient defined in Stein et al (1993).

If the excess shear model defined in equation 2.14 is rewritten in terms of stream power (P), as done in Annandale (2006), the result is:

$$\varepsilon_r = k_d (C'_f \rho)^{0.5} ((\tau_e - \tau_c) / (\tau_e^{1.5} - \tau_c^{1.5})) (P_e - P_c) \quad 6.3$$

where: P_e = applied stream power, P_c = critical stream power, τ_e = applied shear stress, τ_c = critical shear stress, ρ = water density, C'_f = friction coefficient defined in Stein et al (1993), k_d = erosion rate coefficient from excess shear model. The consequence of this is the erosion rate predicted by the stream power model is non-linear in shear stress.

The penetration rate versus water velocity data collected in this study was converted into penetration rate versus stream power and the results are plotted in Figure 6.9. The value used for the frictional coefficient is the value from Stein et al (1993) and presented in Hanson and Cook (2003). The significance of this plot is that the measured penetration rates could never be adequately explained using the excess shear model and while this is not a formal regression, it is suggestive that this might be a more appropriate model for examination in future research.

6.2.4 Penetration Rate dependency on Tip Diameter and Water Table

The behavior of the probe is related to the tip diameter and to the location of the water table. The probe's behavior is related to the tip diameter as a consequence of the fact that smaller tip diameters give a higher jet velocity for a given flow rate. A comparison of penetration rate and water velocity is shown in Figure 6.10 for the 1/2 inch and the 3/4 inch tips used in this study. In this figure, the solid symbols represent the 0.5 inch diameter tip. For the data collected here, the data tend to cluster higher on average than the data collected with the 0.75 inch diameter tip at similar jet velocities. On a practical note, the 1/2 inch tip requires a different hose so the water volume from the tip during a given time is less than that of the 3/4 inch tip and hose in a similar time.

Evaluating the effects of the water table on the penetration rate is straightforward, but is complicated in this study by the data collection method. As mentioned earlier, in Chapter 4, the camera was changed and most of this data was collected with the old camera. The old camera was incapable of capturing the behavior of the probe in saturated sand. The resolution in time was too low and consequently, images were smeared when movement was rapid.

Some data was collected where the water table was known and is shown in Figures 6.11 and 6.12. In Figure 6.11, the penetration rate is plotted against water velocity when the water table is less than 100 cm. In Figure 6.12, the penetration rate is plotted against water velocity when the water table is greater than 100 cm. It was unexpected when the figures were being prepared, but the data collected when the water table was less than 100 cm was all collected with the 0.5 inch tip while all of the data collected when the water table was greater than 100 cm was collected with the 0.75 inch tip. As observed both in the field and in

the lab, when the water table was near the surface, the sand tended to fluidize and the penetration rate is greater than when the water table is deeper.

In Figure 6.13, the data from Figure 6.11 has been fit with a linear regression. In addition, 95% confidence intervals have been determined for this regression. For water velocities between 2 and 7 m/s, the confidence interval for the estimated penetration rate ranges from 2500 to 5200 mm/hr. These values range between 10 to 50% of the values penetration rate values predicted by the regression. This water velocity range corresponds to the range used in the collection of the data. Outside of this range of water velocities, the confidence interval ranges from 7500 to 10500 mm/hr, which is much larger since the estimates and confidence intervals are not constrained by data.

Figure 6.9 Penetration rate predicted as a function of stream power compared to ISEP collected data. The stream power penetration prediction is thought to better represent the erosion caused by the turbulence present in the jet. For several stream powers, multiple values of penetration rate are calculated based on differences in water velocities between the two tip sizes.

In Figure 6.14, the data from Figure 6.12 has been fit with a linear regression. In addition, 95% confidence intervals have been determined for this regression. For water velocities between 3 and 5 m/s, the confidence interval for the estimated penetration rate ranges from 700 to 1600 mm/hr. which is 18 to 38% of the values predicted by the regression. The 95% confidence intervals are not as well constrained on this regression since the data were collected over a more limited range of water velocities and the confidence intervals may

be seen to diverge away from the predicted values outside of the range of the data with the known water velocities.

In Figure 6.15, the penetration rate measured for known tip diameter and water table less than 100 cm is compared with the calculated penetration for each water velocity. Two predictions are shown; one using Annandale's predicted value for the frictional coefficient, C_f , and one using the value presented from Hanson and Cook from Stein, et al (1993). In this figure the predicted penetration rate is less than the data presented but the regression lines and the prediction lines are parallel. Although comparison with entire data set, as shown in Figure 6.9, which used the Hanson and Cook value for C_f , suggests that this approach provides a good prediction, better estimates for K_d defined in equations 2.14 and 6.3, are still required.

In Figure 6.16, the penetration rate measured for known tip diameter and water table greater than 100 cm is compared with the calculated penetration rate for each water velocity. Again the two predictions are shown; one using Annandale's predicted value for the frictional coefficient, C_f , and one using the value presented from Hanson and Cook from Stein, et al (1993). In this figure the predicted penetration rate is less than the data presented and the data regression are not parallel to the predictions. This is a consequence of the limitations in velocity range of this data set.

As a final analysis of the data collected in the lab, the erodibility coefficient calculated from this data is compared to stream power. At first, data from Briaud and Kwak

Figure 6.10 Comparison of Penetration rate and Tip Diameter. All of these data were collected with the 0.5 tip.

were also included in this analysis, but attempts at regression resulted in weak coefficients of determination between K_d and stream power. This comparison is shown in Figure 6.17. After Briaud's and Kwak's data were removed, regressions provided coefficients of determination of about 0.02, very weak at best. In conclusion, while stream power appears to provide a good first approximation to penetration rate, it does not provide an explanation for variations in the coefficient of erodibility, K_d .

Figure 6.11 Penetration rate versus water velocity for water tables less than 100 cm.

All of these data were collected with the 0.5 inch diameter tip.

Figure 6.12 Penetration rate versus water velocity for water tables greater than 100 cm. All of these data were collected with the 0.75 inch diameter tip.

Figure 6.13 Linear regression on ISEP data with water table less than 100 cm with 95% confidence intervals. All of these data were collected with the 0.5 inch diameter tip.

Figure 6.14 Linear regression on ISEP data with water table greater than 100 cm with 95% confidence intervals. All of these data were collected with the 0.75 inch diameter tip.

Figure 6.15 Penetration rate versus water velocity for lab data where water table is less than 100 cm. The tip is 0.5. In this case, the values determined by the regression on the measurements are greater than the values predicted by stream power.

Figure 6.16 Penetration rate versus water velocity for lab data where water table is greater than 100 cm. The tip for these data is 0.75. The values determined by the regression fall within the limits predicted for penetration by the stream power assumptions.

Figure 6.17 Coefficient of erodibility compared to stream power for data where the water table is known. In this case, the data are fully saturated and come from both data measured in this study and Briaud et al (2001) and Kwak (2000)

CHAPTER 7 FIELD APPLICATIONS

To field test the ISEP, a recommendation was made by Dr. Margery Overton (Personal Communication) that a good test site might be on the outer banks of North Carolina in the area around the inlet formed by 2003's Hurricane Isabel. The Isabel inlet severed the outer banks just north of the Village of Hatteras and removed NC Route 12, the fresh water supply pipe, the electrical and the phone lines serving the Village of Hatteras and points south along the outer banks. An outline map showing the location of the field area is shown in Figure 7.1.

In addition, two further field tests were performed as the opportunities presented themselves. One field test was performed at a DOT bridge site near Wake forest and another

Figure 7.1 Map showing location of field area. Field area is outlined by red box. Base map from Freeman et al, 2004.

was performed at the site of the temporary bridge site over the Pea Island breach caused by Hurricane Irene in 2011. Results from these studies will also be presented.

