Proceedings of a Workshop on COASTAL SEDIMENT TRANSPORT

with emphasis on the National Sediment Transport Study

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Sea Grant College Program
University of Delaware

Proceedings of a Workshop on

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with emphasis on the
National Sediment Transport Study

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*Although not appearing on the program, this written project summary was submitted for inclusion in the Workshop proceedings.

FOREWORD

An improved basis for predicting longshore sediment transport is of concern to coastal engineers and scientists in advancing their capability to understand shoreline phenomena and in carrying out designs which will perform in the manner anticipated and with a reasonable amount of maintenance. The quantitative understanding of sediment transport processes has been elusive and today must be considered rudimentary in many respects. Aside from the major problem of quantifying the total transport in terms of wave and current parameters, numerous other problems must be resolved to provide a rational understanding of this process. These include: the transport partition into bed and suspended load for various particle sizes and wave characteristics; the distribution of sediment transport across the nearshore; the effects of rip currents on onshore-offshore transport; the modifying action of a longshore bar on transport; the mobilizing effect of onshore waves with no longshore component combined with relatively small oblique waves as compared to the effect of the oblique waves acting alone; and many other problems of equal importance. The present lack of understanding of these processes has been due, in part, to the difficulties of conducting accurate measurements in the energetic and abusive surf zone environment. Therefore, development of methodologies and instrumentation for surf zone application is essential to the understanding and quantification of these processes.

The impetus for the present workshop was that, under the aegis of the Sea Grant National Project legislation, there existed the possibility of a broad based and coherent effort to be directed toward the nearshore sediment transport problems noted above. The intent of the workshop was, through state-of-the-art summaries, to provide an update and forum for discussion for the Workshop Participants and those reading the Proceedings. It is hoped that this Workshop and those to follow will contribute to more effective investigations leading to an improved rational understanding of nearshore transport processes.

The support for this Workshop by the University of Delaware Sea Grant Program and Department of Civil Engineering is greatly appreciated. The continuing interest of the National Sea Grant Program in this problem area has been instrumental not only in this Workshop but in previous informal meetings that have contributed to the development to date of plans toward a national investigative effort.

Robert G. Dean

NATIONAL SEDIMENT TRANSPORT STUDY--A POTENTIAL SEA GRANT NATIONAL PROJECT

Arthur G. Alexiou

National Sea Grant Program Office

Among the innovations in the National Sea Grant Improvement Act is a provision which authorizes the Secretary of Commerce to make grants up to 100 percent of the total cost of projects, if they address specific national needs or problems with respect to ocean and coastal resources. This is in contrast to the 66-2/3 percent maximum permissible for conventional Sea Grants. One of the main reasons for this new authority stems from the realization that it is unrealistic to expect local matching funds to be made available to support an initiative of major significance having national rather than local impact. Another reason is recognition that talent exists throughout the Sea Grant network that can make a major contribution to national needs and this new authorization would make it possible for teams of investigators to address such problems under the more appropriate funding structure the authorization provides.

Since the appropriations act under which Sea Grant is currently operating was passed before the authorization, no funds were made available for national projects. Availability of such funds in the future will depend on how successful NOAA is in the supplemental budget request process.

Thus, we stand betwixt and between. We have an authorization, but no guarantee of any funding. Obviously, major projects are not turned on and off at the drop of a hat. A great deal of preparation is necessary and since the future of a national projects program under Sea Grant depends upon what we get started initially, it behooves us to prepare with great care.

Under these circumstances, we, in the National Office, plan to pursue the national projects concept along either path, i.e., with or without special funding. If new funds are appropriated this year, it is our intention to issue a set of formal guidelines and priorities, and through a competitive selection process award substantial grants for several national projects funded fully with Federal dollars. On the other hand, if such funds are not forthcoming, we will proceed prudently with one, more-modest grant, that has all the attributes of a national project, except that one-third matching will be required. Clearly, this first project must be an outstanding one.

We believe the National Sediment Transport Study embraces the ingredients we are looking for. It addresses a solid problem that is clearly defined. There is a good prognosis that concentrated effort will yield a definitive result in a reasonable, predictable time. Sea Grant has been supporting related research by investigators on an individual basis at Scripps, Delaware, Florida, and other places. They can make more effective contributions as part of a team. The proposed research is susceptible to being broken down into separate identifiable tasks if properly managed. We have an excellent group of advisors that will act as a steering committee. They include members from appropriate interests of the Federal Government, a state government representative, universities and industry.

Under the modest funding option mentioned previously, we expect to begin with a nucleus of investigators from Scripps, Delaware, and the California Department of Navigation and Ocean Development. This initial team will develop a comprehensive proposal with certain identified tasks, only a limited number of which will be initially pursued because of the present limitation of funds.

If additional funds are provided by the Congress later on, then the project can be expanded and the door will be open for additional participation if it makes good sense. Proposals to undertake new tasks will be entertained by Sea Grant which will rely on the steering committee. Where appropriate, the steering committee will make recommendations to the National Sea Grant Office to embrace any new research in the overall study.

The mechanism integrating the research tasks will depend upon a strong management scheme, which generally means a strong manager. It will also require workshops at appropriate times and prompt dissemination of information and data.

I am personally optimistic about the future of national projects and am confident that the National Sediment Transport Study can become the precedent setter for more projects of this scope in the future.

OBJECTIVES, SCOPE AND FORMAT OF NATIONAL SEDIMENT TRANSPORT STUDY

R. J. SEYMOUR

The overall objective of the National Sediment Transport Study (NSTS) is to perfect relations for the prediction of sediment transport by waves and currents in the nearshore environment. Initially, this program will deal with the problems of sediment transport along straight coastlines. Detailed studies of the mechanics of water-sediment interaction will be coordinated into a working model, and tested by two or more "large scale" field experiments along the coast of the United States.

Results of the study will provide coastal engineers with a model that will allow useful prediction of the magnitude and direction of sediment transport under waves and currents on relatively straight sections of coastlines. This model will utilize measurement schemes that provide the necessary data in a practical and economical fashion. To develop a model which will have sufficiently universal applicability, it is necessary to understand the physics of sediment motions under a range of conditions. Therefore, intermediate objectives of this program are to characterize those physical processes and parameters that have significant effects on sediment motion. Only those investigations that continue to demonstrate this significance will be pursued under the study.

Program Organization

The National Sediment Transport Study is proposed as a multi-university program under the sponsorship of the National Sea Grant Program with cooperation by the Coastal Engineering Research Center, the Navy Postgraduate School and the California Department of Navigation and Ocean Development. Other interested State and Federal agencies will be invited to cooperate. Research will be funded by means of supplementary grants through the State Sea Grant Programs.

The direction of the program will be through a Steering Committee with membership from the sponsoring agencies and from the universities. The Steering Committee will: 1) review all proposals for research under this program and advise the National Sea Grant Office on their acceptability, both on technical merit and on meeting the objectives of this study; 2) sponsor conferences and workshops to acquaint the academic and engineering communities with this program and to report on progress; 3) review each research project annually for progress toward pertinent objectives and advise the National Sea Grant Office on the advisability of continuing the effort for the following year; 4) arrange for and coordinate the writing of several authoritative survey papers in areas relevant to program objectives; 5) assume the responsibility for editing a synthesized final report which will be in the form of an engineering-oriented handbook for applying the results of the study to actual field conditions; and 6) assume responsibility for organizing and conducting a series of workshops, short courses and other appropriate training activities for the purpose of transferring the knowledge accumulated to those responsible for designing, building, reviewing plans for and approving coastal engineering works.

The individual investigators or research teams will:

1) submit proposals for research under NSTS through the appropriate State Sea Grant office;

2) undertake approved research leading to an understanding of certain elements of the sediment transport phenomena, as outlined in the objectives, and/or;

3) participate in the large scale field experiments which will provide the data to develop and to test the physical and engineering models for sediment transport; and,

4) in addition to normal reports and publications, agree to the synthesis of all findings under this study, with appropriate acknowledgment, into a single report or reports under the general editorship of the Steering Committee.

STATUS OF SURF ZONE SEDIMENT TRANSPORT RELATIONS

by

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A large number of relations for the prediction of the longshore transport of sand in the surf zone have been reported in the literature (e.g., Das, 1971; King, 1972). A formula for predicting longshore transport from wave energy flux has been in general use in the United States since 1947, when it was first suggested in Wave Report 68 of the Scripps Institution of Oceanography that the energy flux of waves should be related to the littoral drift of sand. Following this report the concept was applied extensively by the Los Angeles District, U. S. Army Corps of Engineers, where it was referred to as the "littoral drift factor". The relation was generally given by the Corps of Engineers (Eaton, 1951) as

$$Q = k(ECn)_\infty \frac{s_\infty}{s_b} \sin\alpha_b \cos\alpha_b \qquad (1)$$

where Q is the "littoral drift factor", assumed to be proportional to the longshore volume transport rate of sand, ECn is the energy flux of the waves $E = \frac{1}{8} \rho g H^2$ is the wave energy per unit surface area, ρ is the density of water, g is the acceleration of gravity, H is the wave height, C is the phase velocity of waves, Cn is the group velocity, the subscripts "∞" and "b" refer to waves in deep water and at the breakpoint respectively, s_∞/s_b is the change in energy per unit length of wave crest due to wave refraction, α is the angle the wave ray makes with an orthogonal to the beach, and k is a dimensional factor of proportionality with units of $ft^3\ lb_f^{-1}$ when Q is in $ft^3\ sec^{-1}$ and ECn in $ft\ lb_f/(sec\ m)$.

The formula was first applied to rough estimates of wave energy and littoral drift in the field by Watts (1953b) who decided that Q was proportional to $P_\ell^{0.9}$, where $P_\ell = (ECn \sin\alpha \cos\alpha)_b$. Caldwell (1956) reviewed Watts' data, and data from Anaheim Bay and concluded that $Q \sim P_\ell^{0.8}$. Also Manohar (1962) summarizing field and laboratory data found $Q \sim P_\ell^{0.9}$.

This relation was first placed in dimensionally correct units by Inman and Bagnold (1963) who expressed the empirical relation as

$$I_\ell = K'(ECn \sin\alpha \cos\alpha)_b \tag{2}$$

where I_ℓ is the immersed weight longshore transport rate, and K' is a dimensionless constant, first evaluated from field measurements by Inman, et al (1968) as 0.7, and later from additional field measurements as equal to 0.77 by Komar and Inman (1970) as shown in Figure 1. The "at rest" volume transport rate of sand Q (e.g., $m^3\ sec^{-1}$) is converted to immersed weight transport rate I_ℓ (e.g., newton sec^{-1}) by the relation

$$I_\ell = (\rho_s - \rho)\, gN_oQ_\ell$$

where ρ_s is the density of the sand (e.g., kg m^{-3}), and, N_o is the "at rest" volume concentration of sand.

Inman and Bagnold (1963) also developed a theoretical model for predicting longshore transport of sand, given as

$$I_\ell = K''(ECn)_b \frac{v_\ell}{u_m} \tag{3}$$

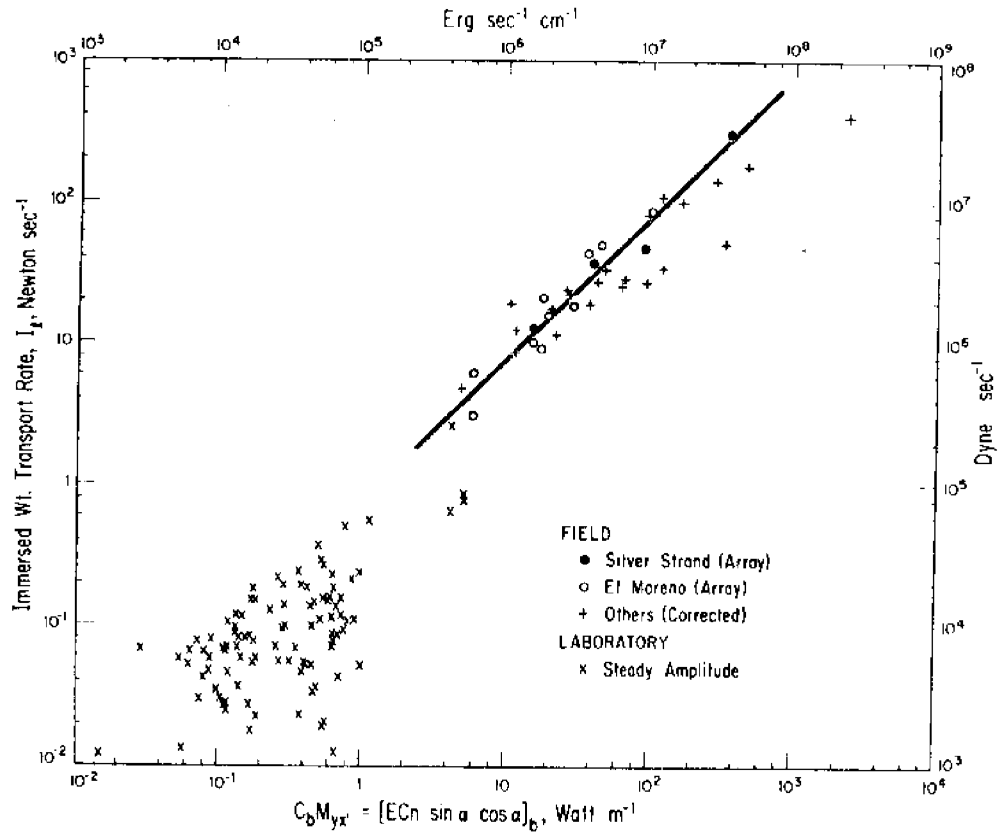


Figure 1. The immersed weight longshore transport rate of sand as a function of the longshore transport parameter (ECn sinα cosα)$_b$ = $C_bM_{yx'}$. The wave data from directional "arrays" is from Komar and Inman (1970), other data are given in Inman, et al (1969), and other field data has been corrected from significant wave height to rms wave height.

where v_ℓ is the velocity of the longshore current, u_m is the maximum horizontal component of the near bottom orbital velocity at the breakpoint, and $K'' = 0.28$ is a dimensionless constant evaluated from field measurements by Komar and Inman (1970).

The U. S. Army Corps of Engineers still recommend using relation (1) for predicting the longshore transport of sand. Over the years the recommended value of k has varied by more than a factor of four. When placed in the dimensionless form of relation (2) for purposes of comparison, the various values recommended for K' are: 0.2 (Savage, 1959; BEB, 1961); 0.4 (CERC, 1966); 0.7 Inman, et al, 1968); 0.77 (Komar and Inman, 1970); and 0.8 (CERC, 1975).

Longuet-Higgins (1972) points out that the term $ECn \sin\alpha \cos\alpha$ is not the longshore component of energy flux (or power) as previously claimed, since energy flux $E\vec{C}n$ is a vector and its longshore component is simply $ECn \sin\alpha$. The full relation $(ECn \sin\alpha \cos\alpha)_b$, because of its common occurrence in longshore transport relations (e.g., equations 1, 2, 4) is preferably termed the "longshore" sand transport parameter". This parameter is most conveniently expressed as $(ECn \sin\alpha \cos\alpha)_b = C_b M_{yx}$, where $M_{yx} = En \sin\alpha \cos\alpha$ = constant is the onshore flux of longshore directed momentum per unit length of shoreline, and n is that portion of the wave energy moving forward with phase velocity C. M_{yx} is conserved from deep water to the breakpoint of the waves, and is therefore a constant for any given wave condition. It has also been shown that M_{yx} is the driving term for the longshore current, $\bar{v}_\ell$ (Bowen, 1969; Longuet-Higgins, 1972).

It is of interest to note that the only evaluation of relations (2) and (3) that compared simultaneous measurements of wave energy flux and sand transport is the study by Komar and Inman (1970) and this is based on only 14 data points. Further, in no case was suspended load measured, nor was there any attempt to consider the actual dynamics in the surf zone where the transport was occurring.

There are a number of other derivations for the longshore transport of sand that warrant investigation (e.g., Dean, 1973; Madsen and Grant, 1976). In particular the first reference is of interest because it gives an expression for the longshore transport of suspended sand in the surf zone. Dean (1973) assumes that suspension is the dominant mode of transport and derives a relation that can be written as

$$I_\ell = K''' \frac{(\gamma_b g H_b)^{1/2}}{c_f W} \tan\beta \, (ECn \sin\alpha \cos\alpha)_b \cdots \tag{4}$$

where $\gamma = H/h$, H is the wave height, h is the water depth, β is the beach slope, c_f is the coefficient of bottom friction, W is the mean fall velocity of the suspended sand, and K''' is a dimensionless constant that includes the fraction of wave energy consumed by falling sand grains. This relation has not been tested, but Saville (1969, p 5) states that "the majority of the sand in the prototype probably moves in suspension". Further, Fairchild's (1972) data indicates that the suspended concentrations he measured would account for the entire longshore transport of sand using the prediction relation given by CERC (1966; $K' = 0.4$).

On the other hand, Komar (1976, p 216), using field data from Watts (1953a) and Fairchild (1972), calculates that the suspended load transport would contribute only 20% (or less) of the total sand transport as given by relation (2) using a value of $K' = 0.77$. Komar also cautions against the use of data collected in close proximity to piers as giving artificially high values for suspended sediment. It is apparent that better understanding of the role of suspended sediment in the longshore transport of sand is crucial to any improvement of the prediction relation.

SUSPENDED SEDIMENT TRANSPORT

The possible importance of suspended load transport in the surf zone has been long recognized (Das, 1972), and a number of observations have been made and reported in the literature (e.g., Watts, 1953a; Homma, et al, 1958; Fairchild, 1972; Jensen and Sorensen, 1972; and, Basinski and Lewandowski, 1974). Unfortunately, none of these studies has been definitive in terms of aiding our ability to predict suspended load and to include it as a component of the general transport relation. The deficiencies of previous studies generally fall into one or both of the following categories: artificial contamination by the sampling device and/or its supporting structure; and, absence of simultaneous measurements of the velocity and turbulence fields that suspend and move the sediment. Many measurements have been made from ocean piers, but it is known that pier pilings induce bottom scour that may extend several pier widths from the pier. The effect this may have on suspension is not known.

