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# ANCHORING STABILITY OF NEW SEAGRASS PLANTINGS

by

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## ABSTRACT


Seagrass beds are among the most desirable systems in our shallow coastal waters. Others are mangroves, mud flats, coral reefs, etc. Revegetation of areas damaged by development and in some cases natural events such as hurricanes should therefore be quite common.

Runners from healthy and well established beds are transplanted to the damaged areas after the bottom elevation has been restored. A spacing of 0.5 to 1 m between the transplanted runners is usually used. Runners may be attached to the bed by use of steel staples or other anchoring devices, and the roots pushed into the sand. Areas revegetated in this way exhibit a lower resistance to flow than heavily vegetated areas under similar flow conditions. Therefore, new beds are exposed to higher velocities and bed shear stresses than the established areas, often resulting in failures caused by disruption of the bond between plant runners and sand bed or simply by a massive erosion of the sand bed itself.

The paper discusses these processes and develops methods for quantification of the flow impact on the bed. Testing of live transplanted seagrasses (Halodule) in the Hydraulic Laboratory's research flume is discussed and a special model law for transfer of flume results to prototype conditions is developed.

Model tests referring to a case near Cudjoe Key in the Florida Keys (Niles channel restoration project) are described.

# INTRODUCTION

Newly transplanted seagrasses such as Halodule are exposed to higher erosive forces than well established beds due to their lower hydraulic roughness and thereby reduced resistance to flow. Failure may be caused by one of two phenomena, a general erosion of the sand around the transplants or flow induced motion of the transplanted runners resulting in disruption of the root/sand contact. A combination of the two is of course also possible. The general erosion of sand beds is considered first.

## Erosion of sand beds

Due to its importance in littoral processes, the question of when incipient motion of sand takes place has been treated in the literature quite frequently during the last couple of centuries. Two schools of thought have evolved, the critical velocity approach and the tractive force approach. Both are used today. However, the tractive force approach does seem to be the most attractive.

The critical velocity approach is based on a *critical velocity* at which a granular cohesionless bed material will begin to move. A formula for this velocity was - according to Forchheimer (1914) - proposed by Brahms in 1753. It is postulated to be proportional to the dry weight of a singular grain to the one sixth power corresponding to Sternbergs (1875) later formula stating that the critical bed velocity is proportional to the square root of the grain-size. Many similar formulas have been presented since then. They are discussed and referenced by Forchheimer in his 1914 book and by Graf (1971). The location of the critical reference velocity is not always well defined in the contributions. Sometimes it is a surface velocity, other times the vertically averaged velocity, the cross-sectional mean velocity or the bed velocity as used by Brahms and Sternberg.

In the early part of this century Fortier and Scobey (1926) published an extensive study on "Permissive Canal Velocities" that is giving such velocities in tabular form. Fortier and Scobey refer to the cross-sectional velocity.

Although Fortier and Scobey's tables still are used today by some engineers and coastal morphologists it is Hjulström's (1935) diagram, Fig. 1, that is used by most of the critical velocity proponents. This diagram which is based on numerous observations of incipient erosion in Swedish rivers gives the critical cross-sectional mean velocity, $v_m$, as a function of the grain-size, $d_e$.

