The Effects of Converting Surface Water into Groundwater on the Stability of the Beach

by

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Abstract

As barrier islands become more developed, an increase in the area of impervious surfaces and reduction of stormwater infiltration occurs. Stormwater drainage systems are installed to control the runoff and have been designed to discharge on local beaches. However, it is apparent that the outfalls affect the natural processes of the beaches by creating scour channels to the ocean.

A percolation riser is intended to drain as much stormwater as possible downward into the beach berm converting the surface water into groundwater. The purpose of this work is to determine, by scaled physical modeling, whether groundwater flow from a percolation riser installed on the beach affects the stability of the beach. An existing percolation riser is being modeled using a 9.05 meter wave tank located in the Surf Mechanics Laboratory at Florida Institute of Technology. Modeling this experiment requires scaling of the beach profile, the percolation riser, sediment, wave period and height, and permeability.

Case 1 (control) is performed without any flow into the percolation riser, and Cases 2 through 5 are performed over a range of flow rates (3.22 liters/min, 7.38 liters/min, 10.79 liters/min, and 12.95 liters/min) through the percolation riser. The beach is constructed to the same shape before each case. Wave conditions representative of a typical Nor'easter are generated until the beach reaches an equilibrium profile. The initial and final profiles of each case are recorded. Cases 2 through 5 are compared to Case 1 to determine if the stability of the beach has been affected by a constant flow rate through the percolation riser.

The results of the model testing verify that the equilibrium beach profile that results seaward of a percolation riser is affected by converting surface water into groundwater. The nearshore slopes become more gradual as the flow rate increases. The induced flow raises the water table within the beach berm, increasing the potential for foreshore erosion. It was also observed that shoreline recession increases with

groundwater flow rate. In addition, as the flow rate increases, the breakpoint moves offshore. This widening of the surf zone likely results to maintain constant energy dissipation per unit volume in the presence of a more gradual slope in the beach profile.

1.0 Introduction

As barrier islands become more developed, an increase in the area of impervious surfaces results from the construction of roads, residences, and businesses. These impervious surfaces do not allow stormwater to infiltrate into the soil, which results in point source discharges of large volumes of runoff or surface water. Installation of a stormwater drainage system becomes necessary to control flooding. These systems often discharge onto local beaches through stormwater outfalls - outlets where discharge occurs. Outfalls discharging onto beaches affect the natural processes of the beaches by creating scour channels to the ocean (Smith, 1997).

In the City of Indialantic, Florida, there are several stormwater outfalls located on the beach. Three of these have been modified in an attempt to reduce the amount of erosion of the surrounding area. Percolation risers were installed on these outfalls. A percolation riser is intended to drain as much of the stormwater as possible downward into the beach berm converting the surface water into groundwater. The outfall located at the end of Watson Drive in Indialantic, Florida, was modeled in this research, because the measurements made by Galli (2002) at this location suggest localized changes to the beach profile in the vicinity of enhanced groundwater flow from an outfall.

The percolation riser at Watson Drive, the design of which is shown in Figure 1, is a 2.74 meter long vertical pipe 1.52 meters in diameter. The pipe extends approximately 1.5-1.75 meters into the sand. The horizontal overflow pipe connected to the vertical percolation pipe discharges directly onto the beach and is 2.44 meters above the mean sea level according to Galli's (2002) work. The portion of the pipe that is buried in the sand has numerous 1 inch (2.54 cm) holes that allow the water to percolate into the surrounding berm. The bottom of the pipe is open ended but is covered with filter cloth (Olsen and Associates Inc., 2002).

1.1 Background

In 2002, a beach nourishment project took place along the Indialantic coastline. After completion of the nourishment, Galli (2002) monitored the outfall located at the end of Watson Drive. Galli intended to determine the impact of the outfall on the natural equilibrium of the beach. One of Galli's desired observations was the scour channel created after a rainfall; yet, the scour channel was not produced. The percolation riser had not been installed at this time. Due to an elevated berm shoreward of the outfall from the nourishment, the stormwater produced a large pool of standing water in front of the outfall as well as north and south along the dune. The standing water ultimately percolated into the beach berm.

