SITE SELECTION METHODOLOGY

AND APPLICATION TO THE PACIFIC COAST

by

R. G. Dean and O. Börekçi

Sponsored by the National Sea Grant Program
Through the Nearshore Sediment Transport Study

Ocean Engineering Report No. 17

June, 1979

Department of Civil Engineering
University of Delaware
Newark, Delaware

SITE SELECTION METHODOLOGY
AND APPLICATION TO THE PACIFIC COAST

by

R. G. Dean and O. Börekci

OCEAN ENGINEERING REPORT NO. 17
Sponsored by the National Sea Grant Program
Through the Nearshore Sediment Transport Study

June, 1979

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF FIGURES
(Continued)

LIST OF FIGURES
(Continued)

LIST OF TABLES

SITE SELECTION METHODOLOGY
AND APPLICATION TO THE PACIFIC COAST

by

R. G. Dean and O. Borekci

INTRODUCTION

During the last several decades, the usage of the coastal zones of the world has intensified considerably. This has resulted from added emphasis on national security, commerce, and recreation and the recognition of the advantages provided by this zone for industrial and residential sites.

In a natural state the coastal zone is dynamic, in which both short and long-term erosional and depositional trends occur. The slow, but persistent, rise in sea level is the major natural factor in the present long-term trend in beach erosion. Sea level is rising at a rate of approximately one foot per century thus causing an associated shoreline recession on the order of one to three feet per year depending on the characteristics of the sediments forming the beach. Short-term storm erosion from a single severe storm may cause an abrupt erosional event amounting to the equivalent of several hundred years due to sea level rise. Erosion, due to storms however, usually recovers to the pre-storm or nearly the pre-storm condition within a period of weeks, months, or years depending on the magnitude of the storm. In addition to natural factors, man has caused perturbations on the natural processes and shoreline fluctuations. Construction of residences and industrial facilities in the coastal zone has resulted in the need to protect these investments against the encroachment of the sea due to erosion resulting from both the slow rise in sea level and fluctuations due to storms. In many cases, the approach to protecting this investment has been by structures designed to limit the erosion resulting from the above mentioned effects. These structures fall into two classes including seawalls and groins and in many cases, these structures have accomplished their purpose. However, there are numerous examples where they have also induced severe erosion of the adjacent shorelines. In addition to shore protection structures, navigational structures (jetties) have been built to provide for efficient commerce and safe refuge to calm waters through many natural and artificial channels along the coastal zone of the United States. These channels may be on the order of 15 or so meters deep and represent extreme barriers to natural sand transport processes.

The cost of coastal structures and the evident need to be able to predict their effects on the stability of adjacent shorelines is well documented in much of the available engineering, scientific and governmental literature.

The costs of coastal structures and/or beach erosion remedial measures is very substantial. As examples of costs, the Corps of Engineers has recently designed two "weir-jetty" projects along the South Carolina coast (one is now under construction). Each of these projects costs on the order of $15,000,000. Other weir-jetty projects have been designed costing up to approximately $60,000,000. As an example of the cost of a non-structural approach to beach erosion control, a beach nourishment project is presently being installed at Miami Beach, Florida over the segment between Baker's Haulover and Government Cut. This ten mile segment will comprise the placement of approximately 10,000,000 cubic yards of sand dredged from offshore locations and will cost upwards of $60,000,000.

In recognition of the need for a greatly improved basis to calculate the impact of structures on adjacent shorelines, the maintenance costs associated with a shoreline project and the establishment of national regulatory measures in the coastal zone, the Sea Grant Office established the National Sediment Transport Study (NSTS) as the first national program. This program identified an important, but somewhat limited, objective of the overall coastal sediment transport problem discussed above. In particular, the specific objective defined was, through a series of extensive field investigations, to allow accurate quantification of sediment transport along a straight unobstructed shoreline. As part of the investigative program of the NSTS, longshore sediment transport rates and the relevant wave characteristics are to be measured at several field sites. One mode of measurement, where deemed possible and representative, is to be through impoundment at physical "traps" along the shoreline. The purpose of the trap experiments is to provide a long-term impoundment of the total material moving within the surf zone and also to measure the "forcing function" by a means that could be readily deployed at many locations along the shoreline of the United States. Through the correlation of the directional wave characteristics and the impounded volumes, a substantially improved capability to predict longshore sediment transport should result.

PURPOSES OF STUDY

The general purposes of this study were to develop and apply methodology for the selection of a site for a large-scale field trap experiment along the west coast of the United States. The development of methodology includes consideration of wave and sediment characteristics at various candidate sites, and also logistics. The latter factor is quite important since the construction of a sediment trap at a location considered to be ideal from other considerations could cost from one-half to several million dollars. Therefore, the need was clear to attempt to take advantage of physical characteristics and/or utilize existing operations which could complement the trap studies. An additional, very important, characteristic of a trap is that the measurements conducted therein should be free from any bias or the bias should be small and quantifiable.

For this first field trap it was considered important that the wave characteristics be as simple as possible to allow development and testing of the field methodology and also to provide a proper basis for the interpretation of later field trap studies which will be conducted under more complex wave and sediment conditions. Ideally, the trap would be located in an area where the wave direction was reasonably constant so that the trap impoundment results and the need for field measurements would not include the effect of transport reversals. In addition, the waves at the site should be of moderate and reasonably constant wave height. Other factors of importance that will be discussed in greater detail later for the west coast, included whether the beach profile and sand characteristics are reasonably representative, a straight beach alignment, availability and reliability of previous experience and data at the site, and the accessibility of the site from the standpoint of field experiments and installation of instrumentation.

REVIEW OF GENERIC TYPES OF TRAPS

In this section the various types of possible "traps" will be reviewed and their suitability for use in various wave climates and sediment conditions will be discussed.

Offshore Breakwater

An offshore breakwater such as shown in Figure 1 offers one possibility for the entrapment of longshore sediment transport. The Coastal Engineering Research Center of the U. S. Army Corps of Engineers has recently completed a four-year field study using the Channel Islands Harbor (CIH) offshore breakwater shown in Figure 2 as an impoundment basin. Preliminary results from this experiment are presented by Bruno and Gable (1976). In the CIH study, the sediment was allowed to accumulate for intervals of approximately two years after which it was dredged and placed on the beaches downdrift (to the east) of Port Hueneme. The preliminary results from this study indicate that there could be an increase of approximately two in the presently accepted sediment transport relationship. Preliminary results also show that whereas the waves at this site are dominantly from the west southwest, there are periods of reversal during which sediment is transported updrift from the trap.

One of the unknown, but presently unquantified, problems with a trap of this type is its characteristic of introducing a bias (by overtrapping) in the material impounded. A very simple demonstration of this overtrapping characteristic is that if an offshore breakwater is placed parallel to shore in an area where waves propagate only perpendicular to shore, then through wave diffraction around the breakwater, the waves

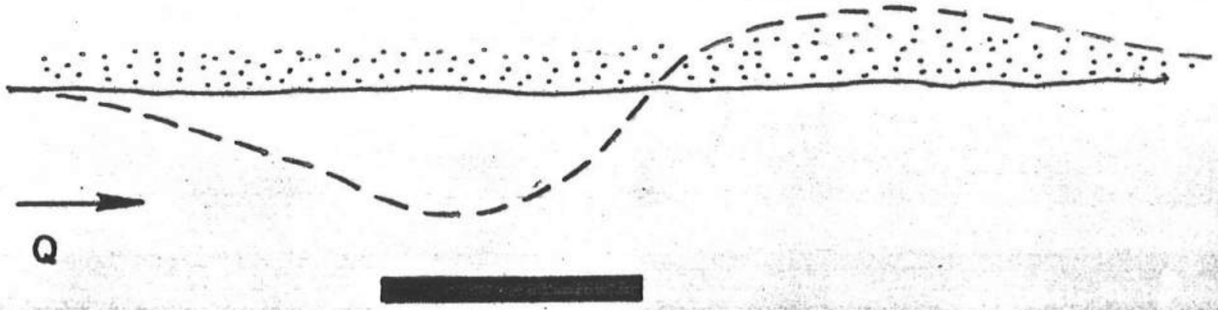


Figure 1. Response of Shoreline to Offshore Breakwater with Unidirectional Longshore Transport.

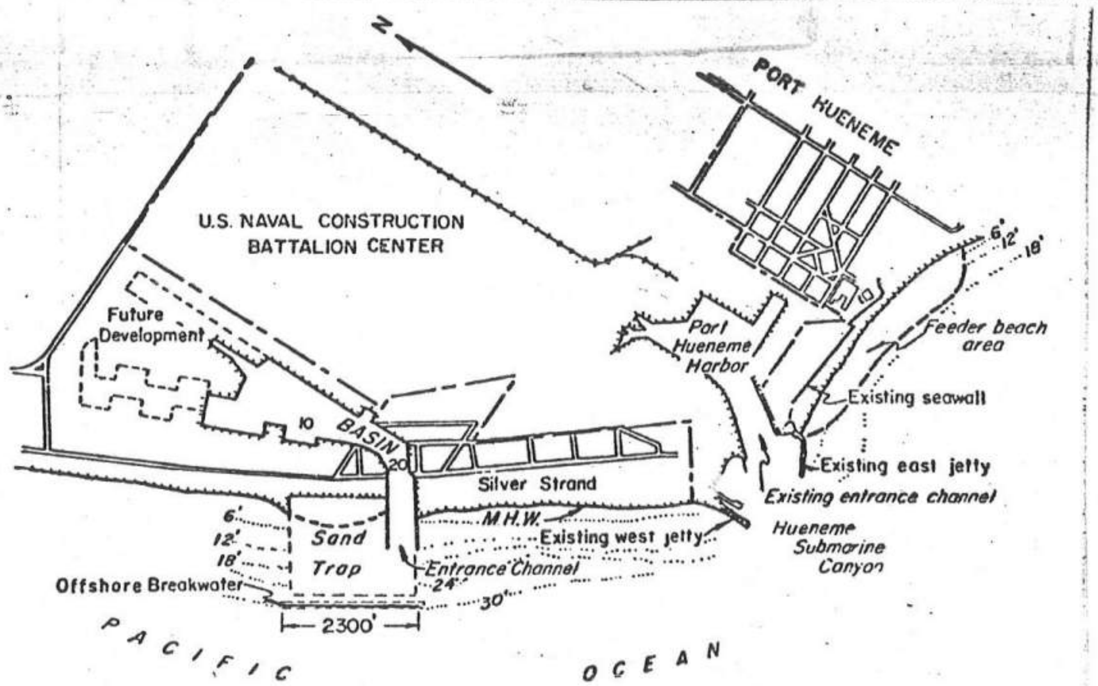


Figure 2. Channel Islands Harbor to Port Hueneme, California (U. S. Army Coastal Engineering Center).

will arrive at shore with an angle directed toward the centerline of the breakwater and will transport sediment to form a cuspate or tombolo planform, see Figure 3. Consider the effect of such a structure if an equilibrium condition were allowed to occur or if the impounded material behind the structure were continually removed. First consider the case where an equilibrium condition results. The equilibrium planform landward of the breakwater will be the aforementioned cuspate or tombolo which will stabilize when the wave crests at breaking are approximately everywhere parallel to the beach planform. At this stage, by definition, there will be no net transport as a result of the presence of the offshore breakwater. The local beach slope and wave orientation have adjusted such that the entire bottom is everywhere in equilibrium with the incoming waves. It is instructive to examine the result on the parameter, K, of relating the volume, $\Delta\forall$, of sand trapped behind the offshore breakwater to the longshore energy flux, ΔP, which occurs along the (apparently) unaffected shoreline during the time of impoundment through an equation of the form

$$\Delta\forall = K\Delta P \tag{1}$$

Then it is clear that an infinite value for the constant K would result since ΔP is zero. One could argue (correctly) that this obviously erroneous result is due to the fact that erosion has been caused on the shores adjacent to the breakwater and the correct volume, $\Delta\forall$, to be used in Equation (1) must include changes a sufficient distance away from the breakwater to account for the total effect of the structure (which would yield a <u>net</u> volume, $\Delta\forall$, of zero). In this idealized case, it is clearly impossible to determine the total $\Delta\forall$ with any accuracy, since the distance is infinite and the effect at any one cross-section is infinitesimal.