Oscillatory motion over a plane sand bed may give rise to four distinct modes of sediment transport, which in the order of increasing intensity are: incipient motion following the onset of grain motion, a more intense but orderly sequence of motions over vortex ripples; sheet flow where several grain layers are in motion; and suspension. The first two are the most common modes of motion outside the surf zone, while the latter two are more common in the surf zone. It is of interest to note that swell waves obeying potential (inviscid) theory have no mechanism for suspending sand above the vortex sublayer which extends only about one ripple wavelength above the bed. This is shown graphically by the data of Homma, et al (1965) where all measurable grain "suspension" vanishes about 1 to 1.2 ripple wavelengths above the bottom (Figure 2). The wave motion above this point is irrotational and cannot suspend sediment. Of course, other sources, such as wind may induce turbulence, causing the water outside the surf zone to be turbulent, but this is not the norm. Many references to turbulence outside the surf zone are artifacts resulting from the scour, turbulence and suspended sand induced by the sampler. A cloud of suspended material placed in suspension by a sediment trap and then trapped by the return flow is shown in Figure 3, and resulted in the abandonment of this type of sampler.

On the other hand, the surf zone has a number of mechanisms that cause the water to be highly turbulent. The "turbulent boils" associated with plunging breakers, the white water of the cascading bore as it crosses the surf zone and finally the broad spectrum of sea level fluctuations (Figure 4) all contribute to turbulence. The measurements of Inman, et al (1971) show that the most pronounced turbulence occurs in the upper portions of the surf zone, and decrease near the bottom. Understanding of this stratification of turbulence in the surf zone will be of considerable significance to understanding surf zone suspension.

There are three general types of suspended samplers in use: in situ samplers that obtain a bulk sample of water and sand; filter systems that separate and retain the sand (Inman, 1948; Watts, 1954; and, Fairchild, 1972); and indirect measures such as gamma absorption, and light attenuation and the backscatter of light (Homma, et al, 1965; Horikawa and Watanabe, 1970; Basinski and Lewandowshi, 1974; and, Brenninkmeyer, 1974). A meter employing the combined principle of light attenuation and backscatter has been in use at the Scripps Institution of Oceanography for several years. As discussed above (Figure 3), artificially induced suspension has been a severely limiting factor in previous measurements, and one that now appears best overcome by using portable in situ

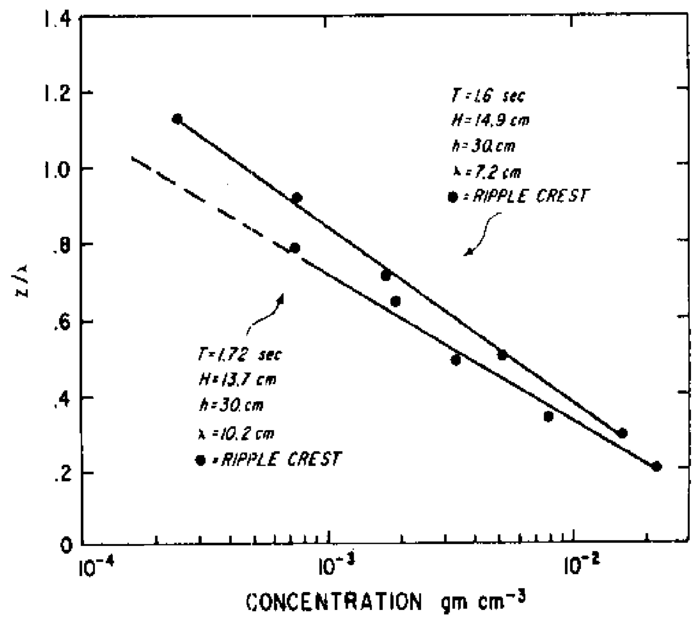
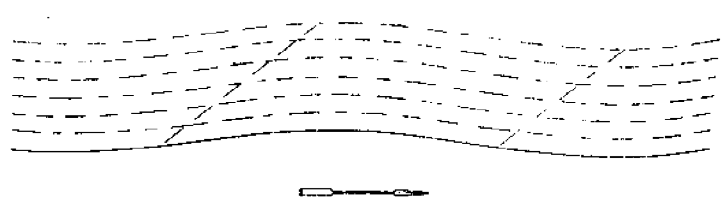


Figure 2. Showing that suspended sediment is limited to one ripple wavelength, λ, above a rippled sand bed under wave action. z is height above the bed, and $z/\lambda = 1.0$ is one ripple wavelength above the bed. Data from Homma, et al (1965).



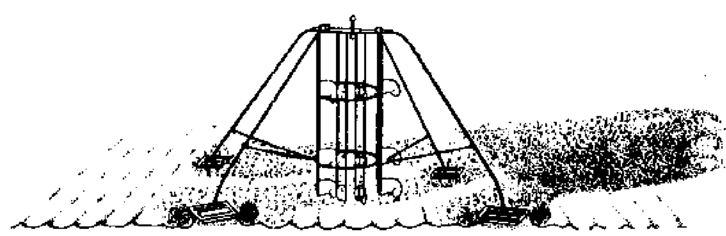


Figure 3. Artificially entrained sand caused by scour and turbulence around a sediment trap placed outside the surf zone. The return flow associated with the wave trough causes the cloud of sediment from the passage of the crest to be retained in the sample bags. It should be noted that naturally suspended sediment was restricted to a height of one ripple wavelength above the bottom, except in the vicinity of the sand trip. Use of this trap has been discontinued.

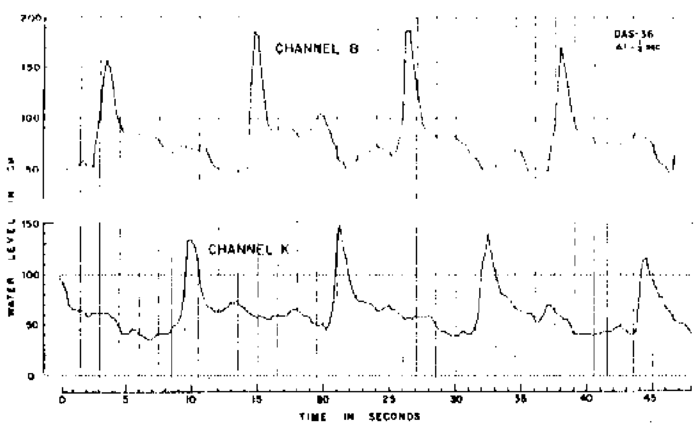


Figure 4. Time history of irregular water level as bores and reformed waves traverse the surf zone. Profiles of bores were measured by two digital wave staffs placed 30 meters apart in the surf zone. The true water depth at each staff is obtained by adding 72 and 69 cm to the water level scale of staffs B and K respectively (from Inman, 1967).

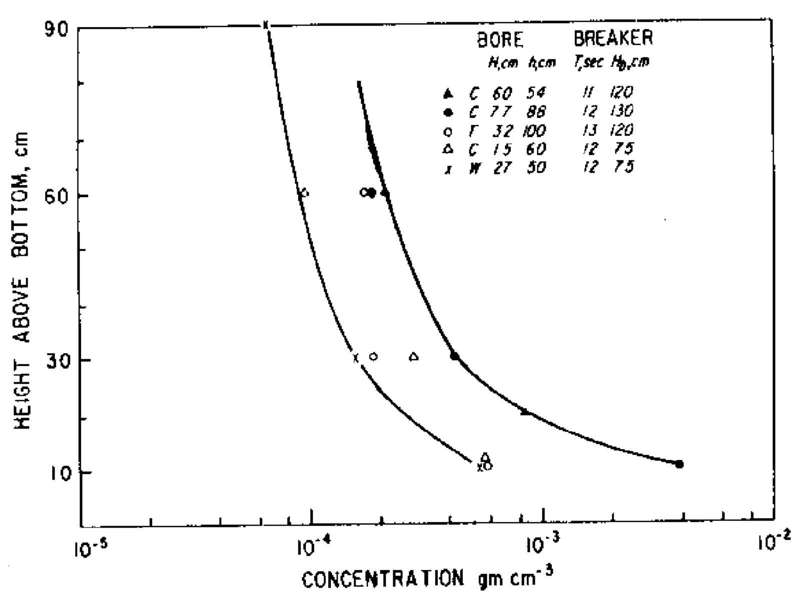


Figure 5. Measurement of suspended sediment in the surf zone using in situ samplers of one liter volume at distances of 10, 30, 60 and 90 cm above the bottom. Legend indicates position sampled relative to bore: C at instant of crest passage; T, trough following bore; and, W reformed wave.

samplers activated by swimmers to calibrate the results of mounted sensors. The results of in situ samplers used in the surf zone at Scripps are shown in Figure 5.

It is imperative that accurate measurements of suspended and bedload sediment transport be made in such a manner that they can be related to simultaneous measurements of the velocity and turbulent fields in the surf zone. Past experience has shown that accurate measures of sediment suspension in a rigorous environment such as the surf zone is a most difficult undertaking. However, accurate measurement of suspended load which can be related to the driving forces are essential to provide the necessary background for formulating a predictive relation for suspended sediment.

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ACTIVE SEA GRANT PROGRAMS CONCERNING

COASTAL SEDIMENT TRANSPORT

1976-77

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A highly probable result of this Workshop will be the reality of a National Sea Grant Project on sediment transport. Such an effort ultimately could involve, to a greater or lesser degree, the present Sea Grant network (people, groups, schools). Therefore, it would be instructive to review the objectives and scale of the present Sea Grant program in this general field.

Any synthesis or attempt to condense the thoughts of numerous people requires being arbitrary to a considerable degree. In that context, my interpretation of the major thrust of current (FY 1976, 1977T, and 1977) research in coastal sediment transport and its ramifications condenses all projects to have one or more of four major objectives: 1) to describe present conditions, 2) describe or determine how change takes place, 3) determine or describe when change takes place, and 4) what to do about that change. Efforts which address these objectives I choose to group into 6 specific and one catch-all categories briefly described in the following outline.

I. COASTAL MORPHOLOGY. The Game. Studies in this category are quite varied, ranging from research of a site specific nature which examine measures for the mitigation of shore erosion; others which provide comparisons between various shoreline types; and study of fluidization of sand as a way of reducing inlet shoaling rates. Studies of a more regional nature are examining changes in, and rates of change in, coastline configuration, evaluation of various litteral drift models, drift computation, rip currents, and barrier island evolution.

The Players. University of Maine: Fink; SUNY: Calkin; Delaware: Dalrymple, Dean, Kraft, Lai; University of North Carolina: O'Connor; University of South Carolina: Hayes; SUS Florida: Smutz; TA&MU: Morton; University of Wisconsin-Madison: Edil; Oregon State University: Komar; SIO: Inman.

II. WATER MOTION. The Game. Waves and currents are the fundamental topic under study where in some cases historical data is collated, "atlases" prepared, or wave refraction technology improved and applied. In other cases actual measurements are made of flow, tide, wind, and waves. Additionally, some projects direct effort toward the modeling of circulation of the water mass (velocity and direction in X, Y, Z) and modeling the advecting particles in that mass. Some studies are research in new technology; others are application of existing technology.

The Players. UNH: Celikkol; MIT: Conner; University of Rhode Island: Spaulding, White; Princeton: Mellor; VIMS: Goldsmith; University of North Carolina: Pietrafesa; University of Miami: Wang; TA&MU: Mungall; University of Michigan: Green; SIO: Inman, Isaacs; University of Hawaii: Lee, Bretschneider.

III. SEDIMENT TRANSPORT PREDICTION. The Game. Only one project is presently funded which has the sole purpose of developing, evaluating, and perfecting an analytical model for predicting sediment transport. This one is unique in its inclusion of expressions for unsteady flow and sediment grain size.

The Players. MIT: Madsen (see abstract by Madsen for details).

IV. BEACH NOURISHMENT. The Game: Many plans for shore erosion protection, with or without engineering structures, call for placement of large volumes of fill. Maintenance of beaches calls for periodic nourishment with sand. A potential source of sand is the seafloor offshore and another is inlet deltas. Several projects are searching for sources of sand, others are examining ways of managing that resource.

The Players. SUNY: Schubel; University of Wisconsin/Madison: Meyer; University of California/Berkeley: Berry; California State/ Northridge: Fischer.

V. STRUCTURES. The Game. Many structures emplaced to halt erosion of a coastal sector or stabilize an inlet have been unsuccessful to varying degrees and most are expensive to install and maintain. Present projects are directed toward inventory of types, evaluation of their effects upon coasts, and costs of installation of different structures. Several projects are researching types of floating breakwaters.

The Players. University of New Hampshire: Savage; University of Rhode Island: Kowalski; University of Georgia: Oertel; University of Michigan: Brater; Braden; University of Washington: Richey.

VI. INSTRUMENTATION. The Game. Support of instrumentation research is this year directed toward development of means for obtaining current measurements using a spar buoy which would provide a vector summed over the vertical length of the spar. Another project seeks to develop the spar as a directional wave gage.

VII. OTHER. The Game. Lumped into this imprecise category are several projects relating to legal and economics aspects of coastal sediment transport, (such as methods for zoning and costs of mitigating erosion); sea floor stability (instrument calibration/evaluation and pipeline stability); and vegetation as a shoreline stabilizer (plantings in diverse areas as dune, marsh, and dredge spoil islands).

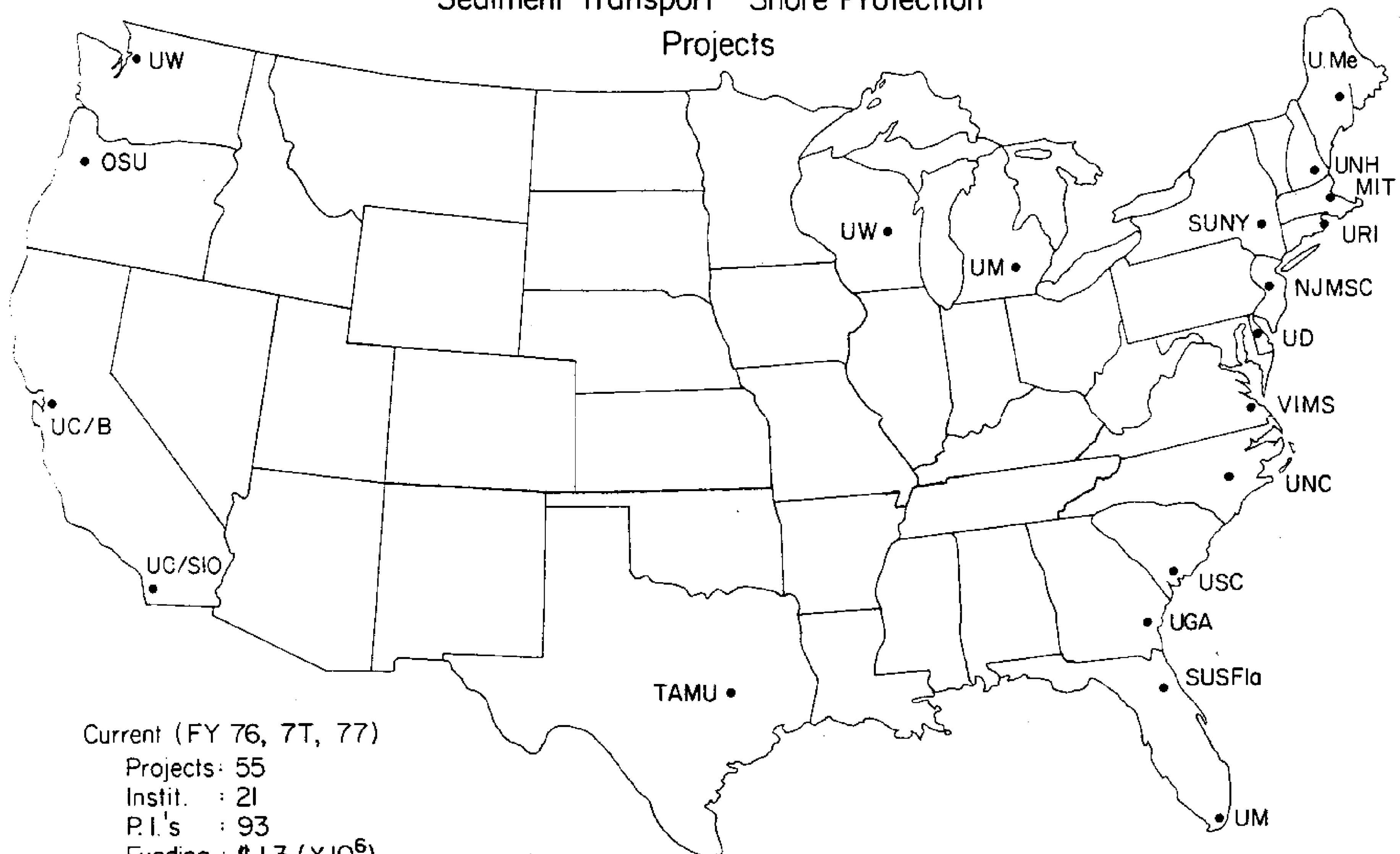
The Players. MIT: Baligh (seafloor stability); SUNY: Churchill (vegetation); University of North Carolina: Seneca (vegetation); SUS Florida: Maloney (legal); TA&MU: Herbich (pipeline stability); University of Michigan: Armstrong (mitigation costs); and University of California/Berkeley: Barbour (vegetation).

During introductory remarks to this gathering, Professor R. R. Rumer, Jr., Chairman of the Department of Civil Engineering, University of Delaware observed that erosion along coasts is a persistent problem. I agree, as I assume most persons would. However, it is more than just persistent; it is also a multiple coast problem. As this brief review of Sea Grant effort has pointed support is directed toward lacustrine, estuarine, and marine (Atlantic, Pacific, and Gulf) coast lines. A satisfactory managerial solution to the problem is a multiple discipline responsibility requiring input from engineers, scientists, lawyers, and economists. Considering the different bureaus at the state and federal level with interest and responsibility for different aspects of a solution of this problem, it is also a multiple agency responsibility.

Fig. 1 summarizes the scale of Sea Grant resources and the Sea Grant network presently being directed toward understanding coastal sediment transport and the problems created as a result of that phenomena. However, this present effort is directed toward solution of different aspects of the phenomena and usually of topics with priority established by local or regional conditions. Such an effort is naturally diffuse, and from a national standpoint, uncoordinated and without clear direction.

Clearly there is a base upon which to build a suitable national sediment transport program, and there is justification. Perhaps the time for a national project dealing with coastal sediment transport has indeed arrived despite our present lack of appropriations for projects of national scope.

