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# Beach Nourishment Projects as "Experiments of Opportunity"
# "기회 실험"으로써의 양빈계획

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**Abstract** □ A beach nourishment project at North Redington Beach, FL was monitored and considered as an "experiment of opportunity" to investigate longshore sediment transport processes. The project comprised placement of 405,000 m$^3$ over a shoreline length of 2.6 km. Monitoring was carried out over a two-year period and included seven surveys of twenty six profile lines, wave measurements and sediment sampling. These data were analyzed to evaluate three longshore sediment transport formulae and to determine the best-fit coefficients and the relative goodness-of-fits. These included: (1) the Komar-Inman relationship, (2) an equation relating the total transport to the total longshore thrust on the surf zone, and (3) a recently suggested formulation by Kamphuis. It was found that all three relationships yielded goodness-of-fit values within 4% of each other when compared to the volumetric changes over the two year monitored period, too close to suggest superiority of any of the relationships tested. Calculations carried out using engineering approaches were found to overpredict the rate of evolution.

**要 旨** : 미국 Florida주 North Redington 해변의 양빈계획(Beach Nourishment Project)을 정기적으로 측정함으로써 종방향토사이동과정(*Longshore Sediment Transport Process*)을 조사하였다. 본 계획은 약 2.6 km 해변에 약 405,000 m$^3$의 모래로 이루어졌으며 2년에 걸쳐 26개의 profile을 계측하였고 파고, 파향 그리고 시료 채취를 포함하고 있다. 본 연구는 측정된 자료를 분석함으로써 최적의 토사이동계수와 그에 상응하는 상대오차를 구하는데 주된 목적이 있다. 적용된 토사이동공식은 1) Komar-Inman 공식, 2) Dean 공식(Radiation Stress, $S_{xy}$의 변형식), 3) Kamphuis 공식 등이다. 상기 3가지 공식은 大同小異(±4%)의 결과를 나타내어 어느 공식이 더 적합한지 우열을 가늠하기 어려웠다. 횡방향토사이동량을 고려한 상세모델과 고려하지 않은 단순모델의 두 가지 형태로 적용되었으며 일반설계를 위한 Model(단순모델)은 실측치보다 다소 큰값을 보여준다.

## 1. INTRODUCTION

Beach nourishment comprises the placement of large quantities of good quality sand on the beach to advance the shoreline seaward, thus resulting in a shoreline protuberance which nature tends to diminish through longshore sand transport. Additionally, because the sand is usually placed at slopes steeper than equilibrium, seaward sediment transport occurs. Figure 1 illustrates these adjustments. Because this disequilibrium induces sand flows, well-monitored nourishment projects represent "experiments of opportunity" which can yield information pertaining to sediment transport processes. Although beach nourishment has been employed as an engineering measure to forestall beach erosion for more than three decades, uncertainy remains regarding the effects of individual factors on project performance and there is a controversy concerning our capability to predict the performance of such projects. See for example the exchanges by Pilkey and Leonard (1990, 1991) and Houston (1990, 1991). Major reason for such uncertainty is the lack of good monitoring data which include both the sedimentary response and the forcing, principally the wave climate. With an improved ability to predict

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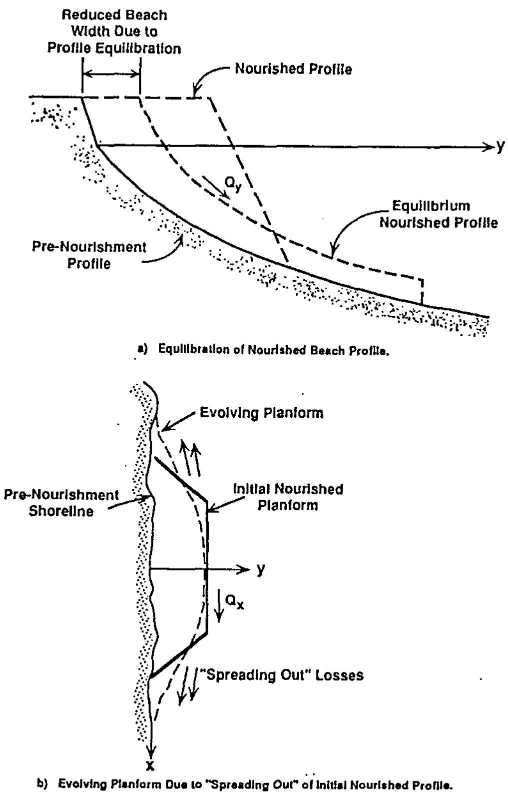



**Fig. 1.** Processes Affecting Beach Nourishment Projects.

beach nourishment performance, more appropriate allocation of project benefits could be established including the additional effects due to longshore spreading of the placed sand to adjacent beaches (Dean, 1988).

## 2. BACKGROUND

The bases for predicting the effects of longshore spreading are the equations of longshore transport and continuity which may be expressed in the "one-line" form. The three alternate transport formulae evaluated include

$$Q_1=\frac{KH_b^{2.5}\sqrt{g/\kappa}}{8(s-1)(1-p)}\sin(\beta-\alpha_b)\cos(\beta-\alpha_b) \tag{1a}$$

$$Q_2=K'\frac{S_{xy}}{\rho g(s-1)(1-p)} \tag{1b}$$

and

$$Q_3=K''6.4\times10^4H_{sb}^2T_p^{1.5}m_b^{0.75}D_{50}^{-0.25}$$
$$[\sin(2(\beta-\alpha_b))]^{0.6}sign(\beta-\alpha_b)\ (m^3/yr) \tag{1c}$$