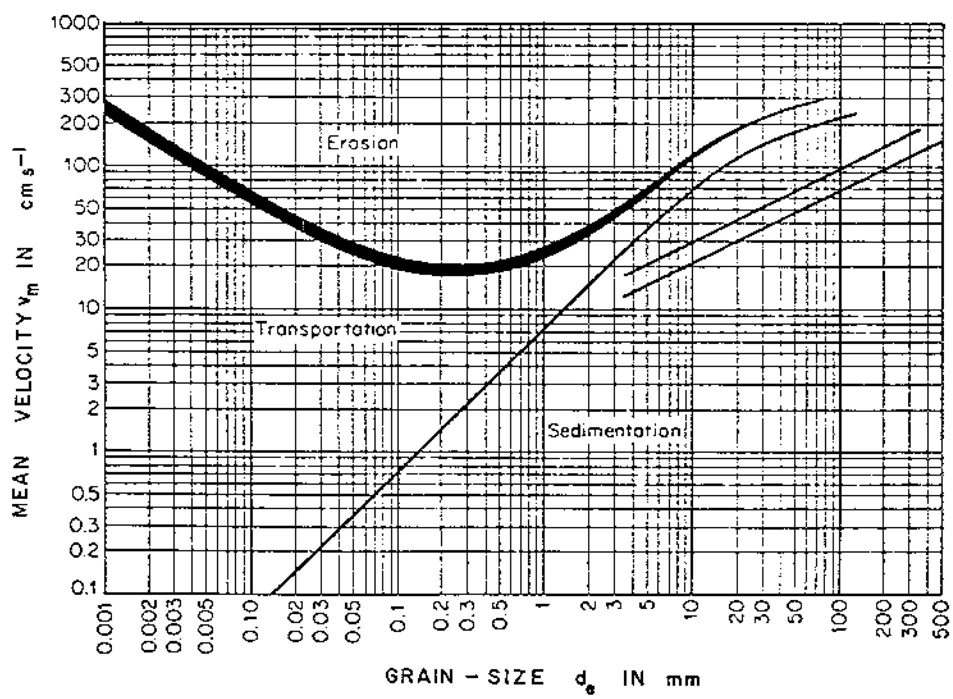


Figure 1. Hjulström's diagram for critical mean velocity as function of grain-size of bed material. Horizontal bed. Hjulström (1935)

The *critical tractive force* approach was initiated by du Buat (1786) (Forchheimer (1914)) at about the same time as Brahms did his pioneering work with critical velocities. Du Buat states that a bed material can withstand a certain bed shear stress, called the critical bed shear stress, beyond which erosion will take place. This critical shear stress is considered a property of the bed form and material while the bed shear stress it must be compared to of course is a flow property proportional to the local depth and slope of the energy grade line.

The best known results of critical tractive force research are those provided by Shields (1936) and given in his now famous diagram shown in Fig. 2. In Shield's diagram the entrainment function for a horizontal bed $E_h$ is defined as indicated on the ordinate axis where $\gamma_s$ = unit weight of the grain material, $\gamma$ = unit weight of water, $d_e$ = grain-size and $\tau_{cr.h}$ = the critical shear stress of a horizontal bed. The abscissa, the wall Reynolds number, is defined as the product of the friction velocity $v_f$ and grain-size $d_e$ divided by the kinematic viscosity $\nu$ of water. $v_f$ is a measure of the bed shear stress and defined as the square-root of the ratio of bed shear stress to density of water.

The critical bed shear stress seems to be the most rational measure for a bed's ability to resist scour. However, a majority of the formulas for this quantity are deterministic. Since both the turbulent flow that causes scour and the composition of the bed material are of a stochastic nature it should be expected that the formulas for the critical bed shear stress must be stochastic rather than deterministic and relate the critical shear stress to a probabilistic risk of erosion.