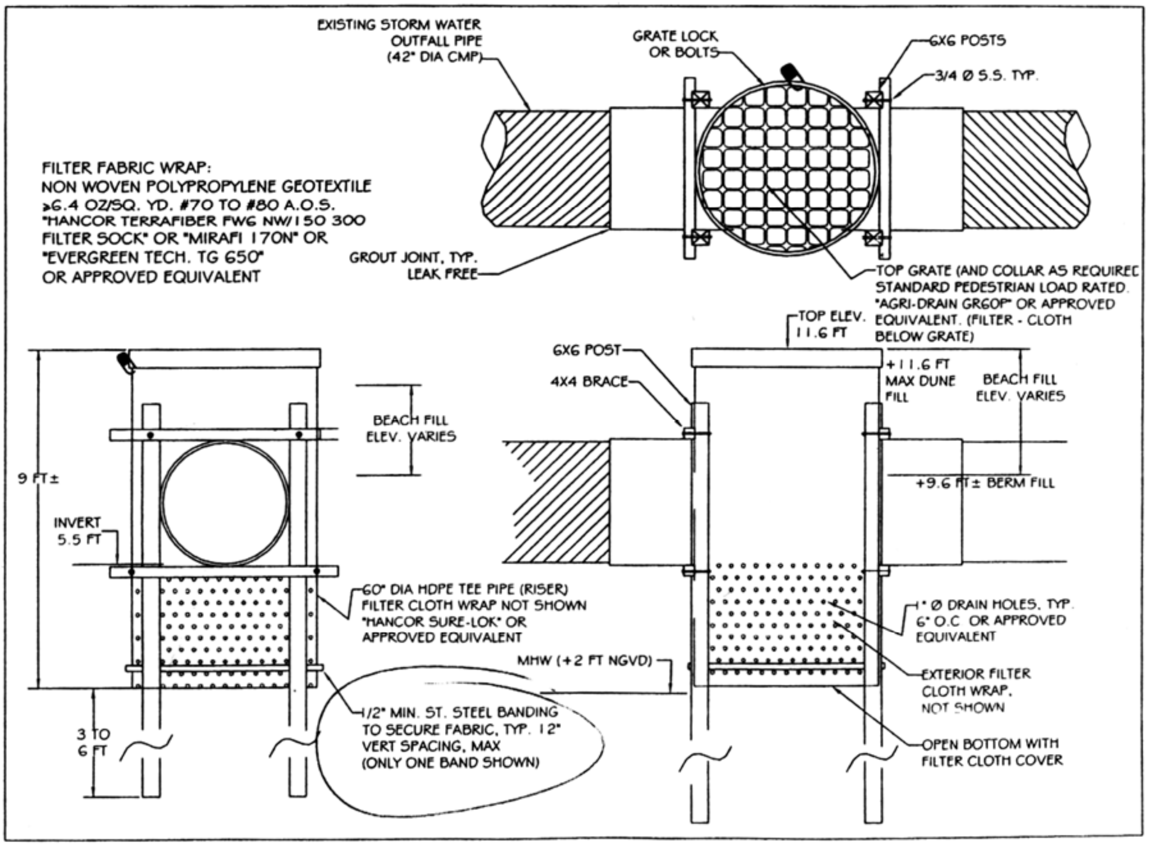

Figure 1: Diagram of Watson Drive percolation riser, from Olsen Associates Inc., 2002.

Galli surveyed the beach profile at three locations; 45.72 meters north of the outfall, 45.72 meters south of the outfall, and directly in front of the outfall, hereafter referred to as north, south and site respectively. The additional profiles taken to the north and south allow changes occurring seaward of the outfall to be interpreted relative to the changes occurring updrift and downdrift of the site. Three-dimensional graphical representations of the beach surveys can be seen in Figure 2. On October 14th and 15th of Galli's study, a heavy rain occurred. The results of Galli's studies showed that the nearshore submerged beach slope at the outfall site after a storm was gentler than before the event, which is also seen in Figure 2. Prior to the storm, the north and site profiles have similar slopes with the south profile having the steepest slope of the three. Four days after the storm event occurred, beach profiles were again measured. These show that the steepness of the north and south profiles increased whereas the site profile became gentler. The study concluded that the gentler slope of the nearshore submerged beach resulted from the increased outflow from the seabed. No further storm events occurred during Galli's study, so after several weeks of recovery the north and site profiles have similar slopes with the south profile being the steepest of the three.

