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Ocean City, Maryland, Wave Runup Study

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ABSTRACT

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An investigation of wave runup using video camera technology at Ocean City, Maryland, is discussed. Past studies on wave runup statistics are reviewed and practical problems of the wave runup prediction problem are noted. Results are provided from a subset of the runup experiment and differences between wave runup level probability density functions and wave runup amplitude probability density functions are detailed.

ADDITIONAL INDEX WORDS: *Wave runup, beaches, wave overtopping, wave spectra, wave groups.*

INTRODUCTION

Wave runup on beaches is a subject of major importance to coastal engineering and beach design. An improved ability to predict wave runup on beaches will lead to better estimation of necessary beach berm design height for storm protection as well as improved estimates of costs and benefits for various alternative beach template designs in a nourishment project.

Although considerable effort has been expended on addressing the problem of very long wave (tsunami) runup over the past century, only in recent years (≥ 1950) has the subject of wind and swell wave runup on beaches begun to be addressed. In the 1950's a series of small scale linear slope laboratory experiments on wave runup due to monochromatic wave forcing was carried out in wave tanks at the Beach Erosion Board (predecessor to the Coastal Engineering Research Center) in Washington, D.C. and at the Waterways Experiment Station in Vicksburg, Mississippi. These experiments along with additional data obtained in runup experiments in a French laboratory became the primary data base utilized in a classic wave runup paper by Major Ira HUNT (1959). SAVILLE (1956) utilized some of the same smooth slope runup data along with additional laboratory data to compile the first widely published wave runup curves for use in engineering design. These curves are still in use today and published in the *Shore Protection Manual* (1984). SAVAGE (1958), in another set of small scale laboratory wave runup experiments with monochromatic wave forcing, looked at both smooth slope wave runup and roughened slope wave runup. Results of his findings showed that wave runup on roughened slopes was reduced from that on smooth slopes as might be expected. The experiments of SAVILLE (1956) and SAVAGE (1958) have been the primary engineering guidance by which many coastal engineering structures in the United States have been designed. WALTON and AHRENS (1989) and WALTON *et al.* (1989a,b) relooked at the early monochromatic wave forcing runup data of SAVILLE (1956) and SAVAGE (1958) as well as limited data sets from other countries and developed a simplified uniform methodology for assessing adequacy of structures against overtopping during design storm scenerios. Only a limited amount of this data pertains to mild slopes consistent with typical natural beach slopes.

In more recent times, both VAN DORN (1976) and GUZA and BOWEN (1976) have addressed the physics of wave runup in limited laboratory testing using monochromatic wave forcing. Laboratory testing of wave runup using irregular wave forcing is limited to mostly site specific studies, although MASE and IWAGAKI (1984) and MASE (1989) in Japan made a generic set of small scale wave runup tests and produced empirical curves for predicting wave runup on smooth linear slopes utilizing a power law relationship. WALTON (1992b) has reassessed and provided a simplified method for predicting the statistics of wave runup based on the data set discussed in MASE and IWAGAKI (1984) and MASE (1989).