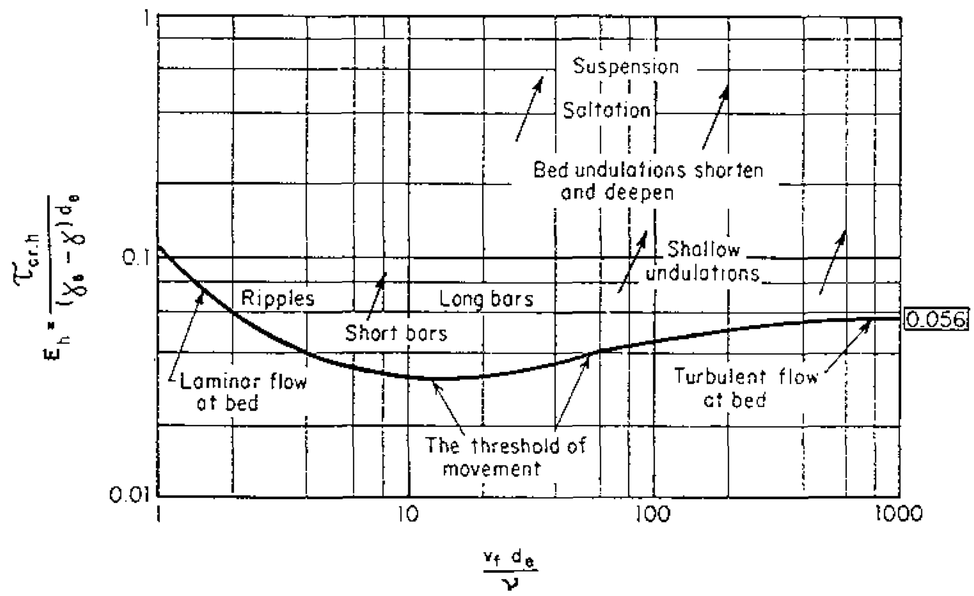


Figure 2. Shield's diagram for critical value of entrainment function as function of Wall Reynolds number. Horizontal bed. Shields (1936)

In the case of nonuniform deposits an effective grain-size, $d_e$, of the bed material may be used. This grain-size is defined as the particle size of a bed of a uniform material which will experience incipient motion at the same bed shear stress as the nonuniform material it represents. When the grain-size distribution of the nonuniform material is known the effective grain-size $d_e$ may be evaluated. According to Christensen (1969) this may be done by the integration

$$d_e = \frac{1}{\int_0^1 \frac{df}{d_s}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

where f = fraction of weight of the bed material which is smaller than the size $d_s$.

## CRITICAL SHEAR STRESS OF A SAND BED

Erosion of a sandy horizontal ocean bed will take place when the local bed shear stress exceeds the critical bed shear stress $\tau_{cr.h}$, i.e. when

$$\tau_{cr.h} = \gamma dS > \tau_{cr.h} = E_h \ (\gamma_s - \gamma) \ d_e \quad \ldots\ldots\ldots\ldots (2)$$

where d = the local water depth and S = the local slope of the energy grade line. As indicated for instance by Streeter and Wylie (1975) S may be related to the mean flow velocity $v_m$ and depth by the classic Manning formula

$$S = \frac{v_m^2 \ n^2}{d^{4/3}} \quad \text{(S.I. units)} \ldots\ldots\ldots\ldots (3)$$

where Mannings n may be found from the roughness k of the bed by

$$\frac{1}{n} = \frac{25.8}{k^{1/6}} \ \text{(S.I. units)} \ldots\ldots\ldots\ldots (4)$$

according to Christensen (1983a).

At values of the wall Reynolds number in excess of about 500, Figure 2 indicates that $E_h$ is constant and equal to 0.056. Since erosion is caused by turbulent flow of a highly stochastic nature it does not seem likely that $E_h$ should be the same constant for all cases. Dependency on the probability of erosion appears to be more realistic. This was shown by Christensen (1983b).

The stochastic formula for the entrainment function found in that study is plotted in Figure 3 where the indicates values of shape factors $\beta_1$, $\beta_2$ and the angle of repose $\phi$ are characteristic for sandy beds in Florida's coastal waters.

A *longitudinal* bed slope in the direction of flow does have an influence on $E_h$, but this influence is usually negligible in Florida's coastal waters.