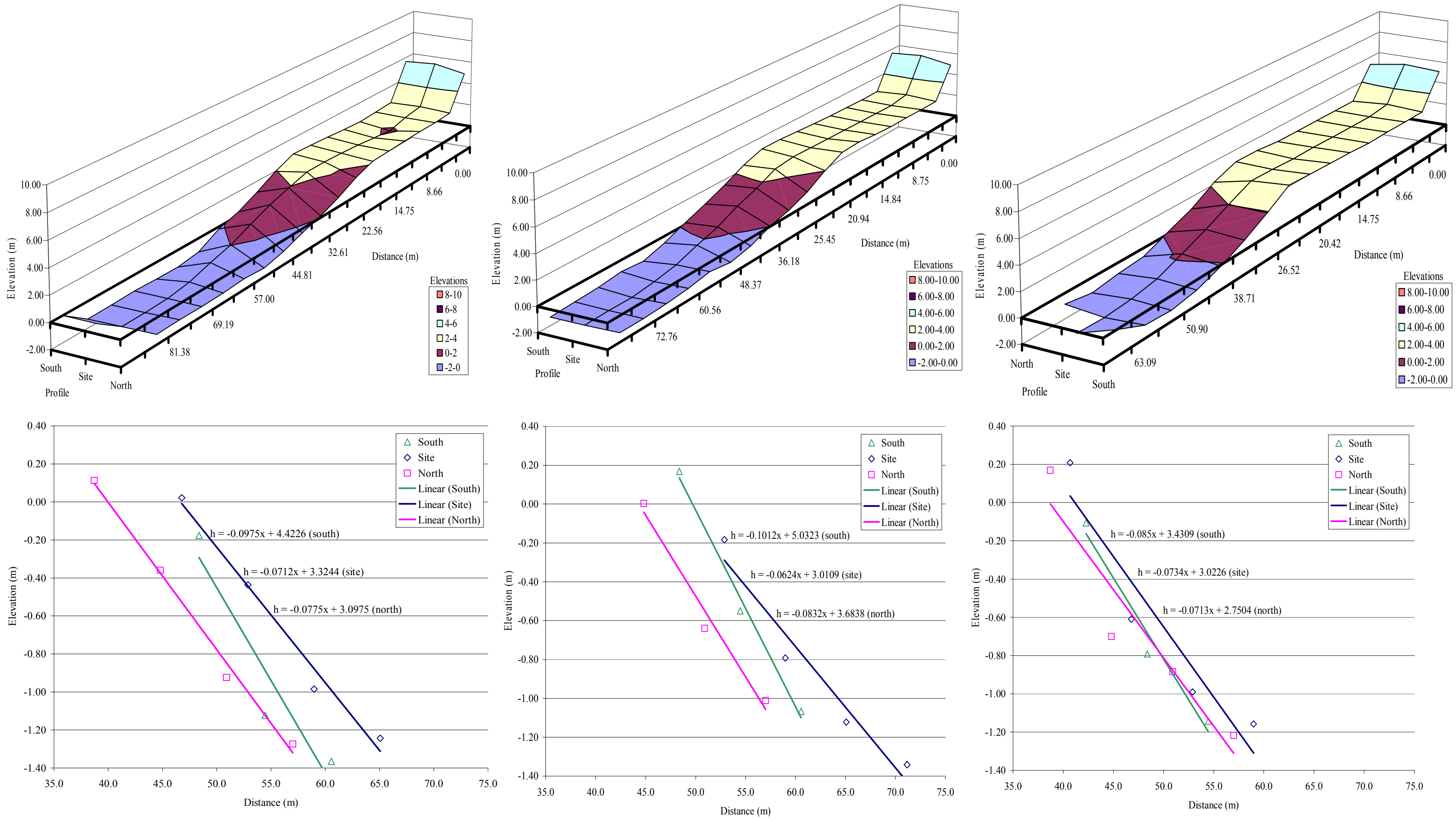

Figure 2: (top row) North, site, and south profiles from 9/19, 10/19, and 11/12/2002. Zero on the vertical axis is sea-level; zero on the horizontal axis is a temporary benchmark near the crest of the dune. (bottom row) Profile data truncated to show submerged nearshore slopes. Linear trend lines have been added. Unpublished data from Galli, 2002.

The purpose of this work is to use scaled physical modeling to assess the extent of the change of the beach profile due to groundwater flow from a percolation riser installed on the beach. An existing percolation riser will be modeled in a 9.05 meter wave tank located in the Surf Mechanics Laboratory at Florida Institute of Technology. Although the model is based on an existing field installation, no field testing at the site will be performed or verifications made between model and field. Optimizing the design of the outfall is also beyond the scope of this research.

Modeling this experiment requires scaling of the beach profile, the percolation riser, sediment, wave period and height, and permeability. Four different flow rates will be tested to conclude whether the stability of the beach is affected. Case 1 (control) is performed without any flow into the percolation riser whereas Cases 2 through 5 are performed with the different flow rates (3.22 liters/min, 7.38 liters/min, 10.79 liters/min, and 12.95 liters/min) through the percolation riser. The beach is constructed to the same shape before each case. Generated waves are run until the beach reaches its equilibrium profile. The initial and final profiles of each case are recorded. Cases 2 through 5 will be compared to Case 1 to determine if the stability of the beach has been affected by a constant flow rate through the percolation riser.

Based on Galli's measured effects on the beach profile at Watson Drive, it was hypothesized that the percolation risers would cause the beach profile to destabilize, specifically to adopt a new equilibrium profile of shallower slope. With an increase in flow rate, the extent of this change would be more evident. The extent of the change will be examined.

2.0 Methods

2.1 System Description

The wave tank in which the model is tested is 9.05 meters long, 0.91 meters high, and 0.57 meters wide. The waves are generated by a piston type wave generator. The piston generator is a plastic wave board supported by a carriage that moves along an aluminum track attached to the wave tank. The wave board is driven by an electric motor, feedback controlled to produce sinusoidal motion. The wave generator does not remove the reflected wave component. Due to the piston stroke and the wave absorber behind the wave board, the last 1.22 meters of the tank are not used in designing the model. The wave absorber consists of a screen mesh bag filled with floating polypropylene pellets, and is placed behind the wave board.

2.2 Model scaling

The experimental model represents the prototype beach from the most shoreward to the most seaward points affected by the processes being studied. At this location, the most seaward point is taken to be the depth at which the historical beach profiles coincide. The most shoreward point is taken to be the toe of the dune.

To begin the modeling procedure, beach profiles taken at monument R-121 were obtained from the Florida Department of Environmental Protection website, ranging from years 1972-2003 using the vertical reference datum NAVD88 (North American Vertical Datum) (Florida DEP, 2005). Monument R-121 was chosen because it was the closest benchmark to the Watson outfall, located approximately 150 meters west of it. As seen in Figure 3, the depth at which the historical profiles coincide occurs at approximately 3.33 meters depth. While this is recognized to be at a depth smaller than the expected depth of closure, the historical profiles suggest that at this location, no significant changes in the profile are occurring beyond this depth.

Mean sea level (MSL) was determined as -0.55 meters NAVD from the Sebastian Inlet tide gage (NGS, 2005). Sebastian Inlet is located approximately 29 kilometers south of Watson Drive. MSL was subtracted from the profile elevations and profiles were aligned by shoreline position. With the seaward extent of profile change occurring at a depth of 3.33 meters, the largest distance from the shoreline to this seaward extent in any historical profile is roughly 175 meters from the shoreline.