7.1 Isabel Field Test

The Village of Hatteras was founded in the 1700's as a fishing village (Christian Science Monitor). By the 1930's, it was common knowledge that the barrier island was eroding. To correct this problem, the Civilian Conservation Corps and the Works Progress Administration collaborated to construct a dune system on the ocean side along the entire length of the island. The main road, NC Highway 12, which runs the length of the island, was completed in 1950 behind the dune system. The National Park Service maintained and rebuilt the dune system until the 1970s. Since that time, the North Carolina Department of Transportation has become responsible for the dunes that border NC Highway 12.

Hurricane Isabel made landfall on September 18, 2003. The eye passed about 10

Figure 7.2 Wave height recorded at a bouy 2 miles offshore as a function of time. These data were measured at the USACE Field Research Facility at Duck, NC, 113 km north east of study area. Downloaded from USACE (2003)

miles north of the Village of Hatteras. While it had been a Saffir-Simpson Category 5 hurricane offshore, it had weakened to a Category 3 by the time it hit. Even so, the USACE station at Duck, NC, 80 miles to the north of the Village of Hatteras, recorded waves as high as 39.7 feet, with an average of 24 feet; among the highest ever recorded at Duck, NC, USACE (2003) The wave height graph as a function of time is shown in Figure 7.2. Storm surges over 4.5 feet high and reports of waves up to ten feet high were reported within the Village of Hatteras, Tucker (2003). However, the major part of the damage is that the waves and storm surge created a 1700 ft inlet severing the outer banks just north of the Village of Hatteras. At its deepest, the new inlet was about 24 feet deep and both Route 12 and all of the utilities serving Hatteras were severed. Because there was no access to the Village of Hatteras and points south except by ferry, the decision was made to repair the breach and to rebuild Route 12 and the dunes. The breach formed by Hurricane Isabel consisted of 3 separate channels and 2 small dividers of sand separating the channels. Figure 7.2 shows 3 aerial views of the Isabel breach along with the current view. These photographs form a geo-registered suite and are provided courtesy of Dr M. Overton at NCSU.

Since repair of the breach was considered to be a time critical operation, the decision was made to use sand dredged from ferry channels in the nearby sound. To expedite the replacement process, the sand was hydraulically moved through pipes and emplaced. Surface smoothing and minor contouring using bulldozers followed placement. The goal of the fieldwork in this study was to test the sand at the reclaimed and the undisturbed locations and to see if measurable differences in scour or erodibility are found. The fieldwork was accomplished in 3 visits to the island.

Because the breach occurred entirely within the Cape Hatteras National Seashore (CHNS) a request for a research permit was made to the NPS and the permits were granted. During October of 2010, a first scouting trip was made to the field site for a meeting with the head biological technician of CHNS and for orientation and recognition of protected areas. During this trip, samples were collected of the sand at locations within the breach and from the northeastern sand divider.

Figure 7.3 Before, after, and present views of the Isabel breach and the field area. The location of the first field visit, which is in the reclaimed area, is shown by the red dot. Photos provided courtesy of Dr M. Overton at NCSU.

Sand samples were collected into a 5 gal container and sealed. For each location, a GPS reading was marked onto the lid of the container. Sampling was with a shovel and first the upper 12 inches of soil was removed from an area several times the diameter of the 5 gal bucket into which the samples were to be placed. From the center of the cleared region, shovelfuls were removed to a depth of approximately 18 inches and placed into the bucket. Field inspection revealed a number of differences between the disturbed and the undisturbed site. Visually, the undisturbed site had a much higher proportion of organic material in the form of roots, twigs and stems. The vegetal smell was also more pronounced in the undisturbed sample. In addition, the sample from the undisturbed site contained a higher proportion of fines by field inspection. Soil samples rubbed between the fingers left discoloration streaks while samples from the disturbed sites did not leave streaks.

The differences noted in the field inspections of the samples were also apparent in grain size distributions determined for the soil samples performed at the soils lab located at the Constructed Facilities Lab at NCSU. Austin Key performed the grain size tests and results are shown below. Results from the sample collected at the unbreached and the breached location are shown in Figure 7.4. The sample from the unbreached site had a D_{50} of 0.31 mm and was poorly graded with over 60% of the grains having diameters between 0.25 and 0.4 mm. The sand was relatively clean with less than 0.09% fines. By comparison, sand from the breached site had a D_{50} value of 0.425 mm and the same 60% range for this sand was 0.25 mm to 1.4 mm; a more well-graded sand.

Figure 7.4 Comparison of the grain size distribution from the breached and unbreached areas near the Isabel Breach.

A second trip to the field site was made during January of 2011. The goal of this trip was to test the equipment in the field and examine several alternatives for the data collection methodology. In particular, Hanson style measurements were attempted to determine values of the critical shear stress and the erodibility coefficient at the surface and at depth. In addition, standard penetration tests were run, although camera failures prevented collection of a detailed data set of the penetration testing.

The Hanson-style measurements are a method of determining critical shear stress and erodibility at the surface and was discussed in Chapter 4. Typically, the method is applied to cohesive sediments, but there is no reason precluding its use on non-cohesive sediments as well. During this test, the probe is clamped into place and erosional depth is measured as a function of time. From this data, critical shear stress and erodibility coefficient are

determined as outlined in Chapter 4. Assumptions are necessarily made as to the long term shape of the eroded hole and a least squares curve fit is done to the processed data to determine the best results. A picture of one of the Hanson -style experiments showing the scour hole and the entrained sediments is shown in Figure 7.5. Results from the application of this technique applied at the field site are shown in Figure 7.6.

Figure 7.5 A Hanson-style experiment for determination of critical shear stress and erodibility coefficient. Note the circular scour hole and the entrained sand from the bottom of the hole. The hole is 40 cm in diameter and 9.5 cm deep.

Figure 7.6 Results of Hanson-style determination of critical shear stress and erodibility coefficient. This information was collected at the first field site inside the Isabel breach.

Admittedly, there is some ambiguity in the application of this technique to non-cohesive soils since the equilibrium shape of the eroded hole is achieved much more rapidly than with cohesive sediments. In particular, the results presented here are based strictly on determination of the depth of the hole as a function of time. This was the assumption made here since it is consistent with the hyperbolic approximation of the time versus depth curve upon which the Hanson methodology is based. However, there is greater ambiguity in this measurement. An alternate method examined was to measure the depth to refusal below the nozzle. While there was no ambiguity to this measurement, analysis of the data was more

difficult and currently, Austin Keys is currently working on the solution of this problem.

What is most interesting about this result is that this site is well within the Isabel breach and only the replaced sand is being sampled. In comparison with the sand in the pit, the sand at this location has a lower erodibility coefficient as well as a much higher critical shear stress implying that a lower overall rate of erosion is to be expected. This is consistent with the expectations from the measured GSD revealing a D_{50} value of 0.4 mm for this site.

Regardless of whether Briaud or Shields values are used to estimate the critical shear stress of this sand, the erodibility coefficient from this site should be less than the erodibility coefficient determined for the sands in the test pit. A Comparison of this result with the sands from the pit (shown in Figure 4.2) reveals that this is in fact, true. In addition, this site was 2-3 meters from the water's edge on the sound side so there was no ambiguity regarding the depth to the water table; it was just below the surface and much of the behavior of the sands at this site ate controlled by the water table.

A series of tests were attempted to determine the critical stress and the erodibility coefficient at depth. The idea was to use the surface measurement of critical stress and erodibility as a constraint for erosion rate as a function of depth. As the probe advanced into the sand during an erosion rate measurement, it would periodically be stopped and the depth to the bottom of the hole measured. The probe would be backed upwards in the hole several centimeters and clamped to the tripod. The pump would be energized for a fixed measure of time and the erosion depth would be measured by allowing the probe to progress down the hole when the pump was turned off. From this measurement, a value of critical stress and erodibility coefficient can be calculated for a specific depth. While a single measurement of

erosion depth and time will not uniquely determine critical stress and erodibility coefficient, any values determined by another method should at least be similar.