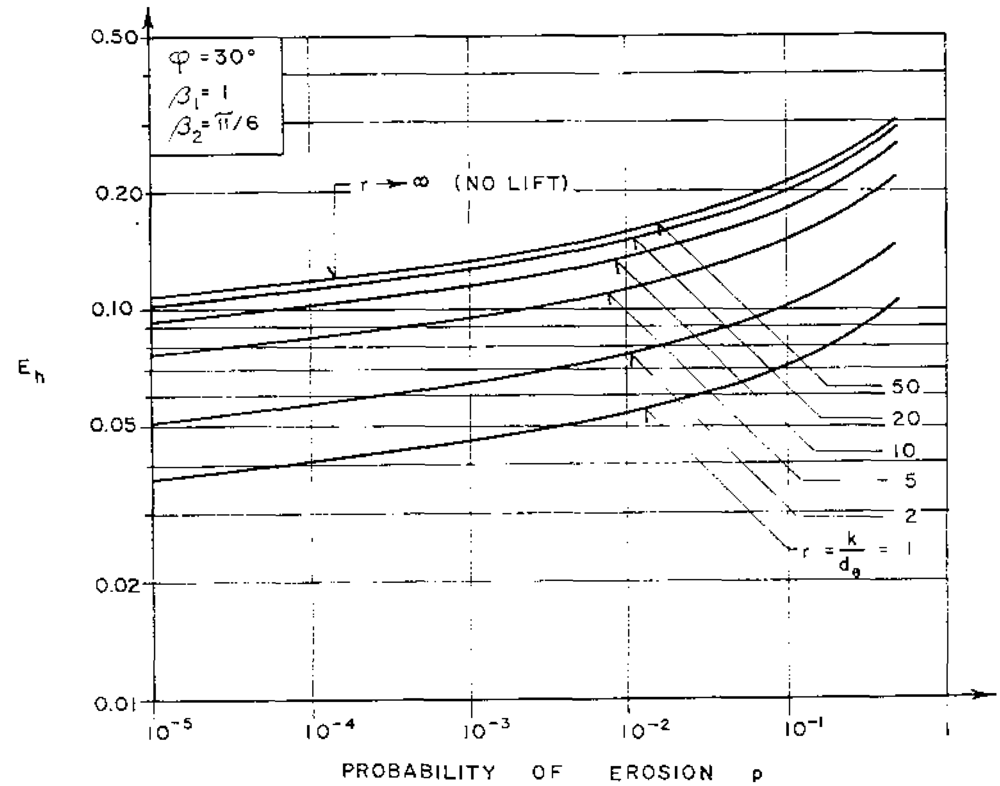


Figure 3 Critical value of entrainment function as function of probability of erosion p and roughness/grain-size ratio r for typical Florida coastal sediments. Horizontal bed. Christensen (1983b).

For a *transverse* slope i.e. a bank slope normal to the direction of flow a correction factor, $E_b/E_h$, may be developed. This factor is plotted for $\phi$ = 30° in Figure 4 as a function of the roughness/grain-size ratio r and the bank slope s here defined as the cotangent of the inclination of the embankment with horizontal (Christensen, 1972).

## EROSION OF TRANSPLANTED RUNNERS

Individual runners and their anchoring devices such as steel staples are acted upon by hydrodynamic forces, i.e. drag and lift, caused by the flowing water, buoyant forces and gravity as indicated in the left part of Figure 5. While the hydrodynamic and buoyant forces are trying to break the bond between the runner and the bed the gravity force is stabilizing this bond.