The common continuity equation is

$$\frac{\partial y}{\partial t}=-\frac{1}{h_*+B}\frac{\partial Q}{\partial x} \tag{2}$$

in which $Q$ is the total volumetric rate of longshore sediment transport, $K$ is the longshore sediment transport coefficient, $H_b$ is the breaking wave height, $g$ is the gravitational constant, $\kappa$ is the ratio of local breaking wave height to local water depth, $s$ is the density of the sediment relative to the water in which it is immersed, $p$ is in-place porosity, $\beta$ is the azimuth of the shoreline outward normal and $\alpha_b$ is the azimuth of the direction from which the wave is coming at the breaking point. In Eq. (2), $x$ and $y$ are the longshore coordinate and shoreline displacement, respectively, $h_*$ is the water depth which is active in the equilibration process, and $B$ is the berm height. Eq. (1a) is the so-called "Komar-Inman" (1974) relationship, Eq. (1b) was evaluated by Dean *et al.* (1982) and Eq. (1c) is a formulation proposed recently by Kamphius based on an analysis of laboratory and field data. The terms in Eq. (1c) are defined as follows: $H_{ab}$ is the significant breaking wave height, $T_p$ is the peak period of the offshore wave spectrum, $m_b$ is the beach slope at the breaking zone, $D_{50}$ is the median grain size, and $\alpha_b$ is the breaking wave angle. In Kamphius' formulation, $K''=1$, and we have included this coefficient as a measure of the difference between his result and that obtained here.

In modelling the evolution of beach nourishment projects, Eqs. (1a) and (2) may be combined by considering the change of planform alignment due to nourishment to be small, which allows the transport equation to be linearized yielding, as found by Pelnard-Considere (1956),

$$\frac{\partial y}{\partial t}=G\frac{\partial^2 y}{\partial x^2} \tag{3}$$

which is the familiar heat conduction equation and the coefficient $G$ is referred to as the "longshore diffusivity" parameter written in its simplest form as

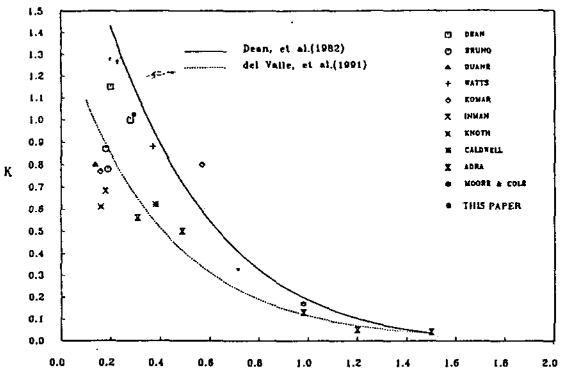



**Fig. 2.** Variation of Sediment Transport Coefficient with Sand Size. Results Include Previous Field Studies and That of This Paper.

$$G=\frac{KH_b^{2.5}\sqrt{g/\kappa}}{8(s-1)(1-p)(h_*+B)} \tag{4}$$

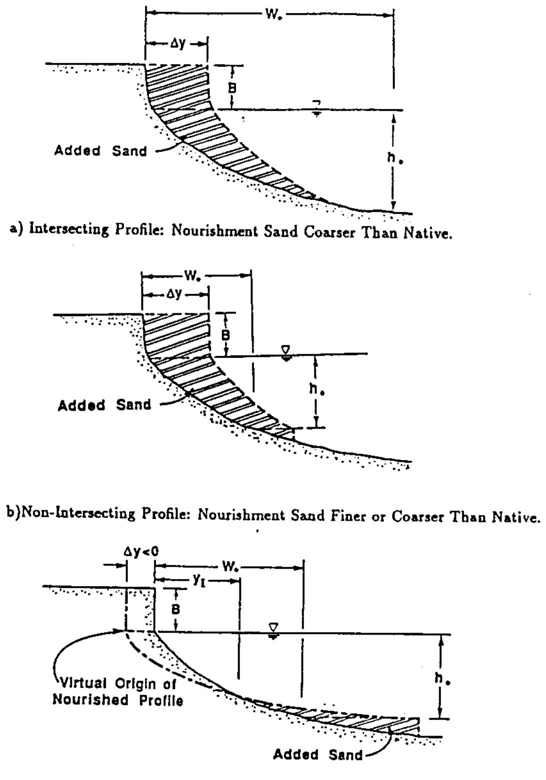
Eq. (3) has a number of solution for various initial and boundary conditions. Many solutions are of interest in the field of coastal engineering and have been summarized by Larson *et al.* (1987). A solution of particular interest here is for the evolution of a beach nourishment project of an initially rectangular planform of uniform displacement, $Y$, centered within a length, $l$,

$$y(x,t)=\frac{Y}{2}\left\{erf\left[\frac{l}{4\sqrt{Gt}}\left(\frac{2x}{l}+1\right)\right]-erf\left[\frac{l}{4\sqrt{Gt}}\left(\frac{2x}{l}-1\right)\right]\right\} \tag{5}$$

in which "*erf*" denotes the error function. Based on Eqs. (4) and (5), it is possible to infer general performance characteristics of beach nourishment projects, e.g. Dean (1983), Kraus (1991). The most significant parameters include: project length, wave height, sediment characteristics, and background erosion. Defining the project longevity as the time required for a certain proportion of the sediment to be transported from the placement area, the effect that each of these parameters has on project life is reviewed briefly below.