From the experiment described in the previous paragraph, no useful results were obtained. The biggest reason for this failure to obtain useful information was that the equipment was not designed for this type of measurement. In particular, the clamp used in the experiment that supported 2 sections of the probe in the lab was unable to support 3 sections of the probe in the field. Consequently, the probe tended to slide down the hole when water was flowing rendering depth measurements dubious at best.

The last experiment performed during this field trip was a simple penetration test using the shrouded probe. The original goal of this test was to determine the erosion rate, but camera failure prevented capture of the detail necessary to complete this part of the test. However, several important observations were made. The probe penetrated between 2 and 3 meters at a normal penetration rate. During the last part of the test, the rate of probe penetration visibly decreased and the up-washed sand simultaneously changed color. The sand visible at the surface and what had been up-washed for most of the depth of the hole had been light brown or tan. At a depth of about 2.3 m, the up-washed sand became gray. Movement of the probe ceased just after the sand turned gray. The color change of the up-washed sand is shown in Figure 7.7.

Figure 7.7 Gray colored sand up-washed from depths below 7 feet during the penetration test at the Isabel breach.

Our interpretation of this observation is that the probe penetrated the erosional surface left by Hurricane Isabel. This interpretation is consistent with the available elevation profiles taken shortly after the hurricane (Figure 7.8 is used from Wamsley et al, 2009 and appears as

Figure 7.8 Elevation profiles across Isabel breach from Figure 7 in Wamsley et al 2010.

Figure 7 in their paper). An alternate interpretation is that the probe could be penetrating one of the peat layers underlying much of the island. However, even though this interpretation is possible, it does not appear to be supported as strongly by available data since most of the peat layer was removed from the breach. A subsurface Ground Penetrating Radar (GPR) profile shown in Figure 7.9 (Figure 7.9 here is Figure 16 in Mallison et al, 2008) clearly shows the peat layer to the northeast of the edge of the breach on a survey collected shortly after the breach. Mallison et al, 2008 attribute the formation of the main channel to a location where the peat layer in the form of peat terrace was missing due to previous breaching.

An example of this peat levee on the ocean side of the outer banks is shown in Figure 7.10 (Photo courtesy of Adam Clinch, 2011). While there are bright reflections beneath the test site, the reflections are closer to the surface than the depth penetrated by the probe. If these bright reflections did come from peat, the probe would have penetrated the peat at a lesser depth and a change of color should have occurred sooner.

The third visit to the Isabel breach occurred during May, 2011. The goal during this field trip was to apply the lessons learned during the short, previous field trip and to apply the

Figure 7.9 GPR profile across Isabel breach. Modified from Figure 16 in Mallison et al,

ISEP technique to the question of the sands within the breach. More specifically, the question was to determine whether the technique could differentiate between the sands placed in the breach during the repair in 2003 from those sands that had not breached. The strategy was to re-visit the site visited in January since sand parameters and the geological beach structure were known.

An additional site was located several hundred meters further south on one of the islands that separated the channels by remaining above water during the breach.

Arrangements had been made to meet with the acting head biological technician of

Figure 7.10 Exposed peat layer on the outer banks; on the ocean side directly across the island from the field area.

the Cape Hatteras National Seashore because a nesting site of a pair of birds, Oyster Catchers, was found to be active and the park service wanted to ensure that no birds were disturbed or harmed during the research. The day before we arrived, a bird was hatched and the birds moved away from the nest to the north. Park service rules dictate that all shore birds are to be treated as endangered species and thus we were closed out of the 2 field areas for which we were permitted to provide a buffer zone for the birds. Even though the birds returned back south to their nesting area, where there were still 2 eggs in the nest, the

boundary remained 30 m north of our northernmost field area. E-mails were hurriedly sent to NCSU to obtain coordinates locating the breach boundaries and special thanks are in order for Onur Kurum (NCSU, Personal Communication) for his assistance in delineating the breach boundaries. Three sites were chosen for testing. The first site was between 2 m and 4 m outside the breach boundary, the second site was about 30 m outside of the boundary and the third site was approximately 24 m inside the boundary.

The first site was visited on 5/17/2011 and 9 penetration rate tests were made. Five conventional probe tests, 3 shrouded probe tests and another test that jammed in the tripod and was not recorded. A number of Hanson style tests were planned, but were not run. The tripod had been damaged during packing for the field and during transportation the upper plate had blown off and was lost. Consequently, there was no way to clamp the probe to the tripod.

Data were collected using a 2-segment probe. The data were collected in a 2 rows where each test was at least 1 meter from any other test. For this site, water velocities were either 3400 rpm or 1800 rpm. These numbers were chosen based on results obtained in the lab. The conventional probe was tested first since most of the lab work was done with this configuration and 3400 rpm was chosen to insure adequate penetration. Using the conventional probe, three runs with the pump at 3400 rpm were followed by 2 runs at 1800 rpm. Three runs of the shrouded probe followed this with the pump set at 1800 rpm.

Videos were made of each test run using a Flip Ultra camera operating at 30 frames per second. The videos were imported into Vernier's LoggerPro software. Data recorded in the LoggerPro software from analysis of the videos were video frame number and depth

since LoggerPro gave the opportunity to step through the video on a frame-by-frame basis. Frame number data was collected for penetration intervals of at least 1 cm for some of the tests and 5 cm for other tests although penetration intervals were allowed to increase up to 7 cm if the motion of the probe was fast enough. The data were exported from LoggerPro into a .csv file for importation into EXCEL. Once the data were imported into EXCEL, the frame numbers were converted into differential times and a midpoint for each interval was calculated. The probe penetration rate was calculated as $p=ds/dt$ which had units of cm/s and was then converted to mm/hr.

Results from site 1 are summarized in Figures 7.11 and 7.12. Figure 7.11 shows results from the shrouded probe at 1800 rpm. On Figure 7.11, several trends are visible. Five out of the 5 experiments show a zone of low penetration rate between 16 and 22 cm depth. There is also a zone of low penetration rate near 33 cm depth where 4 out of 5 test show low values. Another zone of low penetration rate can be observed between 50 and 60 cm. and there is also a possible low penetration rate zone near 92 cm. Similarly, depths between 30 and 40 cm show an penetration rate that is higher than normal. Other zones with a high penetration rate are between 48 and 56 cm and between 63 and 72 cm.

Figure 7.11 Data collected with the shrouded probe from Site 1 at the Isabel breach.

The high penetration rate zone near 63 cm is of particular significance. After testing, the probe was carefully extracted and a water tape was inserted into the hole to locate the water table. What was actually measured was the upper surface of the saturated sand that collapsed in the hole marking the general location of the water table. At site 1, the depths to

the collapsed sand ranged between 56 and 72 cm below the surface. Much of the variability in the water table occurred because the tide was going out during this period.

Figure 7.12 shows results collected from site 1 with the conventional probe. In a general sense, the same trends are visible although there appear to be more swings in the data. There are zones of low penetration rate near 22 cm, 32 cm, 42 and 58 cm. Likewise, there are zones of higher penetration rate near 36, 50, 58 and 66 cm. The conclusion is that regardless of the instrument, the penetration rate variations are constant. However, the extreme variability shown by the data suggests that care should be taken with this interpretation.

Because of the difficulty associated with interpretation of the data shown in Figures 7.11 and 7.12, an alternate way of displaying the data was sought. Dr. Gabr made the suggestion to analyze and display the data in the form of cumulative time versus cumulative penetration. One advantage to displaying the data in this fashion is that the averaging, which is built into the process, smoothes some of the variations making patterns easier to observe. Since cumulative time is plotted against cumulative penetration, the slope is the reciprocal of the penetration rate. In other words, flatter curves represent higher penetration rates while steeper curves represent lower penetration rates or more resistant sands. The data are also plotted against a secondary axis on the right side of the graph. The axis labeled depth is the depth to the mid-point of each of the intervals for which an erosion rate was determined.