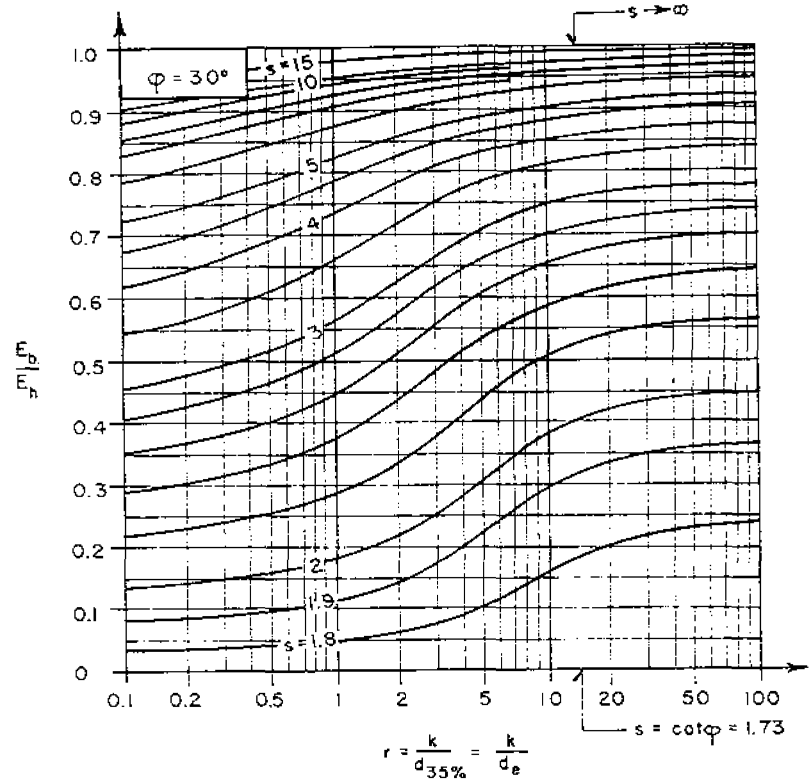


Figure 4 Correction factor for bank slope as function of roughness/ grain-size ratio and bank slope for typical Florida coastal sediments. Christensen (1972).

The local mean velocity at which the bond between transplanted runners or rather their growing root systems and the sand bed is broken may be determined by model experiments in a hydraulic research flume in which live runners are planted in a sand with the same effective grain-size as the sand on the site where the vegetation is to be established.

Since plant material (and thereby sand) must have the same size in model and prototype and a natural depth of 1.5 m to 2.5 m usually cannot be established in research flumes, it is necessary to use a hydraulic model law that allows for reduction of the model depth. Such a model law must be based on the requirement that the velocity fields in model and prototype are identical near the runners so that all hydrodynamic forces that are induced by the flow fields are the same in the two systems. This is illustrated in the right-hand side of Figure 5

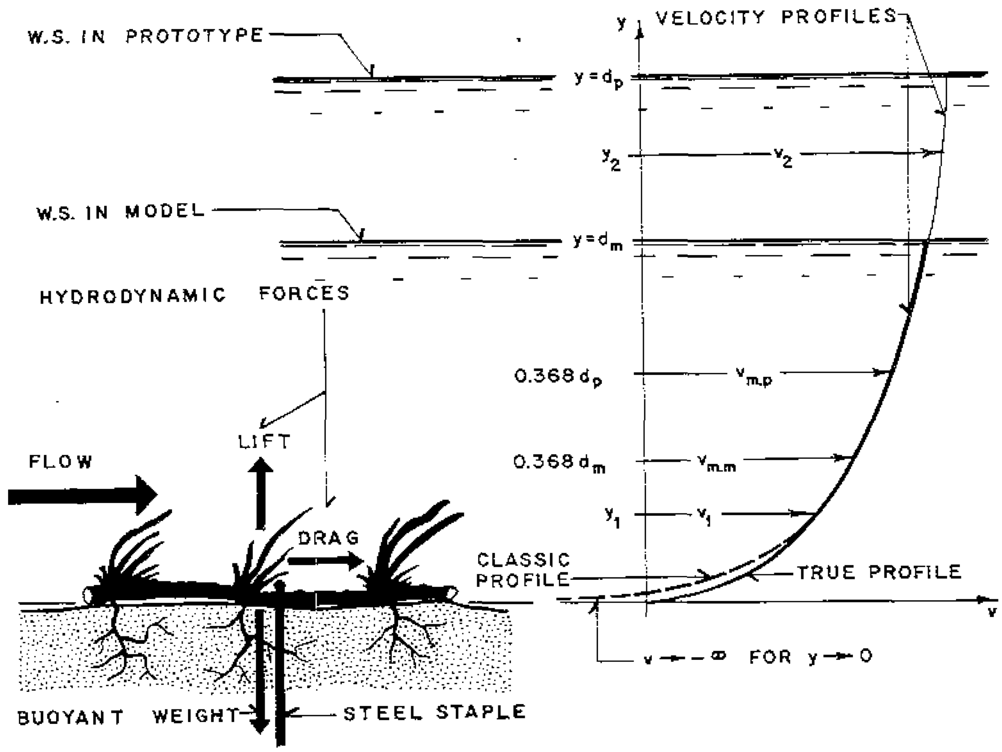


Figure 5 Velocity profiles and hydrodynamic forces acting on a newly transplanted seagrass runner in model (hydraulic flume) and in prototype (nature).

where the velocity profile below the model water surface corresponding to depth $d_m$ is the same as the prototypes velocity profile.