The longevity of a project increases with the square of its length. Thus, other conditions being equal, a project with a length twice that of another project will have a greater longevity by a factor of four. The "spreading out" of the project is due to the gradients in longshore sediment transport and since the transport depends on the breaking wave height to the 2.5 power, the longevity varies inversely with height to the same power. The effect of sediment size is two-fold. The first is through the longshore sediment transport dependency on sediment size. Figure 2 was developed by Dean *et al.* (1982) and has been augmented by del Valle *et al.* (1991) with the addition of field data and interpretation. Thus coarse sediments will experience less transport than finer sediments thereby, increasing longevity in the same proportion as that of the $K$ values of the native and nourishment sediments.

The second effect of sediment size is due to the well-known relationship of increasing beach slope with sediment size. If a sediment size is used which is finer than the native, the resulting profile, when equilibrated, will form a narrower dry beach than a coarser sediment had been used. Based on equilibrium profile concepts, Dean (1991) has shown that, depending on the relative sizes of the native and nourishment sediments and the volume of sand placed per unit length of beach, three types of equilibrium profiles can result as shown in Fig. 3 and described as follows: (1) Intersecting profiles in which the nourished and original profiles intersect. This type requires the nourishment material to be coarser than the native although coarser nourishment material does not ensure intersection, (2) Non-intersecting profiles, which always occur if the sediment is the same size as or finer than the native and may occur if the sediment is coarser than the native, and (3) Submerged profiles characterized by the entire nourished profiles being subaqueous; this type occurs for placement of relatively small amounts of sediment which is finer than the native. Usually the background erosion affecting the nourishment project is assumed to continue at the same rate and with the same distribution as the previous nourishment. However, since the background erosion is usually considered to be due to longshore transport, there may be justification in modifying the background erosion rates by the ratio of the $K$ factors of the nourishment and native sediments.



a) Intersecting Profile: Nourishment Sand Coarser Than Native.

b)Non-Intersecting Profile: Nourishment Sand Finer or Coarser Than Native.

c)Submerged Profile: Nourishment Sand Finer Than Native.

**Fig. 3.** Three Generic Types of Nourished Profiles.

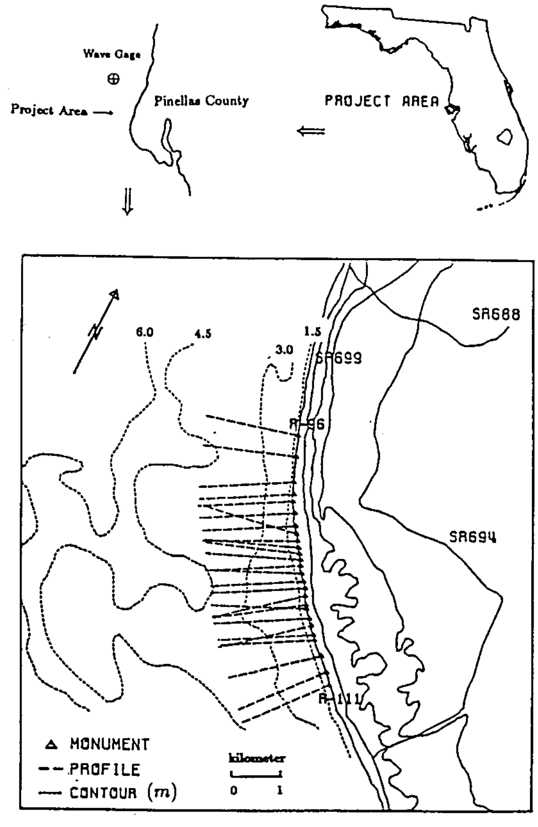



**Fig. 4.** Plan View of Beach Monitoring Profiles in the Sand Key Project and Location of Directional Wave Gage.

Recently "profile nourishment" has been proposed in which the sediment is placed offshore underwater with the expectation that the shallower water will cause damping of the waves, thus protecting the shoreline, and the sediment will eventually be transported shoreward and will become part of the subaerial beach system. Not surprisingly, our capability to calculate the evolution and performance of this type of beach nourishment is even less well developed than for subaerial beach nourishment.

## 3. DATA DESCRIPTION

### 3.1 The Field Data

The Redington Shores beach nourishment project commenced in July, 1988 with the placement of approximately 405,000 m$^3$ of sand along approximately 2.6 km of shoreline. Prior to the nourishment, the shoreline had been receding at an approximate rate of 0.3 m/yr. Project documentation included a wave gage at the location indicated in Fig. 4. During the first year, a pressure gage was installed, thus providing only a one-dimensional spectrum. During the second year, the gage comprised a pressure sensor and a biaxial current meter which provided a basis for determining a measure of wave direction. Monitoring of the beach fill evolution was undertaken jointly by the University of South Florida and the University of Florida. Data collected included beach and offshore profiles and sediment characteristics. The profiles were surveyed a total of seven times with the first and second surveys occurring immediately prior to and following nourishment. Various aspects of the project and its performance have been reported by Davis (1991) and Davis *et al.* (1991).

One interesting aspect of the project that proved beneficial to the analysis presented herein was that