Consequently, the depth and the cumulative penetration are not necessarily the same. Figure 7.13 replots a portion of the data collected with the conventional probe from Site 1 (shown in Figure 7.12 as the black crosses) in the fashion described above. Note that the variations

visible in Figure 12 have been damped in Figure 7.13 and the slope ($1/\text{erosion rate}$) is observed to be relatively constant over the entire depth although there are distinct offsets representing zones where the probe sticks momentarily.

Figure 7.12 Data collected at site 1 with the conventional probe.

In Figure 7.14, the data from the shrouded probe in Figure 7.11 is plotted in the same cumulative time vs cumulative depth format. In contrast to the data collected with the

conventional probe, which is shown in Figures 7.11 and 7.12, the data collected with the shrouded probe has a curve which is described here as "concave upward". Again, in contrast to both the conventional probe as well as the erosion rate versus depth, the detail is more apparent. From 0 to 10 cm of erosion, the erosion rate is high and from 10 to 37 cm slows down and from 37 to 57 it slows even more until 57 cm depth. The sudden high erosion rate at the end, from 57 to 63 cm is misleading since it occurs right at the end of the run and the probe ceases all movement at 63 cm. The interval from 37 to 57 cm is a good display of the stick-slip movement often displayed by the probe. The actual movement visible on the video is for the probe to stop at a depth, erosion to occur, fall into the resulting hole, stop and repeat. This is in contrast to another type of visible behavior where the probe remains in continuous motion, but just at a very slow rate (out to 1 mm/45 min). This type of motion has, to date, only been observed in the lab.

All of the runs from Site 1 that were collected with the shrouded probe and with the pump speed set to 1800 rpm (~ 2.8 m/s) are plotted together in Figure 7.15. Two of the runs are similar and show the concave upward behavior described earlier. The third experiment shows a very different behavior appearing almost linear with several offsets. The reason for this difference in behavior is explained in the field notes where the remark was made that "...no water out top" and "... probe plugged with shells." Since the purpose of the shrouded probe was to allow pressure release, which did not occur, the probe reverted to a behavior more similar to that of the conventional probe. For comparison, the conventional results from Site 1 collected at 1800 rpm are displayed in Figure 7.16.

Figure 7.13 Penetration rate versus depth data shown as black crosses in Figure 11 replotted as cumulative time versus cumulative penetration.

Penetration rates determined with the conventional probe at Site 1 are uniformly higher than those rates determined using the shrouded probe. All of the measurements determined from data using the shrouded probe range from 1100 to 252,000 mm/hr. One measurement of the lowermost interval of v18 of site 1 was 200,000 mm/hr, but all of the other intervals had rates that ranged between 273,000 and 4.5×10^6 mm/hr. This was

completely unexpected since it is opposite to the behavior observed in the lab; in the lab, the shrouded probe penetrates at a much faster rate than the conventional probe.

On the basis of the original data, several interpretations were made regarding zones of low and high penetration rate. On Figure 7.15, low penetration rate zones are found between 15 and 21 cm, 30 and 40 cm, 47 to 53 cm and 90 to 95 cm. High penetration rate zone may likewise be observed between 0-10 cm, 25-30 cm, 45-53 cm and 55 to 63 cm. These compare favorably to the zones discussed earlier and suggest the interpretation in this case is comparable regardless of the display approach.

Figure 7.14 Data collected with the shrouded probe at site 1.

Since results from the analysis of cumulative time vs. cumulative erosion are easier to interpret, Further results from the field studies will be in this format for the remainder of the text. The penetration rate data in "penetration rate vs depth format" is shown for all of the sites in [appendix 4](#).

Figure 7.15 All of the shrouded probe runs from Site 1 collected with the pump set at 1800 rpm. Note run v22, denoted by the triangles, is different from the other shrouded probe runs. On this run, the probe became plugged with shells and behaved more like a conventional probe.

Erosion results from site 2 are shown in Figures 7.17 and 7.18. Site 2 is well outside the breach boundary, so this site was chosen to represent the unbreached region. All 6 of the

runs shown in these 2 figures were collected with the pump set at 1800 rpm. Figure 7.17 shows the data collected with the conventional probe. On Figure 7.18 there is a low penetration rate zone visible between 15 and 20 cm down. Other low penetration rate zones are around 30 cm, 38 cm, 54 cm and 70 cm. There are high penetration rate zones visible near 22, 32, 44 and from 58 to 65 cm. depth.

Figure 7.16 Conventional probe results collected at Site 1 with the pump rpm set at 1800 rpm (~2.8 m/s).

Figure 7.18 shows results collected with the shrouded probe. The pump rpm was set to 1800 rpm for each of the three runs shown in the figure. Again, as observed at site 1, there

are fewer swings visible in the data using the shrouded probe. What is most striking is that two of the runs correspond to comparatively low erosion rates while the third is an order of magnitude higher. The field notes provide an explanation of the difference. During the v30 test, no water was observed to be exiting from the top of the probe. This is unusual behavior for the shrouded probe and in analogy to the results observed in run v21 at site 1, probably corresponds to the annular region between the central tube and the shroud being plugged. The field notes also noted that the amount of sand that washed up around the bottom of the probe is much greater than is normal which also suggests that when the shrouded probe gets plugged, it behaves more like the conventional probe. In addition, it displays more of a stick-slip behavior on the video data. Lastly, the low penetration rate zone observed in data collected with the conventional probe in the depth region of 15-20 cm does not appear visible on data collected with the shrouded probe except on 1 run, v29.

Figure 7.17 Results of the conventional probe at Site 2; the unbreached site. Note the similarities in the slopes for each of the three runs.

The low penetration rate zone near 15 cm on Figure 7.18 warrants additional examination. As mentioned earlier, the probe was carefully removed after every run to measure the depth to the water table. As the depth of the hole was being measured, a color difference was noticed in the side of the hole at a depth of 15 cm. Further investigation showed that there was a layer of shell fragments at a depth from near 15 to about 21 cm. A

picture was obtained showing this layer of shell fragments in the illuminated part of the hole in Figure 7.19. This is significant because in this case, a low penetration rate zone may be observed on experiment v29 on Figure 7.18. A red diamond on Figure 7.18 highlights the low penetration rate zone corresponding the layer of shell fragments observed on figure 7.19.

Results collected from Site 3 are shown in Figures 7.20 and 7.21. Two runs were performed at site 3 at a pump speed of 1800 rpm and these results are shown in Figure 7.20. Due to time and water constraints, only one run was attempted at site 3 using the conventional probe at 1800 rpm and the results of this run are shown in Figure 7.21. On Figure 7.20, there are several characteristics to note. First observation, and perhaps most important, is that the slopes of the 2 runs are similar implying that the same erosion rate is determined for each run. Except for the interval 0 to 20 cm erosion depth, the penetration rate determined for the other intervals ranges from 100,000 to 750,000 mm/hr. What is significant is that the probe was plugged for experiment v36 and yet the erosion rates determined at this location were very close to those determined from the unplugged probe. Moreover, the changes in penetration rate of the sand occur at the same depths; 20, 85, and 115 cm.

Figure 7.18 Shrouded probe results from site 1 run at 1800 rpm. Note the red diamond on experiment v29 that corresponds the layer of shell fragments shown on Figure 7.19.

Figure 7.19 Shell Fragment layer corresponding the low erodibility zone at 15 cm depth at site 2.

At Site 3, runs were also made using the shrouded probe at 3400 rpm. While these were mostly to examine the depth of penetration, it is significant to note that the shape of the curves from inside the breach is very different compared to the curves made from data collected from outside the breach. Data from Site 3 shows a "concave downward" curve, while data collected outside the breach shows a concave upward curve.

Another observation made at Site 3, which is visible both in the field and on the video, is that the stick-slip behavior that causes the behavior apparent on the graphs of erosion rate versus depth. The erosion rate versus depth graphs are shown in Appendix 4. The periodicity of the penetration rate variations was measured and is found to have an average value of 7.75 cm with a lowest value of 3 cm and largest value of 10 cm. The nature

of these variations is not known but might represent variations in relative density caused by the bulldozers contouring the sand after placement.