The classic Prandtl-profile that usually is used to represent turbulent flow over coastal and riverine sand bed reads

$$\frac{v}{v_f} = 8.48 + 2.5 \ln \frac{y}{k} = 2.5 \ln \frac{29.7y}{k} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; (5)$$

in which y = vertical distance from the bed to the point where the velocity is v. See for instance Streeter and Wylie (1975).

While equation (5) certainly represents the true velocity profile in most of the water column it fails near the bed where $v \to -\infty$ for $y \to 0$. Since the hydrodynamic forces acting on the runners depend on the velocities in the proximity of the bed it is necessary to use a modified version of equation (5). Such a profile was proposed by Christensen (1972). It may be written

$$\frac{v}{v_f} = 8.48 + 2.5 \ln \left(\frac{y}{k} + 0.338\right) = 2.5 \ln \left(\frac{29.7y}{k} + 1\right) \quad \ldots\ldots\ldots (6)$$

It represents the true velocity profile near the bed.

Recalling that the mean velocity in a vertical may be measured at a distance from the bed equal to 0.368 times the depth, applying equation (6) in model and prototype, and realizing that $v_f$ must have the same value in model and prototype lead to the ratio

$$\frac{v_{m.p}}{v_{m.m}} = \frac{\ln\left(\frac{10.9d_p}{k} + 1\right)}{\ln\left(\frac{10.9d_m}{k} + 1\right)} \quad \ldots\ldots\ldots (7)$$

in which $v_{m.p}$ and $v_{m.m}$ are the mean velocities in prototype and model verticals, respectively, $d_p$ = prototype depth, $d_m$ = model depth and of course k = roughness which must be the same in prototype and model in this case.

If model tests show that the bond between transplanted runner and sandbed fails at mean velocity $v_{m.m}$ the mean velocity $v_{m.p}$ at which this will happen in the prototype (nature) may be found from Equation (7) when $d_p$, $d_m$ and k are known.

The roughness k may be found by measuring the velocities $v_2$ and $v_1$ at distances $y_2$ and $y_1$ from the bed in the prototype as seen in Figure 5. Applying Equation (6) to these two points of the velocity profile and eliminating $v_f$ yields the iterative formula for k

$$k = \frac{\dfrac{29.7y_2}{\left(\dfrac{y_2}{y_1}\right)^{\frac{v_2}{v_2-v_1}}}}{} \cdot \frac{\left(1+\dfrac{k}{29.7y_1}\right)^{\frac{v_2}{v_2-v_1}}}{\left(1+\dfrac{k}{29.7y_2}\right)^{\frac{v_1}{v_2-v_1}}} \quad \ldots\ldots\ldots\ldots\ldots (8)$$

The above mentioned technique was applied to evaluate the impact of flow on an area to be restored in the Florida Keys as outlined in Tecnical Report No. 8301 from University of Florida's Hydraulic Laboratory. The model was established in the University of Florida's hydraulic research flume where a model bed consisting of 10 tons of sand with the same effective grain size ($d_e \cong 0.6$ mm) as the natural bed was constructed . Halodule runners 15 cm to 30 cm long were placed at 90 cm between centers and anchored by 15 cm steel staples made of 6 mm steel wire. Model velocities were measured with precision laboratory propeller meters and checked by discharge observations provided by the flume's main V-notch weir. A total of five tests were performed and the following observations made:

| | | |
|---|---|---|
| TEST NO. 1: | Model Depth: | $d_m = 0.49$ m |
| | Prototype Depth: | $d_p \cong 1.37$ m |
| | Roughness (apparent): | $k = 0.55$ m* |
| | Model Mean Velocity: | $v_{m.m} = 0.05$ ms$^{-1}$ |
| | Prototype Mean Velocity: | $v_{m.p} = 0.07$ ms$^{-1}$ |

*Observations:* No changes to plants or sand.