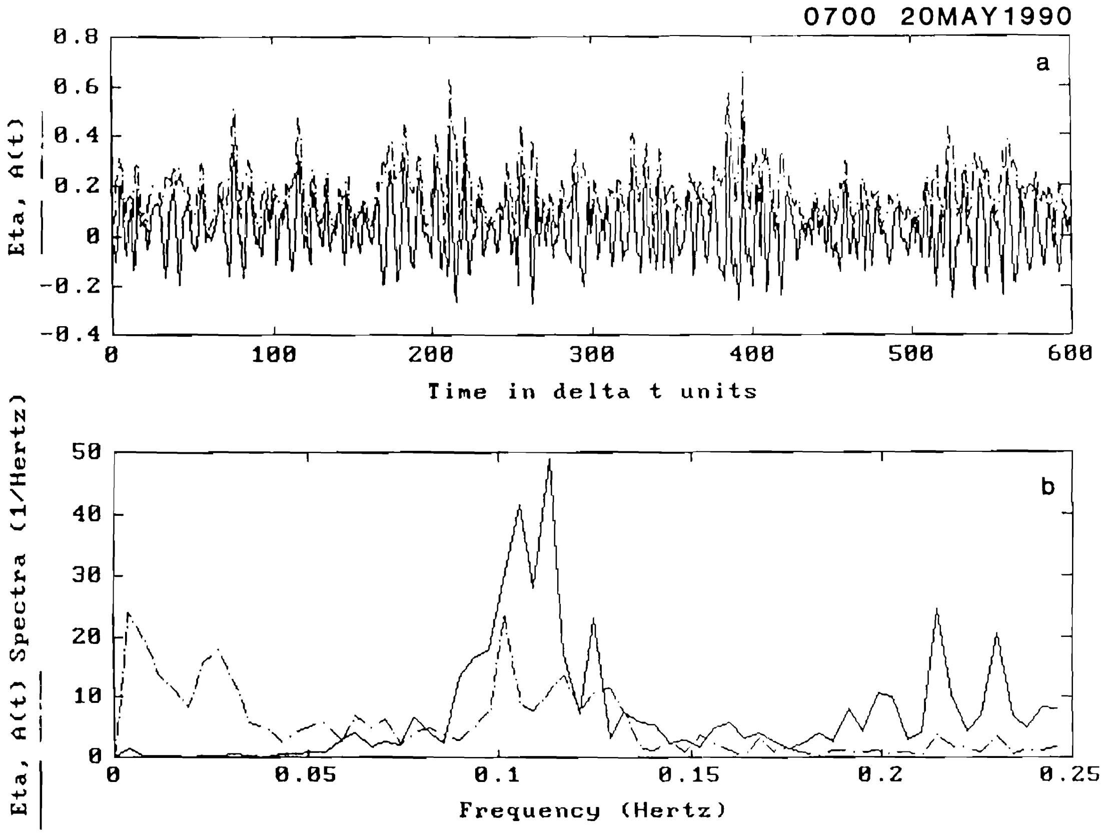

Figure 1. (a) Offshore water level, amplitude envelope. (b) Offshore water level spectra, envelope spectra.

On prototype natural beaches runup data is even more sparse. BRADSHAW (1980) discusses the importance of both the level of wave energy and the beach morphology in the type of runup spectra observed on natural beaches. GUZA and THORNTON (1982), GUZA *et al.* (1984), HOLMAN and GUZA (1984), HOLMAN and SALLENGER (1985), and HOLMAN (1986) have summarized field data results for estimating significant swash height on a limited number of beaches. Considerable scatter in various statistics of the runup process has been noted by these authors in the cited references. Both GUZA and THORNTON (1982) and GUZA *et al.* (1984) note the fact that increasing levels of incident wave energy on dissipative beaches lead to increasing levels of low frequency energy in the runup spectra. SAWARAGI and IWATA (1984) discuss results of field and laboratory tests and the coincidence of the runup oscillations at frequencies corresponding to those of the incident wave envelope developed by connecting the maximas of the incident water level. WALTON (1992a) presented a pragmatic approach to prediction of wave runup based on limited data from a field runup experiment with runup measured and analyzed via video camera as per U.S. ARMY CORPS OF ENGINEERS (1990). NIELSEN and HANSLOW (1991) present the most comprehensive set of field data to date along with an approach to estimating a probability distribution for wave runup on a given beach. Practical problems with the method exist, though, due to: (1) estimation of the empirical distribution function of runup (which is really not an empirical distribution function of runups since the total runup count is unknown and is substituted for by a much larger number of waves), and (2) beach dependent regression coefficients which vary by a factor of 2–3 for the same beach.

An all comprehensive methodology for prediction of wave runup on a prototype "natural" beach is still non-existant which attests to the difficulty of the wave runup problem and the high cost of

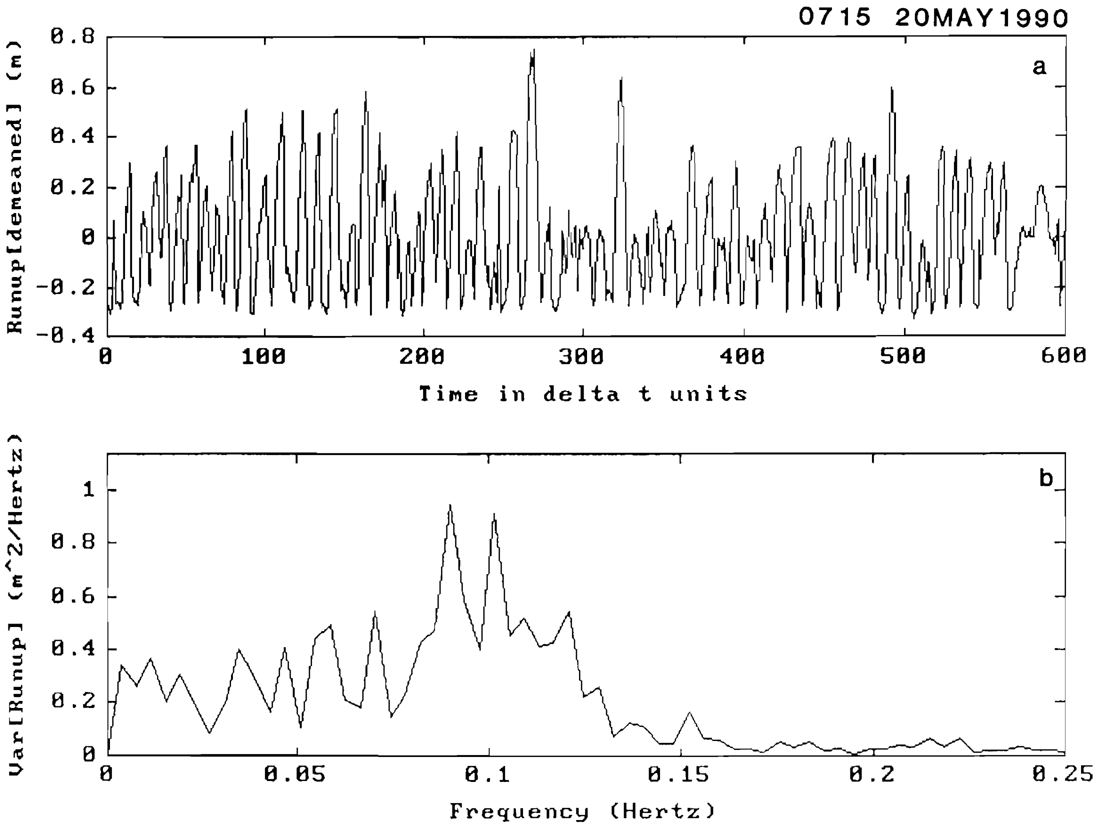

Figure 2. (a) Runup (swash) level. (b) Runup (swash) spectra.

obtaining quality prototype data on wave runup and the offshore wind wave forcing function. The present paper is a basic look at a field data set on wave runup that was collected at a site on the East Coast of the United States. It is hoped that the analysis presented here will shed some light on the runup estimation problem.