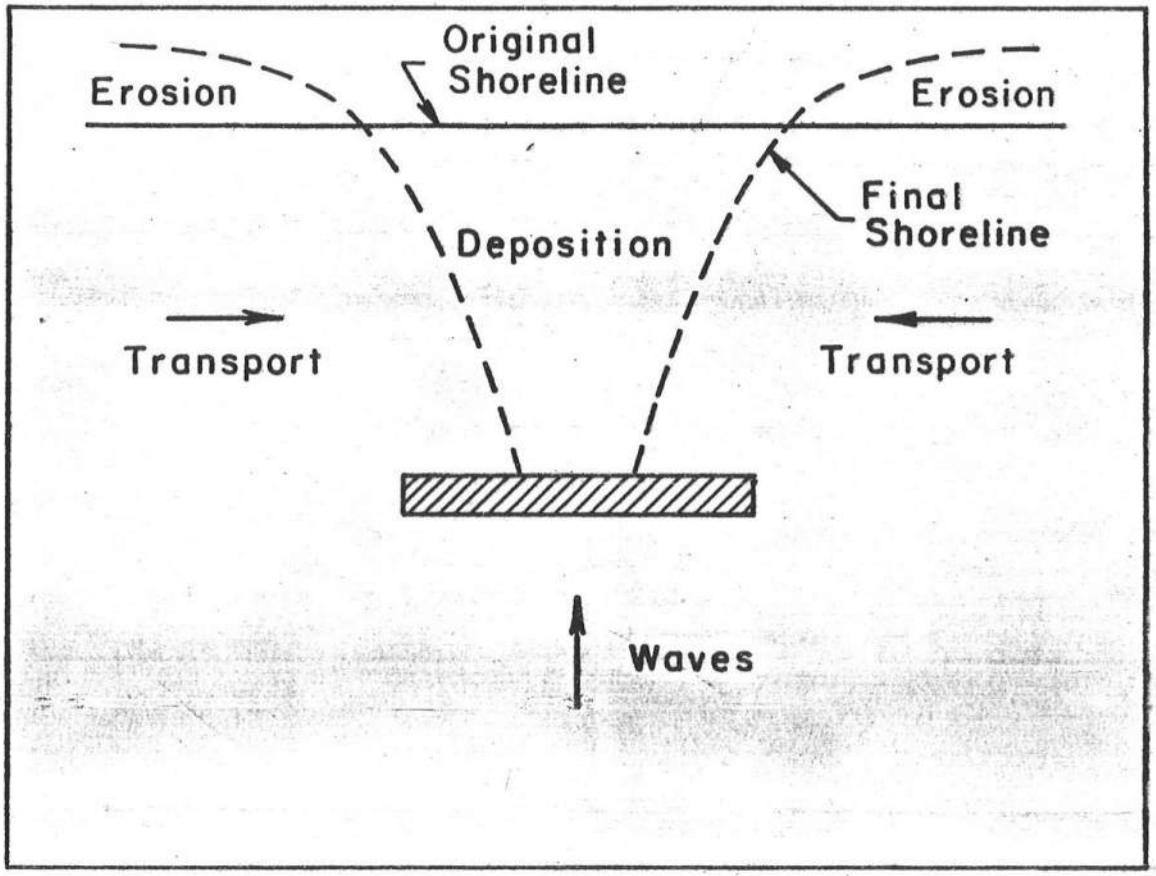


Figure 3. Tombolo Formation Behind Offshore Breakwater Due to Normal Wave Incidence.

As an extension of this scenario, consider the case where sand is now continually removed from the depositional feature behind the breakwater. Clearly, regardless of the mode or location of removal, the equilibrium conditions will no longer exist and the bottom profile and beach planform will adjust such that equilibrium conditions will tend to be restored through the continual influx of additional sand from the adjacent beaches. The rate at which this influx occurs is dependent on the deviation between the equilibrium topography and that which is caused by the withdrawal of material.

The above discussion has shown that in the case of an offshore breakwater and waves arriving normal to shore, the removal of sediment from behind that breakwater will result in sediment being transported from the adjacent beaches and stored behind the breakwater and would provide a bias in the transport from that occurring on the unaffected shoreline.

Turning attention now to the case of an offshore breakwater under conditions of waves propagating obliquely to the shoreline, it is evident that the bias discussed above is present although unquantifiable, especially when it is recognized that most waves break at a fairly small angle to the shoreline. It is interesting to address the problem at a location where there is a predominant net longshore transport and to ask what conditions are required in order that there be no bias caused by the trap. From the preceding discussion it is clear that there will be no bias in the total longshore transport (across the surf zone) only if the three-dimensional profile characteristics are such that the total transport across any cross-section is that which would occur if the breakwater were not present. This poses the question of what characteristics this profile would have for the case of a non-zero net transport. Although the three-dimensional configuration for unbiased transport is not known, it is clear that for a very small net transport, the configuration will differ very little from that for no net transport and for a very strong net longshore transport, the configuration will differ substantially. Stated differently, if sand is removed from behind the breakwater at too small a rate (less than the unbiased transport rate) the <u>local</u> transport will be reduced and accretion will occur. Conversely, if sand is removed at a larger rate than the unbiased transport rate, the local transport rate will be increased and local erosion will occur.

One desirable characteristic of a trap is that the bathymetric perturbations which it causes should be localized. That is, since the quantities of material impounded are to be determined through surveys, then in order that the surveys be reasonably accurate the volume should be definable within a fairly limited zone (since there are some errors in all surveys). Some indication of the possible problems with an offshore breakwater can be obtained by examining impoundment data from the Channel Islands Harbors experiment. As an example, the results of three surveys taken during the periods April 20-May 20, May 20-August 12, and August 12-December 1 in 1976 are presented in Figure 4(a-c). These represent the integrated trap volume, ¥, up to any given point, x, y, as defined by

$$\forall(x,y) \equiv \int_0^x \int_0^y \Delta h(x',y')dx'dy' \qquad (2)$$

in which x' and y' are parallel and perpendicular to the shore-parallel baseline respectively, and $\Delta h(x',y')$ is the change in depth at the location (x',y') between two successive surveys. By definition, the effect of the trap extends up to the point x_*, y_*, beyond which the volume defined by Equation (2) no longer changes. The volume plotted as a function of x and y for $x > x_*$ and $y > y_*$ would then be constant. Figure 4(a-c) presents the results for the aforementioned survey periods and it is seen that at the updrift limit of the offshore structure the integrated volume is still varying with x at the y coordinate corresponding to the offshore breakwater. This variation is most noticeable for the periods May 20-August 12, 1976 and August 12-December 1, 1976. Thus the influence of the offshore breakwater extends updrift of this extremity of the offshore breakwater.

There is an additional problem that can be significant in the analysis and correlation of wave and impoundment data for situations in which there is a reversal of wave direction. Consider the case during

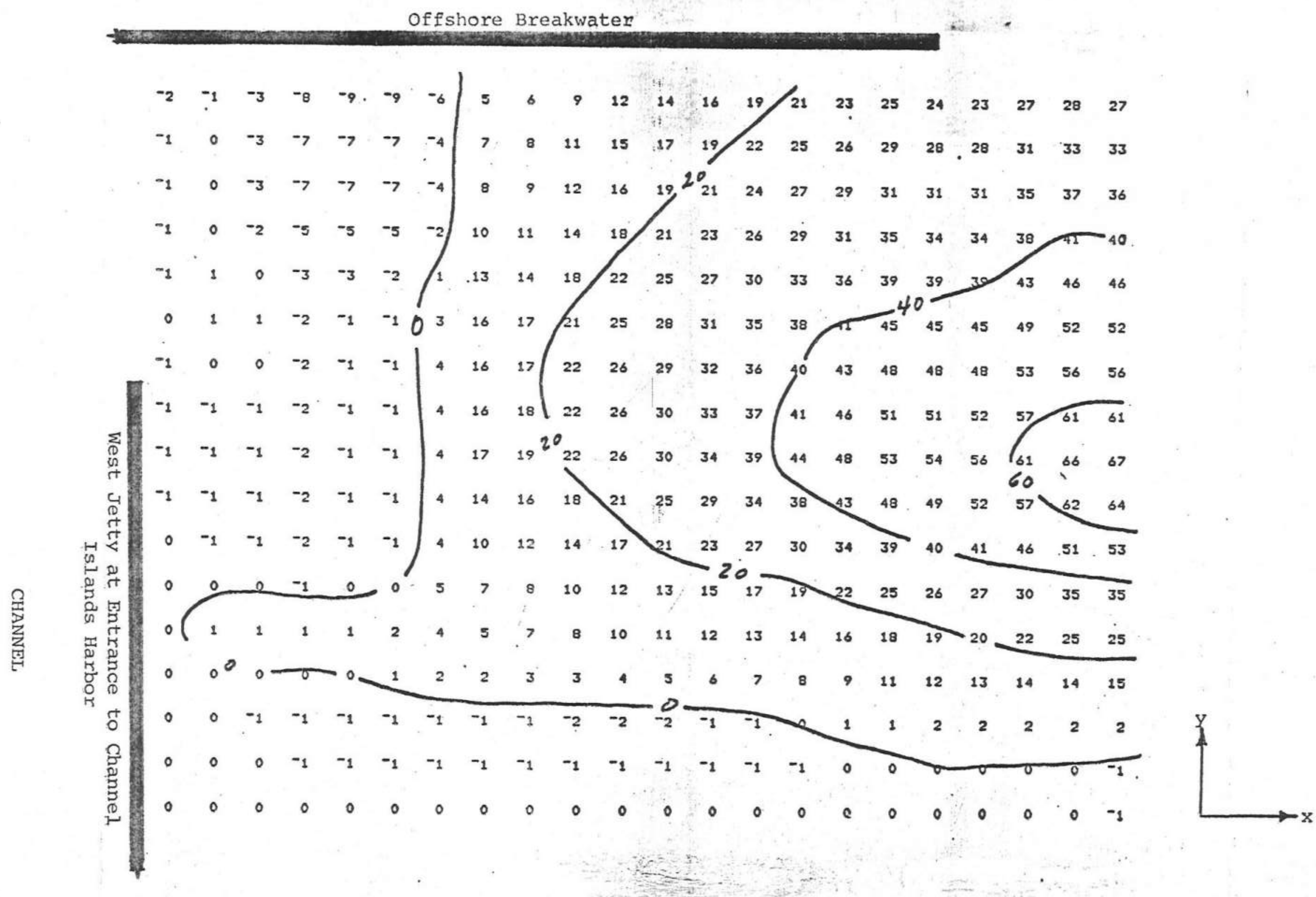


Figure 4a. Integrated Trap Volumes for Survey of April 20-May 20, 1976 for Channel Islands Harbor.