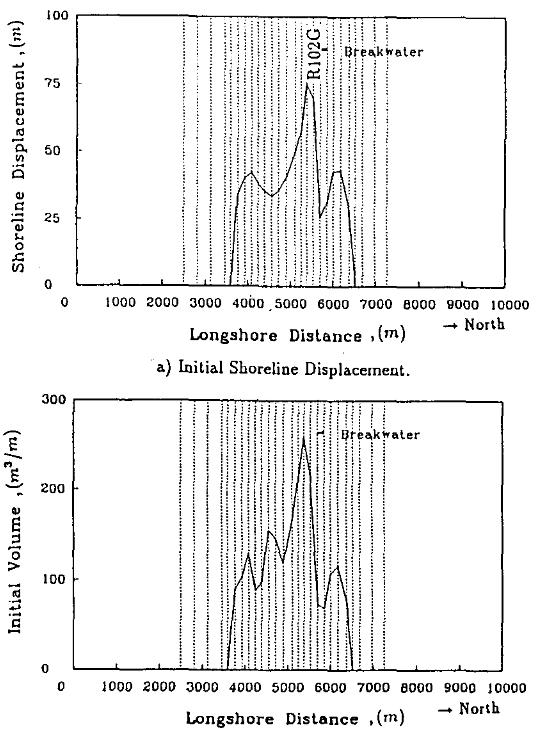



**Fig. 5.** Initial Shoreline Displacement and Placement Volume Density. (Monitored Profiles Shown As Dashed Lines)

for reasons that are not clear, the shoreline displacement and sand placement volume density varied along the shoreline as shown in Figs. 5a and 5b, respectively. This non-uniformity induced gradients in longshore transport that proved useful in determining the sediment transport coefficient, $K$. As is the case in most beach nourishment projects, the sand was placed at a considerably steeper slope than equilibrium. Thus with time, there was a gulfward flow and deposition of sand in water depth greater than normally influenced by the waves.

Figure 6 presents an example of the nourishment and subsequent evolution of a profile at which there was initially an exposed seawall. A feature of the project that caused difficulties in the analysis was the presence of a detached breakwater some 130 m in length located in a water depth of 1.3 m within the project confines near its northern end. This breakwater had been constructed in early 1986 and was the first of five planned elements for the purpose of stabilizing the overall beach nourishment project of which the nourishment project considered here was the first phase. Additional information describing the offshore breakwater is available in Dean and Pope (1987), Terry and Howard (1988), and Terry and Matin (1989).

### 3.2 Wave Data

Wave characteristics were measured by a pressure gage (first year) and a directional gage (second year) located as shown in Fig. 4. Significant wave heights varied from 0.1 m to 0.5 m for most of the period with an occasional maximum of 1.5 m. The variations of wave period and wave direction were relatively small. Although the wave height, period and direction change continuously with time, the effective wave heights were calculated based on the Rayleigh distribution and used in the computations as proposed by Dean and Grant (1989).

$$H_{eff}=\left[\frac{1}{N}\sum_{n=1}^{N}(K_sH_{sn})^{2.4}\right]^{1/2.4} \tag{6}$$

in which $H_{sn}$ is the significant wave height at time, $t_n$ and $K_s$ (=0.735) based on the Rayleigh distribution which relates significantly to effective wave height. The overall effective wave height, $H_{eff}$ and the average wave period and direction were calculated to be 0.27 m, 4.6 s and 95°, respectively where the wave direction is measured clockwise and 90° signifies normal to the shoreline. In the project evolution computations, the effective wave heights were calculated and applied for each intersurvey period. The effective wave characteristics for each intersurvey period are summarized in Table 1. Rosati (1989) reported on the results of a directional wave gauge installation in 5.2 m of water depth some 1250 m offshore Redington Shores. Data were collected over a one year period from February 1986 to February 1987. Although over different time periods, the wave heights were found to be similar to those found here with a range of 0.1 to 1.2 m and a wave period of approximately 5 seconds. The wave direction was generally from the south. Wave heights were compared with data from the University of Florida non-directional gage at Clearwater and were generally found to be within 0.1 m.