STUDY LOCATION

The site at which offshore wave information and beach profile information was collected is Ocean City, Maryland, where the first phase of a major beach nourishment project was completed in October 1988. Beach characteristics of the pre and post fill project are discussed in ANDERS and HANSEN (1990). The beach in the study area does not normally exhibit an offshore bar. During the study period the beach directly landward of the offshore sensor site had a relatively steep nearshore slope of approximately 1:15 out to about 5 meters of water depth (see Figure 9) and a mild offshore slope approximately 1:150 in the deeper water offshore. The grain size characteristics of the beach show predominantly quartz sands ranging from 0.15–0.3 mm in grain size (see ANDERS and HANSEN, 1990). During the pre and post nourishment phases of this beach nourishment project a bottom resting tripod containing a pressure sensor and a bidirectional orthogonal axis electromagnetic current meter was collecting wave and current data at sampling intervals of 1 second with continuous data records of 17 minutes every 1 to 4 hours. The tripod location was approximately 900 meters offshore of the mid portion of Ocean City beach (83rd Street) in approximately 10.8 meters of water. The pressure sensor on the tripod was 0.20 meters above the sea bed while the current sensor ball on the tripod was 0.46 meters above the sea bed. One channel of the bidirectional current meter was directed in the onshore-offshore direction while the other channel of the current meter was orthogonal to the

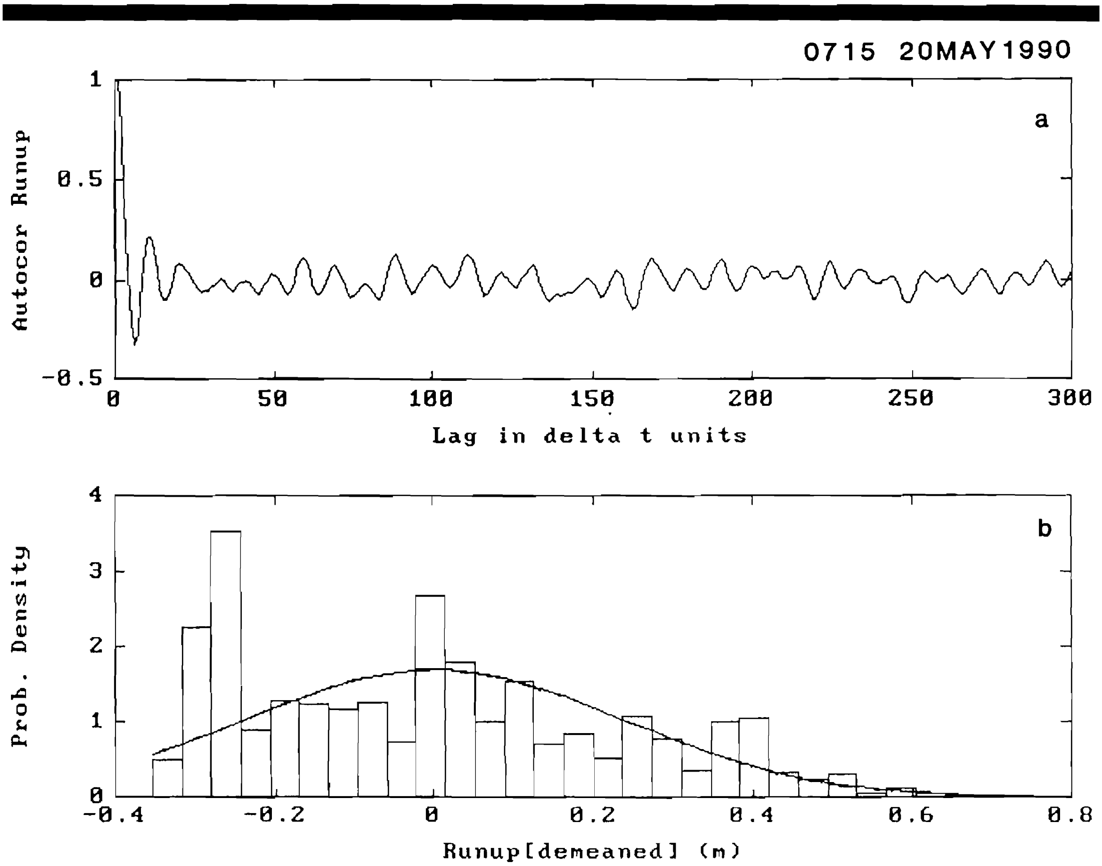

Figure 3. (a) Runup (swash) autocorrelation. (b) Runup (swash) probability density.

first channel and oriented in the longshore direction.