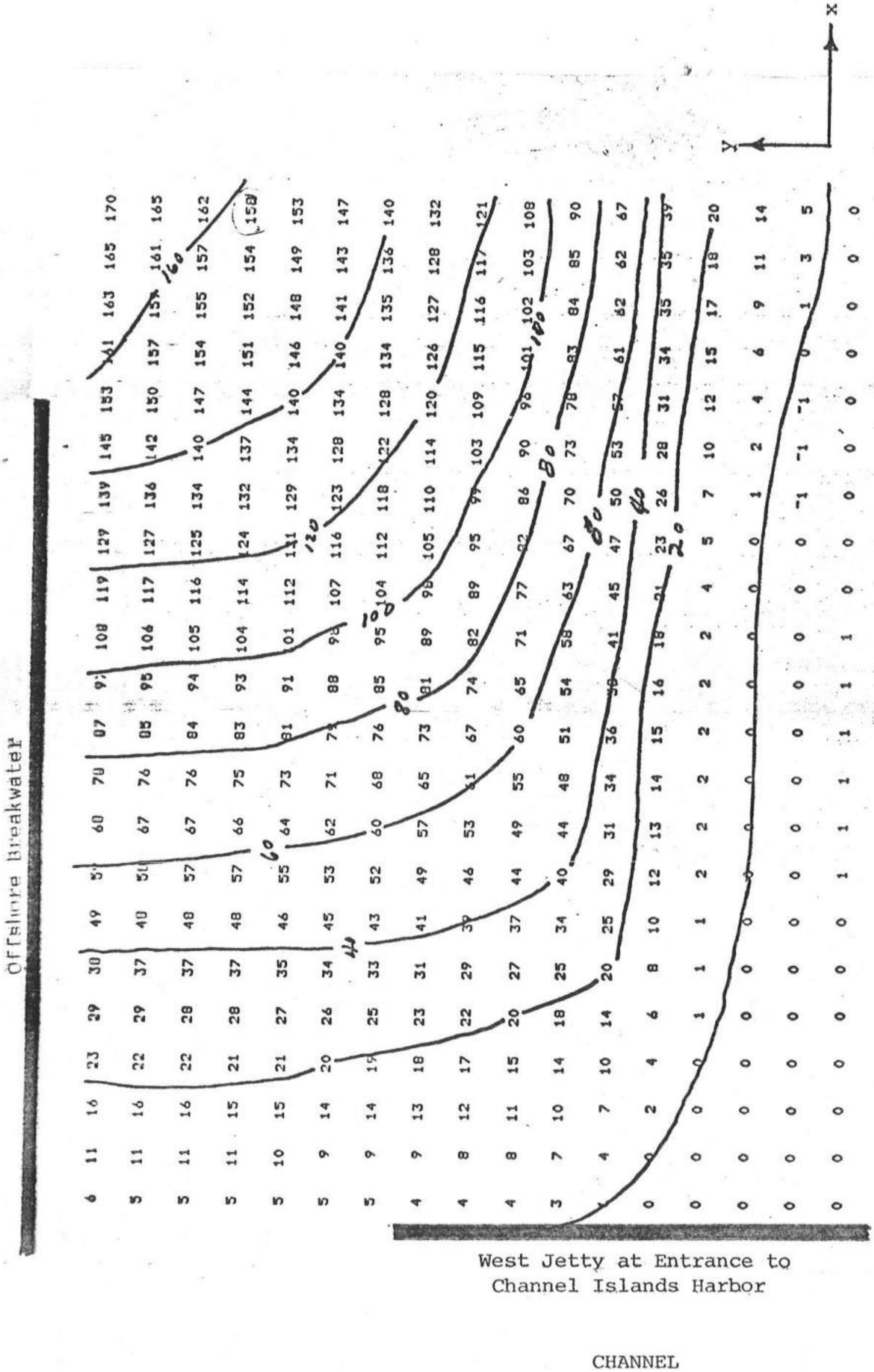


Figure 4b. Integrated Trap Volumes for Survey of May 20-August 12, 1976 for Channel Islands Harbor.

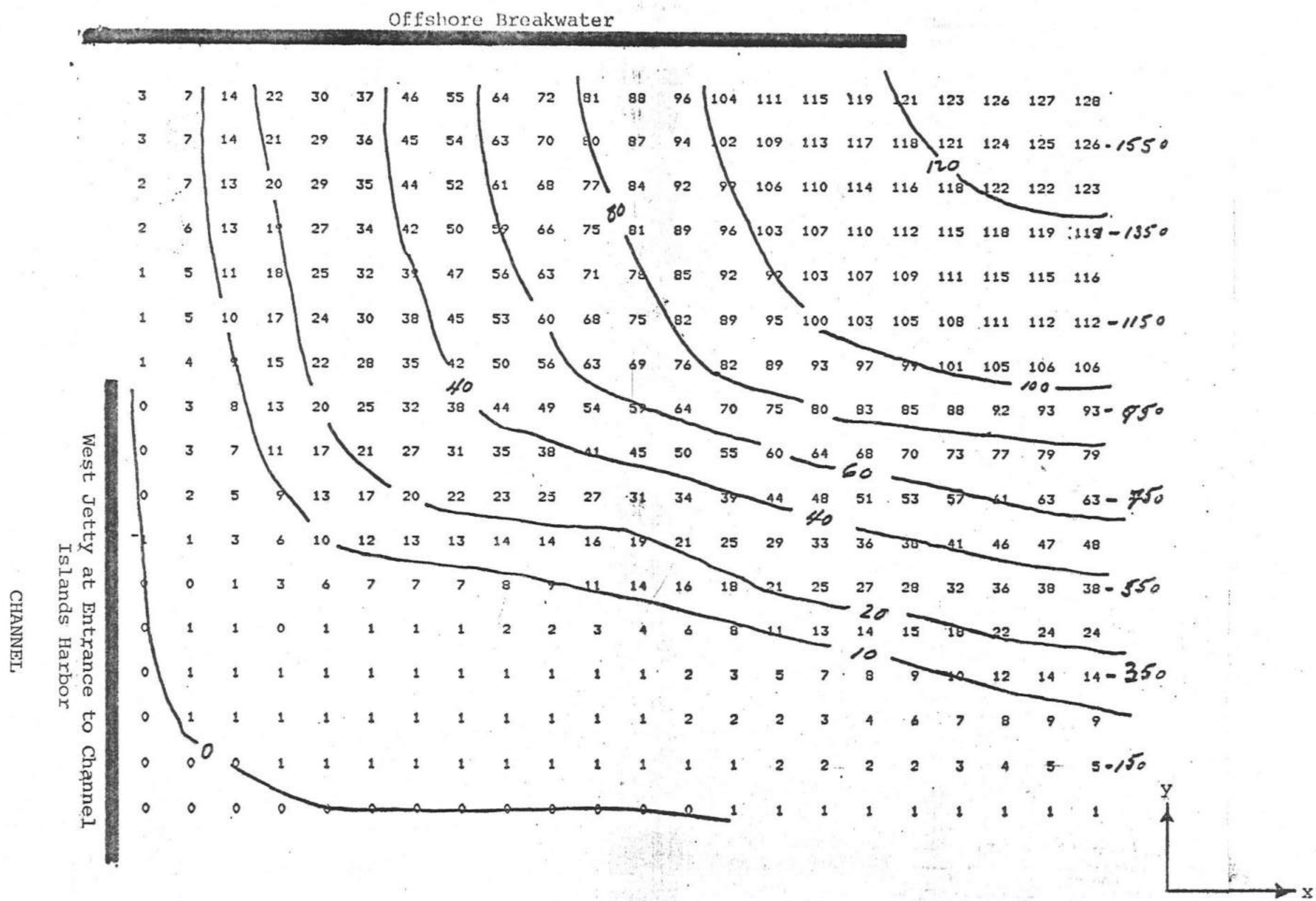


Figure 4c. Integrated Trap Volumes for Survey of August 12-December 1, 1976 for Channel Islands Harbor.

times when waves are propagating so as to transport sediment toward the impoundment area. The quantity of sediment transported is denoted ΔV_L and the integrated longshore wave energy flux is denoted ΔP_L. During the time that a reversal occurs the amount of sediment that will be transported to the right (looking offshore) over a particular time interval is denoted ΔV_R and the associated longshore energy flux, ΔP_R. The amount of sediment that is trapped in the impoundment area would appear to be ΔV_L and the appropriate measure of longshore energy flux would be ΔP_L. Inasmuch as the reversal of wave direction causes an associated erosion of the shoreline updrift of the structure an equally valid correlation could be obtained between the difference between V_L and V_R and the associated difference between longshore energy fluxes in the two directions. This would require the measurement of the region of erosion (updrift); however, this region is not well defined and may extend over a substantial length of beach. Consider now an extension of the above described cycle into a series of such cycles and it is now clear that since there was an erosion updrift of the structure during transport reversals, there will not be as much sediment carried into the trap during periods of downdrift wave energy flux. Over a long period of time it is therefore difficult to establish clearly the proper combinations of impoundment volumes and longshore wave energy fluxes to obtain a valid correlation. If this problem is examined more thoroughly it is found that the basic problem lies in the fact that our present relationship between longshore energy flux and longshore sediment transport is based on a beach configuration in which the contours are straight and parallel and that the effect of the offshore breakwater and impoundment area is to cause a perturbation of the bottom contours such that they are no longer straight and parallel and in fact slopes exist in a direction parallel to the unaffected beach alignment.

It is true in the idealized case that even for the problems discussed above, if it were possible to carry out accurate surveys sufficiently far updrift to a point of "no influence", then the volume of sediment change within this region could be related to the longshore energy flux <u>relative</u> to the orientation of the unaffected beach. The practical problems are that first of all, it is very difficult to define the location of "no influence" and secondly, generally this location is sufficiently far updrift that the cumulative errors involved in the profiling and summation across the individual profiles are too large to be satisfactory for the desired level of accuracies in this study.

In summary of the suitability of an offshore breakwater for purposes of impounding sand for correlation with measured longshore energy flux (or some other appropriate measure), the following are noted. First, it is highly desirable that the areal extent of the perturbation caused by the structure be kept fairly small. Secondly, if this is to be accomplished then one must be able to establish and control the profile within the impoundment region behind the breakwater such that there is little perturbation of the total transport across a section slightly updrift of the breakwater. In particular, in cases where the removal of material behind an offshore breakwater is done rather infrequently it is easily visualized that immediately after dredging, the trap should "overtrap" and influence (draw down) the updrift profiles significantly and as the approximate equilibrium volume behind the breakwater is approached, then the sediment transport rates would approach those of the unaffected shoreline except for the fact that during this period the updrift shoreline would have adjusted to result in less longshore energy transport, see Figure 5. Thirdly, the problems discussed above

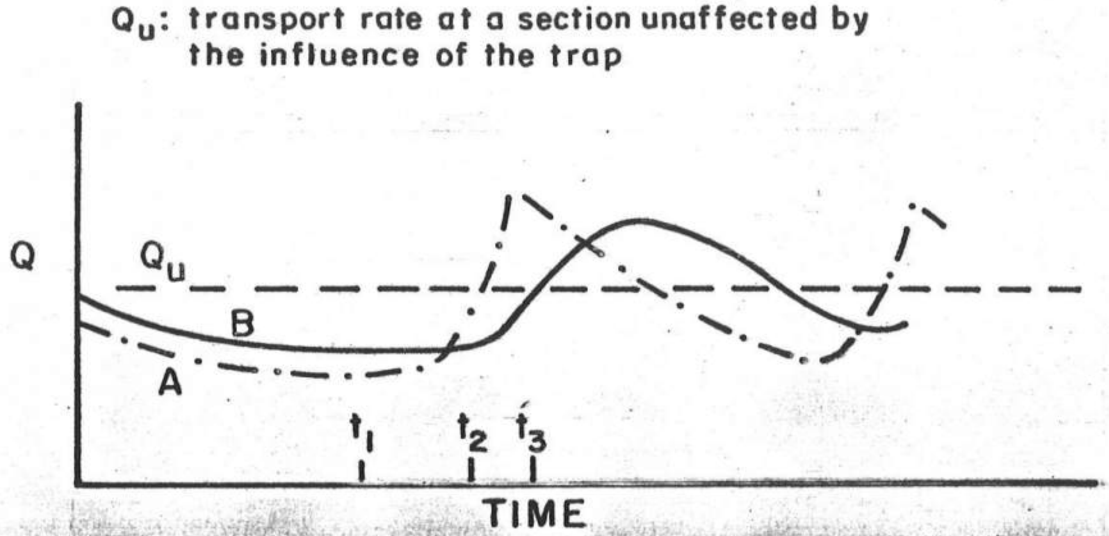


Figure 5a. Longshore Transport Rate at Two Sections, (A, B) Updrift of Offshore Breakwater.