Sea Grant
Sediment Transport - Shore Protection
Projects



Current (FY 76, 7T, 77)
Projects: 55
Instit. : 21
P.I.'s : 93
Funding : \$ 1.3 ($\times 10^6$)

Cummulative Funds for Active Projects \$ 4.1 ($\times 10^6$)

OSG
12/76
DBD

Fig. 1

REVIEW OF SEDIMENT TRANSPORT RELATIONSHIPS AND THE DATA BASE

by

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College of Marine Studies
University of Delaware

INTRODUCTION

Sediment transport processes in the nearshore zone are complex due to the turbulence generated by breaking waves, the extreme variability with time over one wave period of the bottom shear stresses, and other flow parameters, the bottom geometry and the partition of suspended and bed-load transport. These details are important to a rational quantitative description and understanding of the transport; however, this review paper will concentrate on the problem of relating the total longshore sediment transport to the characteristics of the incoming wave system. This review emphasizes only those data obtained from field programs.

GENERAL DISCUSSION OF THE PROBLEM

The noted complexity of the surf zone would suggest that a valid transport relationship should include a number of parameters as variables. In particular, the following parameters are expected to be relevant:

- Sand Grain Size (Fall Velocity)
- Magnitudes of Longshore and Onshore Energy Flux Components
- Wave Breaker Type
- Water Temperature
- Beach Profile Type ("Storm" or "Normal")
- Beach Slope
- Tidal Range
- Currents Generated by Mechanisms Other Than Waves

The earliest equation related the volumetric longshore sediment transport rate, Q_s, to the longshore component of energy flux, $P^*_{\ell s}$, i.e.

$$Q_s = K_1 P_{\ell s} \tag{1}$$

where K_1 was simply a proportionality constant and it is noted that this constant K_1 is not dimensionless. Based on considerations of energy dissipation, Inman and Bagnold (1966) developed the following relationship between the immersed weight transport rate, I, and the longshore component of energy flux

$$I = KP_{\ell s} \tag{2}$$

which has the advantage that K is dimensionless. The immersed weight and volumetric transport rates are related by

$$I = Q_s \rho g (S_s - 1)(1-p) \tag{3}$$

in which p is the in-place porosity of the sediment, S_s is the sediment specific gravity, ρ is the mass density of water and g is the gravitational constant.

It is noted that relating the immersed weight transport rate to the longshore energy flux has intuitive appeal; however, it can be shown that for bathymetry characterized by straight and parallel contours, the onshore component of the longshore momentum flux must exactly equal the total average longshore thrust occurring through shear stresses and may thus be a more appropriate forcing mechanism. Considering a strip of unit length (parallel to shore) with the strip extending from the shoreline to an infinite distance offshore, and assuming no wave energy dissipation sea-

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*$P_{\ell s}$ represents the longshore energy flux and not the longshore energy flux factor used in the Shore Protection Manual.

ward of the surf line, the total longshore thrust is

$$F = \text{total longshore thrust on a strip of unit length} = (S_{xy})_b \tag{4}$$

where

$$(S_{xy})_b = (E\ n \sin\alpha \cos\alpha)_b \tag{5}$$

and n is the ratio of group velocity to phase speed, C. It is noted that $(S_{xy})_b$ and $(P_{\ell s})$ are related through the phase speed at breaking, C_b, by

$$(S_{xy})_b = \frac{P_{\ell s}}{C_b} \tag{6}$$

One of the motivations for a concerted study of longshore sediment transport is the relevance to coastal engineering projects and also the changes in the recommended values of the constant K in Eq. (2) that have occurred with time. Table I shows recommended values in the U. S. Army Shore Protection Manual, and its predecessor, "TR-4-Shore Protection, Planning and Design" and preliminary results presented by Bruno and Gable (1976) of a field impoundment study carried out by the Coastal Engineering Research Center which indicates that further increases of up to 100% above the present coefficient value may be recommended.

TABLE I

VARIATION OF LONGSHORE TRANSPORT PROPORTIONALITY FACTOR, K

Source	Date	Coefficient K
U. S. Army Corps of Engineers, Technical Report No. 4	1966	0.48
U. S. Army Corps of Engineers, Shore Protection Manual	1973	0.88
Bruno and Gable (1976)	1976	≈1.8

FIELD MEASUREMENTS

Several field measurement programs have been conducted to develop data for the correlation of transport rate and longshore component of wave energy flux. These will be reviewed briefly below in chronological order.

South Lake Worth Inlet, Florida (Watts, 1953)

Longshore transport rates were based on quantities of sand transferred by a permanent sand bypassing plant on the north jetty of this inlet. The pressure drop of the bypassing pump was correlated with sand discharge through a series of surveys in a diked disposal area on the downdrift (south) side of the inlet. Thereafter transport rates were inferred from the (calibrated) pump pressure drop. The wave characteristics were based on: (a) the wave height measurements from a staff gage mounted on the South Lake Worth Pier some 10 miles to the north, and (b) visual observations of wave direction at the surf line as obtained from a vantage point approximately three miles north of the inlet.

Anaheim Bay, California (Caldwell, 1956)

Dredge material from the entrance to Anaheim Bay was placed on the downdrift (southeast) shore and repeated surveys of this area were conducted as the material was transported in a southerly direction. The changes in volume were interpreted as longshore transport rates and the longshore component of wave energy flux was based on (a) wave staff measurements from the Huntington Beach Pier, some six miles to the south, and (b) wave directions based on hindcasts and recognition of the sheltering by the offshore islands from waves originating from certain directions.

Cape Thompson, Alaska (Moore and Cole, 1960)

The growth of a spit and the associated waves were observed over a three-hour period. The spit volumes were measured and wave characteristics based on visual estimates.

Santa Barbara, California (Johnson 1952 and Galvin 1969)

These results are based on two different sources. The sediment transport data are developed by Johnson (1952) from impoundment and by-passing around the Santa Barbara breakwater. The longshore energy values were developed by Galvin (1969) and represent a combination of hindcast wave heights and wave directions selected to result in the maximum longshore energy flux and still be realistic.

Silver Strand, California and El Moreno, Baja California (Komar and Inman, 1969)

These data represent transport over fairly short time intervals as determined from sand tracer measurements and wave energy flux derived from an array of wave sensors. Sand transport volumes were usually determined over a fraction of a tidal cycle as the product of the width of the surf zone, the longshore displacement of the center of gravity of the tracer and the thickness of tracer movement. The latter quantity was based on observations of the depth to which a cylindrical "plug" of tracer had been eroded over the observational period. Values of this depth ranged between 2 and 10.5 cm.

Fernandina Beach, Florida (Thornton, 1969)

Mechanical sand traps were placed at discrete locations across the surf zone and used to trap the lower 20 cm. of the water column down to and including material in transit on the sand bed. The traps, which were operated from a pier, could be closed and the trapped material pumped up to the pier for measurement. Wave data (height and direction) were based on two wave gages located at the end of the pier.

FIELD DATA AND CONSISTENCY

General Data Characteristics

Table II summarizes some of the salient features of the 43 field data points of longshore sediment transport. The sediment sizes span a range from 0.18 to 1.0 mm, although there is only one data point for a sediment size greater than 0.60 mm. Much of the wave data is based on observations or hindcasts. The wave data reported by Komar and Inman (1969) and Thornton (1969) are the only sets for which the height and directional characteristics are based on measurements.

Evaluation of Data Consistency

There are several ways of evaluating the data consistency. Das (1972) has presented the field data which can be interpreted as I vs. $P_{\ell s}$ as presented in Figure 1.

A second approach is to consider the internal consistency of the data. The non-dimensional factor K relating the immersed weight transport rate, I, to the longshore wave energy flux has been defined as

TABLE II

SUMMARY OF FIELD DATA AVAILABLE FOR LONGSHORE TRANSPORT CORRELATION

Investigator	Location	Sediment Diameter (mm)	No. of Points	Methods		
				Transport	Waves	
					Height	Direction
Watts (1953)	South Lake Worth Inlet, Fla.	0.40	4	Calibrated Bypassing Pump	M	O
Caldwell (1956)	Anaheim, Calif.	0.40	5	Surveys	M & H	H
Moore and Cole, (1960)	Cape Thompson, Alaska	1.00	1	Measured Spit Growth	O	O
Johnson (1952) and Galvin (1969)	Santa Barbara, Calif.	0.20	5	Surveys	H	Max.
Komar and Inman, (1969)	Silver Strand, Calif.	0.18	4	Tracers	M	M
Komar and Inman, (1969)	El Moreno, Baja Calif.	0.60	10	Tracers	M	M
Thornton (1969)	Fernandina Beach, Fla.	0.20	14	Bed Load Traps	M	M

KEY

M = Measured, O = Observed, H = Hindcast, Max. = Direction Selected to be Realistic and Yield Maximum Possible Transport.

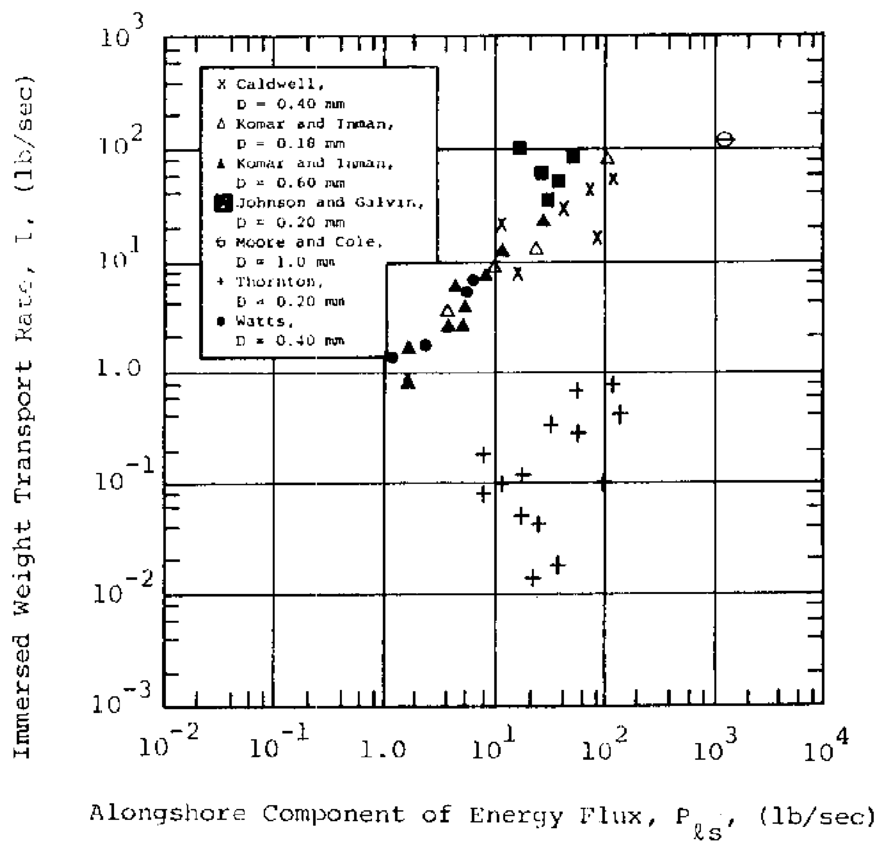

Figure 1. Correlation of Field Data of Immersed Weight Transport Rate, I, Versus Longshore Component of Energy Flux, $P_{\ell s}$.

$$K = \frac{I}{P_{\ell s}} \tag{7}$$

The mean, $\overline{K}$, standard deviation, σ_K, and percentage standard deviation (PSD) are presented in Table III for each of the seven data sets. The percent standard deviation is defined for each data set as

$$\frac{\sigma_K}{\overline{K}} \times 100\% = \frac{\sqrt{\frac{1}{I}\sum_{i=1}^{I}(K_i - \overline{K})^2}}{\frac{1}{I}\sum_{i=1}^{I} K_i} \times 100\% \tag{8}$$

If the quantity, K, was the same for all data in the set, the PSD would be 0, and a value of the PSD equal to or greater than 100% would indicate an extremely poor scatter about the average value.

SUMMARY AND CONCLUSIONS

Summary

Prior to discussing the results presented in Tables II and III, several comments are in order concerning the expected accuracy and variability for the various data sets.

First, all transport rates reported are based on measured quantities. The data of Caldwell, Moore and Cole, Johnson and to some extent, Watts, are based on surveys for which a reasonably high degree of accuracy should be attainable. The data of Komar and Inman are based on defining tracer displacement and those of Thornton on sediment trapped at several discrete locations across the surf zone. These two latter methods are expected to have a relatively high error due to the problems of disturbances to the sand bed by the sand traps (Thornton) and diffi-

TABLE III

SUMMARY OF CHARACTERISTICS OF "CONSTANT" K FOR VARIOUS FIELD DATA SETS

Investigator	Sediment Diameter (mm)	No. of Points	Characteristics of K		
			$\bar{K}$	σ_K	$\sigma_{K/\bar{K}}$ x 100%
Watts (1953)	0.40	4	0.899	0.107	19%
Caldwell (1956)	0.40	5	0.759	0.382	50%
Moore and Cole (1960)	1.00	1	0.246	--	--
Johnson (1952)	0.20	5	1.605	0.702	44%
Komar and Inman (1969) Silver Strand, Calif.	0.175	4	0.77	0.18	23%
Komar and Inman (1969) El Moreno, Baja Calif.	0.60	10	0.82	0.27	33%
Thornton (1969)	0.20	14	0.0475	0.0266	56%

$I = K P_{\ell s}$, I = Immersed Weight Transport Rate, $P_{\ell s}$ = Longshore Energy Flux of Waves.

culties of establishing the depth and width of movement of tracers as well as possible problems of interpretation associated with tracer burial and/or erosion. The data of Watts probably are affected by the short length of the updrift jetty which formed the trap from which the sand was pumped.

With regard to wave height, the data of Caldwell, Moore and Cole and Johnson are based on either observations or hindcasts rather than direct measurements. The wave directions reported by Komar and Inman and Thornton are determined from two or more wave sensors; all others are based on observations or other considerations. The wave directions of Komar and Inman should be considerably more accurate than those of other investigators as they were determined through a number (> 6) of wave sensors.

Finally, strictly from considerations of expected errors in wave and transport measurements, the values of the parameter K associated with the longer time intervals should exhibit less variability than those for short measurement intervals. One advantage of short measurement intervals is that wave and transport conditions should be reasonably constant.

A review of the results of the proportionality factor, K, as presented in Table III provides an indication of the uncertainties to be expected in the prediction of longshore sediment transport and the associated measurement problem to determine K. There is a much greater variability in the average K values as determined by different investigators than the individual values associated with a given data set.

The average K values range from 0.048 (Thornton) to 1.605 (Johnson). This represents a value of 33.5 for the ratio of the highest to the lowest average. Figure 2 presents a histogram of the various reported values in which each average value has been weighted by the number of points. One can question whether this range is due to measurement methodology/accuracy or is truly a result of the different proportionality factors for the various field conditions. As noted previously, it would seem that the sediment size, D, could be an important parameter in affecting the parameter, K. Figure 3 presents a plot of the average K values versus reported diameters. Associated with each data point are the initial(s) of the investigator(s) and the number of data points in the set. If the data of Thornton were not present and if the Moore and Cole K value is weighted equally with the other averages, it would appear that the constant K decreases with sediment diameter. If the Thornton data are included, it is difficult to discern a trend of K versus diameter. The percentage standard deviation for the various data sets range from 19% (Watts) to 56% (Thornton). This range of PSD for the individual data sets is surprisingly small in view of the variation in the average K values associated with the different data sets.

Conclusions

1) There are a relatively few (43) field values of reported longshore sediment transport and longshore energy flux.

2) The variability of average K values for the various data sets is quite large and ranges from 0.048 to 1.605. It is not known whether this range is due to transport/wave measurement errors or due to the true variation in the factor K.

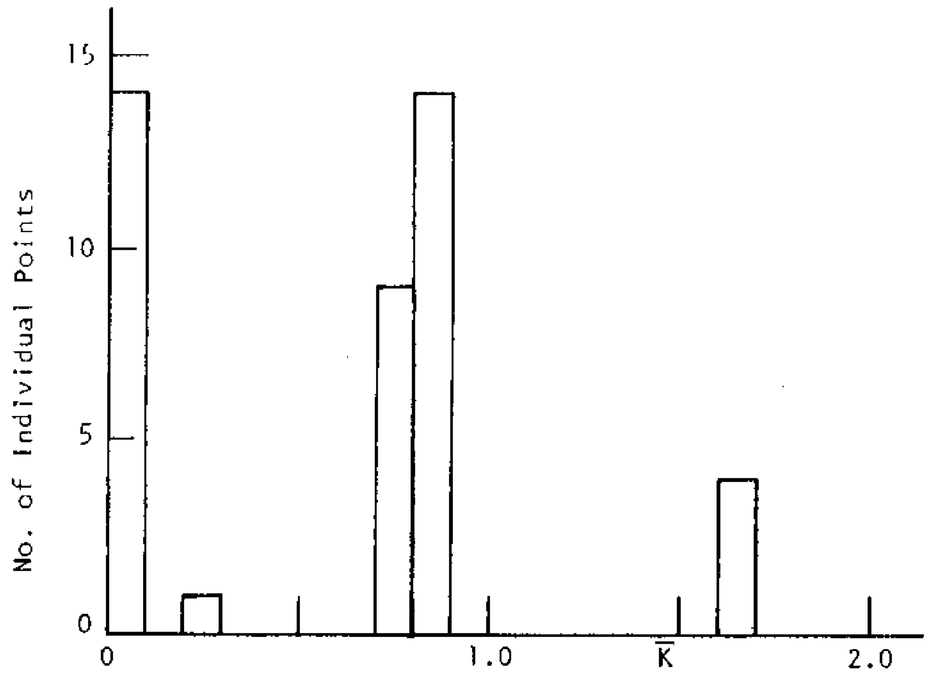

Figure 2. Histogram of Weighted Average, $\overline{K}$, Relating Longshore Energy Flux to Immersed Weight Transport Rate.