What is perhaps most interesting about the results from these experiments is that the values of the erosion rates determined for the sands near the Isabel breach are much larger than expected. The average values for the erosion rate of the sand inside the breach ranges from 900,000 mm/hr to 1,400,000 mm/hr. The average value for the erosion rate of the sand outside of the breach ranges from 150,000 mm/hr to 200,000 mm/hr. In comparison, values for the erosion rate measured for the sand in the lab range from 1000 mm/hr to 40,000 mm/hr. Since the grain size distributions of the sands outside the breach are similar to the sand in the test pit, the expectation is that the erosion rate should also be similar. This expectation arises since the critical shear stress is a function of the D_{50} value and since the erosion rates are so different, another explanation must be sought. Within each site at which data was collected, the consistency of numbers determined for the erosion rate suggests that the measurements are reflecting a truth about the relative values of the erosion rate even if the numbers themselves are confusing.

One difference in the analysis between the field and the lab that must be discussed.

For most of the results collected in the lab, data were collected with a Sony Digital8 camera.

Figure 7.20 Shrouded probe results from site 3 with the pump at 1800 rpm. This site is within the breach.

This camera recorded video images using the NTSC standard implying a nominal frame rate of 29.97 frames per second. However, the 525 vertical scan lines (of which 486 are visible) of the imaged are interlaced and analysis of single frames is hindered by smearing within the frame because of the interlacing of the scan lines. As a consequence, timing of penetration

rates was accomplished manually with a stopwatch and erosion rate measurements were limited to time intervals of at least 1 second. This was chosen to keep errors due to reaction time less than 20% if reaction times of 0.1 second are assumed. In contrast, the FLIP video camera used in the fieldwork uses an NTSC recording standard of 720 progressive, vertical scan lines in a frame. The inherent sharpness of this technology allows time picks (i.e. frame numbers) to be chosen to the nearest 1/30 sec. This process of picking times and depths is greatly simplified using the FLIP technology. However, the consequence of the FLIP's ability to subdivide time is that, given the stick-slip style of motion, the lack of averaging allows one to concentrate on the periods of movement by themselves rather than on an average of both motion and rest periods. The data shown in Appendix 2 have all been processed in this way and reveal rapid variations in calculated erosion rate. The erosion rates are very large but consistent giving erosion rates between 4 and 30 times larger than expected compared to the similar lab sands.

Figure 7.21 Site 3 data collected with the conventional probe at 1800 rpm.

The discrepancy between penetration rates from the beach and from the lab is based on choosing depth intervals of 1-3 cm, which might only require several 1/30's of a second to traverse the interval, but misses the several seconds the probe pauses at the top and bottom of the interval.

Lessons learned during this field trip are specific in terms of the data collection.

- The shrouded probe can be started without a tripod, but it tends to wobble

Figure 7.22 Results from Site 3 using the shrouded probe at a pump speed of 3400 rpm.

Note the characteristic "concave downward shape of the runs within the breached area

without extreme care. The wobbling probe then makes it difficult for consistent depth picking in the near surface. The oscillations damp themselves as the probe embeds itself.

- The annular region of the shrouded probe tends to get plugged when passing through zones of large shell fragments. In fact, it is a good idea to check the annular region after every run when operating in a beach environment. It is recommended that the size of the inner pipe be decreased to lessen the likelihood of this occurrence.
- At the beginning of testing in a new area, extra time should be spent calibrating the water velocity at the tip of the jet. Experiences in the lab suggested that higher velocities should be used in the field. Unfortunately, use of the higher velocity water in the field tended to obscure variations in penetration rate visible at lower velocities. The controlling factor for depth of penetration in the field tends to be the water table. If the water table is at or very near the surface, penetration depths of several meters can be obtained, but there is a fine line between fluidizing the sand and eroding the sand.

If the penetration rate is assumed to reflect the erodibility potential of the tested soil, the first conclusion that can be drawn about the sands in and around the Isabel breach is that the sand is much more erodible than expected. The sands from within the breach that were placed during the repair are 4-5 times more erodible than the sands from outside the breach. This conclusion is supported by results obtained with both the conventional probe and the shrouded probe. The reason for the difference is not fully understood, but the most likely explanation for this observation is mechanical in nature. The truth is that the sands in the uneroded area are more likely to have roots, fines, and some larger shell fragments all mixed together. This observation was noted in field examination, however, ASTM D422 requires

that the roots and shell fragments larger than a number 10 sieve to be removed. Hence, the gross similarities in the grain size distributions between the breached and the unbreached sands. Furthermore, as the sand in the breach was hydraulically placed, it was placed and only minor contouring was performed with bulldozers so one might expect sand layers on the lower side on the minimum density scale.

Another conclusion that can be drawn about the differences in the sands within and outside of the breach is that the penetration rate data forms curves of different shapes. Inside the breach, the erosion rate is lower at the surface and then about 20 cm down, the penetration rate increases dramatically. From there until the maximum depth reached, the penetration rate is high forming a series of sub-parallel curve segments. One interpretation of this behavior is that the sand is fairly uniform in its penetration rate and that the probe sticks and then slips.

Penetration rate data collected outside the breach forms curves that are "concave upward" implying that the erosion rate is highest at the surface and then decreases with depth. On some of the curves there is a small high penetration rate for the last 10 cm of depth, but this is an artifact similar to behavior observed in the laboratory studies. When the probe finally stops and the water pump is shut off, the pressure is dissipated and the probe settles downward. To put this in a different way, the sand becomes mechanically stronger and more erosion resistant as the probe goes deeper when sampling sands in the unbreached areas.

7.2 Other ISEP Field Tests

Discussions with Jerry Beard of the NCDOT led to the opportunity to test the ISEP at several locations of interest to the NCDOT. The results from these tests are presented in the next two sections.

7.2.1 Wake Forest Bridge site

The first site was at a bridge near Wake Forest, NC. The bridge is on Oak Grove Church Road and it crosses Smith Creek, north of the Wake Forest Reservoir. The NCDOT has been monitoring the depth of the creek at the bridge site for years and has estimates of erosion and deposition at the bridge site spanning about 15 years, Figure 7.23. After a scouting trip to evaluate the suitability of the site and site access, plans were made to measure penetration rate data on September 16, 2011.

Based on the initial scouting trip in July, examination of the stream bed appeared to indicate that the bed of the river was composed mostly of silt. When testing commenced, rain-induced flow had washed the bed, which was found to consist of mostly of sand with inter-layered organic material. The organic material consisted mostly of partially-decayed leaves from the previous year. While samples of the sand and the organic material were collected for lab analysis, they were discarded by an overzealous CE 548 student who did not bother to read the paper name tags in or on the sample bowls.

When the site was reached on the morning of the 16th, equipment was erected on a sandbar in the creek, Figure 7.24. NCDOT collected DCPT data on the sandbar before penetration testing started and the penetration testing was planned to be as near to their site as

Figure 7.23 NCDOT Observers on bridge overlooking field site near Wake Forest, NC.

Jerry Beard of NCDOT who helped arrange the experiment is on the right.

possible. The generator and tank were set up at the road level on the side of the road next to the bridge and a transfer pump was hung from the bridge to fill the tank. The pump was set up several meters down the hill toward the creek allowing gravity head to prime the pump.

Testing commenced as soon as the equipment was set up and ready. During the testing, a total of 7 penetration runs were collected and 4 of the runs gave useful data. In one case that did not give useful data, the probe got stuck in one of the shallow pods of the organic material and did not penetrate very deeply. When the probe became stuck, the water coming up through the annular part of the probe was carrying large chunks of organic material. Since all of these tests were performed using the 3/4 inch tip, the decision was made to try the 1/2 inch tip and see if the increased water velocity would provide more of a cutting action and be more successful at penetration the layer of organic material. During this part of the testing, the probe appeared to be less able to penetrate the organic layer accounting for

the last 2 unsuccessful tests suggesting that a combination of flow rate and velocity controls the depth of penetration.