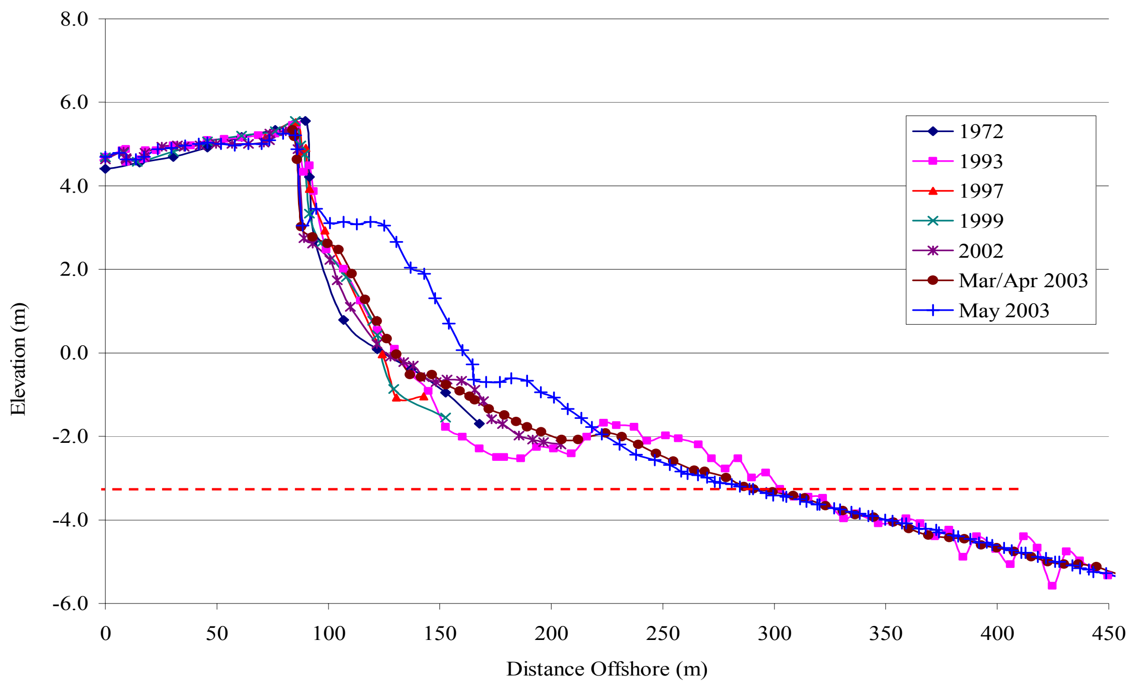

Figure 3: Historical profiles at R-121 indicate that the seaward extent of profile change occurs at a depth of 3.33 meters, indicated by the dashed line. Only a subsection of the profile data is shown.

Based on the historical beach profile data, the distance from the toe of the dune to MSL at R-121 is approximately 31 meters (Florida DEP, 2005). The elevation from MSL to the toe of the dune is roughly 2.5 meters, and MSL to the depth seaward extent of profile change is 3.33 meters, producing an elevation from the toe of the dune to the seaward extent of profile change as 5.83 meters. The distance from the toe of the dune to the seaward extent of profile change is approximately 206 meters, and the available length in the wave tank is 7.83 meters; therefore, a length scale ratio of 1:26 was chosen. The wave tank is 9.05 meters long, but due to the piston motion of the wave board, the last 1.22 meters of the tank will not be considered in the scaling, giving an available length of 7.83 meters. To have an undistorted model, the horizontal length scale and the vertical scale must be the same, restricting the vertical scale ratio to 1:26 as well. An undistorted model is used because there is not a way to model a distorted wave. The ease of extrapolating model flow rates to prototype flow rates is another reason for choosing an undistorted model. The percolation riser at Watson Drive is 3.05 meters seaward from the toe of the dune. In the model, the riser is centered between the walls of the tank and positioned 13 centimeters from the end of the wave tank. The portion of the beach from MSL to the dune is modeled as a rectangular block, which is conservative. A summary of the prototype and model lengths and elevations are shown in Figure 4.

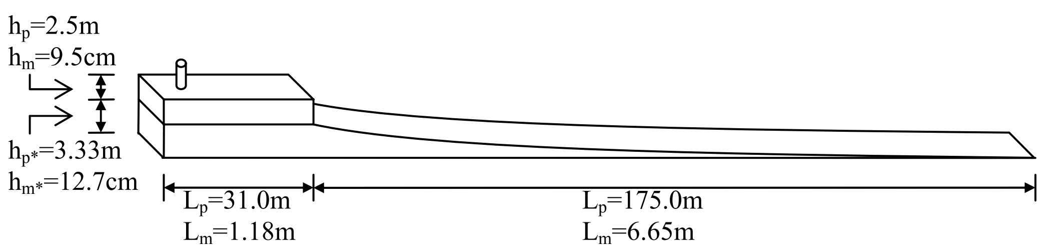

Figure 4: Model and prototype lengths and elevations. MSL is located at the base of the rectangular block. The subscripts *m* and *p* refer to model and prototype, and the subscript asterisk refers to the seaward extent of profile change. The vertical column is the percolation riser.

In movable bed models, the sediment will respond as bedload (moving along the bed), a suspended load (lifted into the water column), or a combination of the two. Perfect similitude for a bedload model to exist would require that the grain size Reynolds number, densimetric Froude number, relative sediment density, relative length, and relative fall speed be equal to one. However, this is not possible, and several similitude models have been developed where two or more scale ratios are not maintained. Hughes describes four models of possible similitude, in which none attempt to maintain the relative fall speed parameter. The more parameters that are fulfilled, the better the model will represent the prototype. Perfect similitude for a suspended load model to exist would require that the grain size Reynolds number, mobility number, relative sediment density, relative length, and relative fall speed equal to one. Again, perfect similitude is not possible. Suspended load models can be approached with or without fall speed dependency. Without fall speed dependency, the focus is on similar beach slopes, and with fall speed dependency, the focus is obviously the sediment fall speed (Hughes, 1993).

Similitude for this research is a combination of the bedload lightweight model and the suspended load without fall speed dependency. In the lightweight model, the grain size Reynolds number and densimetric Froude number are satisfied, and the relative sediment density is constrained to $1.05 < \rho_s / \rho < 2.65$ (Hughes, 1993).

When working with models that involve sand motion, problems occur when trying to scale the sand size. Effects such as cohesion become a factor if the model sand size is too small; less than 0.08 millimeters (Hughes, 1993). In the case of modeling flow rates, the cohesive characteristics do not allow flow, and it changes the physics of the problem, so other methods of modeling sand are needed.