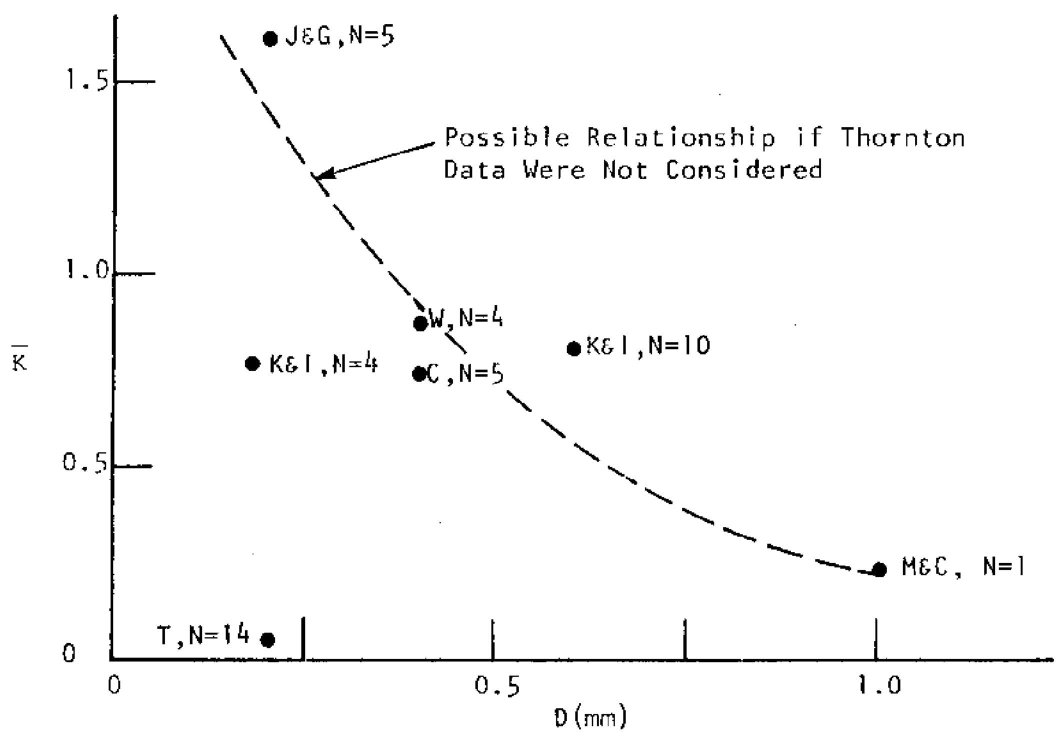

Figure 3. Plot of $\overline{K}$ Versus Diameter, D. Initials and Numbers, N, Refer to Investigators and Number of Data Points Comprising, $\overline{K}$.

3) The variability of the K values within the individual data sets is surprisingly small (ranging from 19% to 56%) compared to the range noted in Conclusion 2.

4) Based on the data, it is not possible to establish with confidence a variation of the parameter, K, with sediment diameter, D, or with other variables which would seem to be relevant.

5) There is a clustering of the proportionality factor, K, around approximately 0.8.

6) Despite the relative small scatter in the individual data sets, the uncertainties in the data are such that, even with exact wave data, it is believed that the longshore sediment transport can only be predicted within approximately -67% to +200%.

7) The distribution of sediment transport across the surf zone, of considerable importance to coastal engineering, is very poorly understood.

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REVIEW OF LONGSHORE CURRENT RELATIONSHIPS AND DATA BASE

by

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Longshore currents are an important forcing function of sediment transport along the coast. The sediments are stirred and set into motion by the waves impinging primarily perpendicular to the shore and the longshore current acts as the net transporting agent along the shore. At the last count there were at least 25 formulae that were available for predicting mean longshore current (see Table 1 for a partial list). In addition, there are at least six formulae for the prediction of variation of longshore current across the surf zone.

The various theories can be categorized in terms of the basic principle upon which the derivation is based. These are conservation of energy, mass, and momentum flux and empirically derived equations. The difficulty with applying conservation of energy principle is that only a very small percent of the energy is needed to drive the longshore current. The distribution of mass transport within the surf zone is very difficult to predict. Strictly empirically developed equations have led to erroneous implications. The most successful approach has been to solve the conservation of momentum flux. A review of the then existing theories, led Galvin (1967) to conclude that at that time there was no quantitative formula for predicting longshore currents.

The modern theories of longshore current are all based on the idea of conservation of momentum flux which was given impetus by the ideas derived by Longuet-Higgins and Stewart (1964) concerning the excess

TABLE I. LONGSHORE CURRENT FORMULAS

Authors	Mean Longshore Current, V	Formulation	Eq. No.
Putnam-Munk-Traylor (1949)	$[6.97g \frac{s}{f} \tan\beta\, H_b^2 \frac{\sin 2\alpha_b}{T}]^{1/3}$	Energy Conservation, Solitary waves	1
Eagleson (1965)	$[\frac{3}{8} g H_b \frac{\sin\beta \sin\alpha_b \sin 2\alpha_b}{f}]^{1/2}$	Momentum Conservation, Asymmetric-periodic waves	2
Putnam-Munk-Traylor (1949)	$\frac{A}{2} [(1 + \frac{4}{A} 2.28gH_b \sin\alpha_b)^{1/2} - 1]$ $A = 20.88 \frac{\tan\beta}{fT} \cos\alpha_b H_b$	Momentum Conservation, Solitary waves	3
Galvin-Eagleson (1965)	$gT\tan\beta \sin 2\alpha_b$	Mass Conservation	4
Inman-Bagnold (1963)	$2.31 \frac{\kappa \ell \tan\beta}{T} \cos\alpha_b \sin\alpha_b$	Mass Conservation, Rip currents included	5
Bruun (1963)	$C_f [\frac{0.95}{\sqrt{g\kappa}} H_b^{3/2} \frac{\tan\beta \sin 2\alpha_b}{T}]^{1/2}$	Mass Conservation	6
Bruun (1963)	$2.31 \frac{\kappa \ell \tan\beta \cos\alpha_b}{T}$	Mass Conservation, Rip currents included	7
Inman-Quinn (1951)	$[(\frac{1}{4A^2} + 2.28gH_b \sin\alpha_b)^{1/2} - \frac{1}{2A}]^2$ $A = 108.3 \frac{\tan\beta H_b \cos\alpha_b}{T}$	Empirical-based on momentum analysis	8
Brebner-Kamphius (1963)	$8.0 \sin^{1/3}\beta \frac{H_o^{2/3}}{T^{1/3}} [\sin 1.65\alpha_o + 0.1 \sin 3.30\alpha_o]$	Empirical-based on momentum analysis	9
Brebner-Kamphius (1963)	$14.0 \sin^{1/2}\beta \frac{H_o^{3/4}}{T^{1/2}} [\sin 1.65\, \alpha_o + 0.1 \sin 3.30\, \alpha_o]$	Empirical-based on energy analysis	10
Harrison (1968)	$0.241 H_b + 0.0318 T + 0.0374 \alpha_b + 0.0309 \tan\beta - 0.170$	Empirical-least square analysis	11
Komar, Inman (1970)	$2.7 u_m \sin\alpha_b \cos\alpha_b$	Empirical	12
Longuet-Higgins (1970)	$B \frac{\tan\beta}{c_f} \sqrt{gh_b} \sin\alpha_b \cos\alpha_b$	Momentum Flux	13

momentum flux due to the unsteady motion in waves which has been termed "radiation stress". A review of the momentum flux derivation of longshore current prediction is given below. The reason for reviewing this derivation is to consider the various assumptions that have been made in order to derive the predictive equations; the assumptions give an indication of the areas of deficiency in the formulation.

Derivation of the Longshore Current using Conservation of Momemtum Flux

The conservation of momentum flux is expressed by separating the total velocity into its wave induced, turbulent and mean contributions. The Naviar-Stokes equation is integrated over depth and averaged in time.

In order to solve for the energy distribution, it is first necessary to solve for the mean water level which results in a set-down outside the surf zone and a set-up inside the surf zone. This can be solved by examining the momentum flux equation perpendicular to the beach (x direction) as expressed by

$$\frac{\partial S_{xx}}{\partial x} = -gD \frac{\partial \overline{\eta}}{\partial x}$$

where S_{xx} is the excess momentum flux due to the unsteady wave motion, D is the total depth of water, and $\overline{\eta}$ is the set-down (up). Longuet-Higgins (1969) solved this equation and showed that outside the surf zone there is a set-down given by

$$\overline{\eta} = -\frac{H^2}{8} \frac{k}{\sinh 2kh}$$

and inside the surf zone there is a set-up

$$\overline{\eta} = K(D_b - D) + \overline{\eta}_b$$

where the subscript b refers to the point of wave breaking. The total water depth is given by the still water depth plus the set-up. The wave height inside the surf zone is expressed as proportional to the total water depth and the energy distribution across the surf zone can be stated.

The solution for the longshore current variation is derived by solving the momentum flux equation in the direction parallel to the beach (y direction) as expressed by

$$\underbrace{\frac{\partial S_{xy}}{\partial x}}_{\substack{\text{change in excess}\\ \text{momentum flux}}} = R_y = \underbrace{\overline{\int_{-h}^{\eta} \frac{\partial \tau_{xy}}{\partial x}\, dz}}_{\substack{\text{lateral shear}\\ \text{stress}}} - \underbrace{\bar{\tau}_{by}}_{\substack{\text{bottom shear}\\ \text{stress}}}$$

The expression states that if there is a <u>change</u> in the momentum flux then there must be a reactive force. Outside the surf zone the excess momentum flux due to the wave motion is essentially conserved. Inside the surf zone there is a change in the excess momentum flux due to wave breaking and hence a reactive force is induced. This force can be equated to the lateral shear stress acting over the water column minus a bottom shear stress. The excess momentum flux tensor (radiation stress) is given by

$$S_{ij} = \int_{-h}^{\eta} \overline{(\rho u_i u_j + p\delta_{ij})}\, dz - \frac{1}{2}\rho g D^2 \delta_{ij} \qquad i,j = 1,2$$

The velocities u_i and pressure, p, must be stated using appropriate wave theory.

Similar formulations of the variation of longshore current across the surf zone were derived by Bowen (1969), Longuet-Higgins (1970), Thornton (1970), Johnson (1974) at approximately the same time. The theories all assume steady state and bottom contours that are straight and parallel. The assumption of straight and parallel contours implies that the distribution of energy and momentum flux is spatially constant along the beach. All the above formulation used linear monochromatic waves to describe the excess momentum flux tensor (radiation stress). Conoidal wave theory has also been used in the formulation but without significant improvements with a considerable addition of complexity. Inside the surf zone there is no analytical statement of the distribution of energy across the surf zone. Hence, it has been assumed that the waves can be described as spilling breakers in which the wave height is approximated as being proportional to the depth of water; the portionality factor most often used is that given by solitary wave theory at breaking of 0.78. The wave speed is also assumed given by solitary wave theory. Hence, the waves inside the surf zone are an amalgamation of both linear and solitary wave theory. In the formulation for the variation of longshore current across the surf zone for plain beaches,it is necessary to include lateral stress in order to obtain reasonable velocity profiles. Hence, the present theories require the specification of bottom and lateral shear stress.

The most accepted formulation is that by Longuet-Higgins (1970) in that he obtained a closed form solution for the longshore current variation as given by

$$\begin{aligned} V &= Ax + B_1 x^{P_1} && 0 < x < 1 \\ &= B_2 x^{P_2} && 1 < x < \infty \end{aligned}$$

where x is a nondimensional distance and p is the function of the lateral shear stress. The mean longshore current, equation 13, was obtained by integrating across the surf zone assuming no lateral shear stress (Longuett-Higgins,1970).

A statistical distribution of linear waves has also been used by Collins (1972) and Battjes (1974) to describe the input of the unsteady motion to the radiation stress term. A comparison of models for two values of p in Longuet-Higgins formulation and the random sea model developed by Collins are shown in figure 1 compared with lab data obtained by Galvin and Eagleson (1965). Also shown is a no eddy diffusivity model. Outside the surf zone the momentum flux is essentially conserved. Inside the surf zone there is a change in momentum flux due to wave breaking. Without the addition of lateral or eddy diffusivity the resulting velocity variation across the surf zone is a discontinuity at the breaker line. The addition of a lateral shear stress term results in velocity profiles which appear much more reasonable. Reasonable velocity profiles were also obtained by assuming a statistical distribution of waves at breaking without having the recourse to lateral eddy diffusivity.

Bottom Shear Stress

The difficulty with the present formulations is that they require a bottom shear stress coefficient which is not very well specified and also a lateral eddy diffusivity coefficient which is even less understood. Most investigators assume a quadratic formulation for the bottom shear stress. The most easily applied formulation for bottom shear stress is that by Thornton (1970) and Longuet-Higgins (1970) as given by

$$\tau_b = \rho C_f |u_w| V$$

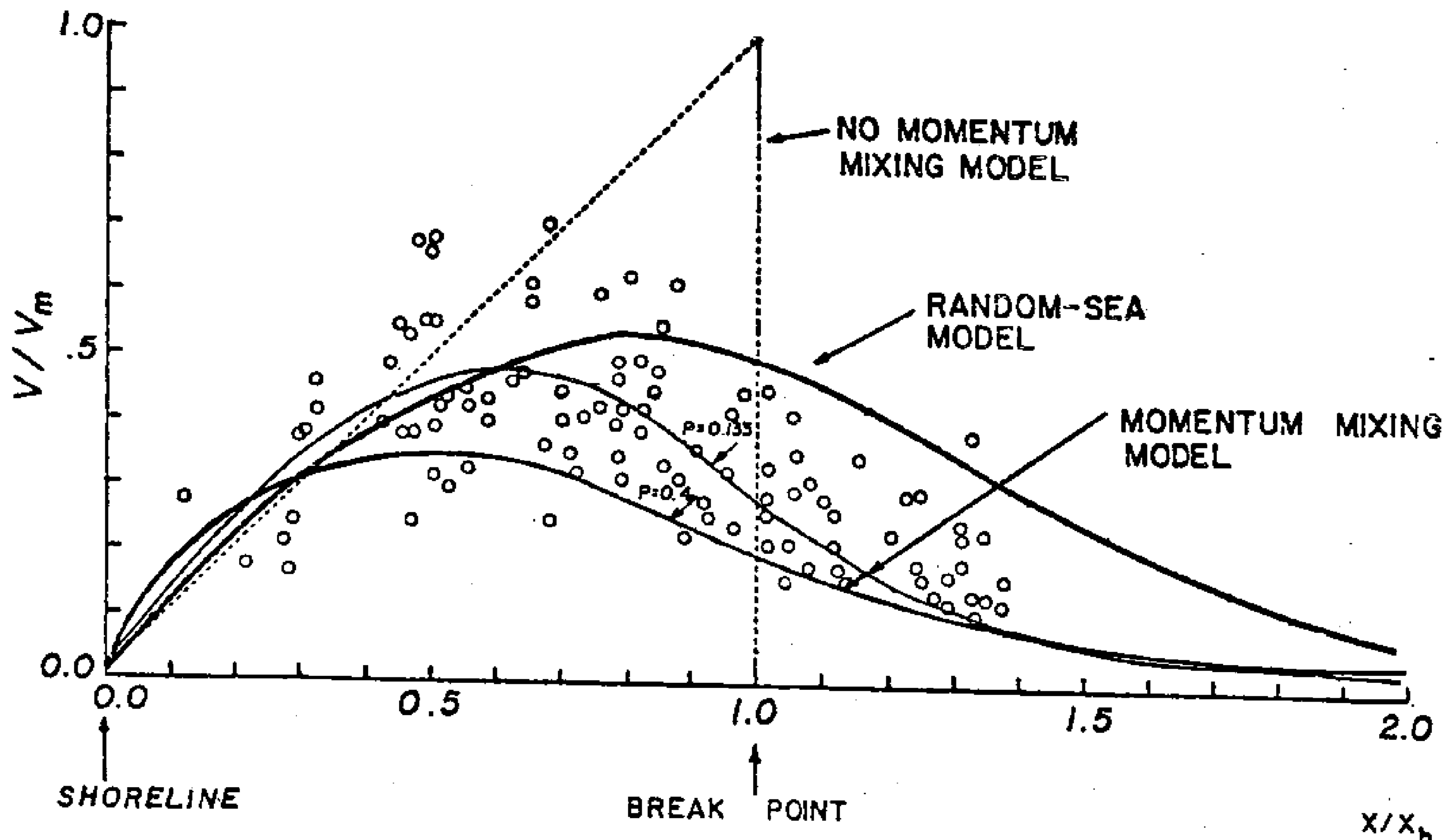

Figure 1 Various Longshore Current Models (from Sonu,1975)

where u_w is orbital wave velocity at the bed and V is the longshore velocity component. This formulation assumes that the wave-induced velocity at the bed is much greater than the longshore current velocity. This formulation linearizes the bed shear stress coefficient and hence is very convenient. Longuet-Higgins (1970) simply assumes that the bed shear stress coefficient was equal to 0.01. Thornton (1977) used a much more complicated formula for calculating the bed shear stress coefficient as derived by Johnson (1966) which is dependent upon the roughness at the bottom and the wave height. This formulation gives comparable values as those assumed by Longuet-Higgins. The difficulty in applying the available values for the bed shear stress coefficient is that they were obtained outside the surf zone or in the laboratory under rippled wave conditions indicative of conditions outside the surf zone (see Table 2).

Inside the surf zone the bed is fluidized. Sheet flow conditions exist at the bed; it would be expected that the bed shear stress coefficient would be different under such conditions. Hence, it is felt that at this time the bed shear stress coefficient is not well specified for conditions inside the surf zone. Another interesting formulation for bed shear stress was obtained by Komar and Inman (1970) who obtained an empirical expression for the longshore current velocity based upon their measurements of longshore current and sediment transport. Their formulation (equation 12) which is empirically derived, parallels the derivation by Longuet-Higgins (1970). In the derivation of Komar and Inman it is suggested that the ratio of $\tan \beta / c_f$ is equal to a constant, where β is the beach slope. The constant being equal to 2.7. Komar (1977) has applied this formulation to a number of data sets assuming that this ratio is a constant resulting in some good correlations for certain data sets

Table 2 Bottom Friction Coefficients Proposed by Various Investigators (from Sonu,1975)

Friction Coefficient	Wave Height (meters)	Wave Period (sec)	Test Conditions	Author
0.01	Arbitrary	Arbitrary	Shallow water steady state wave generation	Bretschneider (1954,a)
0.030-0.089	0.23-0.51	2.88-3.96	Gulf of Mexico; depths 3.4-5.2 m slope 0.00035-0.00-41	Bretschneider (1954,b)
0.030-0.040	2	10	Niigata, Japan; depths 2.25-2.75 m slope ~ 0.018	Kishi (1954)
0.01	5.4*	8.4*	Oscillating water channel, turbulent boundary layer	Jonsson (1966)
0.01 -0.40	0.002-0.100	0.88-2.58	Wave flume, laminar boundary layer	Iwagaki and Tsuchiya (1966)
0.03 -0.18	1.77-2.47	9.1-15.5	Hiyoshizu, Japan; depths 13-10 m slope 0.0060	Iwagaki and Kakinuma (1966)
0.03 -0.15	1.05-1.60	7.4-12.5	Takahama, Japan; depths 10-7 m slope 0.0057	Iwagaki and Kakinuma (1966)
0.09-0.50	$(UT/2\pi\eta) \approx 4 \sim 20$ U: nearbottom velocity η: ripple height		Wave flume study; derived from energy dissipation in sand ripple vortices	Tunstall and Inman (1975)

Note: *) equivalent values at 10 m depth.

and not so good for others.