During the period 18–23 May 1990, a black and white video TV camera was installed on the balcony of the 14th floor of a high rise hotel fronting on the beach. The camera was situated so as to provide a good oblique (almost vertical) view of the beach where wave runup was observed. The wave runup video measurements were taken along a Corps of Engineers monumented beach profile line which was surveyed just prior to and throughout the period of the video measurements. Painted steel pipes and wooden stakes were placed through the beach profile as well as perpendicular to the profile for horizontal and vertical coordinate location of the runup transect in the video image system. The pipes were accentuated for the video camera by placing tires around each marker. Video measurements of the wave runup were made at four hour intervals during daylight hours and syncronized (approximately) to times when wave data was being collected offshore. All video imaging was aquired on SVHS tapes and an SVHS recorder.

The data discussed in this paper is data collected on May 20, 1990. Significant wave height and peak wave period during the analysis were 0.8 to 1.0 meters and 8.5 to 9.5 seconds respectively. Breaking waves during this period were of a plunging type. During the runup analysis periods, analysis of wave direction was made to assess the importance of any strong wave directionality. The method of wave direction analysis for this site is discussed in GROSSKOPF (1981). The predominant wave direction for the records analyzed was approximately perpendicular to the beach. The predominance of waves in a cross shore direction during the discussed study period was confirmed by comparison of the alongshore velocity variance to the cross shore velocity variance. In the cases analyzed, the ratio of the alongshore velocity variance to the cross shore variance

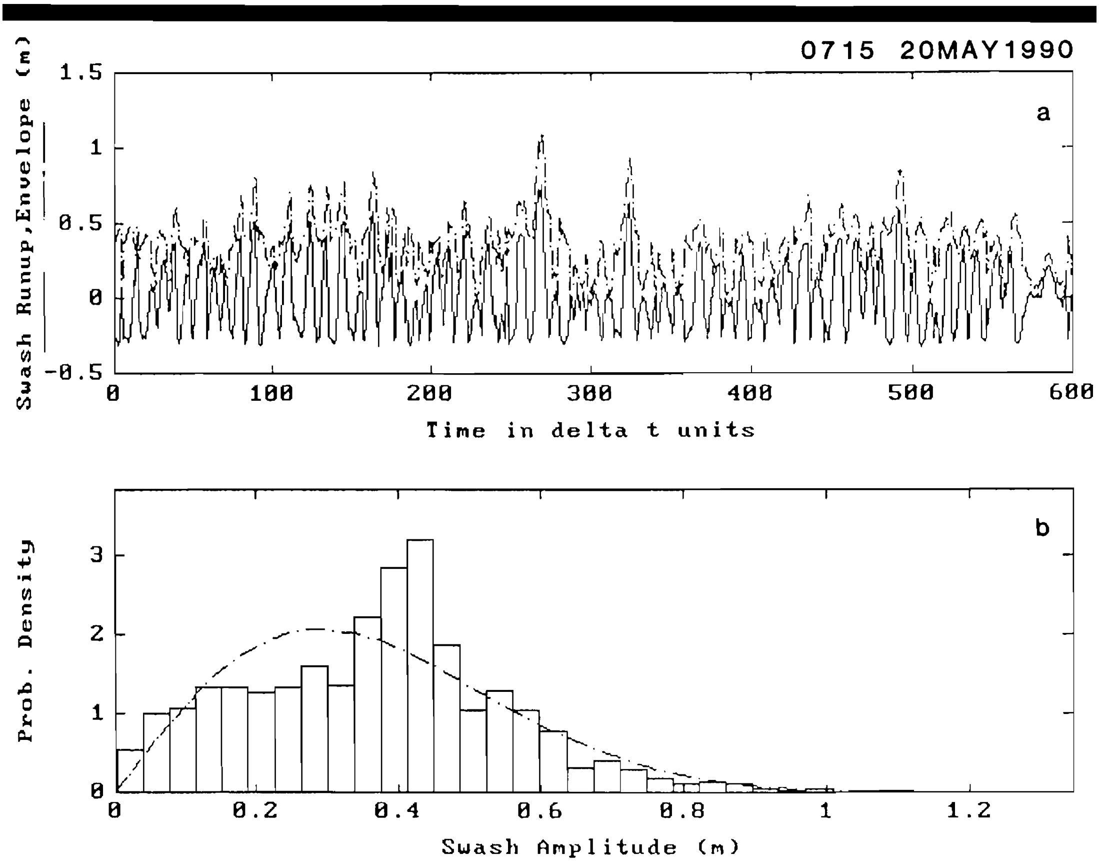

Figure 4. (a) Runup (swash), swash amplitude envelope. (b) Runup (swash) amplitude probability density.

was less than 20% and mean alongshore directed velocity at the gage site was less than 0.15 m/sec.

ANALYSIS OF DATA

The pressure transducer collected continuous pressure data at sampling increments of 1 second for 17 minutes six times per day. Standard linear wave theory (DEAN and DALRYMPLE, 1984) was used to compute the pressure response factor and transfer function for inversing the pressure record to obtain the surface wave elevation time series. A high frequency cutoff of 0.25 Hertz was utilized in analysis to prevent signal contamination by noise from the pressure response factor inverse transform.