Noda developed a graphical representation of model relationships based on horizontal scales, vertical scales, specific gravities, and material size ratios. The representation was compiled from over 130 tests on 14 different materials and 22 different grain sizes (Noda, 1972). According to Noda, when modeling a two-dimension beach profile, horizontal scale, vertical scale, sediment size, and sediment relative specific weight are the four parameters that need to be specified. Using Noda's graphical representation and choosing two of the four parameters, the other two parameters are then defined as seen in Figure 5. From this figure, if an undistorted model is desired (horizontal and vertical length ratio is 1:26), it is seen that Plexiglas (specific gravity of 1.16) with a material size ratio of 12 would be ideal for the experimental model. A material size ratio of 12 and a prototype sediment grain diameter of 0.30 millimeters yield a model grain diameter of 3.6 millimeters. Due to cost and availability, the actual material used in the research was ACRYREX® CM 211 pellets with a specific gravity of 1.19. The average diameter of the pellets was verified as 3.6 millimeters using calipers.

The wave period is scaled according to the Froude model law, which states that for the model and prototype the Froude number must be the same.

$$\left(\frac{U}{\sqrt{gh}}\right)_p = \left(\frac{U}{\sqrt{gh}}\right)_m \qquad 2.1$$

U is the wave related velocity *($U=l/t$),* g is the acceleration of gravity, and h is the characteristic depth (Dean and Dalrymple, 2002). Therefore,

$$t_r = \frac{l_r}{\sqrt{h_r}} \qquad 2.2$$

where r is the ratio of model to prototype. A prototype wave with a 10 second period using Froude modeling would thus require a model wave period of 1.96 seconds. The motor on the wave tank is set to operate at 175 RPM. Taking 175 RPM with a 6:1 gear ratio yields a period of 2.06 seconds; therefore, the period for conduct of this research is 2.06 seconds. Using Froude modeling, a prototype wave height of 2.2 meters would require a model wave height of 8.5 centimeters.

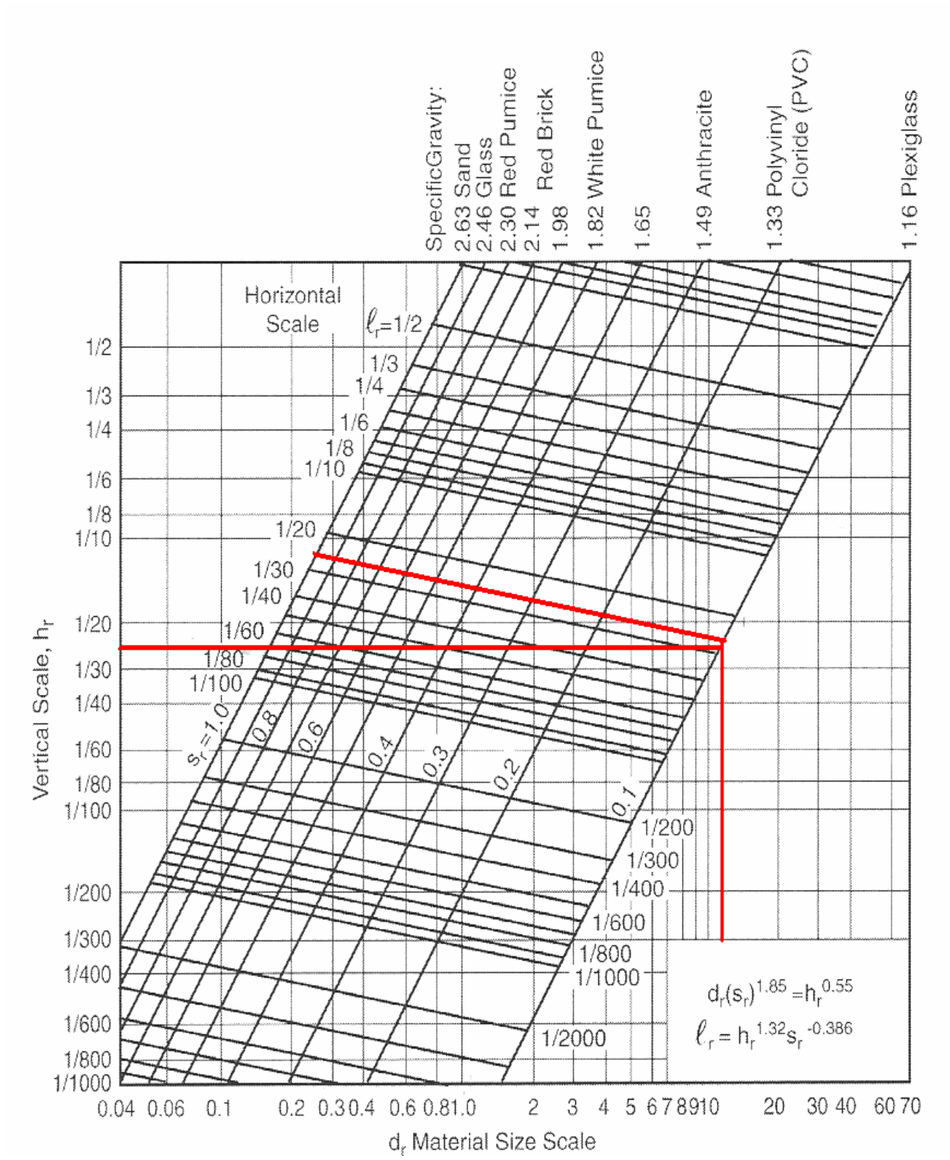

Figure 5: Graphical representation of Noda's model relationships, adapted from Dean and Dalrymple, 2002.