Lateral Shear Stress

The lateral shear stress is equivalent to the integrated Reynolds stresses. An eddy diffusivity formulation is often utilized in order to linearize the equations of motion. Lateral shear stress expressed in terms of the eddy diffusivity is given by

$$\tau_\ell = -\int_{-h}^{\eta} uv\,dz = A\frac{dV}{dx} \qquad u\ell\frac{dV}{dx}$$

Thornton (1977) suggested that the lateral diffusivity coefficient could be calculated using the mixing length hypothesis analagous to Prandtl's original ideas. Battjes (1976) obtained a similar formulation through a much more rigorous and physical derivation of the examination of the turbulence problem. The formulations require that a characteristic velocity and length be specified. Characteristic length ℓ and velocity u scales utilized by a number of investigators is summarized in Table 3. It is safe to say that lateral shear stress is not very well understood within the surf zone. We can then ask ourselves how important is the lateral shear stress actually under field conditions. There are at least two effects that tend to offset the importance of the inclusion of lateral shear stress. First, the breaker location varies under actual conditions when there is a statistical distribution of breaking waves which results in effective momentum mixing. Secondly, actual profiles in nature tend to be parabolic in shape. Since momentum flux is proportional to the bottom slope, the momentum flux changes slowly for small bottom slopes near the breaker line.

TABLE 3. CHARACTERISTIC VELOCITY AND LENGTH SCALES FOR LATERAL SHEAR STRESS FORMULATION

	u	ℓ
BOWEN (1969)	CONST.	CONST
THORNTON (1970)	u_w	ξ_w
LONGUET-HIGGINS (1970)	c_b	x_b
INMAN,TAIT,NORDSTROM(1971)	H_b/T	x_b
JONSSON (1974)	u_w	a_w
HARRIS (1963)	H/T	H
BATTJES (1976)	C_b	h

Review of Data

The available laboratory data for longshore current are summarized in Table 4. This data has been reviewed by Galvin (1967). Available known field data is shown in Table 5. Most of the field data was taken by making visual observations. The wave height is generally estimated visually and would most closely correspond to a significant wave height. The breaker angle was generally determined by some mechanical device such as a protractor or sextant. Some investigations included variation of longshore velocity across the surf zone by making point measurements at various locations across the surf zone and some included variations along shore so that longshore variation could be taken into account. There appears at this time to be no single set of precise longshore current measurements in the field that have been under fairly ideal conditions in order to make a rigorous comparison with the available longshore current formulations.

Conclusions

At present we have a reasonable theoretical framework in which to make longshore current calculations. Deficiencies in application of the formulation are due to the following: (1) The coefficients of bottom shear stress and lateral eddy diffusivity are not well specified. Correct specification of the bottom shear stress coefficient is not only important in predicting the longshore current but also is important in specifying the bed load transport of sand. It is apparent that in order to have a better knowledge of the bottom shear stress coefficient more experiments are going to have to be made. (2) Breaker type should be taken into account in order to have a more realistic formulation of longshore current. This is because

TABLE 4. LONGSHORE CURRENT LABORATORY DATA

AUTHOR	NO. OBSERVATIONS	REMARKS
PUTNAM, MUNK, TRAYLOR (1949)	37	
SAVILLE (1950)	9	SAND BED
BREBNER, KAMPHUIS (1963)	141	
GALVIN, EAGLESON (1965)	38	

TABLE 5. LONGSHORE CURRENT FIELD DATA

AUTHOR	NO. OBSERVATIONS	REMARKS
PUTNAM, MUNK, TRAYLOR (1949)	18	
INMAN, QUINN (1951)	33	LONGSHORE VARIATION
MOORE, SCHOLL (1961)	44	
GALVIN, SAVAGE (1966)	5	
INGLE (1966)	11	SURF ZONE VARIATION
THORNTON (1969)	5	
DETTE (1972)		EM CURRENT METERS
MANAHAR, MOBAREK, MORCOS(1974)	54	SURF ZONE VARIATION
BALSILLIE (1975)	633	LONGSHORE VARIATION

different breakers, such as spilling and plunging, result in very different distributions of momentum flux across the surf zone. (3) The inclusion of the more realistic bottom profile would also improve the formulation---presently only a constant sloping bottom is assumed. Bars have not been adequately taken into account in the formulation of longshore currents. The momentum flux technique runs into difficulty when waves break on a bar and and then reform inside the bar. It may be necessary for this case to include mass tnansport in the formulation. (4) In order to verify the existing longshore current theories and properly specify the various coefficients it will be necessary to have more precise information of wave height and particularly wave direction.

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Review of Status of Energetics and Momentum Fluxes in the Surf Zone: Field Data

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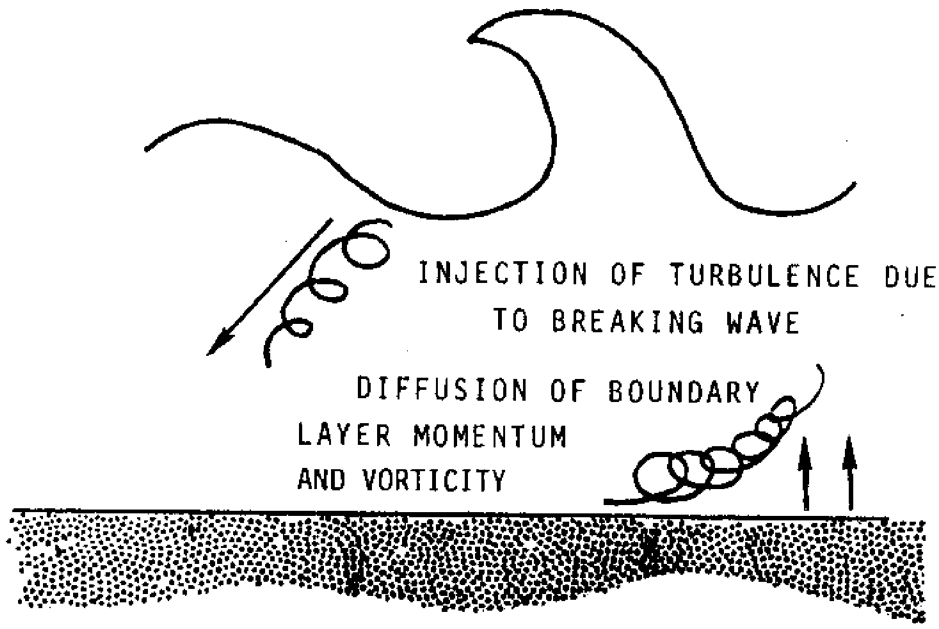
Littoral sand transport is a function of the turbulent and wave energy within the surf zone and longshore current generated by the incident waves. Turbulence is derived from the wave energy in the process of the dissipation of the waves across the surf zone. The longshore current is generated by the coherent wave energy and acts as a transporting agent for the sand along the shore. Hence, sediment transport is a function of the turbulent and wave-induced kinetic energy distributed across the surf zone. Turbulence and wave-induced velocities are examined under breaking waves in an effort to understand sediment transport processes.

The generation of turbulence under breaking waves occurs at both the surface and bottom boundary layer as shown schematically in Figure 1. At the surface there is an injection of turbulence due to the breaking wave and the depth of penetration of the turbulence varies with breaker type. Turbulence penetrates to the bottom under plunging breakers. Under spilling breakers, the turbulence is confined to a surface layer approximated by the trough depth. There is also a diffusion of boundary layer momentum and vorticity due to the bottom shear stress. The bottom boundary layer due to the waves is very weak outside the surf zone and appears to be true also in the surf zone.

The mean turbulent energy production can be assumed equal to the turbulent dissipation over the entire surf zone as a reasonable hypothesis. The production of turbulent energy comes at the expense of the potential energy of the wave due to the dissipation of the incident waves breaking within the



—WEAK BOUNDARY LAYER

—INJECTION OF TURBULENCE DUE TO BREAKING COMES IN BURSTS

—PENETRATION OF TURBULENCE DUE TO BREAKING VARIES WITH BREAKER TYPE

Figure 1. Generation of turbulence under breaking waves

surf zone. Dissipation of incident wave energy can be attributed to the following:

(1) During breaking there is a large amount of turbulence generated with air entrainment. Potential energy is directly converted to potential energy. The amount of potential energy lost is equivalent to the amount of potential energy required to lower the air beneath the surface. Turbulence is also generated by the rising air bubbles to the surface.

(2) When waves break as plunging breakers, vortices, or rollers, are formed; Sawargi and Iwagi (1974) examined plunging breakers in the laboratory and showed that 30 to 50% of the potential energy can be dissipated due to the formation of vortices.

(3) Energy is dissipated due to bottom friction.

(4) Energy is transferred to higher frequencies by a cascade of energy due to nonlinearities. The energy is eventually dissipated into heat at highest frequencies. The cascade of energy is identified by two "saturation regions"; a wave saturation region identified by a -3 slope in the wave spectrum and a -7/3 slope denoting a surface tension region.

(5) Percolation, and

(6) Work is done in moving the sediments.

Assuming that sediment transport is related to the turbulent energy within the surf zone it would be desirable to have an expression describing the turbulent energy. The production of turbulent energy can be equated to the loss of potential energy as identified by the change in wave height. The wave height within the surf zone is generally assumed to be a function of the depth. A linear relationship between wave height and depth is usually assumed; the constant portionality of 0.78 is most often used which is derived from solitary wave theory. A number of other relations have been offered and are reviewed in Meyer (1972).

The turbulent energy density balance can be given by the following equation

$$\frac{\partial \varepsilon}{\partial t} + V_i \frac{\partial \varepsilon}{\partial t} = D - B \qquad i = 1,2 \qquad (1)$$

where ε is the turbulent energy density per unit mass, V_i is the convective velocity of turbulent energy transfer, D is the production of turbulent energy via dissipation of organized wave energy and B is the rate of dissipation of turbulent energy to heat. If it is assumed locally that the mean turbulent production is equal to the turbulent dissipation, the left side of (1) is zero and we then have the local production of turbulent energy due to the dissipation of the incident waves is equal to the rate of dissipation.

A characteristic kinetic energy spectrum under breaking waves is shown in figure 2. The breaking waves within the surf zone are highly non-linear resulting in the transfer of energy to both higher and lower frequencies. The transfer of energy is at the expense of wave energy at the primary frequency. Energy is transferred from the primary to low frequencies in the form of surf beat and edge waves. Non-linear transfer of energy to higher frequencies results in harmonic peaks in the spectra as shown schematically in figure 2. These harmonic peaks are due to the fourier decomposition of Stokes type waves being very peaked and having long troughs and also due to the fact that waves within or near the surf zone can have secondary wave formation which are equivalent to harmonic decomposition. At frequencies above the region of these strong harmonic peaks, the spectrum tails into a -3 slope indicative of the saturation region for waves at breaking; energy cannot be transferred fast enough down the spectrum, i.e., saturated, so that energy is given off by breaking (Thornton, 1977). At even higher fre-

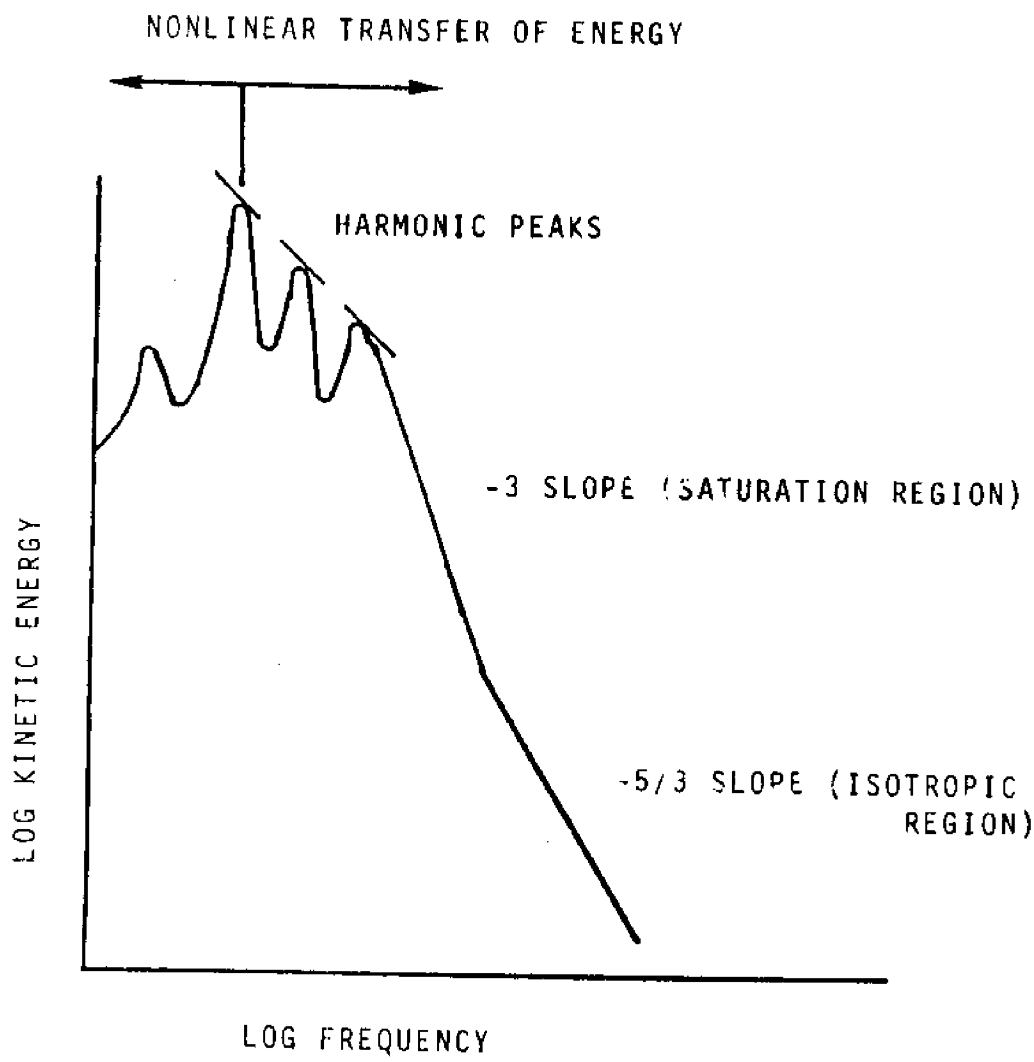


Figure 2. Characteristic velocity spectrum of breaking waves

quencies it is expected that the spectrum will tail into a -5/3 slope indicative of a isotropic turbulent region.

The distribution of kinetic and potential energy and the turbulent energy within the surf zone is described using data collected in the field. Measurements were made across the surf zone of both wave height and velocity using capacitance type wave gauges and electromagnetic flow meters. The instrumentation was mounted on towers made of 2 inch pipe, 12 feet in length that is mounted vertically in the sand and guyed off by cables. The instrumentation and experimental methodology is as explained in detail by Thornton et al (1977). The average potential and kinetic energies can be determined from the measurement of the surface elevation and velocities. A direct measure of the potential energy can be obtained from the variance measurements of the surface. The variance of the surface, σ_η^2 was obtained by integrating the spectrum of the waves, $S_\eta(f)$, across all frequency bands

$$\sigma_\eta^2 = \int_0^\infty S_\eta(f)df \tag{2}$$

where the average potential energy is given by

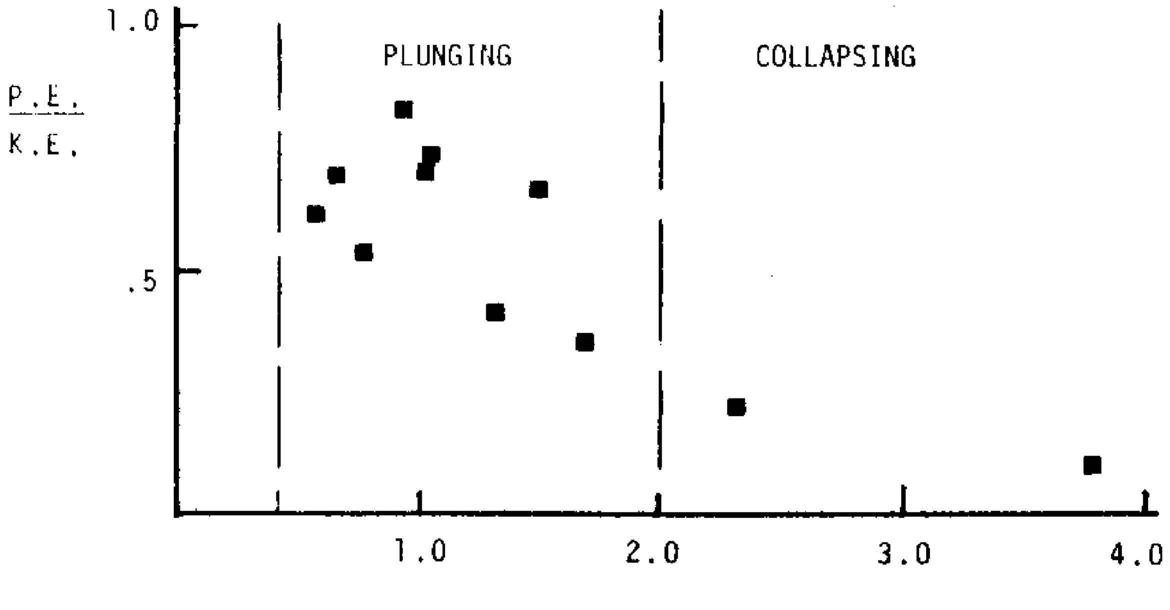
$$PE = 1/2 \ \rho g \sigma_\eta^2 \tag{3}$$