* Roughnesses in excess of the depth have been observed by the authors on many occasions in Florida. These values are apparent roughnesses caused by the flexibility of the bed vegetation.

TEST NO. 2: Model Depth: $d_m = 0.48$ m

Prototype Depth: $d_p \cong 1.37$ m

Roughness (apparent): $k = 0.55$ m

Model Mean Velocity: $v_{m.m} = 0.10\ ms^{-1}$

Prototype Mean Velocity: $v_{m.p} = 0.15\ ms^{-1}$

*Observations:* No changes to plants or sand.

TEST NO. 3: Model Depth: $d_m = 0.48$ m

Prototype Depth: $d_p \cong 1.37$ m

Roughness (apparent): $k = 0.55$ m

Model Mean Velocity: $v_{m.m} = 0.18\ ms^{-1}$

Prototype Mean Velocity: $v_{m.p} = 0.27\ ms^{-1}$

*Observations:* No apparent change to sand but two plants have had one of their ends pulled out of the sand. The other end for each of these two plants has remained in its original position.

TEST NO. 4: Model Depth: $d_m = 0.47$ m

Prototype Depth: $d_p \cong 1.37$ m

Roughness (apparent): $k = 0.55$ m

Model Mean Velocity: $v_{m.m} = 0.26\ ms^{-1}$

Prototype Mean Velocity: $v_{m.p} = 0.37\ ms^{-1}$

*Observations:* Some transport of the sand is apparent; however, the ripples are very small.

Three plants have had one of their ends pulled free of the sand.

Some of the stem portions of the grass which were initially covered with sand have been uncovered.

Two staples are now visible (the top).

Most of the plants are surrounded by very small ripples.

TEST NO. 5: Model Depth: $d_m = 0.46$ m

Prototype Depth: $d_p \cong 1.37$ m

Roughness (apparent): $k = 0.55$ m

Mean Model Velocity: $v_{m.m} = 0.32\ ms^{-1}$

Mean Prototype Velocity $v_{m.p} = 0.46\ ms^{-1}$

*Observations:* There was a tendency for the plants to become uncovered initially; however, ripples then formed around each plant and tended to build up sand around the plant. Two of the plants were almost totally covered at the end of the test and each of the others had a dune built up around it.

These tests seem to indicate that the newly transplanted runners will be affected by the flow at lower mean velocities than the sand bed. Damage to the runners may be expected already at a mean velocity of $v_{m.p} = 0.27\ ms^{-1}$ in the vertical bed.

## PROTECTION OF NEWLY RESTORED AREAS

If prototype mean velocities exceed about 0.25 $ms^{-1}$ in water with a depth of around 1.35 m protection against erosion of a newly transplanted seagrass bed may be necessary. This may be accomplished by installing a small dike upstream of the restored bed and another downstream of the bed. These dikes should be slightly higher than the depth corresponding to high tide.

Using the experimental results of Jensen's (1954) research on shelter effect the length L of a dike needed to protect a bed area of length $\ell$ and width b may be found from the formula

$$L = b + \frac{\ell}{12.5} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

Equation (9) takes into consideration the extend of the shelter zones in the main direction of flow.

## CONCLUSIONS

Methods for quantification of the erosion risk of newly restored grassbeds are presented considering erosion of the sandy bed itself as well as flow induced damage to the bond between roots and bed.

A stochastic diagram for the bed shear stress at which erosion of the bed begins is given together with a correction factor to be applied in the case of transverse bed slope.

A special model to prototype transfer law is developed for transfer of observations made of the resistance of newly transplanted runners in the laboratory. Application of this law to observations made in the University of Florida's hydraulic research flume shows that the growth of Halodule runners may be disrupted when the mean velocity exceeds about 0.25 $ms^{-1}$ corresponding to a depth of around 1.40 m. Damage to the bond between root and sand occurs before the sand bed itself is beginning to erode.

Based on earlier experiments with shelter effect a simple formula for the length of dikes used to protect newly restored areas has been developed.

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