The Attwood V1250 bilge pump was used at 12v, 10v, and 8v for three of the flow rates. The Attwood V500 bilge pump was used at 5.5v (referred to as 6v from here on for simplicity) for the fourth flow rate. The flow rates were determined by finding the time required to fill an 18.9 liter (5 gallon) bucket at each voltage mentioned above, and then converting to liters per minute. Three trials of each voltage were conducted, and the average of three trials was used as the flow rate for that voltage. The flow rates were established as 3.22 liters/min, 7.38 liters/min, 10.79 liters/min, and 12.95 liters/min for voltages 6, 8, 10, and 12 respectively.

2.3 Testing

At the start of each trial, the ACRYREX® CM 211 pellets are placed at the opposite end of the wave maker and constructed to the initial shape as per Figure 6. Both wave gauges are calibrated and set in place. The gauges are calibrated at the start of each day of testing to reduce the effects of calibration drift. The voltage to the bilge pump is adjusted to achieve the flow rate being tested. The initial profile with the location of MSL was drawn onto the tank window panes. The initial profile is recorded every 5 centimeters along the tank starting at -95 centimeters to 95 centimeters. Profiles are taken on both right and left sides of the tank. Once the wavemaker and bilge pump are started, the initial location of the breakpoint is recorded, and the head difference within the percolation riser is measured and recorded.

Each test is run until the beach reaches an equilibrium profile. Equilibrium has been reached when the profile no longer experiences any morphological changes. Even though equilibrium occurs before 30 minutes, each test is recorded for approximately 30 minutes, at which time the wavemaker and bilge pump are stopped. Once the water in the tank is still, the DVD recorder and wave gauges are turned off.

The final profile with the location of MSL is recorded. The final profile is recorded every 5 centimeters along the tank starting at -95 centimeters to 105 centimeters, where the increments then change to 10 centimeters ending just around 315 centimeters. The extent the material in the profile moves offshore varies a few centimeters between each case so the maximum seaward extent of the pellets is recorded. Profiles are taken on both right and left sides of the tank. Cases 2 through 5 are compared to Case 1 to determine if the stability of the beach has been affected by a constant flow rate through the percolation riser.

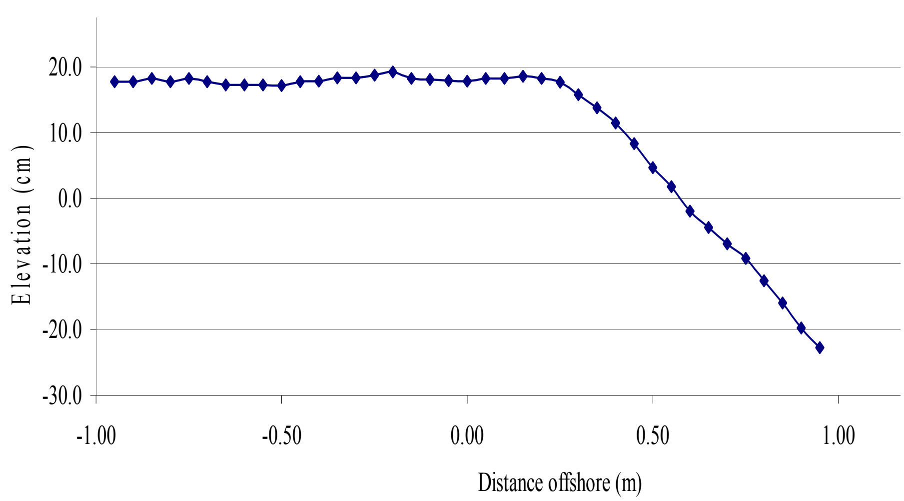

Figure 6: Shape of initial model beach profile.

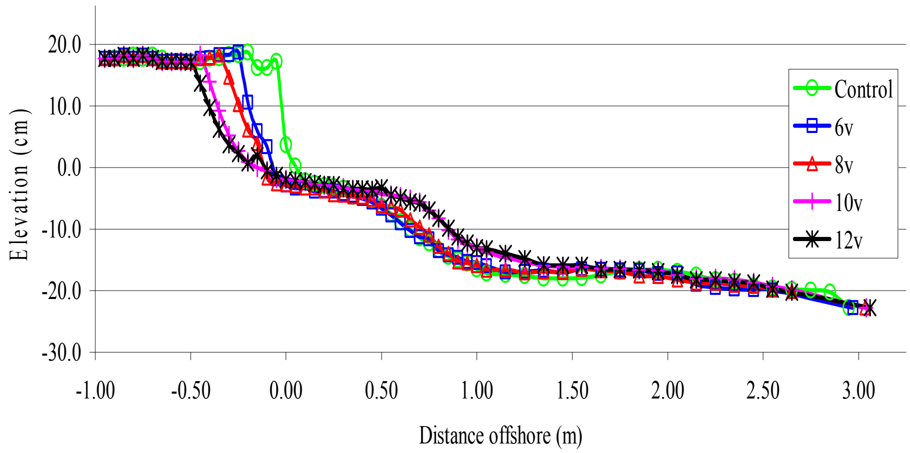

Figure 7: Trial 1-final model profiles for the right side of the wave tank.

3.0 Results

All cases began with the same initial beach shape, shown in Figure 6. The final profile for each case was plotted for both the right and left sides of the wave tank. Figure 7 shows the final profiles from the right side at the end of Trial 1.In Figure 8, the final equilibrium nearshore slopes were plotted from MSL to 0.5 meters. For better comparison to Galli's (2002) study, the data is only plotted to 0.5 meters because Galli's (2002) information is limited to only the nearshore as well. Figure 8 shows, overall, as the flow rate increases the nearshore slope decreases creating a gentler slope. The gentler slope of the berm suggests that the beach was destabilized in this area by the increased outflow from the seabed.