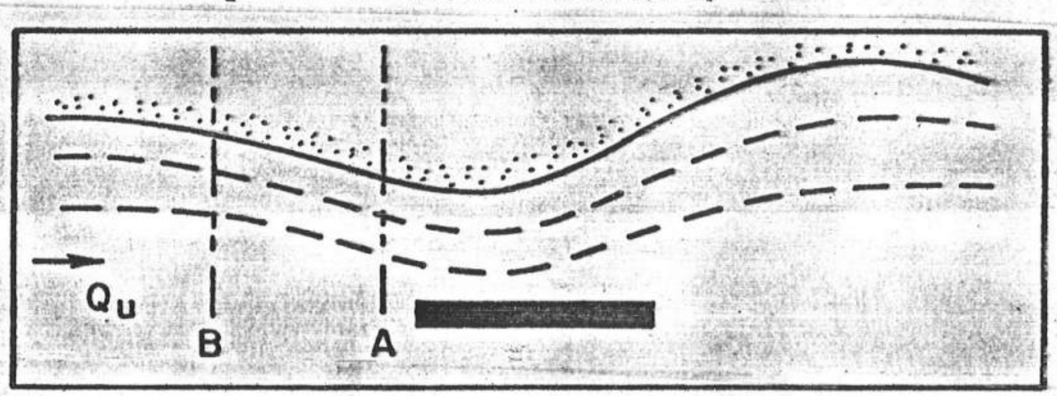


Figure 5b. Trap Filling at Rate Q_u Prior to Dredging, Time t_1.

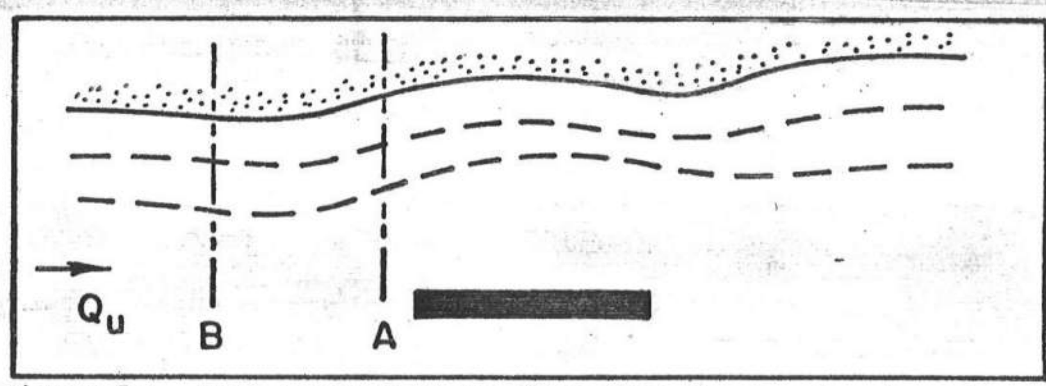


Figure 5c. Trap Dredged to Configuration Shown. Overtrapping Begins, Time t_2.

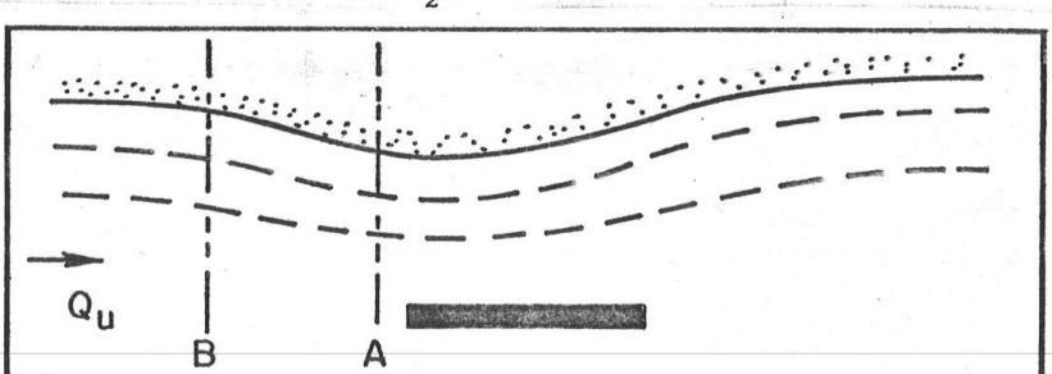


Figure 5d. Drawdown of Updrift Contours Due to Overtrapping Immediately After Dredging, Time t_3.

are even further complicated for the case in which transport reversals occur. Finally, it is concluded that the effects of an offshore breakwater are such that unless dredging control is available to significantly reduce the artificial fluctuations in longshore sediment transport through infrequent dredging, the uncertainties in the correct impoundment volumes and appropriate longshore energy flux values will be so great that the level of accuracy of interest to this study will not be attainable.

Long Groin

One approach to establishing a sediment trap for longshore impoundment would be the construction of a long impermeable groin which would collect material on the updrift side and cause an associated erosion on the downdrift side, see Figure 6. A desirable form of this construction could be the so-called "king pile" groins in which the spaces between adjacent vertical king piles are spanned by wooden panels. Such structures can be made adjustable and the effect of the structures can be eliminated readily by removal of the wooden panels for short periods of time. In principle, an impoundment cycle could be initiated and the accretion and erosion patterns monitored along with the offshore wave climate to determine the required quantities for correlation (Eq. 1). In this phase it would be desirable to continually add wooden panels maintaining the panels only a sufficient distance above the adjacent updrift beach level so that water but no significant quantities of sand would readily pass over the panels. In this way the tendency for a rip current to be caused by the structure would be reduced substantially. These rip currents can cause significant transport of fine material beyond the limits of the surf zone. A significant advantage of this type of structure if it were adjustable is that the

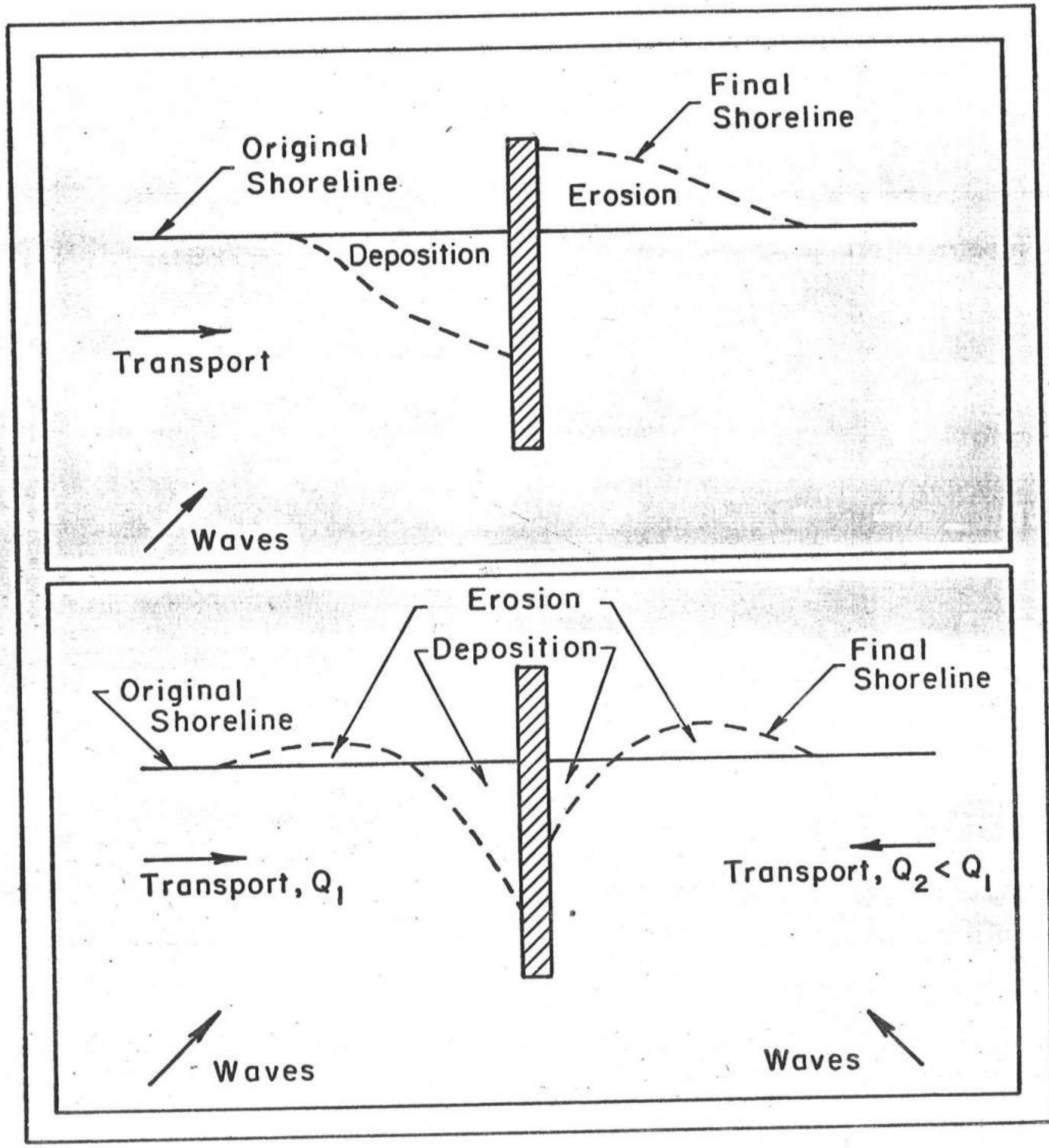


Figure 6. Effects of Long Groin Trap for Unidrectional and Reversing Wave Direction.

structure could be actuated for short periods of time allowing sand to accumulate and when a reversal occurred (such that the updrift extent of the shoreline perturbation would be increased) or when it appeared that bypassing of material around the tip of the structure was commencing, then that phase of the monitoring program could be stopped by removing the panels and allowing the shoreline to equilibrate. It is noted that a considerable amount of interesting qualitative information could be obtained by monitoring the equilibration phase.

Although no experiments of the above type have been carried out to the writers' knowledge and there may be unforeseen difficulties, it appears that a long, adjustable groin which is kept low during the impoundment phase offers a number of substantial functional advantages. An attractive feature is that the cost to install this type of structure should be on the order of $100/ft. of structure length.