Figures 1a and 5a provide the water level time series and wave amplitude (envelope) time series for time periods 0700 and 1500 (EST) on 20 May 1990. The wave amplitude time series $A(t)$ was found via wave envelope analysis utilizing Hilbert transform techniques. Assuming that sea surface elevation $\eta(t)$ is a realization of an ergodic Gaussian process, it can be defined in the following manner:

$$\eta(t) = \sum_{m=1}^{N} A_m \cos(2\pi f_m t + \theta_m) \qquad (1)$$

where N = number of discrete Fourier components (amplitudes), A_m = amplitude of mth component, f_m = frequency of mth component, and θ_m = phase of mth component (assumed random and uniformly distributed over a 2π interval). BENDAT and PIERSOL (1986) define the Hilbert transform of $\eta(t)$ as follows:

$$\hat{\eta}(t) = \sum_{m=1}^{N} A_m \sin(2\pi f_m t + \theta_m) \qquad (2)$$

which is basically Eq. 1 shifted by $\frac{\pi}{2}$. They also define the analytic signal as:

$$z(t) = \eta(t) + j\hat{\eta}(t) = A(t)\exp j(\theta(t) + \phi) \qquad (3)$$

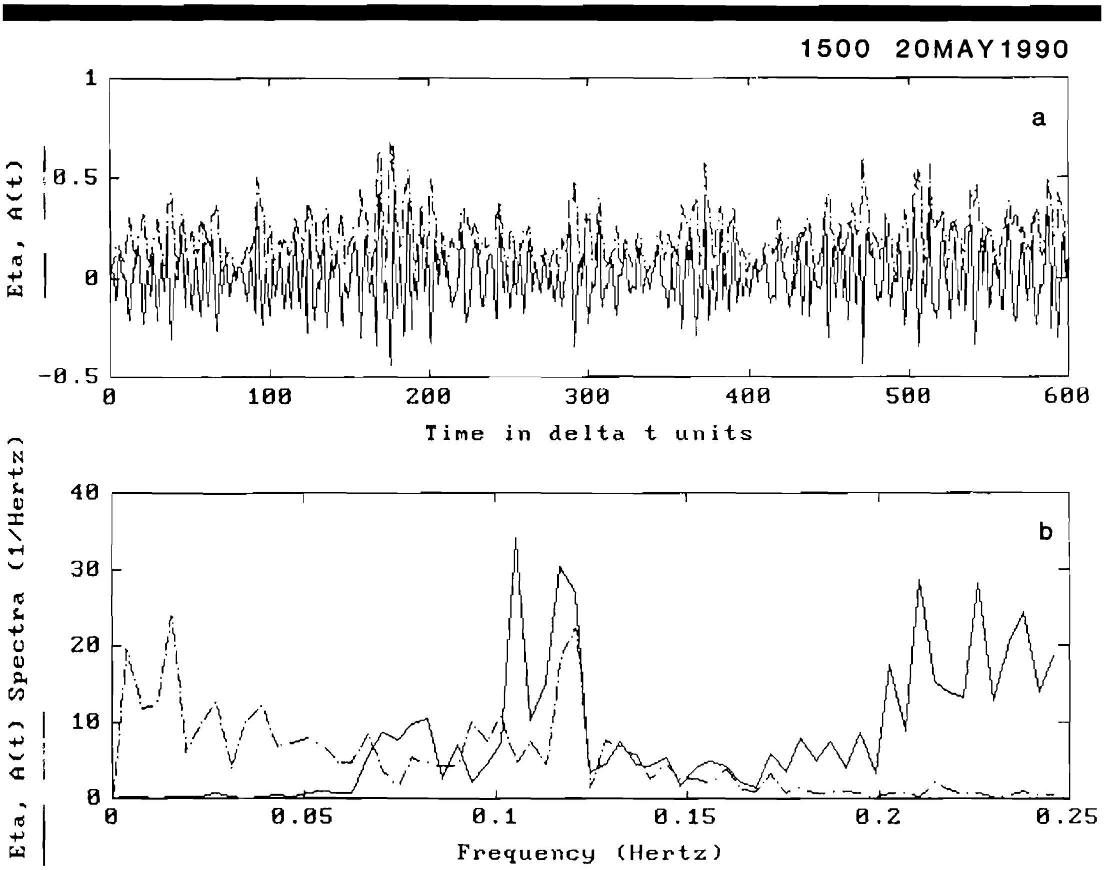

Figure 5. (a) Offshore water level, amplitude envelope. (b) Offshore water level spectra, envelope spectra.

where $j = \sqrt{-1}$, $A(t)$ = amplitude of the envelope, and $\theta(t) + \phi$ = phase angle. The instantaneous function of wave amplitude $A(t)$ is then defined as follows:

$$A(t) = \sqrt{\eta^2(t) + \hat{\eta}^2(t)} \tag{4}$$

To calculate the instantaneous function of wave height using Eq. 4 requires the Hilbert transform of $\eta(t)$ from Eq. 2. The most efficient means of calculating the Hilbert transform is via the frequency domain method as discussed in BENDAT and PIERSOL (1986). This approach was utilized in the following calculations.