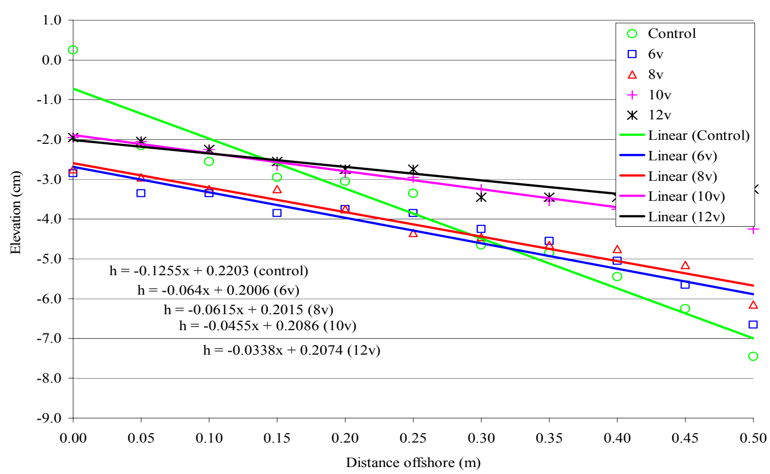

Figure 8: Nearshore slopes for Trial 1-right side showing destabilization of the beach. Data has been truncated, and linear trendlines have been added.

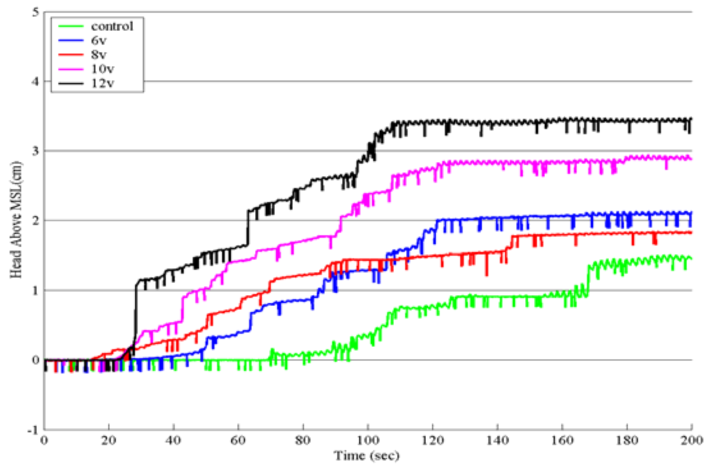

Figure 9: Water level within beach berm recorded from wave gauge for Trial 3. Data has been truncated, and a 16 point running average has been applied.

A capacitance wave gauge was used to monitor the water level within the beach berm. The wave gauge was calibrated before each day of testing to minimize the effects of sensor drift. The gauge was 10 centimeters to the right of center of the percolation system. The data from Trial 3 is represented in Figure 9. The extent of shoreline recession was found from the initial and final MSL positions. The recessions of Cases 2 through 5 were relative to Case 1. Figure 10 shows that the amount of shoreline recession increased as the flow rate increased.

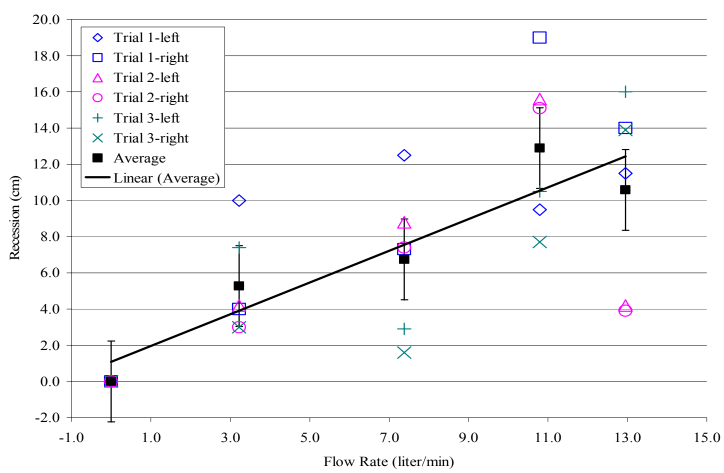

Figure 10: Shoreline recession increased as the flow rate increased. A standard error and linear trendline have been applied to the average at each flow rate.

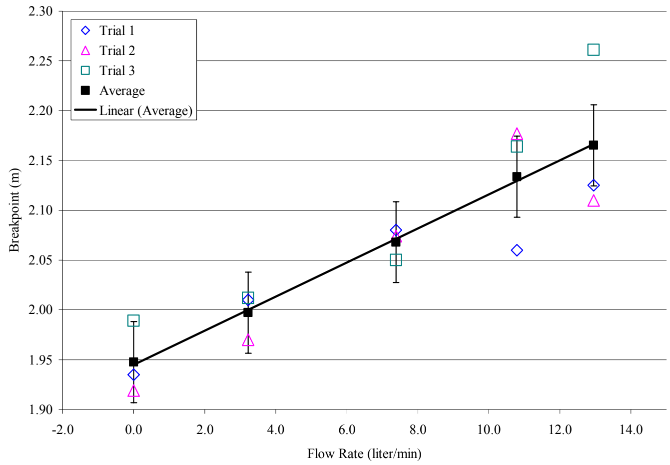

Figure 11: As the flow rate increased the surf zone widened. A standard error and linear trendline have been applied to the average.

The point where the waves were breaking was marked on the side of the wave tank once the breakpoint reached an equilibrium position. It was observed that as the flow rate increased, the breakpoint moved further offshore; widening the surf zone. This is shown in Figure 11.