Since the velocities were measured at only one point over the vertical it is necessary to make an assumption involving the velocity distribution in order to calculate the kinetic energy of the water column. It is assumed, as a first approximation, the velocities have a uniform distribution over depth. The kinetic energy is then given by

$$KE \simeq 1/2 \ \rho h \sigma_u^2 \tag{4}$$

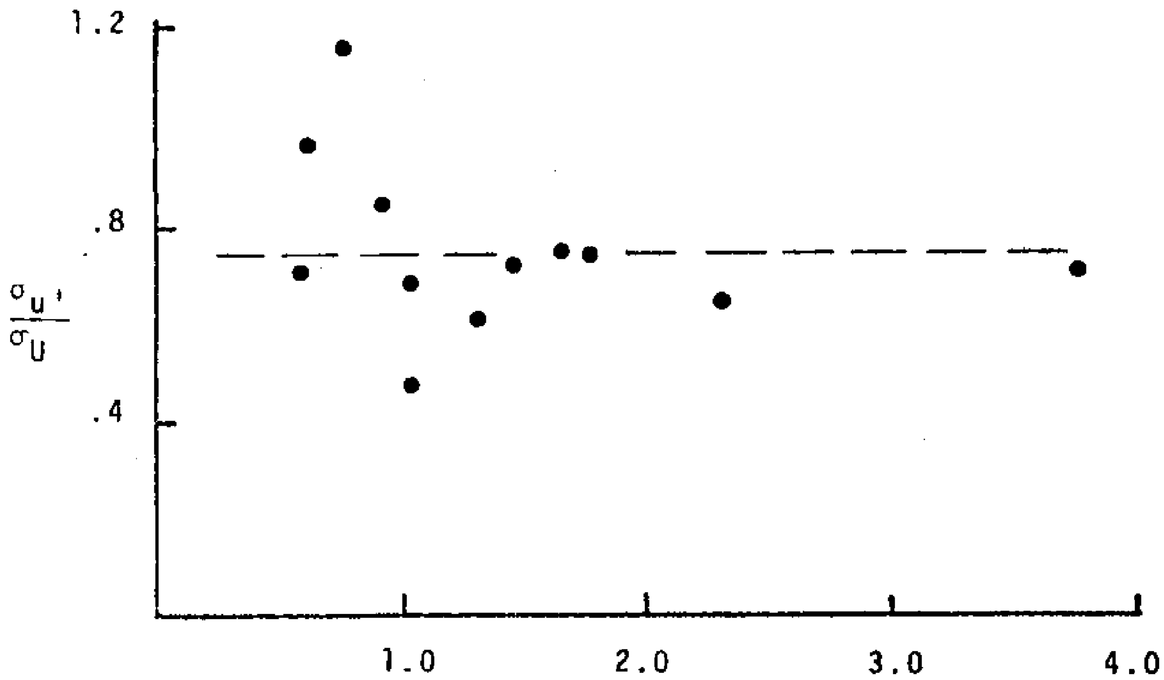
where h is the water depth and σ_u^2 is the velocity variance. The ratio of the average potential to kinetic energies calculated in this manner is shown in figure 3. The ratio is plotted against a parameter that is indicative of the type of breaking wave where β is the bottom slope, $4\sigma_\eta$ is equivalent to the significant wave height and ω is the radial frequency. The difficulty of making point measurements of breaking waves within the surf zone or near breaking is that the waves do not always break at that point. The depth of breaking is a function of the breaker height. Since the breaking waves have a distribution of height, the point of breaking will wander accordingly. The data plotted in figure 3 represents cases of waves near breaking or waves within the surf zone. Waves outside the surf zone have been excluded. No differentation between values at breaking or within the surf zone could be made within the scatter of the data. There is approximately an equal partition of energies (as indicated by the ratio of potential to kinetic energy being equal to one) for the case of spilling breakers. There is an increase of kinetic energy under the plunging breaker at the expense of the potential energy reducing the ratio. The ratio of potential to kinetic energies is expected to decrease with the increasing breaking parameter and then increase again as the waves change to the classification of surging breakers. For highly reflective waves cases, point measurements of the ratio potential to kinetic energy are not indicative of the true processes. The two low values in the region of collapsing breakers appear to have been measured at near nodal points of highly reflected waves on a very steep beach. The nodal point for the surface elevation would be at an anti-node for the velocities; hence, the ratio of potential to kinetic energy would be small.

Longshore transport of sand can be considered due to the turbulent velocities acting as a stirring agent and the longshore current generated by the coherent wave energy acting as a net transporting agent for the sediments



$$\frac{\tan\beta}{\left[4\ \sigma_\eta\ \frac{\omega^2}{g}\right]^{1/2}}$$

Figure 3. Partitioning of Potential and Kinetic Energies in the surf zone.



$$\frac{\sigma_{u'}}{\sigma_U}$$

$$\frac{\tan\beta}{\left[4\sigma_\eta\ \frac{\omega^2}{g}\right]^{1/2}}$$

Figure 4. Ratio of turbulent to wave-induced velocity intensities

which have been stirred up. It is of interest then to have a measure of the wave and turbulent velocities separately. The total velocity can be separated into components of a mean, plus wave-induced, plus turbulent velocity:

$$u = \bar{u} + U + u' \tag{5}$$

The wave induced and turbulent velocity spectral components are assumed statistically independent which can be shown to be a reasonable assumption. The cospectra between waves and velocity is then given by

$$S_{u\eta}(f) = S_{U\eta}(f) \tag{6}$$

Using the assumption of statistical independence, the horizontal velocity spectrum is given

$$S_u(f) = S_{u'}(f) + S_U(f) \tag{7}$$

It is further assumed that the waves and wave-induced velocities are described by a constant parameter linear process where the coherence is identically equal to unity

$$\gamma^2_{U\eta}(f) = \frac{|S_{U\eta}(f)|^2}{S_U(f)S_\eta(f)} \equiv 1 \tag{8}$$

The substitution of (6), (7) and (8) into the definition of coherence between the total horizontal velocity and waves results in

$$\gamma^2_{u\eta}(f) = \left[1 + \frac{S_{u'}(f)}{S_U(f)}\right]^{-1} = \frac{S_U(f)}{S_u(f)} \tag{9}$$

Increasing lack of coherence is due to an increasingly high ratio of turbulence (noise to coherent wave-induced velocity fluctuations (signal). The coherence values indicate the percent of total velocity which is associated with the wave surface.

The wave induced velocity can be calculated using (9)

$$S_U(f) = \gamma^2_{u\eta}(f)S_u(f) \tag{10}$$

The wave-induced velocity spectral component computed in this manner should under-predict the actual value. The linear coherence between waves and velocities will always be underestimated due to nonlinearities which are always strong in a breaking wave and because of directional spreading of the incident wave energy, Battjes (1974).

The turbulent velocity spectrum can then be obtained by subtracting the calculated velocity spectrum from the measured velocity spectrum given by

$$S_{u'}(f) = S_u(f) - S_U(f) \tag{11}$$

The ratio of the turbulent velocity intensity to the wave-induced velocities intensity is given in figure 4. A measure of the velocity intensity is obtained by the square root of the variance which was calculated from the area under the velocity spectrum given by

$$\sigma_u = \int_0^\infty S_u(f)df \tag{12}$$

The measurements show a surprisingly small spread of values for the ratio of turbulent to wave-induced velocity intensities. The average ratio obtained is approximately 0.75. This would indicate that, to a first approximation, the wave-induced kinetic energy is approximately twice the turbulent kinetic energy. It should be recalled that the spectral computations are a time averaging process. Hence, the velocities in the surf zone on the average are highly wave-induced.

An alterante method to calculate the wave-induced kinetic energy is to use linear wave theory to calculate the transfer function between the surface and horizontal velocity spectrum. Such that

$$S_U(f) = |H(f)|^2 S_\eta(f) \tag{13}$$

The transfer function is given from linear wave theory

$$H(f) = 2\pi f \frac{\cosh k(h+z)}{\sinh kh} \tag{14}$$

The ratio of the standard deviation of the wave-induced wave velocity calculated using the linear theory transfer function, $\sigma_{u\ell}$, to the standard deviation of the wave-induced velocities calculated using the coherence method (equation 10) is given in figure 5. It would be expected that if linear theory is a good approximation in a spectral sense, the ratio would be equal to unity. As can be seen from the figure some of the values go above the value of one; this is because the coherence technique under-predicts the amount of wave-induced energy because the coherence is decreased by such factors as non-linearities and wave energy spreading. Thus, the true values should be slightly lower. Figure 5 indicates that linear theory under-predicts the spectral wave-induced velocity components; but it should be recalled in the discussion on long shore current theory that monochromatic linear wave theory used in calculating potential energy over-predicts the potential energy. Hence, linear theory used in a spectral sense under-predicts the potential energy and monochromatic linear theory over-predicts the potential energy.

Available field data of direct measurements of kinetic energy and momentum fluxes is reviewed in Table 1. There has been only a very few direct measurements that have been made in the field. This is because the instrumentation to make such measurements has only been available recently. There are several other investigators that have made simultaneous wave and velocity measurements within the surf zone and these are also given in Table 1. The

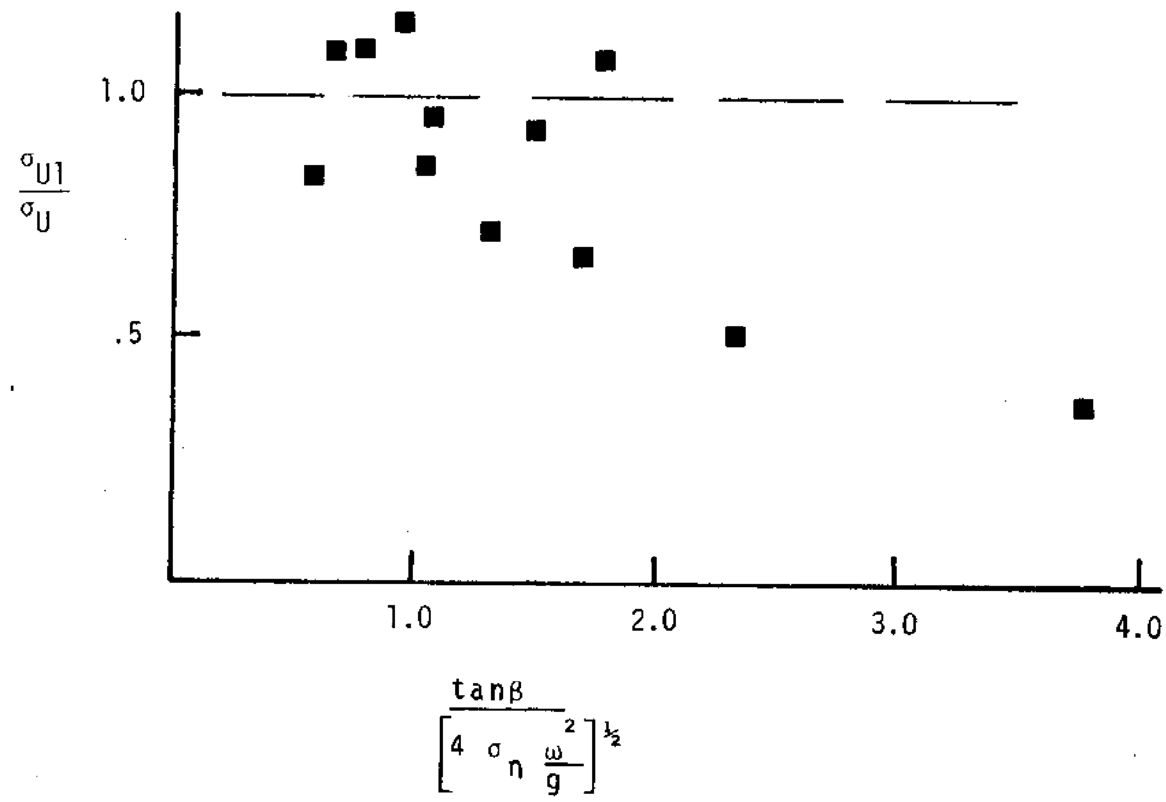


Figure 5. Wave-induced velocity intensity calculated using linear wave theory.

FIELD DATA

KINETIC ENERGIES AND MOMENTUM FLUXES:

INVESTIGATOR	LOCATION	BEACH SLOPE
HUNTLEY AND BOWEN (1974)	NOVA SCOTIA	0.01 - 0.05
DETTE AND FUHRBOTER (1974)	GERMANY	
THORNTON (1977)	MONTEREY, CA	0.02 - 0.15

WAVES AND VELOCITIES:

TELEKI *ET AL*. (1977)	LAKE MICHIGAN
WOODS (1977)	LAKE MICHIGAN
MILLER ZEIGLER (1964)	CAPE COD

measurements of Huntley and Bowen were made using only electromagnetic flow meters. The measurements by Frühböter were made using electromagnetic current meters and pressure wave transducers. The measurements by Thornton were made using electromagnetic flow meters and capacitance wave gauges. It is important to make a direct measurement of the wave surface directly above the veloctiy meters; the wave surface cannot be accurately inferred by pressure measurements. Field measurements by Thornton (1977) showed that using pressure meters to infer the wave surface within the surf zone under-predicts the potential energy by 50%. Van Dorn (1977) found comparable errors in laboratory comparison of wave measurements using pressure transducers and surface piercing staffs.

Historically the study of turbulence has progressed primarily by means of experimental measurements. Turbulence phenomenon can be given a theoretical physical setting but a complete definition requires empirical data because turbulence can only be viewed as a statistical phenomenon. Hence, if we are going to proceed in our understanding of turbulence and energetics within the surf zone which are important for the study of sand transport, it will be necessary to make significantly more field measurements of turbulent and wave-induced velocities.

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NONLINEAR WAVES IN THE SURF ZONE

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ABSTRACT

Wave shoaling is an inherently nonlinear process resulting in a transfer of energy away from incident wave frequencies. Frequencies higher than the incident waves probably receive their energy via strong, but nonresonant, interactions. This transfer to high frequencies must enhance the overall wave velocity shear and dissipation, and may be intimately related to the onset of wave breaking. The energy transfer to low frequencies appears to be dominated by resonant excitation of long edge waves.

TEXT

It has long been recognized that the wave height and velocity field of progressive, shoaling waves cannot be accurately predicted by linear theory, given the bathymetry and deep water incident wave parameters. This is true even for the most idealized laboratory conditions using undirectional, monochromatic waves normally incident on a beach of constant slope. Linear theory generally underpredicts the wave height by a substantial amount (Figure 1).

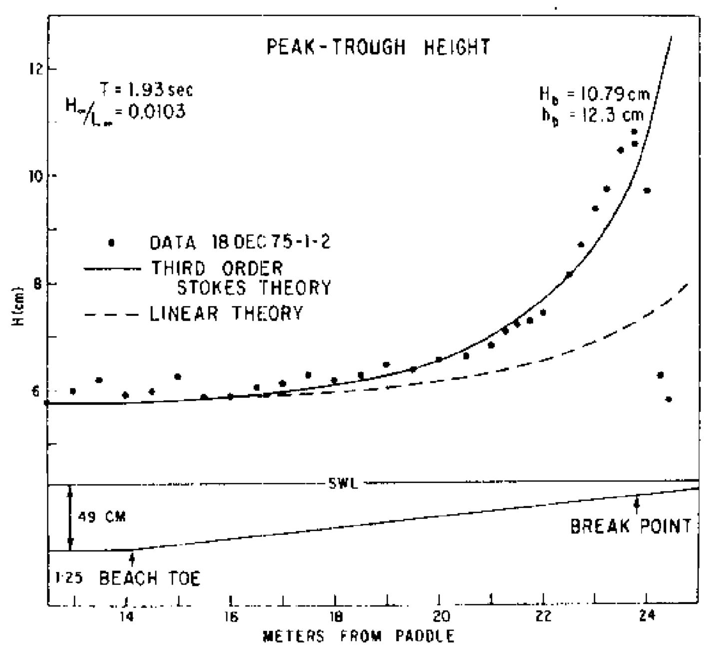

Figure 1. Shoaling wave height data compared to linear and 3rd order Stokes theories. Subscripts b, ∞ indicate break point and deep water respectively; L_∞ is the deep water wavelength (Flick, 1977).

Although the offshore variation of sea surface elevation of waves strongly reflected at the shoreline (forming standing waves) are well described by linear theory (Figure 2), strong reflection occurs only on steep beaches with relatively small amplitude incident waves. Thus, the existing longshore current and sediment transport theories which rely heavily on linear, shoaling theory for progressive waves are necessarily subject to large errors, although ad hoc adjustments of various constants may correct some of the errors for some range of parameters. In order to obtain models for important nearshore processes based on a detailed understanding of the driving forces, and therefore applicable to a wide range of field situations, it is clearly necessary to improve our understanding of wave shoaling and the associated fluxes of momentum and energy.

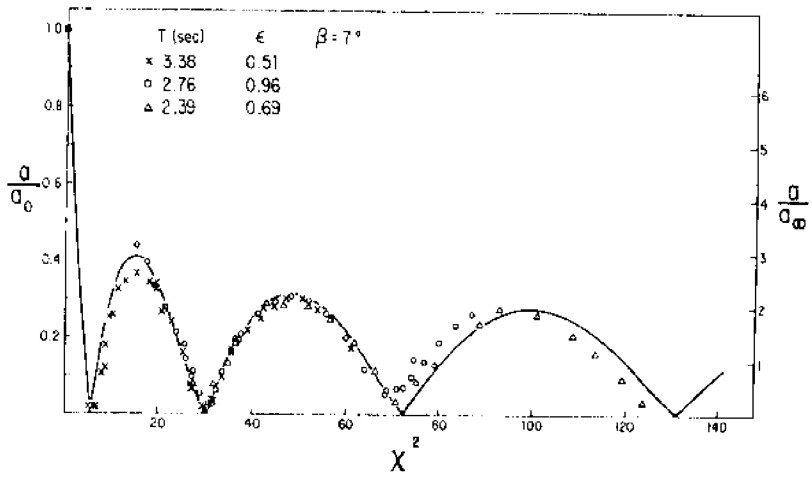

Figure 2. Standing waves on a plane beach are well described by linear theory; a, a_0, a_∞ are the local, run-up, and deep water amplitudes respectively (Guza and Bowen, 1976).