<u>Short Groin With Frequent Bypassing</u>

Figure 7 presents the case of a short groin which extends to the limit of active sediment transport with more or less continuous bypassing around the groin. The principle feature of this arrangement is that the areal extent of the updrift impoundment could be kept small through the frequent measurement and bypassing of material impounded. This type of installation would be well-suited only for those areas in which the longshore energy transport is very nearly unidirectional and the transport occurs within a fairly narrow zone. For cases of bi-directional transport the updrift areas would tend to experience erosion and then upon transport reversal again, the initial conditions would not be well-defined. Therefore, if there were not a strong predominance of longshore sediment transport the uncertainties in the correct impoundment quantities would result in an unsatisfactory level of uncertainty in the resulting correlations.

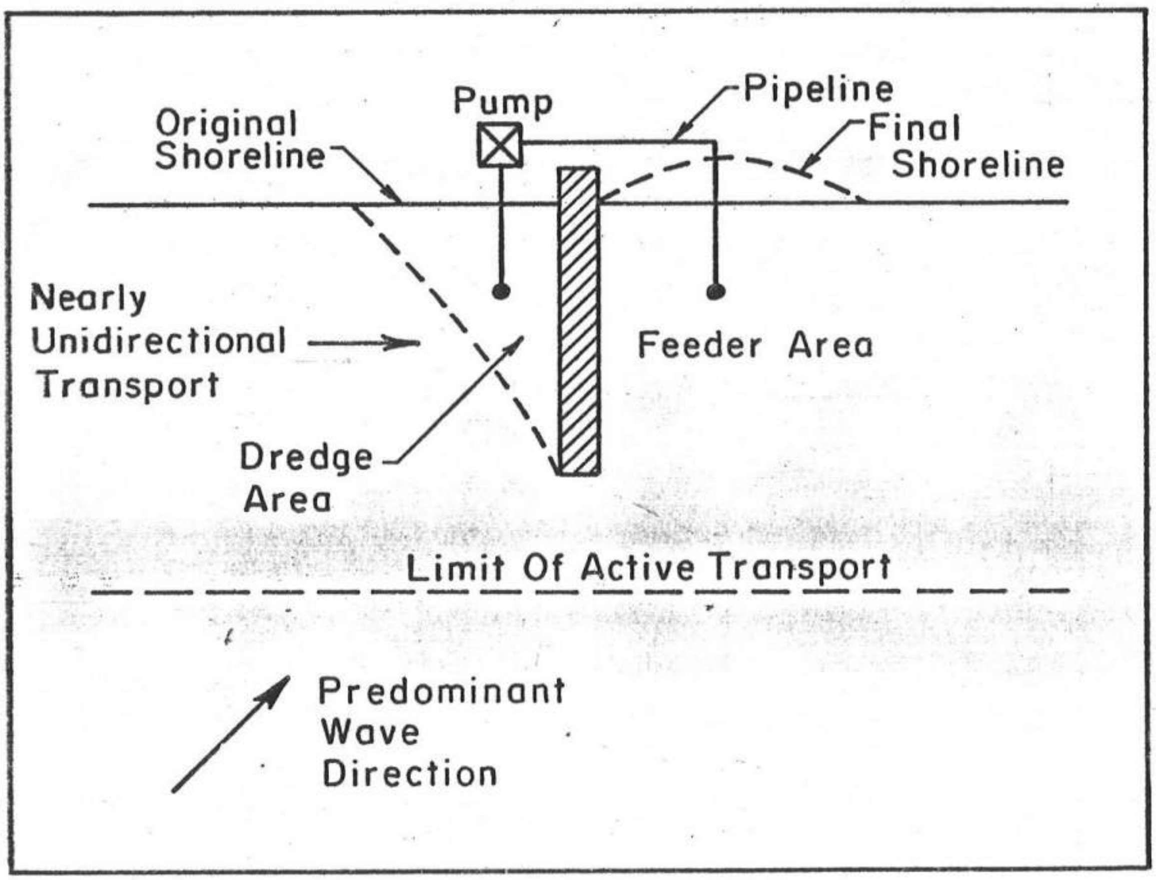


Figure 7. Short Groin Trap With Bypassing System.

Channel Dredged Across the Surf Zone

A possible type of trap would be that of a fairly wide channel dredged across the surf zone with the resulting deposition and erosion on the updrift and downdrift sides of the trap monitored to determine the longshore sediment transport. This type of trap suffers from many of the same features discussed previously for the offshore breakwaters.

That is, even with waves propagating directly toward shore the trap will experience filling and unless the profiles extend a considerable distance in both directions along the coast, the filling would result in a false indication of the longshore sediment transport. Also, as in the case of the offshore breakwater, these erroneous indications of longshore sediment transport would be reduced if the transport were unidirectional and if the longshore gradients of bottom topography were reduced through very mild side slopes of the channel.

In summary, this particular approach does not appear to offer sufficient potential to pursue the construction of this type of trap for impoundment measurements. However, if such a dredged feature were available in an area where the drift were known to be substantially unidirectional, it would appear sufficiently advantageous to monitor the filling of the trap and to carefully attempt to establish the uncertainties in the measurements as a result of the longshore component of bottom slope.

Sand Tracing

Although sand tracing, the injection and monitoring of an identifiable material with the same transport characteristics as the native sand, does not represent a physical trap it is considered here under the generic discussion of sediment traps. Sand tracing offers intrinsic advantages over other types of traps since it is completely passive and there are no physical obstacles which could cause undesirable side effects of the "trap". It is envisioned that in the present National Sediment Transport Study, sand tracing studies will usually be carried out in conjunction with measurements made at the physical traps. There are a number of rather substantial problems which are inherent in the application of tracer methodology for quantification of longshore sediment transport; several of these problems are discussed below.

Equilibration of Tracer Transport Across the Surf Zone - Consider the case in which there is an unknown distribution of sediment transport across the surf zone and that the transport rate in the longshore direction is very large compared to any transport rates across the surf zone. The advection of sediment injected at a particular location would indicate the transport rate at that particular point across the beach. If we now consider that in addition to the longshore sediment transport there is a certain "mixing" of bottom material across the surf zone then it is clear that in order to obtain a reliable measure of the total longshore transport, it may be necessary to conduct observations a great distance down the beach. The probability that the wave conditions including direction would remain constant over such a period required for the transport down the beach could be unlikely. An alternate approach to this problem is to attempt to establish a tracer injection rate across the surf zone which approximates the transport rate. The result downdrift will be tracer of uniform concentration across the surf zone.

Depth of Effective Motion in Sand Bed - One approach to the use of tracers is illustrated by the field studies of Komar and Inman (1970) in which sediment is trenched across a portion of the surf zone during periods of low tide and the sediment distribution sampled on the following low tide after the waves had acted upon and transported the sediment in the downdrift direction. An example distribution of tracer concentration is presented in Figure 8. It appears reasonable that the velocity distribution within the sand bed has the general form shown in Figure 9. The resulting tracer distribution within the sand bed would be very thin and close to the surface near the downdrift limit of the tracer concentration patterns and would be thicker and relatively deeply located in the sand bed near the updrift end of the tracer concentration patterns and would be more or less reasonably mixed over the upper portions of the active sand bed somewhere around the mean position of the tracer displacement. In sediment transport studies to date, the concentration of tracer existing within a certain depth in the sand bed has been measured and it has been assumed that there is a fixed depth within which the dominant portion of the tracer concentration occurs. If the hypothesized three-dimensional tracer concentration dispersal pattern shown in Figure 9 is correct, then it follows that sampling a fixed "thin" upper layer would tend to over-emphasize the displacement (and hence transport rate) of the tracer material.

Tracer Burial - It is well-known that the sand level at any location within the surf zone is not constant but fluctuates due to gradients in onshore-offshore and alongshore sediment transport. In tracer studies, the possibility exists that the tracer material may experience burial or abnormal transport due to these localized depositional and erosional patterns. If deposition occurs for a short period of time the result would be an apparent transport which is less than that actually occurring. Conversely, if the sediment is introduced in a location where erosion is temporarily occurring due to longshore gradients in transport, it follows that an excessively large value (compared to the spatial average) of longshore sediment transport rate will be indicated.

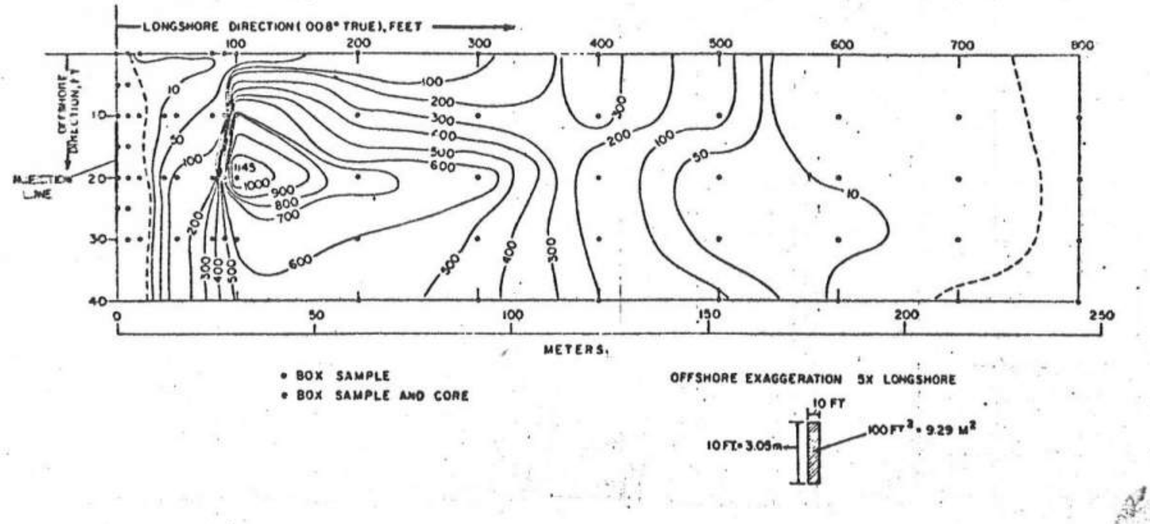


Figure 8. Example of Distribution of Sand Tracer Along El Moreno Beach. After Komar and Inman (1970).