Wave and amplitude (envelope) spectra for the same time periods are shown in Figures 1b and 5b. The wave and amplitude (envelope) spectra are computed by detrending and block averaging with 16 degrees of freedom (BENDAT and PIERSOL, 1986). Both wave and envelope spectra were normalized by their respective variances.

The optical video runup image (30 frames per second) was transformed to a vertical runup signal (1 sample per second) via a computer video image reduction system at the U.S. Army Corps of Engineers Coastal Engineering Research Facility (CERC-FRF) at Duck, North Carolina. The image transformation methodology involves internal transformation of the coordinate geometry of the runup site which must be input to the transformation program via physical surveying of established points on the beach that are also in the video image of the runup measurement site. The system is basically similar to that discussed in U.S. ARMY CORPS OF ENGINEERS CETN II-23 (1990). The runup time series consist of 17–34-minute records with runup frequency sampling of 1 Hertz. The runup consists of a mean component measured from still water level (often referred to as the setup), and a dynamic component (oscillation from the mean), typically referred to as the swash.

Figures 2 and 6 show the runup (swash) time

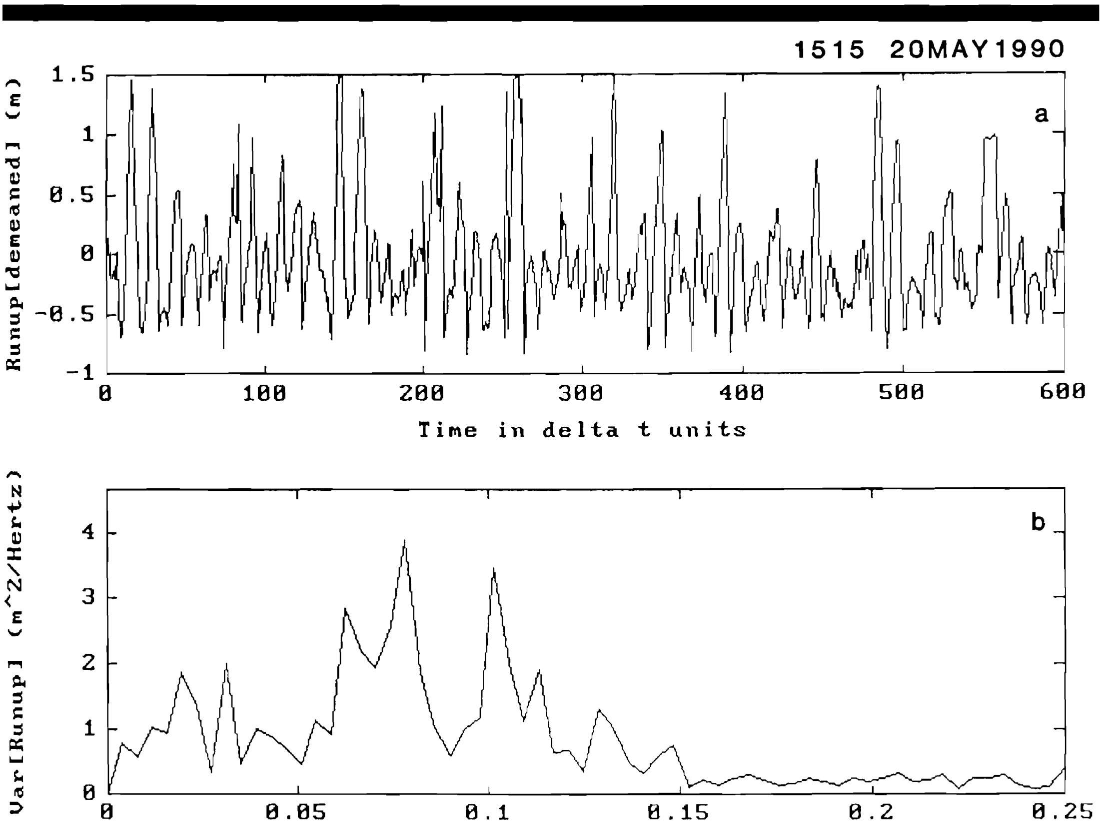

Figure 6. (a) Runup (swash) level. (b) Runup (swash) spectra.

series and runup (swash) spectra for the two time periods. As can be seen in the figures, the runup (swash) spectra have energy content at both the incident wave predominant frequency and at the longer modulated wave amplitude envelope frequency, although the high frequency incident energy ($\geq$ 0.2 Hertz) appears to be totally absent in the runup spectra.

The runup (swash) autocorrelation is shown in Figures 3a and 7a for the two time periods documented. It is apparent in these figures that no strong cyclic low frequency ($<$ incident wave period) is apparent in the runup signal autocorrelation. In fact, the fluctuations with lag $\tau \geq 15$ appear to fall well within the noise catagory $\rho(\tau)$

$$\leq \pm \frac{2}{\sqrt{n}} \approx 0.12 \text{ for a 95\% confidence interval as}$$

per BENDAT and PIERSOL (1986), where $\rho(\tau)$ is the autocorrelation and n = number of lags utilized in the sample computation. The swash autocorrelation plots thus reconfirm the swash spectral plots in that the swash energy at low frequency is broadbanded and not peaked (*i.e.* cyclic).