Many workers have suggested the inclusion of nonlinear effects as a step towards an improved shoaling theory. One of the conceptually simplest and intuitively most appealing theories represents the wave, at each local depth, as a high order Stokes wave. The conservation of energy as the wave propagates shoreward relates the local wave height to deep water incident waves parameters (Koh and LeMehaute, 1966). Gravity waves in a constant depth fluid have been studied extensively using similar Stokes type nonlinear expansions, with considerable success in predicting experimental observations, notably the resonant interactions among wave trains of differing frequencies (McGoldrick, et al, 1966). It would be most surprising, therefore, if nonlinear Stokes waves were found to have no relevance to wave shoaling. The essence of the nonlinear Stokes solution for a single wave train is a separation of the local wave energy between motions at the primary wave frequency σ, and its harmonics having frequencies 2σ, 3σ, ... The order of the approximation determines how many harmonics enter into the solution; in n^{th} order theory the sea surface fluctuations and velocity field contain fluctuations with frequencies σ, 2σ... $n\sigma$. The theory generally predicts increasing amounts of energy in the harmonics (relative to the primary) for both increasing wave height and decreasing depth. It would therefore be expected that wave shoaling will involve dramatic enhancement of the harmonics. Small amplitude waves in relatively deep water are predicted to have almost all their energy at the primary frequency, σ, and the sea surface is a simple sinusoid. The success of the theory in predicting peak to trough wave height is shown in Figure 1, the data is from a laboratory wave train which broke by spilling (Flick, 1977). A similar experiment is shown in Figure 3, here the sea surface elevation was analyzed to give Fourier amplitudes (a_1, a_2, a_3) at the primary and harmonic frequencies (σ, 2σ, 3σ, ...) as a function of the nondimensional depth h/L_∞. Notice that both the measured and predicted potential energy at the primary frequency (proportional to a_1^2) decrease throughout the shoaling process even though the wave height increases. Loosely speaking, this occurs because as the wave shoals an ever larger percentage of the local potential is at the harmonic frequencies, and since energy is conserved the increase in energy at harmonic frequencies results in a decrease at the primary frequency. Experiments on various beach slopes show good agreement between measured and predicted wave heights and Fourier amplitudes right up to the break point, so long as the waves break by spilling (Flick, 1977). Laboratory work is underway to see whether the harmonic content of the velocity field is also well predicted.

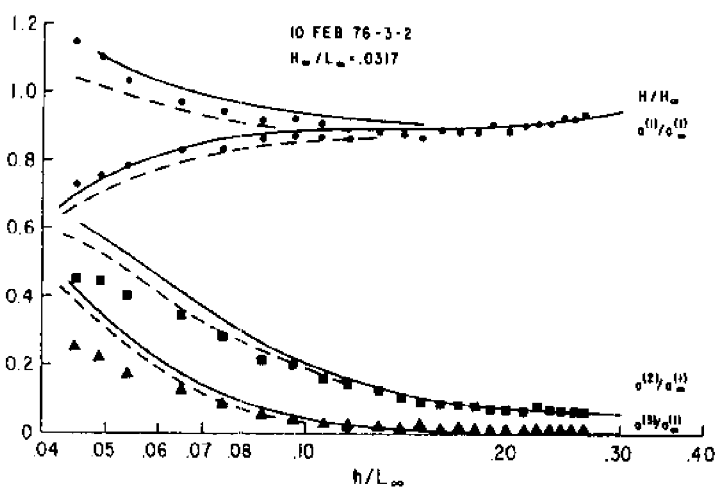

Figure 3. Measured and theoretically predicted Fourier amplitudes of fluctuations of sea surface elevation ($a^{(1)}$, $a^{(2)}$, $a^{(3)}$) at frequencies (σ, 2σ, 3σ) normalized by the deep water value at the primary frequency $a_\infty^{(1)}$ plotted against the nondimensional depth, h/L_∞ where h is the local depth (Flick, 1977).

Waves which are of low steepness in deep water progress into relatively very shallow water before they break by plunging or collapsing. The nonlinear shoaling Stokes theory does not cover these waves because the Ursell number ($ak/(kh)^3$) becomes too large. Nevertheless, the successful prediction of wave behavior up to the point of spilling is encouraging. However, when the restriction of a single unidirectional monochromatic incident wave train is relaxed, both theory and experiment suggest a very complex process of nonlinear interaction. Figure 4a shows a typical amplitude spectra for a single deep water incident wave train of frequency σ_2; in shallow water energy appears at the harmonics $2\sigma_2$, $3\sigma_2$, ..., as discussed above. Figure 4b shows a similar plot with two incident wave frequencies, σ_1 and σ_2. The nonlinear nature of wave shoaling results in energy transfer to the harmonics of each wave ($2\sigma_1$, $2\sigma_2$, $3\sigma_1$, $3\sigma_2$, ...) and also to frequencies forced by the interaction of the wave trains with each other ($\sigma_1 + \sigma_2$, $\sigma_1 - \sigma_2$, $2\sigma_1 + \sigma_2$ ...). Although the Stokes theory might be extended to cover two (or several) incident wave trains, the extension to a random wave field composed of a continuum of frequencies and directions seems a very difficult task. Qualitatively, however, energy must be spread over a very broad range of frequencies. There is a possibility that inside the surf zone nonlinear and dissipative effects are so strong that the spectra (of velocity or sea surface level fluctuation) reaches an equilibrium (saturation) level, generally independent of the details of the incident wave field. Frequencies higher than the dominant incident wave frequency are simply 'filled up' to some level. This seems to be the case for run-up spectra (Huntley, et al, 1977). A typical run-up spectrum is shown in Figure 5; this particular spectrum resulted from an incident wave field with a maximum in energy at about 0.1 Hz. The run-up spectrum is smoother than the incident wave spectrum, and falls off as σ^{-4} for $\sigma > 0.1$ Hz.

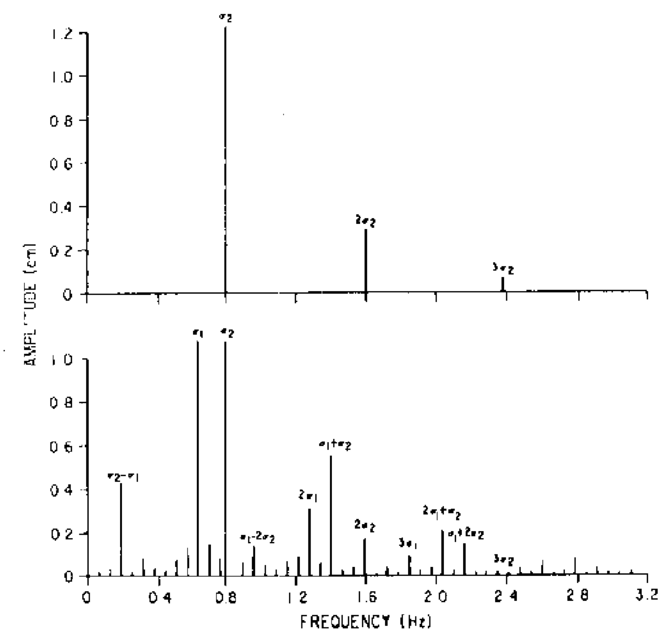

Figure 4. Experimentally measured Fourier amplitudes of sea surface elevation (a) monochromatic incident wave train (frequency σ_2) in deep water transfers energy to harmonics ($2\sigma_2$, $3\sigma_2$) in shallow water; (b) bichromatic (σ_1, σ_2) deep water train transfers energy to harmonics and also to frequencies which are sums and differences of σ_1, σ_2.

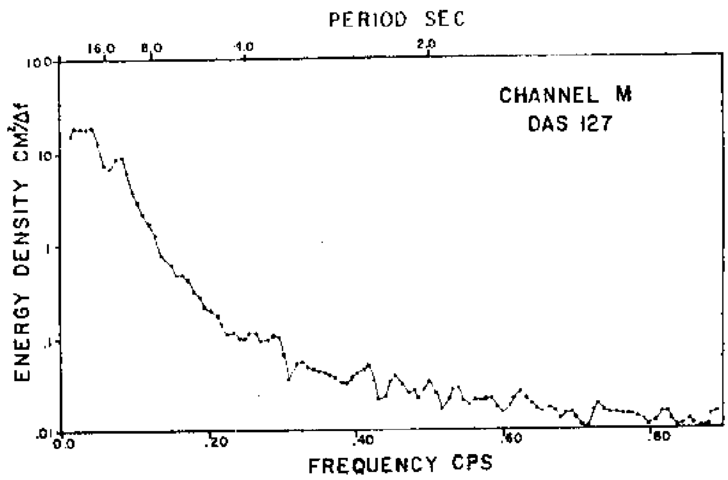

Figure 5. Run-up spectrum (Suhayda, 1972).

The maximum of energy density at relatively low frequency (Figure 5) is characteristic of both run-up and velocity spectra taken in very shallow water and falls into the class of oscillations dubbed 'surf beat'. Longuet-Higgins and Stewart (1962) have suggested that these low frequency waves are essentially the forced low frequency corrections generated by the difference interactions between incident waves of higher frequency. Figure 4b shows just such an oscillation at $\sigma_2 - \sigma_1$. Gallagher (1971) has pointed out that if the interacting wave trains are nonnormally incident, then the nonlinear forcing term at $\Delta\sigma = \sigma_2 - \sigma_1$ has longshore wavenumber $\Delta k = k_2 - k_1$ where k_1, k_2 are the incident wave longshore wavenumbers. Furthermore, if

$$(\Delta\sigma)^2 = g\Delta k(2n + 1)\beta \quad \text{where} \quad n = 0, 1, 2, \ldots \tag{1}$$

g is gravity, and β the plane beach slope, then the forcing satisfies the dispersion relation for a free edge wave, and resonance is expected. The existence of this resonance has been demonstrated in laboratory experiments. Incident waves of two different frequencies, nonnormally incident on a plane beach, were generated in the SIO wave basin. The frequency of one of the incident waves was varied from experiment to experiment (thus varying $\Delta\sigma$, Δk) while keeping the incident wave amplitudes approximately constant. Figure 6 shows the measured low frequency wave heights in very shallow water as a function of $\Delta T = 2\pi/\Delta\sigma$. When the incident wave and beach slope parameters satisfied the resonance condition (equation 1) there is indeed something resembling a resonant response. When equation 1 is not satisfied, only a small forced correction results. Recent field measurements (Huntley, 1977) suggest that surf beat is indeed a combination of edge waves, but much further work is needed to definitively identify their energy source, and to assess their overall dynamic importance.

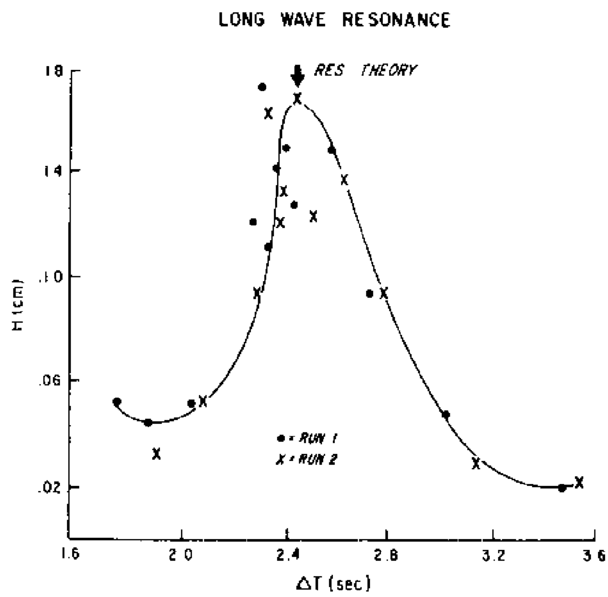

Figure 6. Resonant long wave response observed when nonlinear difference interactions among incident waves satisfy the edge wave dispersion equation (after Bowen and Guza, in preparation).

In summary, wave shoaling is an inherently nonlinear process resulting in a transfer of energy away from incident wave frequencies. Relatively high frequencies probably receive their energy via strong, but nonresonant, interactions. The energy transfer to high frequencies must enhance the overall velocity shear and wave dissipation, and may be intimately related to the onset of wave breaking. The energy transfer to low frequencies appears to be dominated by resonant excitation of long edge waves.

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PRELIMINARY DESCRIPTION OF THE NATIONAL SEDIMENT TRANSPORT STUDY

R. J. SEYMOUR

There are many aspects to the study of sand transport in the nearshore environment, some of which are well understood and others which have not been considered in existing models. Topics which must be individually studied in order to understand the overall process and will become areas of investigation in the NSTS include:

1. CHARACTERIZATION OF THE FORCING FUNCTION:

A. Relate the three-dimensional velocity field produced by shoaling and breaking waves on a straight coastline to the directional wave spectrum.

B. The three-dimensional velocity field produced by tides along a straight coastline.

C. The interactions between incident waves, other wave modes, wind and tidal and other currents which are important to sediment transport.

2. CHARACTERIZATION OF THE RESPONSE FUNCTION:

A. Bed load and suspended load of sediment as functions of sediment characteristics for various combinations of unidirectional and oscillatory flows.

B. On-offshore sediment transport and beach profiles, resulting from the effects of waves, winds and currents.

C. Effects of breaker type on sediment transport.

3. DEVELOP USEFUL MODELS:

A. Engineering models for two-dimensional sediment transport based upon the simplified engineering measurements and the predictive model described below.

B. Predictive models for sediment movement in various nearshore zones as functions of the velocity field models for straight coastlines.

C. Simplified, affordable engineering measurement schemes for characterizing the significant attributes of the forcing function.

In the conduct of the experimental programs it is imperative that certain forcing and response functions be investigated simultaneously.

It is contemplated that intensive study of the above individual topics be through a number of parallel, coordinated investigations, culminating in field experiments, carried out by scientists of proven ability in each specialty as schematically shown in Figure 1. The proposed time phasing of the tasks is shown in the schedule. It is expected that as the results of the individual investigations become known they will be incorporated into an overall predictive model for the transport of sand, and compared to existing field data. After synthesis of ideas into a coherent model, large scale field experiments will be designed for extensive field testing along the coasts of the United States. These field experiments will be extremely valuable in their own right, providing the quantitative measurements so sorely needed, and conspicuously absent.

Completion of these experiments will refine the physical sediment transport model so that it can be utilized as an engineering sediment transport model through the use of affordable engineering measurement schemes which characterize the velocity field sufficiently to allow reasonable predictions of transport. The engineering sediment transport model will be the final product of the National Sediment Transport Study and can then be applied by coastal engineers to the solution of sediment management problems.