Figure 7.24 View from the bridge of the sandbar location at Wake Forest Bridge site.
From left are M. Kayser, C. Styffeler, A. Clinch and the author.

Several Hanson-style tests were performed during the site visit. These tests followed the same format as described earlier during tests of the pit sand and during the visit to the Isabel breach. Processing also followed the same procedure previously described. Results from this analysis are shown in Figure 7.25. Values from this test are found to support observations made in the field, but they are not conclusive since they cannot be compared to

the soil samples. From this analysis, K is found to be $8.25 \text{ cm}^3/\text{N-s}$, which is lower than K values found for the test pit sands or sands from the Isabel breach.

Figure 7.25 Results of a Hanson style experiment performed at the Wake Forest Bridge site.

Data collected during this field experiment were processed in the same way as described during the Isabel breach experiment and the final comparison of the penetration data from each of the four tests are shown in Figure 7.26. Three separate discontinuities may be observed here. After 5 cm, the data show a faster penetration rate after starting slowly. Near 22 cm, the penetration rate slows down on two runs and then speeds up near 30 cm. One of the runs slows down near 15 cm and speeds up near 20 and slows down again near 30 cm. Lastly, of the two runs that penetrate deeper than 60 cm both slow down at 50 cm and speed up again at 60 cm.

Analysis of this data set, which was collected with the 3/4 inch tip, reveals the most consistent set of values for penetration rate yet obtained. Penetration rates ranged from a low of 8,650 mm/hr for the discontinuity at 15 cm in test V2 to a high of 650,000 mm/hr for depths between 20 and 33 cm of test V4. More typically, resistant layers show penetration rates of about 13,000 mm/hr (test V3 at depths between 49 and 52 cm) and softer layers show penetration rates of about 150,000 mm/hr. This suggests that the data collected with the ISEP is differentiating between 5-10 cm thick layers found in the bed of Smith Creek. As might also be expected from the layered bedding of a sandbar in a fluvial environment, the layers vary in depth across the bar. What is gratifying about the results of this experiment is the consistency of the results both in the numerical values as well as with the agreement with the geological expectations.

7.2.2 Pea Island Bridge site

The opportunity to test the ISEP at the site of the temporary bridge over the Pea Island Breach caused by Hurricane Irene was offered by Jerry Beard of the NCDOT. The tests were performed on October 5 and 6, 2011. One of the significant features of this site is that there is a significant amount of geotechnical data available due to the construction efforts associated with the temporary bridge site. Another significant feature of this test is that all experimental equipment was carried in one pickup truck. This significantly reduces the cost of the experiment.

During the two afternoons occupying the site, penetration testing and Hanson style experiments were performed. The average values of penetration rate found at this site range

between 180,000 mm/hr and 1,800,000 mm/hr, which is comparable to the rates found further south at the Isabel breach site. On both afternoons, the tide was coming in so the water table was always within 10 cm of the surface. Since the water table was so high, the

Figure 7.26 Comparison of results from field testing of the ISEP at the Wake Forest Bridge site.

expectation was that the ISEP would penetrate well and, in fact, it did. On one penetration run, the ISEP penetrated about 3 meters. It appeared that it would have kept penetrating, but a bolt had broken off in one of the collars so only 2 sections were useable. However, this was the deepest penetration to date. The surface itself consisted mostly of fine sand with inter-

bedded silts and organic materials. Some of the inter-bedded organic materials were large and on several of the penetration runs, the probe would stop and upon removal of the probe, sticks greater than 1 inch in diameter were recovered by hand from the hole.

Figure 7.27 Aerial view of the temporary bridge over the Pea Island breach. The field site is located by the red circle. The view is north and this photo is by Brian J. Clark of The Virginian-Pilot. Picture downloaded from:

<http://hamptonroads.com/2011/09/temporary-bridge-hatteras-island-nearly-complete>

Penetration field data were processed as previously described and results are presented in Figure 7.28. In two out of the three runs displayed here, the penetration rates are comparable. At cumulative penetrations greater than 40 cm, the rates are almost identical as evidenced by the similar slopes observed in experiments V15 and V22. The values for tests V15 and V22 range between 360000 and 418000 mm/hr while the average erosion rate for V20 is 94900 mm/hr. The high values measured for V15 and V22 may be suggestive that at

some velocity greater than 2.6 m/s, the sand is fluidized. The low values measured at V20 might also be caused by buried organic material, as discussed earlier. However, the sand at the Pea Island breach is very easily removed by water. This conclusion is supported by the results of the Hanson-style experiments performed at the site. Results from the Hanson style experiments are shown in Figure 7.29 and what is significant here is that the critical shear stress is the lowest determined in either the field or the lab by the ISEP. While there is

Figure 7.28 Penetration rate results from the Pea Island Bridge site. Experiment V15 used a water velocity of 4.5 m/s, experiment V20 used a water velocity of 2.6 m/s and experiment V22 used a water velocity of 7 m/s.

insufficient data to support the conclusion that this is a more erodible sand than others measured to date, results gathered in this part of the research are consistent with this conclusion.

Figure 7.29 Results from a Hanson-style analysis of the sand at the Pea Island Breach. The characteristics of the sand at this location suggest that it is the most erodible sand measured by the ISEP to date.

A vertical probe employing a water jet has been developed for measuring erosion potential of non-cohesive sediments. The focus of this thesis was on the development of prototype device and testing its functionality in an attempt to investigate its use to estimate the erodibility potential of sand. The probe, termed ISEP, has been tested in sand across a range of water velocities and flow rates. Water velocities have been correlated to a range of shear stresses applied to the surface below the tip and a scour rate has been calculated. While more work is needed to correlate penetration rates measured from the ISEP data to scour rates measured by other means, penetration rates determined using the method described herein are similar those measured and reported in the literature for similar sands.

Results on the test sand with mean particle diameter (D_{50}) ~ 0.3 mm suggest that the rate of advancement of the probe can be empirically correlated to the vertical velocity of the water at the tip of the probe raised to a positive exponent. For the sand used in testing, the exponent appears to be 1.32. Only one other attempt to make this correlation was found in the literature (Niven and Kalili, 1999) and the exponent given there was 0.995. This was an average value for the penetration depths collected over a wide range of sands and while not directly comparable is certainly similar.

Penetration rate, as might be estimated from the probe data, based on the assumptions made herein, is found to be a function of depth. For the sand used here with a $d_{50} \sim 0.3$ mm, lower scour rates are found as depth increases with a assessed value of 2300 mm/hr for depths between 55 and 60 cm. The decrease in the scour rate with depth has 2 major causes

which, while are not mutually exclusive, tend to dominate the behavior in different regimes. One of the major causes is that as the probe penetrates deeper into the soil, there is an increase in the effective stress. In the unsaturated case for the sand tested here, much of the increase is related to the suction stress increasing the effective stress. In the saturated case, the increase in effective stress is similar to the classical view. Regardless of the cause of the increase in effective stress, the result is that the friction between the sand grains is increased.

The end of the probe's penetration comes from the formation of closed, fluidized zones beneath the tip of the probe. This is assumed because once the jet is turned off and the water pressure allowed to dissipate, the probe was observed to drop some distance into the sand. Within these zones, the water becomes over-pressured when a sand plug forms plugging the hole next to the probe and the combination of friction along the side of the probe and the force over-pressured region brings the probe to a halt. This observation is consistent with the observations of Niven and Khalilli (1999). In this research, this phenomenon is indirectly observed by the collapse of the roof of the fluidized zone caused by the weight of the probe when the overpressure is removed after the water is shut off.

The mechanics of using jetting to correlate to a “scour rate” is complicated and work is still in progress to assess applicability of the data reduction approach and the use of this technique to obtain a scour rate with depth. However, a preliminary model suggests that the decrease in penetration rate observed with depth is consistent with increasing effective stress. More recent results suggest that a modeling approach, based on the fully turbulent nature of the jet, which estimates the size of the pressure fluctuations provides a better estimate of the erosion rate.