Based on the model flow rates, the prototype head difference between the water level in the percolation riser and MSL is calculated. Determining the head difference helps confirm whether the flow rates used in the model are likely to occur at the prototype. Darcy's Law written in terms of flow rate is (Holtz and Kovacs, 2003)

$$q = Ak\frac{dh}{dl} \qquad 3.1$$

From Froude modeling, the permeability for the model scales as follows

$$\left(\frac{dh}{dl}\right)_{\text{model}} = \left(\frac{dh}{dl}\right)_{\text{prototype}} \qquad 3.2$$

The prototype head difference between the water level in the percolation riser and MSL is then

$$h_p = \frac{q_m L_p}{k_m A_m} \qquad 3.3$$

4.0 Discussion

Many of the model results warrant further explanation and are addressed in this section, such as the validity of the modeling in deeper depths with the use of such a highly permeable seabed. The increased erosion potential of the foreshore, the change in nearshore profile slope, recession of the shoreline, and widening of the surf zone are also discussed.

The pellet seabed used in this model is highly permeable. At the more seaward locations in the profile, groundwater flow is confined within a thin layer of pellets. This confinement results due to the presence of an impervious glass bottom in the tank. If the thickness of the bottom layer of pellets were increased, then this restriction to groundwater flow in this region would be removed. However, this would be at the expense of the linear wave theory. Linear wave theory assumes irrational flow (no friction) and an impermeable bottom. With a larger depth of pellets, the wave motion would penetrate deeper into the permeable bottom (violating the assumption of an impermeable bottom) and would further enhance energy dissipation, which in turn affects the final equilibrium profile. Since the equilibrium profile equation is based on linear wave theory, the relation no longer holds true. This is a recognized limitation of model testing with a highly permeable bed material.

The capacitance wave gauge was able to detect a difference in the water level within the beach berm. Before a test was started, the water level in the wave tank was at 21.0 centimeters, and after the test, the water level in the wave tank was at 22.75 centimeters. The increase in water level is due to the water displacement by the pellets forming the equilibrium profile and to the amount of water required to fill the higher groundwater volume. The 12.95 liters/min flow rate had the largest change in water level where the water level in the berm was 2.5 centimeters above the mean open water level in the tank. A higher water level in the beach berm compared to MSL drives the groundwater flow and results in a higher potential for the foreshore to erode.

Gentler equilibrium profile slopes in the nearshore resulted with increased flow rates. These results agree with those of Galli's study. The data from Galli's study only extended to the nearshore, which is one reason for limiting the slopes of the model research to the nearshore. The confinement of the groundwater flow by the impermeable glass bottom offshore is another reason.

A change was noticed in the shoreline position. Overall, shoreline recession increased as the flow rate increased. Being a wave channel study, the longshore effects could not be assessed. The volume of pellets removed from the berm contributed to the greater seaward extent of the profiles. As the flow rate increased, the further the seaward extent of the nearshore profile. Having the nearshore profile extend further offshore then caused the waves to break further offshore. The more the flow rates increased, the further offshore the waves would break. This in the end resulted in the widening of the surf zone. Stated in other terms, the milder nearshore slope reduces the overall water volume for energy dissipation, pushing the breakpoint further offshore to maintain the equilibrium energy dissipation per unit volume.

The flow rates at the prototype are unknown at this time; however, the maximum head difference is known. To determine whether the flow rates used in the model were likely to occur at the prototype, the model flow rates were converted into the prototype head difference instead of prototype flow rate. The outcome of the conversion showed that the 3.22 liters/min flow rate would relate to a realistic value of 2.83 meters head difference between the water level in the percolation riser and MSL. The maximum head difference is limited to 2.44 meters at the prototype outfall by overflow. For the highest flow rate tested - 12.95 liters/min, the resulting 11.39 meter head difference appears to be excessive.

5.0 Conclusions

The purpose of this work was to use scaled physical modeling to assess the extent of the change of the beach profile due to groundwater flow from a percolation riser installed on the beach. The nearshore beach slopes show that as the flow rate increased, the nearshore equilibrium profile slope decreased. With increasing flow rates, the water level within the beach berm was elevated above the sea level in the tank making the foreshore more susceptible to erosion. A receding shoreline and shallower nearshore slope was present due to the induced flow rates. Additionally, the surf zone widened with increasing groundwater flow rate.

This research has shown that the equilibrium beach profile in the vicinity of a storm water outfall is flow rate dependent. To minimize the effects on the beach profile, creation of more drainage systems would allow less flow per outfall. Moving the drainage system further from the shoreline would spread the influence of the outfall over a greater stretch of the beach in the longshore direction. Further testing should be conducted to assess longshore effects.

Noda's equations show that small changes to the composition of the material can change the scale of the modeled beach profile significantly (1972). In this research, the specific gravity of the material used was different from that desired by only 2.6%. This small difference resulted in a change in scale by almost a factor of two. The sensitivity to sediment composition demonstrated by Noda's relations suggests that these relations might be useful in assessing the affects of beach fill composition at prototype scale.

If increased groundwater outflow results in a wider surf zone, then does groundwater inflow to the seabed cause the surf zone to become narrower? Narrowing of the surf zone results in waves breaking closer to the shore, which might be undesirable in extreme wave conditions. This should be explored further, particularly in conjunction with technologies that intentionally remove groundwater as a mechanism of shoreline stabilization.

The results of the model testing verify that converting surface water into groundwater influences the equilibrium profile that results. The results of this research demonstrate that larger scale modeling or field testing is warranted, in order to provide better engineering design guidance to coastal engineers. The qualitative effects observed were in agreement with those seen earlier in Galli's field study. Further testing on a larger scale could validate the relation between model and prototype and result in more reliable quantitative results for design guidance.

Acknowledgements

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