The measured probability density of the demeaned runup (swash) water level is provided in Figures 3b and 7b where the probability density is in the form of a histogram with the computed (via method of moments) estimate of the corresponding Gaussian density curve superimposed. As can be seen from the figures, a Gaussian assumption for the swash level is not unreasonable. A Chi squared test of the assumed distribution at an 80% level of confidence suggests that the null hypothesis of Gaussian swash levels not be rejected.

Figures 4a and 8a provide the runup (swash) water level and the swash amplitude (envelope) function as discussed before. The probability density of the swash amplitudes is provided in Figures 4b and 8b where the probability density is in histogram form and a theoretical Rayleigh proba-

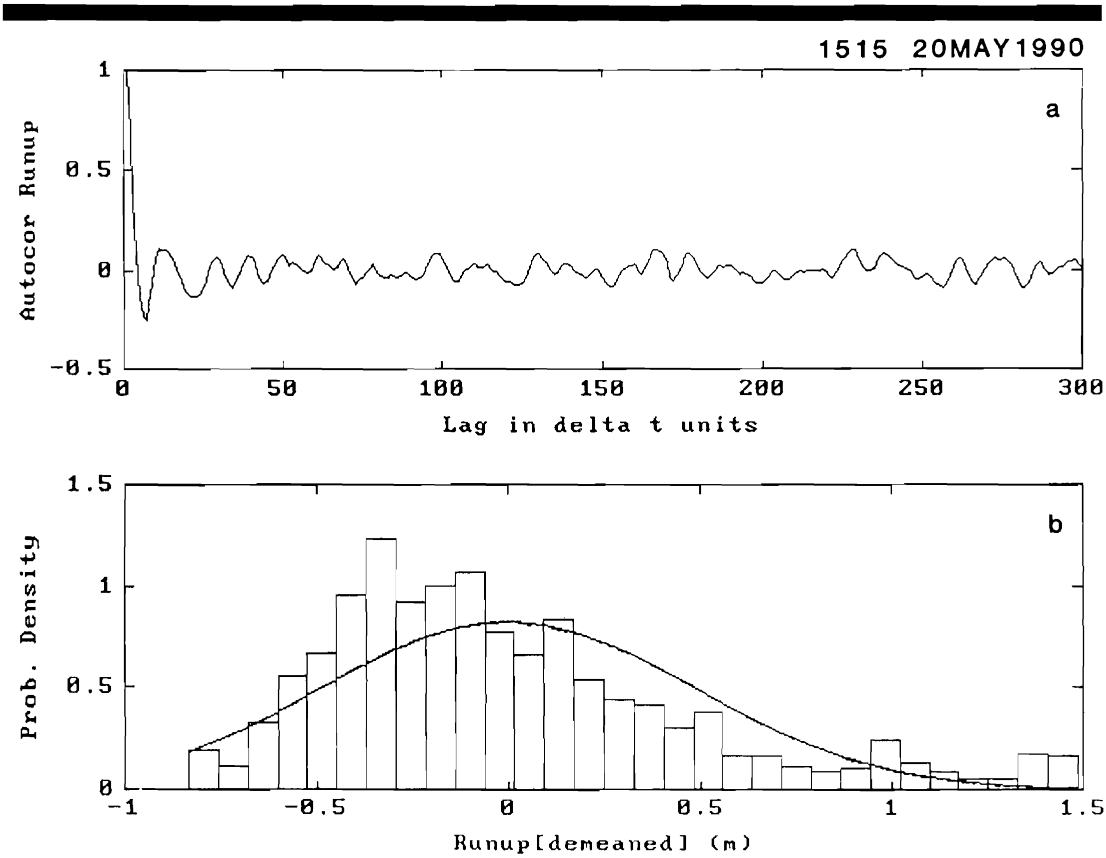

Figure 7. (a) Runup (swash) autocorrelation. (b) Runup (swash) probability denity.

bility density function curve is superimposed. The Rayleigh density function was fit to the measured data via the method of moments. A Chi squared test of this assumed distribution at an 80% level of confidence again suggests that the null hypothesis of Rayleigh distribution swash amplitudes not be rejected.

CONCLUSIONS

Although only two data intervals are provided herein, the general findings in this field data set were similar for most all data recording periods. General statements that apply to the entire data set include the following: (1) Autocorrelation plots of the wave runup (swash) time series show no significant cyclic low frequency (< incident wave period) wave runup activity in this data set. Low frequency or long period in this context must be related to the series length analyzed. In accord with statistical practice (BENDAT and PIERSOL, 1986), significant autocorrelations have lags less than the length of the series divided by 5 (*i.e.* 1,024/5 ≈ 200 sec); hence, periods longer than 3 minutes (6 minutes in the 34-minute runup records) would not be capable of being assessed in the present data. (2) Significant wave runup energy is found at both the incident and broad band modulated frequencies in the present data sets. This is suggestive that at least part of the runup series may be driven by the offshore groupiness of the waves, although no definitive conclusions can be reached in the present limited analysis. (3) Wave runup (swash) water levels may be treated as Gaussian to a first approximation, at least in the present data set. In a similar result, the wave runup (swash) amplitudes may be treated as Rayleigh distributed to a first approximation, at least in the present data set.