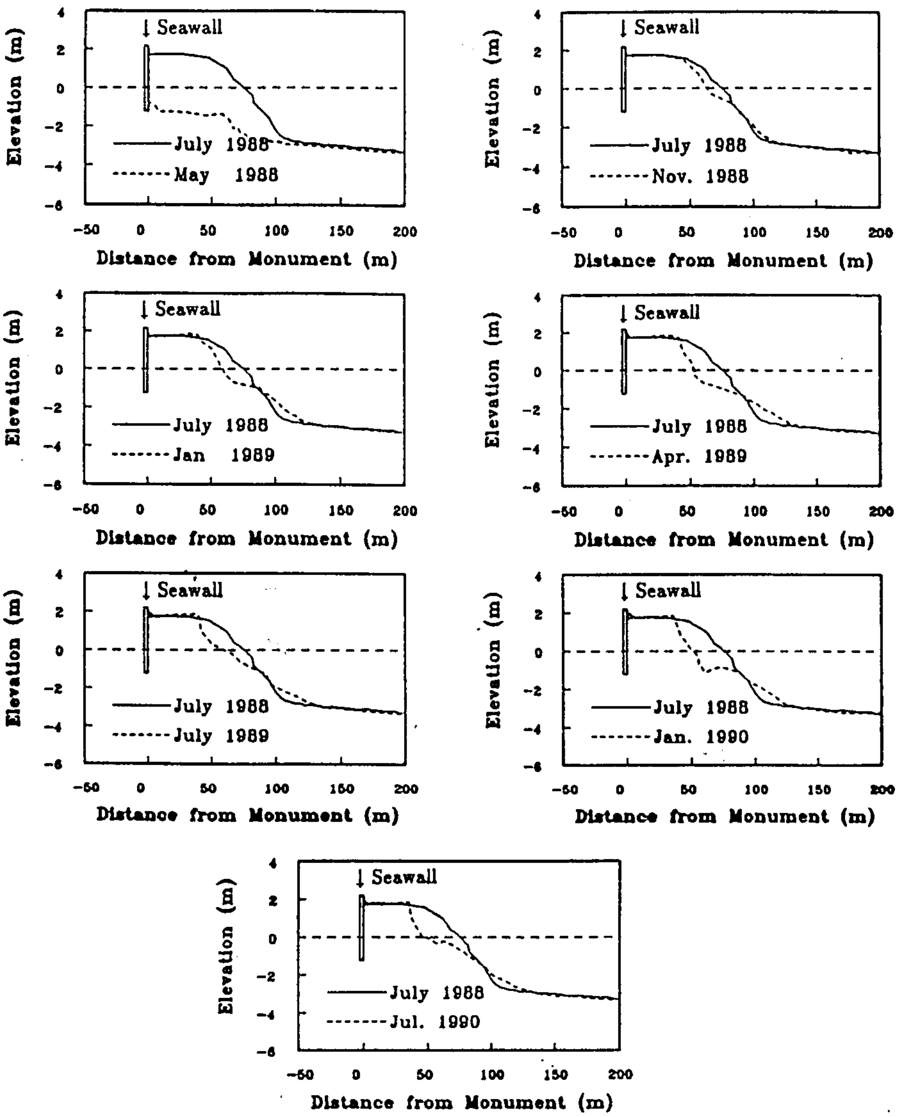



**Fig. 6.** Illustration of Nourishment and Subsequent Evolution Over a Two Year Period for Profile R102G. Note That Initially There Was an Exposed Seawall at This Profile.

**Table 1.** Wave Characteristics

| Intersurvey Period | Duration (Month) | Effective Wave Height[1] (m) | Effective Wave Period (Sec) | Wave Direction (Degrees)[2] |
|---|---|---|---|---|
| 1 | 3 | 0.26 | 4.5 | Not Available |
| 2 | 3 | 0.27 | 4.8 | Not Available |
| 3 | 3 | 0.30 | 4.6 | Not Available |
| 4 | 3 | 0.29 | 4.6 | Not Available |
| 5 | 6 | 0.26 | 4.5 | 96.2 |
| 6 | 6 | 0.24 | 4.5 | 94.9 |

[1] Calculated Deep Water Values. [2] Measured Clockwise With 90° Representing Normal to Nominal Shoreline Orientation.

## 4. MODEL DESCRIPTION

The shoreline evolution model employed here is known as a one-line model in which Eqs. (1) and (2) are applied to predict at sequential times, the changes in volume and the position of the National Geodetic Vertical Datum (NGVD) contour within the model domain. The transport equation given by Eq. (1) is expressed in terms of the wave characteristics at the breaking point. To apply to the one dimensional model, it is convenient to modify this equation in terms of deep water wave conditions. In this paper, the algebraic steps are omitted and only the final results presented. For detailed derivations, the reader is referred to Dean and Grant (1989) or Dean and Yoo (1992). Using linear wave theory and idealized contours, Eq. (1) can be expressed as

$$Q=\frac{KH_o^{2.4}C_{Go}^{1.2}\,g^{0.4}\cos^{1.2}(\beta_o-\alpha_o)}{8(s-1)(1-p)C_*\kappa^{0.4}}\sin(\beta_s-\alpha_*) \qquad (7)$$

in which the subscripts "o" and "*" denote that subscripted variables are to be evaluated at deep water and landward of the depth transition of the nourished profiles, respectively, and $\beta_s$ is the azimuth of the outward normal of the nourished contours.

The transport and continuity equations (Eqs. (1) and (2)) were solved using an explicit method with the grid system shown in Fig. 7. The shoreline displacements are held fixed while the transport is computed and in the second part of the same time step the transport values are held constant while the shoreline displacements are computed. To maintain stability, the limiting time step was taken as 1 day. In conducting numerical modeling of generally straight shorelines where a large perturbation has been introduced, it is recommended, in general, that the original shoreline and offshore bathymetry be represented by straight and parallel contours and that the historical background erosion rate be taken into account (Dean and Yoo, 1992).

Referring to Fig. 7, the azimuth, $\beta_s$, of the shoreline normal at the $n^{th}$ time level, is established to represent the value at the grid line associated with $Q_i^n$

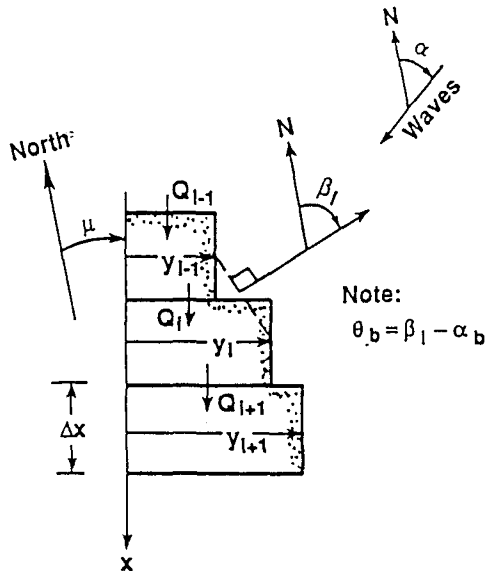



Fig. 7. Definition Sketch for Numerical Model.

$$\beta_i^n=\mu+\frac{\pi}{2}-\tan^{-1}\left(\frac{y_{i+1}^n-y_i^n}{x_{i+1}-x_i}\right) \qquad (8)$$

Finally, the longshore position is updated from the $n^{th}$ to the $(n+1)^{th}$ time level as

$$y_i^{n+1}=y_i^n+\frac{\Delta t}{\Delta x(h_*+B)}(Q_i^n-Q_{i+1}^n) \qquad (9)$$

## 5. CHARACTERISTICS OF PROJECT EVOLUTION

Figure 8 illustrates the general variations in transport during evolution of a nourishment project. In the case of our field data, through examination of the monitored profiles, several features were noted that were relevant to and influenced the analysis and interpretation of the data and that may also be important to the prediction of performance of a beach nourishment project. Three such features are discussed below.