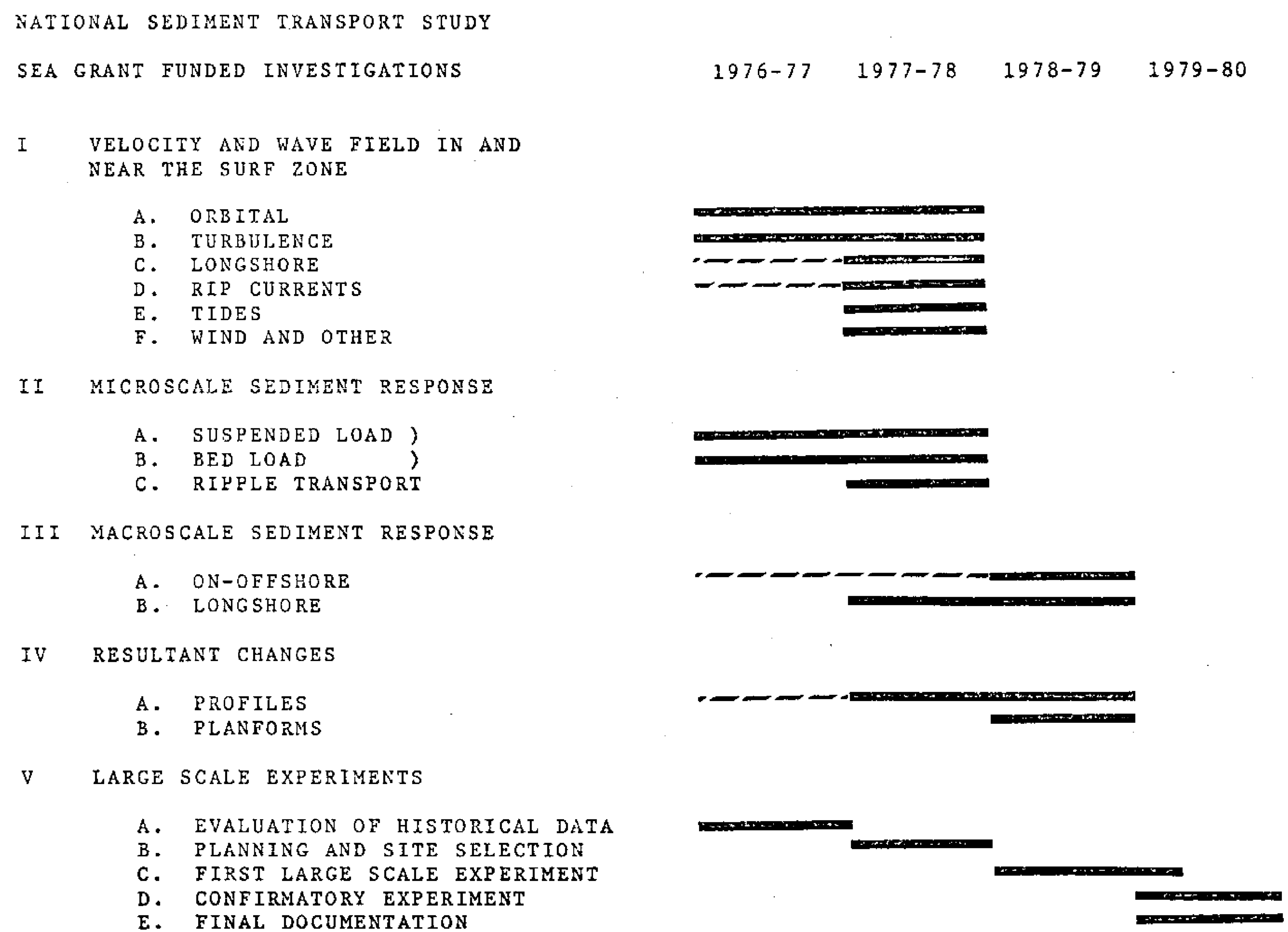
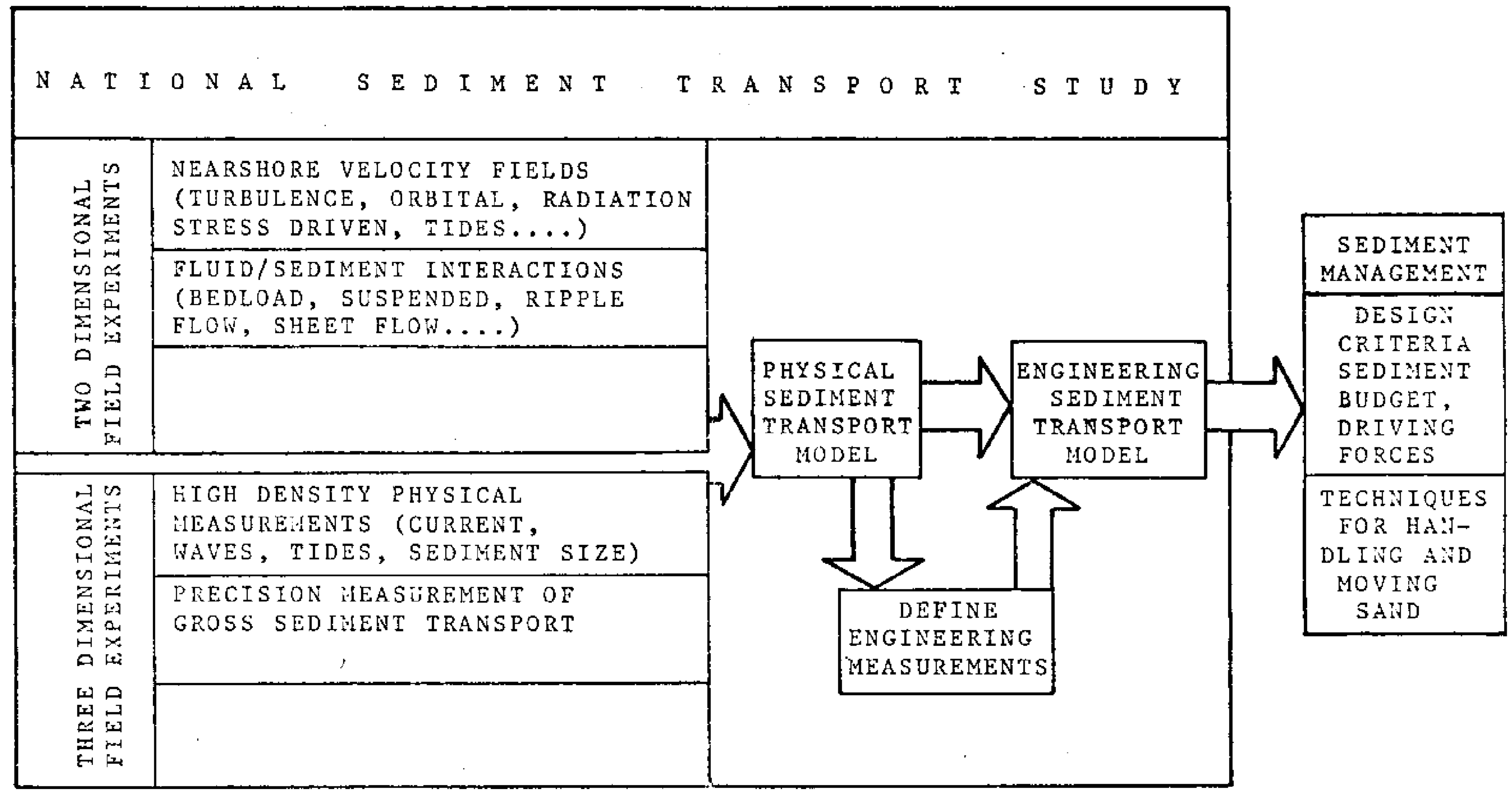

NATIONAL SEDIMENT TRANSPORT STUDY
SEA GRANT FUNDED INVESTIGATIONS
1976-77
1977-78
1978-79
1979-80
I VELOCITY AND WAVE FIELD IN AND NEAR THE SURF ZONE
A. ORBITAL
B. TURBULENCE
C. LONGSHORE
D. RIP CURRENTS
E. TIDES
F. WIND AND OTHER
II MICROSCALE SEDIMENT RESPONSE
A. SUSPENDED LOAD)
B. BED LOAD)
C. RIPPLE TRANSPORT
III MACROSCALE SEDIMENT RESPONSE
A. ON-OFFSHORE
B. LONGSHORE
IV RESULTANT CHANGES
A. PROFILES
B. PLANFORMS
V LARGE SCALE EXPERIMENTS
A. EVALUATION OF HISTORICAL DATA
B. PLANNING AND SITE SELECTION
C. FIRST LARGE SCALE EXPERIMENT
D. CONFIRMATORY EXPERIMENT
E. FINAL DOCUMENTATION

FIGURE 1



NATIONAL SEDIMENT TRANSPORT STUDY
TWO DIMENSIONAL FIELD EXPERIMENTS
NEARSHORE VELOCITY FIELDS (TURBULENCE, ORBITAL, RADIATION STRESS DRIVEN, TIDES....)
FLUID/SEDIMENT INTERACTIONS (BEDLOAD, SUSPENDED, RIPPLE FLOW, SHEET FLOW....)
THREE DIMENSIONAL FIELD EXPERIMENTS
HIGH DENSITY PHYSICAL MEASUREMENTS (CURRENT, WAVES, TIDES, SEDIMENT SIZE)
PRECISION MEASUREMENT OF GROSS SEDIMENT TRANSPORT
PHYSICAL SEDIMENT TRANSPORT MODEL
DEFINE ENGINEERING MEASUREMENTS
ENGINEERING SEDIMENT TRANSPORT MODEL
SEDIMENT MANAGEMENT
DESIGN CRITERIA SEDIMENT BUDGET, DRIVING FORCES
TECHNIQUES FOR HAN- DLING AND MOVING SAND

FIGURE 2

ABSTRACT - SEDIMENT TRAP AT PANAMA CITY

by

T.L. Walton

In August 1975, the Coastal Engineering Laboratory at the University of Florida constructed a large scale sediment trap in the Panama City area to determine the direction and magnitude of sand transport along Panama City beaches. Little evidence presently exists as to direction of sand transport in this area. Court suits have arisen in which evidence has been presented suggesting sand transport along the Panama City beaches in both an eastward and a westward direction. The sediment trap and an associated instrumentation package was built and installed to resolve some of the conflict as well as provide engineering information on littoral drift magnitudes and quantities in this area.

The sediment trap consisted of a nylon PVC coated bag type groin, 450 feet in length and 4.5 feet high. The groin was built with a filter cloth underneath to prevent the structure from sinking. The individual bags of which the structure was built measured 9 feet x 4 feet x 1.5 feet and weighed about 3,000 lbs. when pumped full with sand. The groin was constructed just to the west of Phillips Inlet in an area with high dunes (15 - 20 feet) and no coastal structures encroaching on the shorelines. It was desired that no man-induced changes other than those caused by the nylon bag groin would be present. In the area of construction two offshore bars existed, a somewhat migratory inner bar with crest depths varying from 1 - 4 feet offshore and a stationary outer bar ranging from 800 - 1200 feet offshore with crest depths over the bar of 8 - 10 feet.

At the site a tower was built (20 feet in height) on the primary dune overlooking the groin and a time lapse camera was installed with an intervelometer for taking a picture of the structure and associated sand buildup and wave climate every 15 minutes. An anemometer was also installed on the tower for taking continuous records of wind speed and direction.

A LEO observer was taking surf zone measurements of longshore current velocity, wave height, period, and direction, as well as distance to the breaker line(s), and longshore current distribution across the surf zone. A site was also planned for taking wave directional spectra measurements outside of the outer bar by electromagnetic current meter and pressure transducer signals.

Two weeks after completion of the structure, Hurricane Eloise made a landfall about 20 miles west of the site and destroyed the groin and tower as well as doing 200 million dollars worth of structural damage to the heavily developed Panama City Beach area. Over 60 feet of primary and foredunes were virtually destroyed along a 10 mile section of coast centered near the groin site, and considerable damage extended over a 50 mile section of coast. The tower was also destroyed when wave activity undermined the primary dune on which the tower was sitting. Prior to failure of the tower, a wind speed of 100 mph was measured by the anemometer on the tower (33 feet MSL).

Prior to the storm the structure performed quite well and it was felt that the structure had potential as an effective low cost sediment trap when combined with periodic surveys and proper instrumentation for taking wave data which might be quite useful in meeting some of the objectives for monitoring sand transport.

AN ANALYTICAL MODEL OF

LONGSHORE SEDIMENT TRANSPORT

by Ole Secher Madsen, Associate Professor of Civil Engineering, R. M. Parsons Laboratory, MIT, Cambridge, MA 02139

Sediment Transport Relationship

Previous research (Madsen and Grant, 1975, 1976a and b) has established the general validity of Shields Criterion for the onset of sediment movement in unsteady, oscillatory flow as well as in steady, unidirectional flow. The important parameter quantifying the fluid-sediment interaction is the Shields Parameter

$$\Psi_m = \frac{\tau_{om}}{\rho g(s-1)d} \tag{1}$$

in which ρ is the fluid density, g is the acceleration due to gravity, s is the relative density, ρ_s/ρ, of the sediment, d is the grain diameter. τ_{om} is the maximum bottom shear stress associated with the fluid motion. For a purely oscillatory flow such as the flow above the bottom associated with small amplitude waves, the work of Jonsson (1967) may be used to evaluate the wave friction factor, f_w, relating τ_{om} to the maximum wave orbital veloctiy above the bed, u_b,

$$\tau_{om} = \frac{1}{2}\rho f_w u_b^2 \tag{2}$$

Several previous investigations have been unsuccessful in obtaining a general criterion for the initiation of sediment movement under waves mainly because the friction factir, f_w, was not included in the analysis.

Based on the physical significance of the Shields Parameter Madsen and Grant (1976a and b) reanalyzed the Berkeley data on sediment transport rates in oscillatory flow (summarized by Einstein, 1972). The validity of a quasi-steady application of the Einstein-Brown formula (Brown, 1950) for the purpose of predicting sediment transport rates in oscillatory flow resulted from this analysis. The resulting sediment transport relationship

$$\vec{\phi}(t) = 40\,\vec{\Psi}^3(t) \tag{3}$$

therefore has some experimental support. In Eq. (3) $\vec{\phi}(t)$ and $\vec{\Psi}(t)$ are the instantaneous values of the nondimensional transport function and the Shields Parameter, respectively. $\vec{\phi}(t)$ is defined by

$$\vec{\phi}(t) = \frac{\vec{q}_s(t)}{wd} \tag{4}$$

in which $\vec{q}_s(t)$ is the sediment transport rate (volume per time per unit width) and w is the fall velocity of the sediment grain. The Shields Parameter $\vec{\Psi}(t)$ is given by

$$\vec{\Psi}(t) = \frac{\vec{\tau}_o(t)}{\rho g(s-1)d} \tag{5}$$

where the bottom shear stress vector, $\vec{\tau}_o(t)$, is given by a generalized expression

$$\vec{\tau}_o(t) = \frac{1}{2}\,\rho f_{cw}\left|\vec{u}_w(t) + \vec{v}\right|(\vec{u}_w(t) + \vec{v}) \tag{6}$$

In Eq. (6) $\vec{u}_w(t)$ may be regarded as the unsteady wave component, i.e., $\vec{u}_w(t) = \vec{u}_b \cos\omega t$ if linear wave theory is assumed valid, whereas $\vec{v}$ is a superimposed steady (or slowly varying) current. Since Eq. (6) is taken to express the shear stress resulting from the combined action of waves and currents the friction factor is denoted by f_{cw}. It should be emphasized that our ability to estimate the appropriate value of f_{cw} is extremely limited. Only in the limiting cases of either a purely steady current, $f_{cw} = f_c$, or a purely oscillatory flow, $f_{cw} = f_w$, can f_{cw} be estimated with some confidence from knowledge of the boundary roughness. As shown by Madsen (1976) the difference between f_c and f_w may be as much as a factor of ten (10), thus, underlining the importance of an improved understanding of and ability to estimate the value of the friction factor for the combined action of waves and currents.

Analytical Model of Longshore Sediment Transport

In the surf zone on a long, plane beach the effect of incident monochromatic waves will produce a longshore current. This current, v, may be estimated from the theory advanced by Longuet-Higgins (1970), provided the assumptions (spilling breakers, small longshore currents relative to the wave orbital velocities, etc.) made in the theoretical development are valid. A modified version of Longuet-Higgins' longshore current theory, correcting a few inconsistencies in the original formulation of the wave behavior outside the surf zone, leads to a formula for the variation of the longshore current with distance from shore, x.

$$v(x) = F(P, \frac{x}{x_B}) \frac{5\pi \tan\beta \sin\theta_B}{4 f_{cw}} u_{b,B} \tag{7}$$

where $F(P, \frac{x}{x_B})$ gives the general shape of the longshore velocity with distance from shore relative to the breaker line x/x_B. P is the mixing parameter used by Longuet-Higgins which accounts for the influence of lateral friction. In Eq. (7) β is the beach slope, θ_B is the angle of incidence at breaking, f_{cw} is the friction factor, and $u_{b,B}$ is the maximum wave orbital velocity at the breaker line.

Retaining the same assumptions inherent in the longshore current model, in particular that of $v << u_b$ and the applicability of linear long wave theory, the sediment transport relationship, Eq. (4), may be time averaged in the longshore direction leading to a relatively simple formula for the longshore sediment transport rate, q_ℓ, with distance from the shore

$$q_\ell(x) = 1.7 \text{ wd } \left[\frac{f_{cw}}{g(s-1)d}\right]^3 u_b^{\,5}(x)\ v(x) \tag{8}$$

in which $u_b(x)$ is the maximum orbital velocity associated with the wave motion as a function of distance from shore and $v(x)$ is given by Eq. (7).

Introducing $v(x)$, taking the mixing parameter $P = 0.25$, in Eq. (8) with $u_b(x) = \alpha\sqrt{gh}$ with $\alpha = 0.4$ and $h = x \tan\beta$ the variation of longshore sediment transport with distance from shore may be evaluated. Integrating $q_\ell(x)$ from the shoreline, $x = 0$, to the breaker line, $x = x_B$, the quantity, Q_s, generally referred to as the longshore sediment transport rate may be obtained.

$$Q_s = \int_0^{x_B} q_\ell(x)\ dx \tag{9}$$

Examination of Eqs. (7), (8) and (9) shows that the resulting expression for Q_s will depend on the value of d, the grain size of the sediment, the incident wave characteristics, H_B and θ_B, the beach slope β, the value of the mixing parameter, and on the value of the friction factor, f_{cw}. In fact, it may be shown that Q_s is proportional to f_{cw}^2 and H_B^4, the latter dependency deviating somewhat from popular empirical relationships.

The friction factor, f_{cw}, is taken as $f_{cw} = 0.02$, which corresponds to the value suggested by Longuet-Higgins. A comparison of predicted longshore sediment transport rates (from Eq. 9) and those measured by Komar and Inman (1970) from Silverstrand Beach is presented in Table 1. Since several wave observations were made during the measurement of the longshore transport rate, several predictions can be made for each measured value of Q_s.

Table 1: Comparison of Measured and Predicted Longshore Sediment Transport Rates. Measured Rates by Komar and Inman (1970) on Silverstrand Beach (d=0.0175 cm, s=2.6, $\tan\beta$= 0.034). Predicted Rates (α=0.4, f_{cw}=0.02).

Measured Q_s cm^3/sec. (excluding pore volume)	Predicted Q_s cm^3/sec. (excluding pore volume)
762	1,020
	930
18,060	34,800
	36,800
	23,200
2,808	2,090
	3,040
	3,500
	2,830
2,256	2,890
	3,660
	1,860
	2,880

The data obtained by Komar and Inman (1970) from Silverstrand Beach appears to be the only data obtained under conditions approaching those assumed in the development of the longshore current model, i.e., $v << u_b$ and spilling breakers. The wave data appear to be of comparatively high quality. The measured sediment transport rates are probably the most accurate available, although their accuracy, in an absolute sense, is somewhat uncertain.

Conclusions

The analytical model of longshore sediment transport outlined in the preceeding Section rests heavily on the accuracy of the model employed to predict the wave induced longshore current. Thus, improving the total model would first of all call for an improved analytical model of wave induced longshore currents. To this end, more reliable data on wave induced longshore currents must be obtained so that the important parameters (the bottom friction factor for combined wave and current action, the parametrization of the lateral friction) may be determined.

The sediment transport relationship used in this investigation is supported by some experimental data. Additional experimental data on sediment transport rates (notably giving transport rates due to the combined action of waves and currents including the effects of bed forms, the explicit determination of suspended sediment transport rates) must be obtained to improve the sediment transport relationship.

A better and more comprehensive data set on longshore sediment transport rates must be obtained against which any proposed analytical model can be tested.

The model presented here is relatively successful in reproducing Komar's and Inman's data from Silverstrand Beach. This may be fortuitious, but nevertheless it is encouraging. The basic elements going into the model (the longshore current model and the sediment transport relationship) have some experimental support. For this reason the model is felt to be physically more sound than presently available empirical relationships. Whether the model is superior to existing empirical relationships can only be answered by a comparison with a more extensive data base. At least the present model could serve as the framework for the rational design of an experiment and the analysis of the data.

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Project Title: On the Incipient Motion and the Transport of Shells in the Benthic Boundary Layer

Principal Investigators: B.A. Christensen and A.J. Mehta*

Institution: University of Florida

Brief Project Description:

This research project is concerned with the incipient motion and bed load transport of shells in a uniform turbulent flow field, with the aim of deriving a predictive stochastic mathematical model. This model is intended to do two things. One is to predict the flow conditions under which a bed of shells will fail and establish a bed load motion. The other is to predict the rate of bed-load transport, given the shell geometry and the flow field.

Through experimental and analytical investigations, the existing stochastic laws for the transport of coarse grains, such as sand and gravel, will be extended to the case of transport of shells in a uniform turbulent flow field.

The incipient motion and bed load transport of shells is a phenomenon of interest to environmental engineering for the same reasons as the transport of sediment.

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