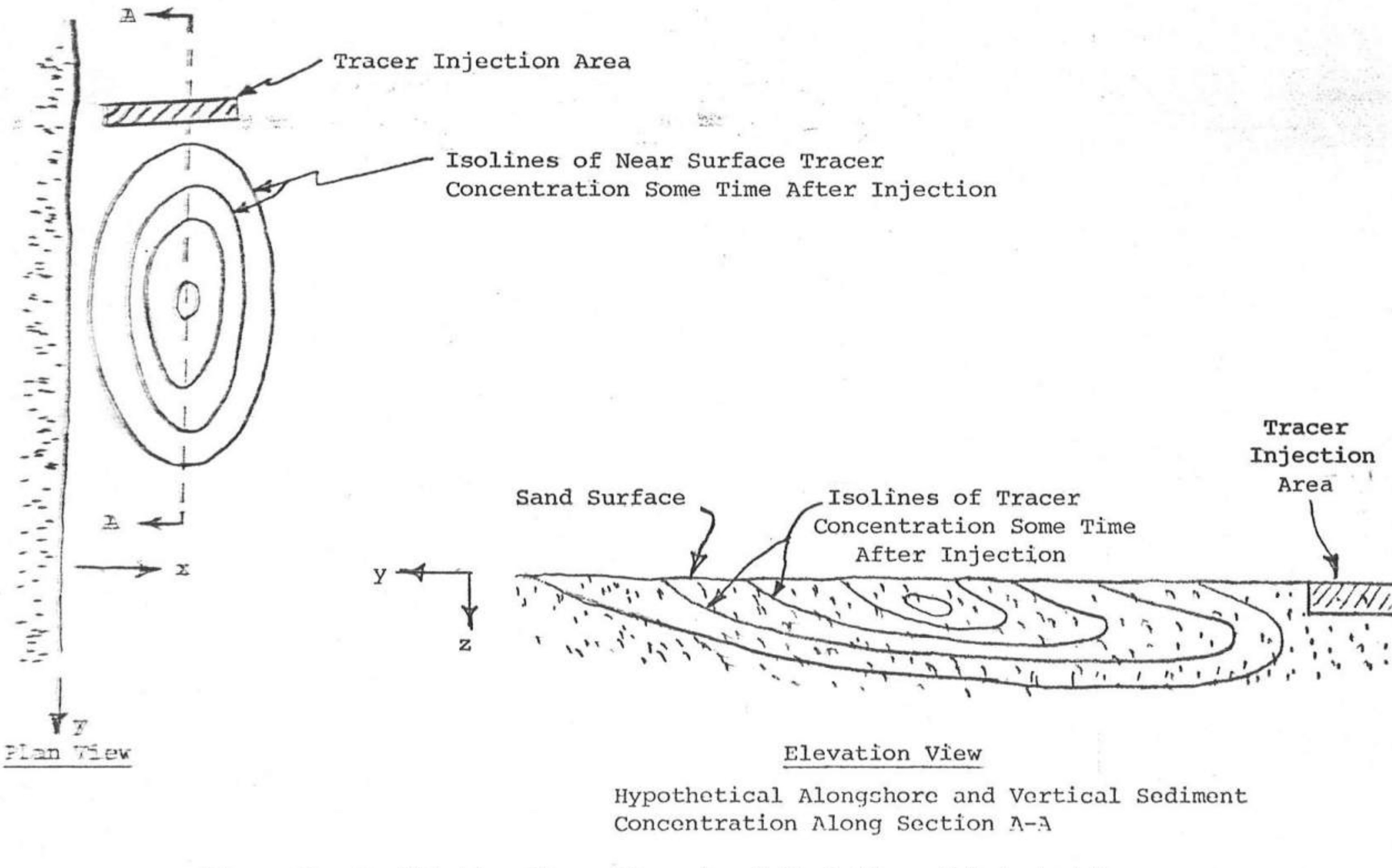


Figure 9. Qualitative Three-Dimensional Evolution of Injected Tracer Concentrations with Time.

SITES CONSIDERED AND AVAILABLE DATA

Based on the criteria described earlier the two Pacific Coast sites considered as locations for the first large-scale field trap of the National Sediment Transport Study were Channel Islands Harbor, California and Santa Barbara Harbor, California, as shown in Figures 2 and 10.

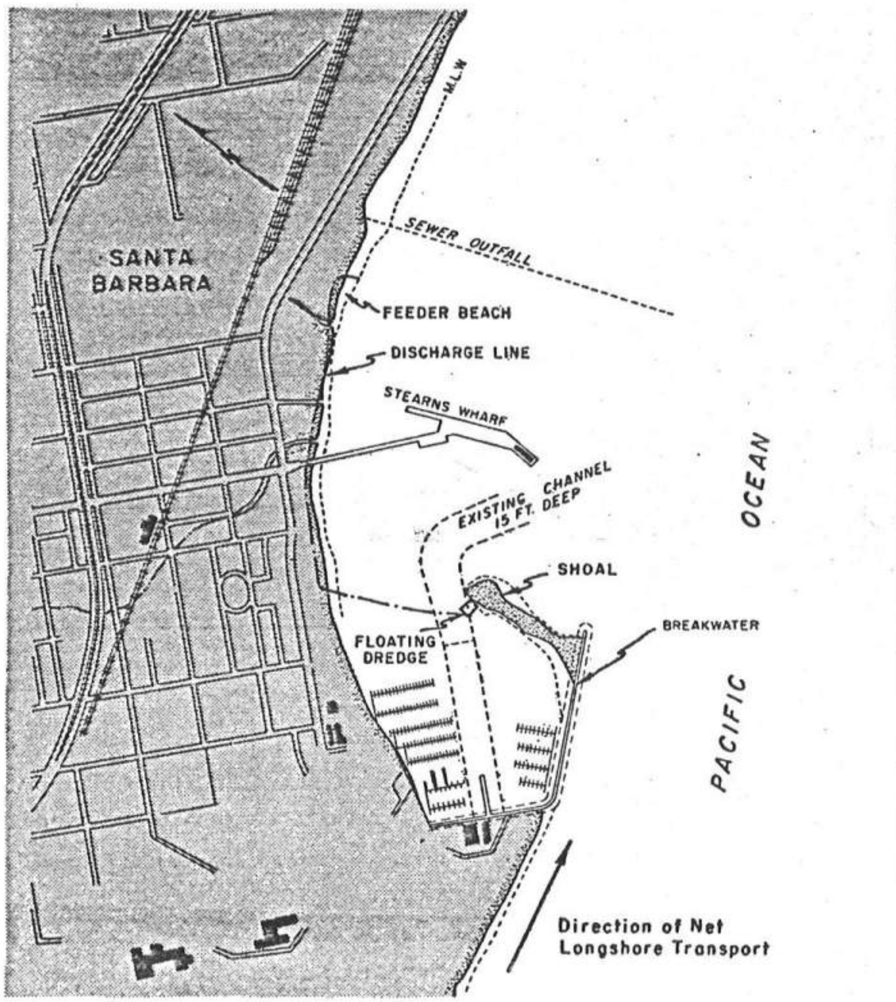


Figure 10. Santa Barbara Harbor, California. (U.S. Army Coastal Engineering Research Center).

The logistics available at each of these two sites weighed heavily on their consideration. At Santa Barbara Harbor, the material accumulated in the spit is transferred by dredge from the updrift (west side) to the downdrift (east side) of the harbor, see Figure 10. This transfer is scheduled on an annual basis but may be more frequent if severe storms cause undue shoaling of the channel. Volumes accumulated in the spit could be obtained by profiling between dredging. Channel Islands Harbor has been discussed earlier and provides a complete trap for the longshore sediment transport. In addition, the studies carried out by the Coastal Engineering Research Center at Channel Islands Harbor provide useful background information.

Santa Barbara Harbor

The history and general character of the waves and shoreline at this harbor have been described in a number of documents (U. S. Congress, House Document No. 522, 1938 and No. 761, 1948, Wiegel, 1964). In brief, the construction of the harbor was initiated in 1927 as an offshore breakwater to shelter the harbor and a gap of some 250 meters left between the breakwater and the shoreline to allow the longshore sediment transport to pass unobstructed along the shoreline. Within a few months following the completion in early 1929, it became evident that the sheltering provided by the offshore breakwater was so significant that the longshore sediment transport was being deposited in the lee of this breakwater and very soon would represent a significant impoundment within the harbor causing excessive shoaling in the areas where deeper water was required.

In early 1930 the breakwater was connected to the shoreline as shown in Figure 11 and immediately a fairly large beach began to accrete on the western side of the breakwater and an associated erosion commenced on the eastern side (downdrift of the breakwater). Later sand accumulated in a spit inside the distal end of the breakwater (see Figure 10). J. W. Johnson (1953) has estimated the accumulation rate between 1932 and 1951 as approximately 280,000 cubic yards per year. The U. S. Army, through its Beach Erosion

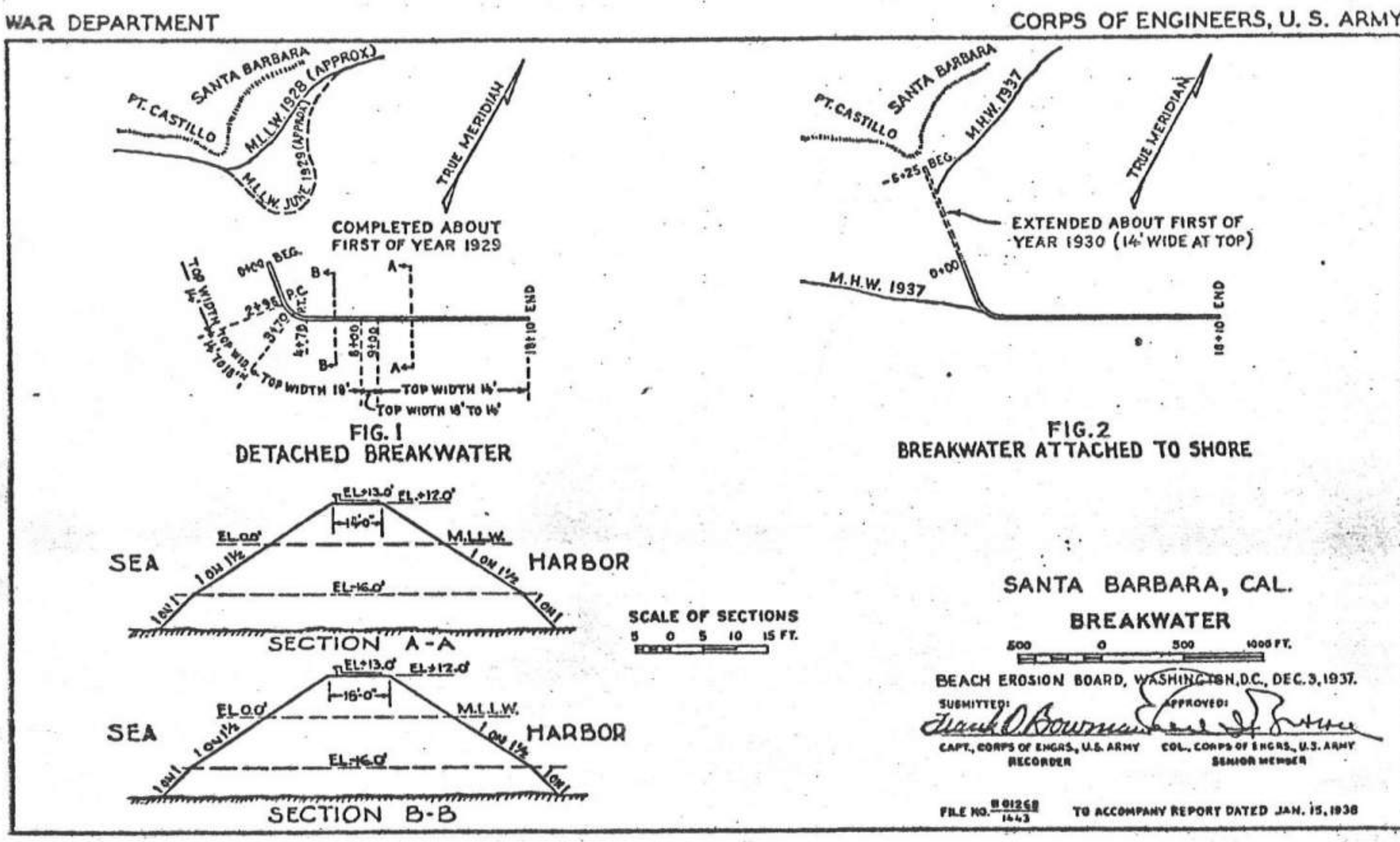


Figure 11. Offshore Breakwater Entrapment of Sediment at Santa Barbara Harbor (U. S. Congress, 1938).