As expected, the more gulfward portions of the nourishment (Fig. 5a) retreated most rapidly with a portion of the sand transported to the adjacent profiles. Because the profiles within the nourished region were oversteepened relative to the equilibrium, the paths of the transported sand were chara-

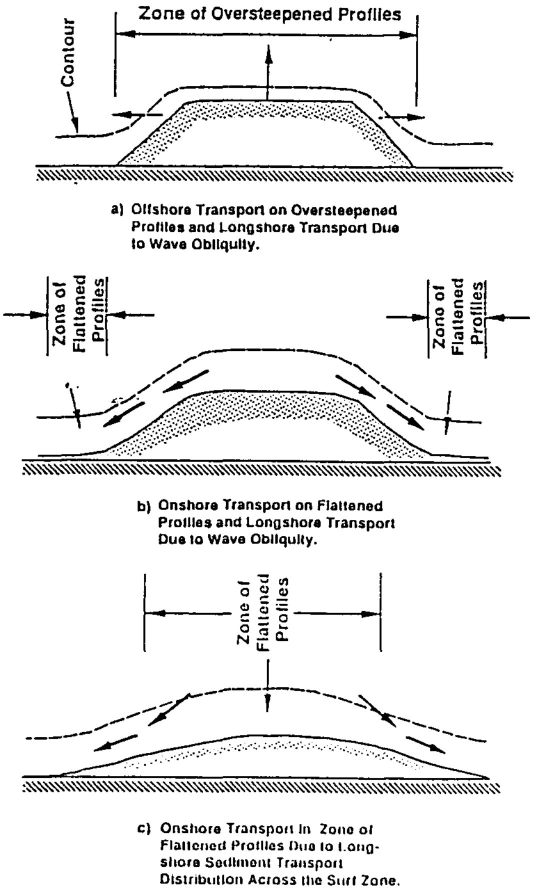



**Fig. 8.** Three Phases of Observed Sediment Transport in Vicinity of Nourished Projects. Note: Cross-Contour Transport Due to Profile Disequilibrium (from Dean *et al.*, 1992).

cterized by an offshore component. Thus profiles adjacent to the more gulfward shoreline segments experienced less offshore deposition than did the oversteepened profiles. This effect is illustrated in Fig. 8a, where the sediment transported offshore.

Realignment of the initially irregular nourished shoreline occurred in part due to the seaward transport of sediment. Thus, even without longshore sediment transport, the tendency for alongshore sediment transport was reduced and it was necessary to account for this effect when modeling the longshore sediment transport.

## 6. ANALYSIS PROCEDURE

The primary objective of the analysis procedure was, through the correlation of calculated and measured of project evolution, to develop estimates of the longshore sediment transport coefficient, $K$. The steps involved in this procedure are described below.

The cross-shore sediment transport at each profile and for each intersurvey period was determined directly from the data for each intersurvey period. In the calculations, the rate of change over the intersurvey period due to cross-shore transport was assumed constant. Thus at each time step, the shoreline change, $\Delta y_c$, due to cross-shore transport only was calculated from the integrated equation of continuity, Eq. (2), as

$$\Delta y_c = \frac{\Delta V_c}{h_* + B} \tag{10}$$

in which $\Delta V_c$ was the prorated change involume over the time step due to measured cross-shore transport above the depth, $h_*$. The corresponding change due to gradients in longshore transport was calculated based on Eqs. (1) and (2). This process was repeated for all profiles and all six intersurvey periods using the effective wave heights, periods and directions associated with those intersurvey periods (Table 1). For the first and second transport models, the wave directions measured during the second year were assumed to apply in the same sequence as the first year when they were not measured. It can be shown that this is approximately correct since in the absence of structures, the evolution of a beach nourishment project is very weakly dependent on wave direction (Dean and Yoo, 1992). The third (Kamphius) model is quite dependent on wave direction and thus it was possible to apply the procedure only over the second year. The effect of background erosion was included by comparing the calculated and measured volumes of sediment remaining within the monitored area after the final (two year) survey. The background erosion was considered to account for this difference and the calculated results adjusted assuming that the background erosion was uniform over the region. It is noted that the background erosion thus determined was generally found to be relatively small (equivalent to approximately 0.3 m/yr compared to the root-

**Table 2.** Comparison of Best Fit Coefficients and Their Respective Non-Dimensional Goodness of Fits, $\varepsilon^2_{mn}$. Based on Two years of Data

| Transport Relationship | Best Fit Coefficient | $\varepsilon^2_{mn}$ |
|---|---|---|
| $Q=\frac{KH_b^{2.5}\sqrt{g/k}}{8(s-1)(1-p)}\sin(\beta-\alpha_b)\cos(\beta-\alpha_b)$ | $K=1.02$ | 0.147 |
| $Q=K'\frac{S_{xy}}{\rho g(s-1)(1-p)}$ | $K'=2.01$ | 0.150 |

mean-square shoreline change of 10.4 m/yr). The pattern of the thus calculated volumetric changes due to longshore transport was then compared with the measured for varying $K$ values and the process repeated until a best fit $K$ value was obtained. This procedure may be viewed as using the overall volume change to determine the background erosion (for each candidate transport coefficient) whereas the pattern of volumetric change provides a basis for determining the longshore sediment transport coefficient, $K$.

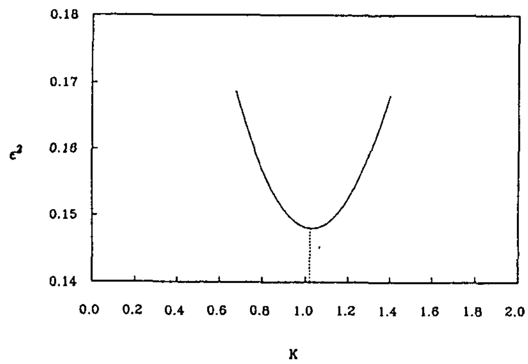



**Fig. 9.** Variation of Error Measure and Best Fit Transport Coefficient ($K=1.02$) for Eq. (1) and Southern Half of Monitored Region.