The proof of the concept of the ISEP has been established. This technique shows promise as a quickly deployable method for estimating the scour potential of non-cohesive soils. Results from use of this technique have been found to be consistent with previously published measurements of penetration rate (Briaud et al (1999) and Kwak. (2002)).

The probe has been tested in the field at the site of the 2003 Isabel breach and has shown the ability to differentiate between sand used by the USACE to repair the breach and the existing sand of the outer banks. During the course of testing, the probe penetrated the subsurface erosional discontinuity and the difference in penetration rate was detected.

On the basis of this study, the following recommendations are made for ease of use and for further study.

1. The equipment needs to be streamlined for ease of use in the field. In particular, the finely machined threads are sensitive to clogging by silt, salt, and sand. In addition, many of the threads joining sections of the probe are too long and need to be trimmed. Field trials revealed this weakness since fines in the naturally occurring sands clogged the threads on the probe as well as internal threads and the fines dried with the salt forming a layer within the threads that was not easily removed.

2. Another consideration is that the addition of the shrouding, makes the probe rather massive. That coupled with the trend to more segments in use means that the original design goal of a lightweight, easily portable device has been violated. Smaller diameter tubing for the inner part of the probe should be considered in a new model probe.

3. Based on manually-controlled experiments, more work needs to be done on computer control of the water-flow. Experiments show that the deepest penetrations have all

come when the pump is started at a low rpm and the speed is increased as the probe penetrates the soil. This increasing of flow rate is hard to control manually and since the pump controller has the ability to be externally controlled, it should be utilized.

4. A wider variety of soils need to be tested. In particular, the sand in the pit and the sands at the beach are fine sands, my recommendation would be to next test the ISEP in coarse sand. Following those tests, controlled mixes of fines and coarse sands would be in order. For more versatility in local field-testing, my suggestion would be to start testing mixes of silts and sands. After this, the complexities of clay-silt or clay-sand mixtures should be tackled.

5. While progress has been made on automating the data collection process, this direction needs to be continued since the amount of time necessary to analyze results from a single experiment takes longer than the experiment to run. In particular, as depth and time measurements are automated, pump rpm and water flow rate should be added as well.

6. Since the driving force for this mechanical system is the weight of the probe itself, a necessary experiment to determine the exact relationship between driving force, rate of erosion and skin friction is necessary. Early experiments showed an increase in penetration depth with increasing weight, tested with both more probe segments and with attached weights, but it is unknown how this affects erosion rate. The skin friction on the probe increases linearly as a greater depth of the probe is embedded.

7. Based on the behavior of lab and field experiments, it appears that relative density of the sand, as well as the D_{50} of the sand, plays an important role in determining the erosional resistance of the sand; especially at the surface. Testing of this hypothesis should

be attempted once the behavior of the probe is completely understood.

Based on the experience gained in the field tests, the following recommendations are made for processing the field data. The data should be collected with a fully digital video camera with a frame rate of at least 30 frames per second. Logger-Pro or another data analysis package is then used to extract penetration depth of the probe and frame number. These data should then be exported to a spreadsheet or another processing package. Once in a spreadsheet, the data can be used to calculate depth below the surface of the tip and relative time elapsed. Once the penetration rate is calculated in mm/hr, Reynolds number, frictional coefficient, and initial shear stress at the surface, t_0 are calculated, and applied shear stress at the surface are calculated. Once these are calculated, J_p and stream power are calculated. From these, K_d may be calculated and compared to calculated penetration rates.

Since data collected in the field has large numeric variations, the penetration rate when plotted as a function of depth tends to be difficult to interpret. This is still a good plot to produce in order to QC the data. To smooth out the data for easier interpretation, it has been useful to plot cumulative time against cumulative depth in the fashion outlined in Chapter 7. If this processing is performed, the averaged velocity over a depth interval is simply $1/\text{slope}$.

The goal for future research should be to find a relationship between K_d and other measured parameters. The reasoning for this suggestion is that K_d includes both material and environmental factors and determination of K_d doesn't require a priori knowledge of the material or its depth of burial. By determining strong correlations between K_d and other parameters, many of the questions that are left unanswered in this research may be answered.

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Appendices

Appendix 1 - Experimental matrix

Table A.1 Table of penetration and erosion rate experiments.

The following abbreviations are used in this table: LST = Long Shrouded Tip, SST = Small Shrouded Tip, UK=unknown, NA=Not applicable, FC=0.125 " cone tip, FT=Flat Top w/6 Orifice ,TC=Truncated Cone - 0.5 " Orifice, C=Truncated Cone - 0.75 " Orifice, Q=Quickened, NP=No Penetration, P=Penetration, ER=Erosion Rate, ND=No Data, SW=Standing water, SC=Soil Catch, ST=Shroud test, H=Extra Weight.

Test	Tip	Pump RPM	Valve	Flow Rate	Tip Velocity	Water Table	P/ER/ND	Notes
1	FC	3439	NA			UK	ND	NM
2	TC	3439	NA			UK	P	
3	FT	3439	NA			UK	P	
4	FT	3439	NA			UK	P	
5	FC	3439	NA			UK	P	
6	FC	3439	NA			UK	P	
7	TC	3439	NA			UK	P	
8	TC	3439	NA			UK	P	
9	FT	3439	NA			UK	P	
10	FT	3439	NA			UK	P	
11	FC	3439	NA			UK	P	
12	TC	3439	NA			UK	P	
13	TC	3439	NA			UK	P	
14	FT	3439	NA			UK	P	
15	FT	3439	NA			UK	P	
16	TC	3439	NA			UK	P	
17	TC	3439	NA			UK	P	
18	FC	3439	NA			UK	ND	
19	FC	3439	NA			UK	P	
20	FC	3439	NA			UK	ND	
21	FT	2900	NA			UK	P	
22	FT	2900	NA			UK	P	
23	FT	2900	NA			UK	P	
24	FT	2900	NA			UK	P	
25	FT	3439	NA			UK	P	
26	FT	3439	NA			UK	ND	
27	FT	3439	NA			UK	ND	

28	TC	3439	NA		UK	P	
29	TC	3439	NA		UK	P	
30	TC	3439	NA		UK	P	
31	TC	3439	NA		UK	P	
32	TC	2900	NA		1" SW	P	
33	TC	2900	NA		UK	P	
34	TC	2900	NA		UK	P	
35	TC	2900	NA		UK	ND	
36	FC	2900	NA		UK	P	
37	FC	2900	NA		UK	P	
38	FT	2900	NA		UK	P	
39	TC	2900	NA		UK	P	
40	FC	3150	NA		UK	ND	
41	TC	3150	NA		UK	P	
42	TC	3150	NA		UK	P	
43	TC	3150	NA		UK	P	
44	TC	3150	NA		UK	P	
45	TC	3150	NA		UK	P	
46	TC	3150	NA		UK	P	
47	TC	3150	NA		SW	P	
48	TC	3150	NA		SW	P	
49	TC	3150	NA		SW	P	
50	TC	2900	NA		SW	P	
51	TC	3439	NA		SW	P	
52	TC	2900	NA		SW	P	
CHANGED SUCTION HOSE							
53	TC	UK	NA	15	UK		
54	TC	1400	NA	11.6	UK	ER	2-SEG
55	TC	1400	NA	11.6	600 cm	ER	2-SEG
56	TC	1400	NA	11.6	600 cm	ER	2-SEG
57	TC	3439	NA		UK	ER	2-SEG
58	TC	2400	NA		UK	ER	2-SEG
59	TC	2400	NA		UK	ER	2-SEG
60	TC	3439	NA		UK	ER	2-SEG
61	TC	2800	NA		UK	ER	2-SEG
62	TC	2800	NA		UK	Q/ND	2-SEG
63	TC	2800	NA		UK	Q/ND	2-SEG
64	TC	3439	NA		UK	Q/ND	2-SEG
65	TC	3439	NA		UK	Q/ND	2-SEG
66	TC	3439	NA		UK	Q/ND	2-SEG
67	TC	3439	NA		UK	Q/ND	2-SEG
68	TC	1400	NA		UK	ER	2-SEG
69	TC	3000	NA		UK	ND	2-SEG
70	TC	3000	NA		UK	ND	2-SEG
71	TC	3000	NA		UK	ER	2-SEG
72	TC	3000	NA		UK	ND	2-SEG
73	TC	3000	NA		UK	ER	2-SEG
74	TC	2800	NA		UK	ER	2-SEG
75	TC	2800	NA		UK	ER	2-SEG