It is worthwhile to note that flooding typically is a problem concerned with water level excursion as opposed to amplitude excursion. In this regard, most practical engineering attempts at describing

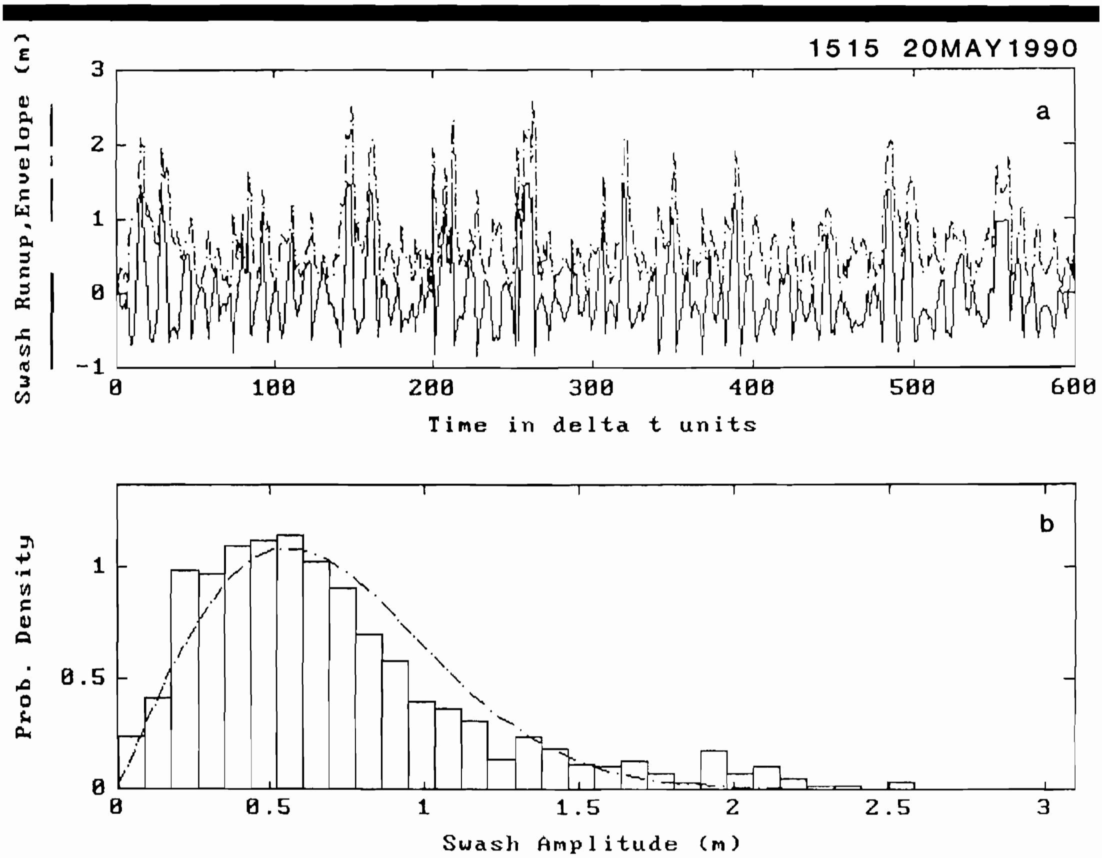

Figure 8. (a) Runup (swash), swash amplitude envelope. (b) Runup (swash) amplitude probability density.

and predicting wave runup probability distributions for beaches (or mild slopes) have addressed the amplitude problem (*i.e.* MASE, 1989; WALTON, 1992a,b; NIELSEN and HANSLOW, 1991). With the ability to obtain prototype real time runup data via video camera imaging, future studies will provide increased emphasis on the runup water level excursion statistics.

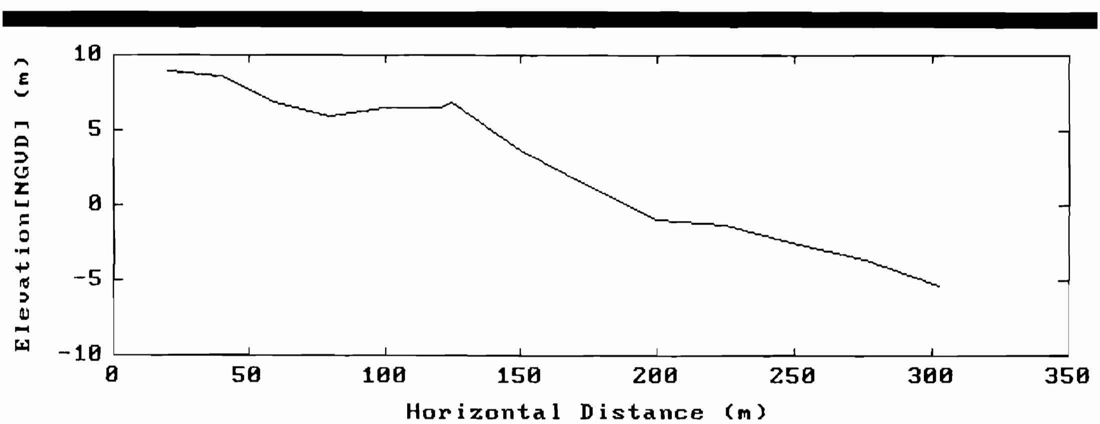

Figure 9. Beach profile at video transect.

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