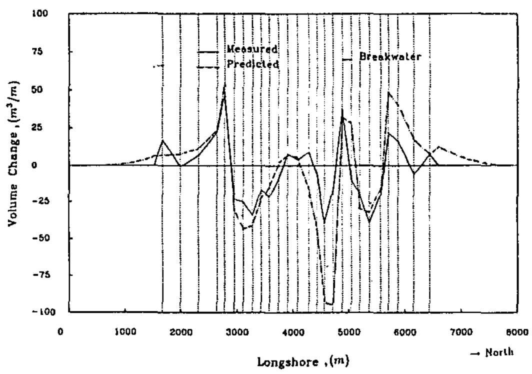



**Fig. 10.** Volumetric Change Distributions at Two Years Over Monitored Area for Best-Fit Coefficient ($K=1.02$). Measured vs Predicted. Cross-Shore Transport Taken Into Account.

## 7. RESULTS

In order to provide a quantitative basis for evaluating fits, the following measure of normalized error was defined.

$$\varepsilon^2=\frac{\sum_{j=1}^{6}(\Delta V_{mi,j}-\Delta V_{pi,j})^2}{\sum_{j=1}^{6}\Delta V_{mi,j}^2} \tag{11}$$

in which $\Delta V_{mi,j}$ and $\Delta V_{pi,j}$ are the best fit volumetric changes at the $i^{th}$ profile and the $j^{th}$ intersurvey period. It is seen that $0<\varepsilon^2<1$, with $\varepsilon^2=0$ indicating a perfect fit. It was found that the patterns of calculated and measured shoreline changes agreed quite well in the southern half of the monitored region; however, the fit attainable was not as favorable in the northern portion. We interpret this to be due to the influence of the detached breakwater in the latter area as shown in Fig. 6. Thus the best-fit transport coefficients are based on the southern one-half of the monitored area. The variation $\varepsilon^2$ ($K$) is presented in Fig. 9 and shows a well-defined minimum at $K=1.02$. The measured and predicted distributions of volumetric changes for $K=1.02$ and the full two year period are presented in Fig. 10. It is seem that although the fit is reasonable throughout the entire region, it is far better in the area least affected by the detached breakwater.

The best-fit $K$ value and the associated sediment size were compared with a composite relationship developed by Dean *et al.* (1982) and augmented by del Valle *et al.* (1991) as shown in Fig. 2. The sediment characteristics at the Redington Shores beach nourishment project were characterized by examining the size on the beach fach and at the 3 m contour within the nourishment area. The approximate averages for these two locations were 0.45 mm

**Table 3.** Comparison of Best Fit Coefficients and Their Respective Non-Dimensional Goodness of Fits, $\varepsilon^2_{min}$. Based on Second Year's Data

| Transport Relationship | Best Fit Coefficient | $\varepsilon^2_{min}$ |
|---|---|---|
| $Q=\dfrac{KH_b^{2.5}\sqrt{g/k}}{8(s-1)(1-p)}\sin(\beta-\alpha_b)\cos(\beta-\alpha_b)$ | $K=0.32$ | 0.333 |
| $Q=K'\dfrac{S_{xy}}{\rho g(s-1)(1-p)}$ | $K'=0.61$ | 0.336 |
| $Q=K''6.4\times10^4H_{sb}^2T_p^{1.5}m_b^{0.75}D_{50}^{-0.25}$ $[\sin(2(\beta-\alpha_b))]^{0.6}$ | $K''=1.10$ | 0.329 |

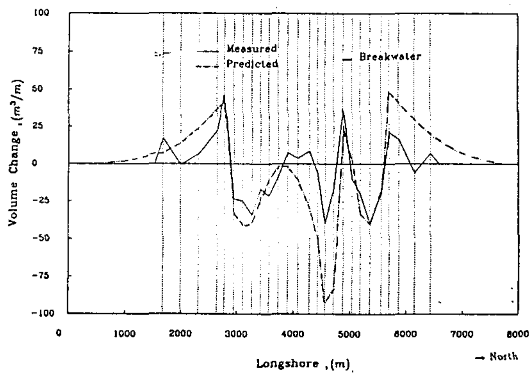



**Fig. 11.** Volumetric Change Distributions at Two Years for "Engineering Approach". No Consideration of Cross-Shore Transport, and $K=0.77$. Measured vs Predicted.

and 0.16 mm, respectively. A size of 0.30 mm is considered as an approximate representative size. This $K$ coefficient was also compared with on earlier plot of $K$ vs. $gH/w^2$ by Dean and Walton (1985) ($w$=sediment fall velocity defined earlier) and was found to agree well.

Several approaches were employed to extracy values of transport coefficients for the three transport formulae discussed earlier. These involved determining the best fit coefficient based on the volume change distribution over the entire two year period and over the first and second years, separately. In addition, the coefficients were determined which agreed with the net volumetric changes inside and outside the nourished areas. It is worth noting that the project evolution associated with transport relationship given by Eqs. (1a) and (1b) are very insensitive to wave direction (Dean and Yoo, 1992) and thus computations could be carried out over the full two year period. Project evolution due to transport given by Eq. (1c) is quite sensitive to wave direction and thus evolution was possible only over the second year. Finally, as a measure of our capability to predict project evolution, standard engineering approaches were applied. Because planform evolution based on Eq. (14) is sensitive to wave direction, the best fit coefficients for all three relationships were recalculated for the second year monitoring data for which wave direction data are available, and the results are presented in Table 3.