76	TC	3200	NA		UK	Q/ND	2-SEG
77	TC	2400	NA		UK	ER	2-SEG
78	TC	3200	NA		UK	ER	2-SEG
79	TC	1800	NA		UK	ER	2-SEG
80	TC	1800	NA		UK	ER	2-SEG
81	TC	2100	NA		UK	ER	2-SEG
82	TC	2100	NA		UK	ER	2-SEG
83	TC	2600	NA		UK	ND	2-SEG
84	TC	2600	NA		UK	ER	2-SEG
85	TC	2600	NA		UK	ER	2-SEG
86	TC	1800	NA		UK	ER	2-SEG
87	TC	2100	NA		UK	ER	2-SEG
88	TC	2600	NA		UK	ER	2-SEG
89	TC	3000	NA		UK	ER	2-SEG
90	TC	1400	NA		UK	ND	3-SEG
91	TC	1400	NA		UK	ER	3-SEG
92	TC	1400	NA		UK	ER	3-SEG
93	TC	1400	NA		UK	ND	3-SEG
94	TC	1800	NA		UK	ER	3-SEG
95	C	1400	2X2O-4X4O		UK	ER	2-SEG
96	C	1400	2X2O-4X4O		UK	ER	2-SEG
97	C	2300	2X2O-4X3C		UK	ND	2-SEG
98	C	2300	2X1C-4X3C		UK	ER	2-SEG
99	C	1800	2X1C-4X3C		UK	ER	2-SEG
100	C	1800	2X2O-4X3C		UK	ND	2-SEG
101	C	3400	2X2O-4X3C		UK	ND	2-SEG
102	C	3000	2X2O-4X3C		UK	ND	2-SEG
103	C	2900	2X1C-4X3C	3.7	UK	ER	2-SEG
104	C	2900	2X1C-4X3C	3.7	UK	ER	2-SEG
105	C	1900	2X1C-4X3C	2.7	UK	ER	2-SEG
106	C	2900	2X2O-4X3C	2.5	UK	ER	2-SEG
107	C	1950	2X2O-4X3C	1.8	UK	ER	2-SEG
108	C	1900	2X1O-4X3C	1.6	UK	ER	2-SEG
109	C	1900	2X1O-4X3C	1.2	UK	ND	2-SEG
110	C	2400	2X1O-4X3C	1.5	UK	ND	2-SEG
111	C	1400	2X1C-4X3C	0.9	UK	ND	2-SEG
112	C	1400	2X1C-4X3C	0.9	UK	ND	2-SEG
113	C	1400	2X1O-4X3C	1.3	UK	ND	2-SEG
114	C	1800	2X1O-4X3C	2.7	UK	ND	2-SEG
115	C	1800	2X1O-4X3C	2.7	UK	ND	2-SEG
116	C	1800	2X1O-4X3C	2.7	120	ND	2-SEG
117	C	1800	2X1O-4X3C	2.7	110	SC	2-SEG
118	C	1800	2X1O-4X3C	2.7	90	SC	2-SEG
119	C	2250	2X1O-4X3C	4.5	45	Q/ND	2-SEG
120	C	2250	2X1C-4X3C	4.5	105	SC	2-SEG
121	C	2900	2X1C-4X3C	3.75	153	ER	2-SEG
122	C	2900	2X1C-4X3C	3.75	150	ER	2-SEG
123	C	2900	2X1C-4X3C	3.75	130	ER	2-SEG
124	C	2200	2X1C-4X3C	3	142	ER	2-SEG
125	C	2900	2X1C-4X3C	3.75	157	ST	2-SEG

126	C	2200	2X1C-4X3C	3	130	ER	2-SEG
127	C	2200	2X1C-4X3C	3	171	ER	2-SEG
128	C	2650	2X1C-4X3C	3.5	150	ER	2-SEG
129	C	2650	2X1C-4X3C	3.5	146	ER	2-SEG
130	C	2650	2X1C-4X3C	3.5	123	ER	ST
131	C	3150	2X1C-4X3C	4	141	ER	2-SEG
132	C	3150	2X1C-4X3C	4	132	ER	2-SEG
133	C	3450	2X1C-4X3C	4.46	103	ER	2-SEG
134	C	3450	2X1C-4X3C	4.46	140	ER	2-SEG
135	C	2800	NA		150	ND	2-SEG
136	C	2800	NA		170	ND	2-SEG
137	C	1400	NA	2.2	154	ER	2-SEG
138	C	1800	NA	2.7	149	ND	2-SEG
139	C	1800	NA	2.7	130	ER	2-SEG
140	C	1600	NA	2.43	126	ND	2-SEG
141	C	1600	NA	2.43	114	ER	2-SEG
142	C	3400	NA		170	ER	2-SEG
143	C	1400	NA	2.2	160	H	2-SEG
144	C	1800	NA	2.7	150	H	2-SEG
145	C	2200	NA	3.2	140	H	2-SEG
146	C	2600	NA	3.7	130	H	2-SEG
147	C	3000	NA	4.2	120	H	2-SEG
148	C	3400	NA	4.9	110	H	2-SEG
149	C	UK	NA		19	SC	2-SEG
150	C	UK	NA		UK	SC	2-SEG
151	C	UK	NA		138	SC	2-SEG
152	C	3430	NA	35	40	ST	
153	C	3430	NA	35	30	ST	
154	C	3430	NA	35	0	ST	
155	C	3430	NA	35	0	ST/ND	
156	C	3400	NA		0	SST/Q	4-SEG
157	C	3400	NA		0	SST/Q	4-SEG
158	C	3400	NA		0	SST/Q	4-SEG
159	C	3400	NA		0	LST/Q	4-SEG
160	C	3400	NA		0	LST	4-SEG
161	C	3400	NA		0	SST/Q	3-SEG
162	C	3400	NA		0	SST/ND	3-SEG
163	C	1800	NA		133	SST	2-SEG
164	C	1800	NA		115	LST	2-SEG
165	C	1800	NA		99	LST	2-SEG
166	C	1400	NA		0	LST	2-SEG
167	C	1400	NA		0	LST	2-SEG
168	C	2200	NA		0	LST	2-SEG
169	C	2200	NA		0	LST	2-SEG
170	C	2600	NA		0	LST/ND/Q	2-SEG
171	C	2600	NA		0	LST	2-SEG
172	C	2600	NA		0	LST	2-SEG
173	C	3000	NA		0	LST	2-SEG
174		3000	NA		0	LST/ND/Q	2-SEG
175		3000	NA		0	LST/ND	2-SEG

176	3000	NA	0	LST	2-SEG
177	3400	NA	0	LST/ND	2-SEG
178	3400	NA	0	LST/ND	2-SEG

Appendix 2 - Field Data

Figure A.1 penetration rate vs depth collected with conventional probe, at Site 1 - Isabel breach.

Figure A.2 Isabel breach penetration rate vs depth collected with shrouded probe.

Figure A.3 Isabel breach penetration rate vs depth collected at site 2 with the conventional probe.

Figure A.4 Results from penetration vs depth measurements collected with the shrouded probe at site 2 with the pump at 1800 rpm.

Figure A.5 Results from penetration vs depth measurements collected from the conventional probe at 1800 rpm.

Figure A.6 Penetration rate vs depth at site 3 using the shrouded probe with the pump at 1800 rpm.

Figure A.7 Penetration rate vs depth using shrouded probe at site 3 with pump at 1800 rpm.