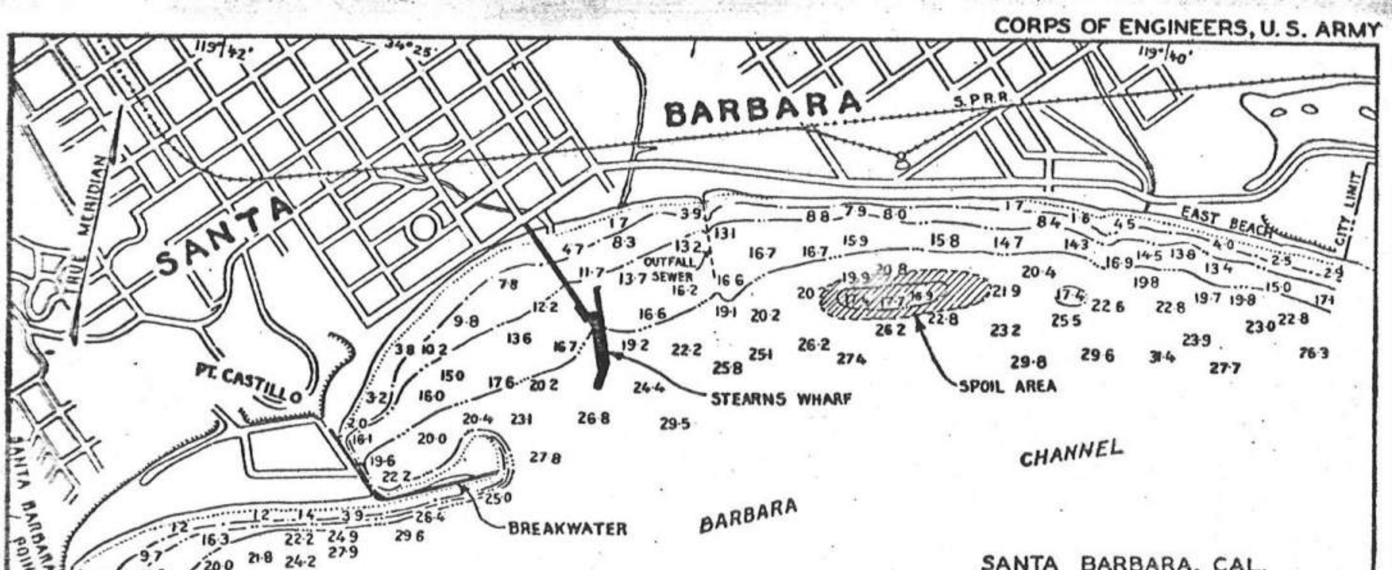


Figure 12. "Feeder Mound" of Sediment Used to Compensate for the Erosion of the Beach After the Breakwater was Connected to the Shoreline. (U. S. Congress, 1938).

Board and local district (Los Angeles) carried out a number of studies to determine the proper remedial action. The first such remedial action was carried out in September 1935 and consisted of the placement of 202,000 cubic yards of material in a "feeder mound" east of the Harbor (at East Beach) extending over a 2200 ft. length in a water depth of 22 ft. about 1000 ft. offshore as shown in Figure 12. There then followed a series of periodic dredging events and studies to establish the effectiveness of this means of bypassing sand material in reducing the downdrift beach erosion. A report summarizing the results of these studies was issued in 1948 and it was established that the downdrift beaches especially near the area where the beach nourishment material was placed had nearly recovered to their conditions prior to the construction of the breakwater. In recent years, a contract dredge has operated at the terminus of the spit formed by sediment transported around the tip of the breakwater. Moreover, observations have shown that the beach updrift of the breakwater has achieved a long-term stability.

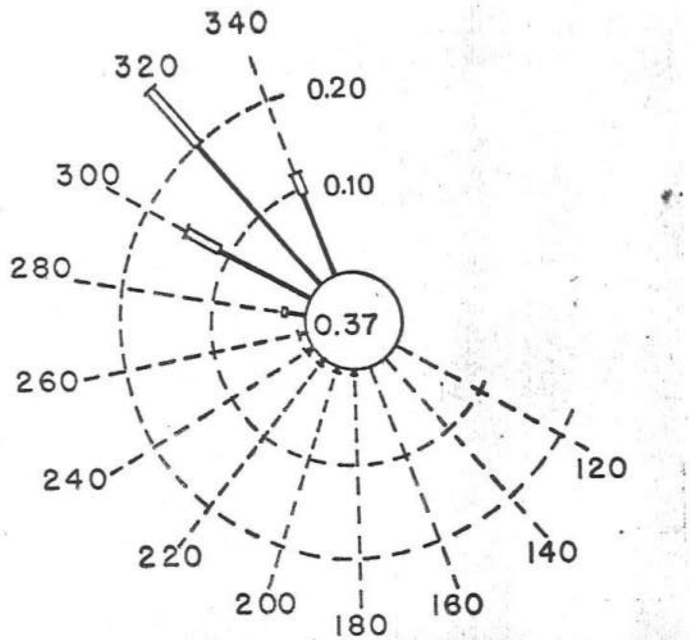
In summary, the Santa Barbara Breakwater and navigational channel represent a system in which the beaches have stabilized reasonably well and at which a complete barrier to natural sand transport processes occurs and has been replaced by a floating dredge which bypasses material once or twice a year. This bypassing operation requires approximately one month per year.

Tides - The tides at Santa Barbara are of the mixed type with a mean range of 1.10 meters and a diurnal range of 1.62 m.

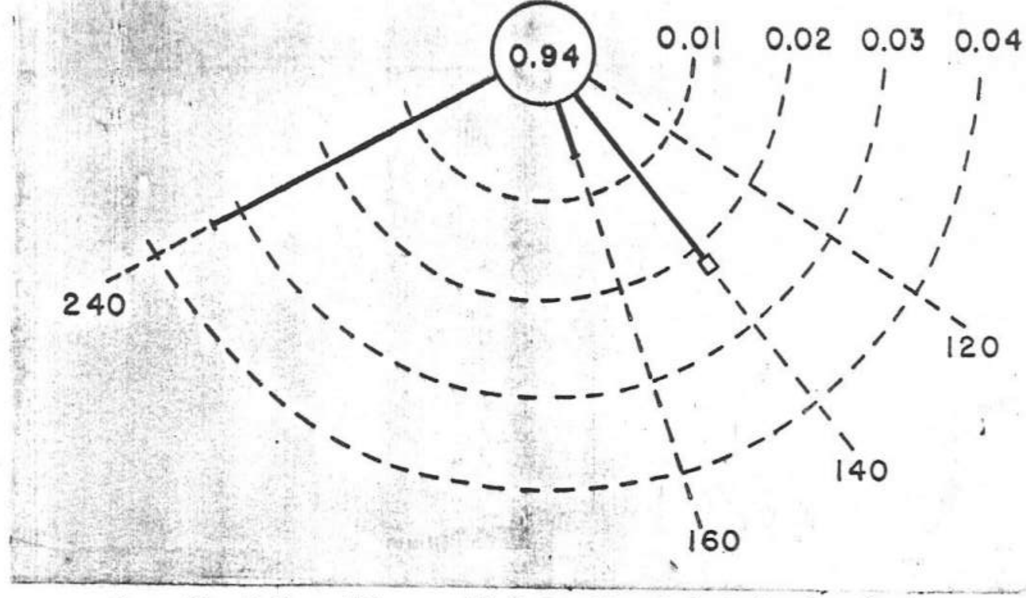
Sediments - The beach sand in the vicinity of Santa Barbara is fairly uniform with a mean size in the range of 0.2 mm.

Waves and Wave Exposure - The wave conditions along the California coast have been investigated in a number of studies--O'Brien (1950), National Marine Consultants (1960), Marine Advisers (1961), and Meteorological International, Inc. (1977). The dominant, unaffected, deep water wave direction in this portion of the Pacific is from the west and northwest although significant deep water wave energy does originate from the southwest. Figure 13 presents annual deep-water sea and swell "wave roses" for Station 5 as determined in the 1976 study carried out by Meteorological International Incorporated for the Department of Navigation and Ocean Development. The offshore islands (San Miguel, Santa Rosa, Santa Cruz, and Anacapa Islands) which are separated from the mainland by the Santa Barbara Channel exert a substantial modifying influence on the deep water wave climate noted above. Figure 14 shows these islands and it is evident that they provide a very significant screening to the incoming waves from the sector ranging from south-southeast to southwest and for the sector west to southeast (proceeding clockwise). The waves from the northwest are modified very substantially such that the effective deep water direction at the Santa Barbara site is limited to a fairly narrow range.

As an approximate measure of the effect of the Island chain described previously in modifying the deep water climate, the sector windows shown in Figure 14 represent the direct exposure of the Santa Barbara Site to waves from various deep water directions. If, as a first approximation, waves coming from directions other than those directly exposed to deep water are eliminated from the deep water wave climate, then the effective deep water wave climate is as shown in Figure 13 where it is compared with the deep water climate at Station 5 as determined in the reference above. It is seen that the effective deep water wave climate is such that the waves approach the shoreline with a definite predominance from the west and that



a-1) Deep Water Wave Height Sea Rose.



a-2) After "Screening" By Channel Islands Chain.

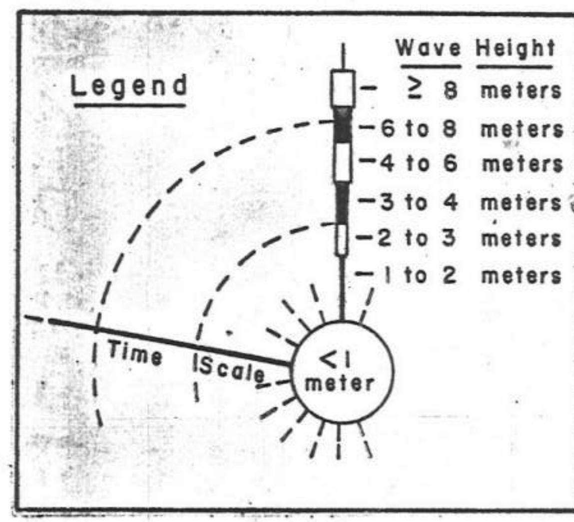
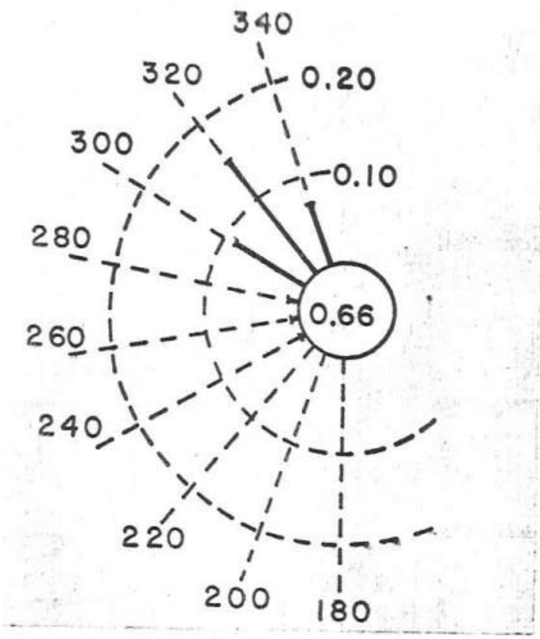
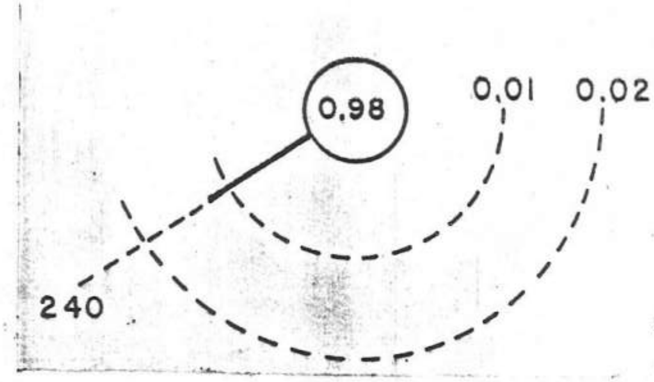


Figure 13a. Annual DNOD Station 5 Sea "Wave Height Roses" in Deep Water and at Santa Barbara Harbor Location After Screening by Channel Islands Chain.



b-1) Deep Water Wave Height Swell Rose.



b-2) After Screening by Channel Islands Chain.