Although Eq. (14) yields a slightly superior least squares fit (Table 3) to the data compared to Eq. (1) to the data, the measureds of fit are very nearly the same, (within 2%) and thus not considered significant. The reasons appear to be that the wave directions relative to the shoreline $(\beta-\alpha_b)$ occurring in these data were in the approximate range $|\beta-\alpha_b|<5°$. It can be shown that $\sin 2(\beta-\alpha_b)$ and $\delta[\sin 2(\beta-\alpha_b)]^{0.6}\mathrm{sign}(\beta-\alpha_b)$ do not differ significantly over this range if $\delta$ is allowed as a free variable. Since $K''=1.10$, this indicates good quantitative agreement with Kamphuis.

## 8. ENGINEERING APPROACH

In addition to the analysis of sediment transport relationships to determine $K$, as described above, it is of engineering interest to evaluate how well the usual approach (based on Eqs. (1) and (2)) for calculating nourishment performance would apply. Differences here are three-fold: (1) the time varying berm height was neglected and based on the data, a constant $h_*+B=2.4$ m was used, (2) cross-shore sediment transport was not taken into account, and (3) a $K$ value of 0.77 was selected as that recommended by Komar and Inman (1970) and the most frequently referenced.

The results in terms of the volumetric and shoreline changes are presented in Figs. 11 and 12. The volume change distribution in Fig. 11 is to be com-

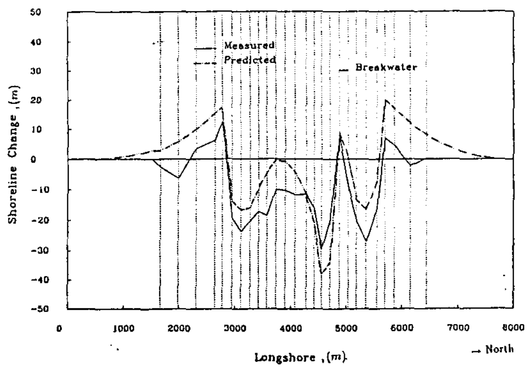



**Fig. 12.** Shoreline Displacement Change Distributions at Two Years for "Engineering Approach". No Consideration of Cross-Shore Transport, and $K=0.77$. Measured vs Predicted.

pared with that in Fig. 10 for which cross-shore sediment transport was included. It is seen that on an overall basis, the fit is somewhat better for the more detailed calculations ($\varepsilon^2=0.147$) than for engineering calculations ($\varepsilon^2=0.23$). Within the nourishment area and in the southern half of the project, the volumetric changes are in reasonalbly good agreement (Fig. 11). South of the nourishment area, the volumetric changes are overpredicted substantially. The shoreline changes (Fig. 12) are not predicted well inside or outside the nourished area. The reasons for the difference are due, in part. to the effect of cross-shore transport which diminished the transport from the nourished are and also caused an associated shoreline retreat within the project area. The smaller $K$ factor (0.77 vs 1.02) caused the volumetric changes in the nourished area to be approximately the same as measured.

## 9. SUMMARY AND CONCLUSIONS

1. A beach nourishment project at North Redington Beach, Florida was monitored through seven surveys over a period of two years. In addition to monitoring a total of twenty six profile lines both within and outside of the nourishment area, wave characteristics were measured by a pressure gauge (first year) and a directional gauge (second year). Sediment samples were also collected.

2. It has been shown that carefully monitored beach nourishment projects, including measurement of the wave characteristics, can provide good "experiments of opportunity" and a basis for determining the longshore sediment transport coefficient, $K$. Irregularities in the planform placement density induce longshore transport and are beneficial to the determination of the transport coefficient.

3. Several characteristics, not reported previously, but of importance in the analysis, include cross-shore transport which diminishes planform irregularities by processes other than longshore transport and a combination of longshore and crossshore transport to lower active portions of those profiles adjacent to the more protruding segments of the project.

4. The best-fit transport coefficient, $K$, for the data here was found to be 1.02 which, for the approximate representative sand size of 0.3 mm, is reasonably consistent with an earlier relationship found by Dean *et al.* (1992) and as modified slightly by del Valle *et al.* (1991) through the addition of more data. The results are also consistent with a plot of $K$ vs $gH/w^2$ presented by Dean and Walton (1985).

5. A linear relationship between the transport and $S_{xy}$, the longshore wave thrust applied to the surf zone yielded a value of $K'=2.01$ m/s (Eq. (12)) compared to the value of 2.63 m/s found by Dean *et al.* (1982) from the Santa Barbara and Rudee Inlet data. The recent transport relationship (Eq. (13)) presented by Kamphuis (1991) was tested and found to yield a slightly better agreement than that proposed by Komar and Inman (1970). The associated transport coefficient determined here (1.10) is only 10% greater than the value (1.00) proposed by Kamphuis.

6. As a test of usual engineering methods of predicting nourishment performance which do not account for cross-shore transport, calculations carried out with $K=0.77$ did not agreed well with the sand transported out of the nourished area (14983 $m^3$ measured vs 25848 $m^3$ calculated). In addition the shoreline advancement and recession were found to be overpredicted outside the nourished area and underpredicted inside the nourished area, respectively. The average NGVD shoreline displacements within the project area were 5.1 m (measured) vs-

11.3 m (calculated).

7. Since sand is usually placed in nourishment projects at slopes steeper than equilibrium, monitoring profile evolution should provide a good basis for investigating the less well understood cross-shore transport characteristics. Such a study is now in progress with the data employed here.

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