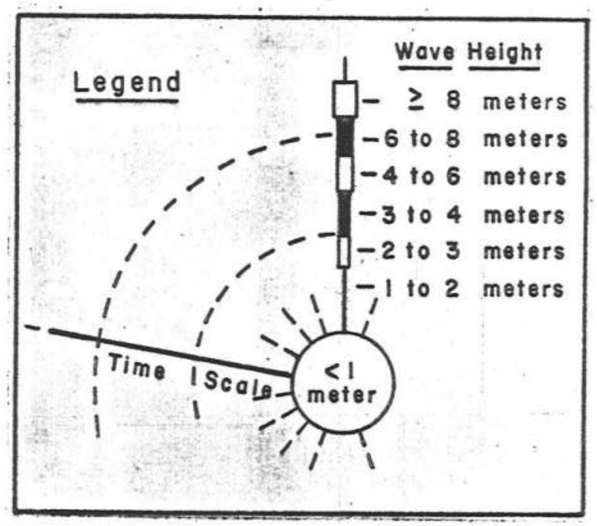


Figure 13b. Annual DNOD Station 5 "Swell Height Roses" at Santa Barbara Location After Screening by Channel Islands Chain.

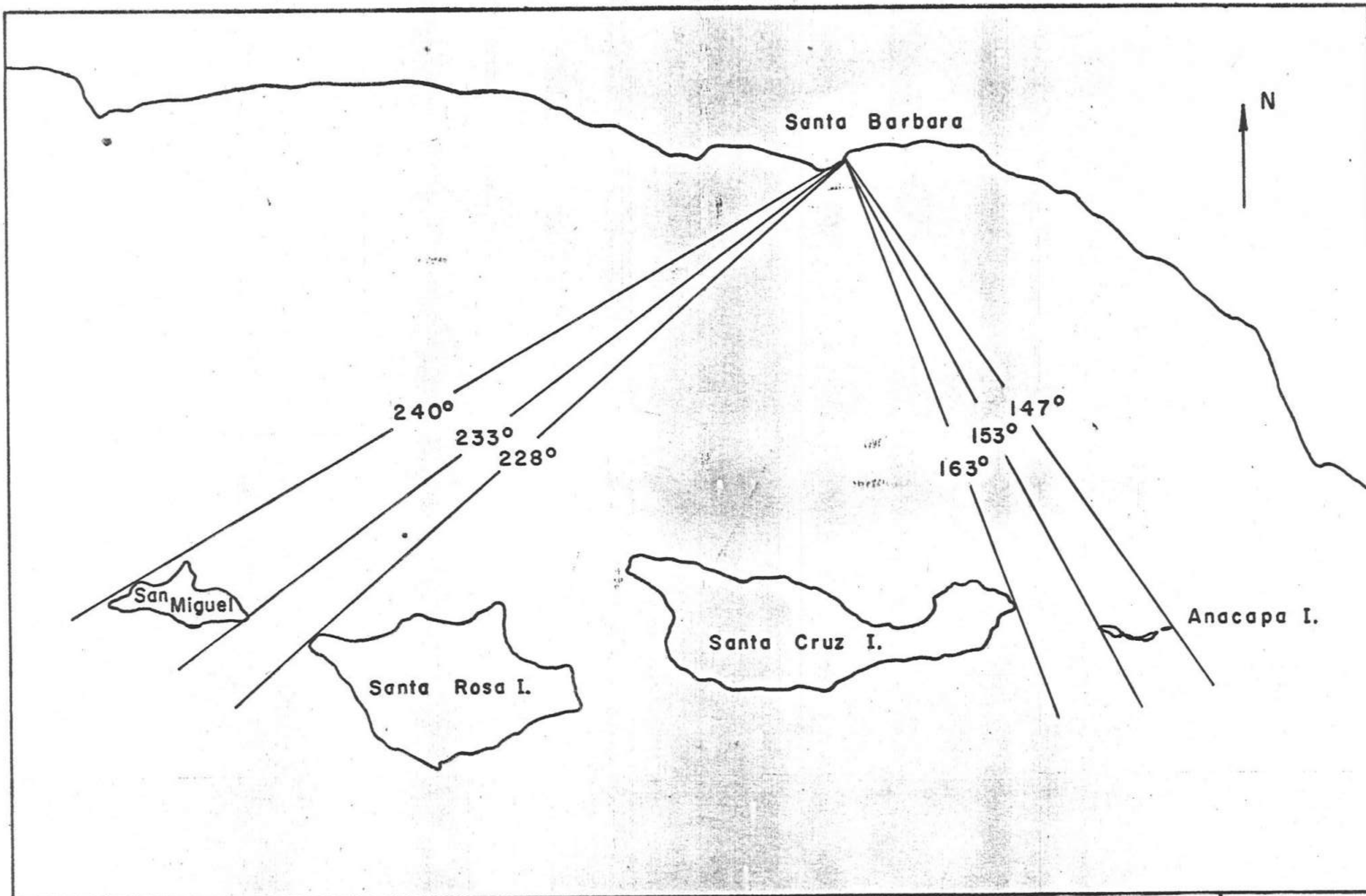


Figure 14. Direct Exposure of Santa Barbara Harbor to Deep Water Waves.

the longshore sediment transport at the Santa Barbara site should be almost unidirectional. This is in accordance with observations made at the site both from the standpoint of longshore currents and sediment transport.

The longshore transport characteristics based on the deep water wave calculations and the sheltering effects of the islands have indicated that there is a strong predominance of longshore transport from west to east.

Longshore Sediment Transport Estimates Through Impoundment - Estimates of net longshore sediment transport at Santa Barbara have been based on the rates of sand accumulation developed by Johnson (1953) and the dredged quantities, information for which was kindly made available by the Los Angles District of the U. S. Army Corps of Engineers. Prior to 1952, an attempt was made to maintain the full width of the entrance through dredging the spit that accumulated at the breakwater tip. In 1952 a decision was made to let the spit develop as additional wave protection; therefore, all dredging was discontinued until 1956. At the present time the spit is approximately 300 meters long and is stabilized by a wooden structure along approximately 275 m of that distance. Table 1 presents the estimates by J. W. Johnson of the volumes accumulated inside the Santa Barbara Harbor from 1932 to 1951 and Table 2 presents the times and quantities of documented dredging events. From 1956 to approximately 1972, dredging was accomplished by a small dredge operating on a more-or-less continuous basis. Attempts to quantify the dredging quantities during this period were not successful. Estimates of the annual net longshore sediment transport quantities based on the periods 1932-1951 and 1972-1979 yield an average value of 217,600 m^3/yr.

TABLE 1

ESTIMATED ANNUAL VOLUMES OF ACCRETION IN SANTA BARBARA HARBOR OVER THE PERIOD 1932-1951 (Based on Johnson, 1953)

Year	Accretion (m^3)	Year	Accretion (m^3)	Year	Accretion (m^3)
1932	172,000	1939	214,000	1946	306,000
1933	203,000	1940	237,000	1947	253,000
1934	298,000	1941	200,000	1948	283,000
1935	153,000	1942	187,000	1949	253,000
1936	172,000	1943	161,000	1950	230,000
1937	157,000	1944	180,000	1951	217,000
1938	180,000	1945	226,000		

TABLE 2

TABULATION OF DOCUMENTED DREDGING EVENTS, SANTA BARBARA HARBOR

Year	Quantity (m^3)	Year	Quantity (m^3)	Year	Quantity (m^3)
1933	464,000	1947	492,000	1974	291,000
1935	155,000	1949	641,000	1975	88,100
1940	534,000	1952	898,000	1976	308,000
1942	459,000	1956 ↓	Not Known	1977	357,000
1945	549,000	1971	↓	1978	473,000
		1972	279,000	1979	157,000

Channel Islands Harbor Offshore Breakwater

The offshore breakwater at Channel Islands Harbor (CIH) as shown in Figure 2 serves two basic purposes. The extension of the breakwater across the opening to Channel Islands Harbor results in a sheltered zone and allows entering small craft to first seek shelter landward of the breakwater before navigating the channel through relatively protected waters. The breakwater also serves to provide a sheltered depositional zone updrift of the western jetty of the entrance to CIH. The predominately easterly longshore transport is impounded in this zone from which it is transferred by pipeline dredge, on a more or less biennial basis, downdrift of the entrance to Port Hueneme, some 1.6 kilometers to the east. The quantities of bypassed sand over the period 1960-1978 since the construction of the breakwater are presented in Table 3.

TABLE 3

QUANTITIES OF SAND BYPASSED AT CHANNEL ISLANDS HARBOR

Year(1)	Volume Bypassed (m^3)	Cumulative Volume Bypassed (m^3)	Running Annual Average Bypassed (m^3)
1961	2,008,486	2,008,486	2,008,486
1963	1,518,406	3,526,892	1,763,446
1965	2,696,585	6,223,476	1,555,869
1967	2,293,665	8,517,141	1.419,524
1970	2,159,103	10,676,244	1,186,249
1971	1,840,284	12,516,528	1,251,653
1974	1,337,971	13,854,499	1,065,730
1975	1,242,402	15,096,901	1,078,350
1978	1,911,387	17,008,288	1,000,488

(1) Completion of bypassing.

As noted previously, the Coastal Engineering Research Center has conducted a four-year study at Channel Island Harbor and has kindly provided the information presented in the following subsections. In addition, portions of the following are based on Bruno and Gable (1976).

Waves - Two pressure sensors were installed 1.3 km up-coast from the deposition basin in a water depth of six meters; however, to date these data have not been analyzed. In addition, Littoral Environmental Observations (LEO) were conducted at the locations shown in Figure 15. These observations include visual estimates of breaking wave height, period and direction. These observations, transferred into longshore energy flux, show that the long-term ratio of net to gross transport is 72% and that the reversals occur primarily during the months of June-August.

Sand Characteristics - Many sand samples have been taken in conjunction with the CERC study at CIH. For purposes here it is adequate to note that the median sediment size is approximately 0.25 mm and that the sediment collected in the depositional area tends to be finer near the deeper areas and toward the distal end of the deposition basin.

Beach Profiles - Profiles along the beaches updrift from the CIH breakwater typically have a berm on the order of 5 m above MLLW, a rather steep beach face and a milder slope offshore. Figure 16(a-e) presents the average and extremes of five profiles measured at Stations 123 + 00, 125 + 00, 127 + 00, 129 + 00 and 131 + 00. During the winter months, there typically is a bar present off these beaches.

Longshore Currents - The LEO observational program includes longshore currents determined through measurement of the advection of dye patches resulting from dye pellets thrown into the surf zone. As might be expected from the previously presented information on wave characteristics, the longshore currents are generally directed toward the east; however, there are periods of reversal. Table 4 presents the averages of longshore currents obtained during April 17, 1974 to April 14, 1975.

Tides - The tides at CIH are also of the mixed type with a mean range of 1.10 meters and a diurnal range of 1.62 m.

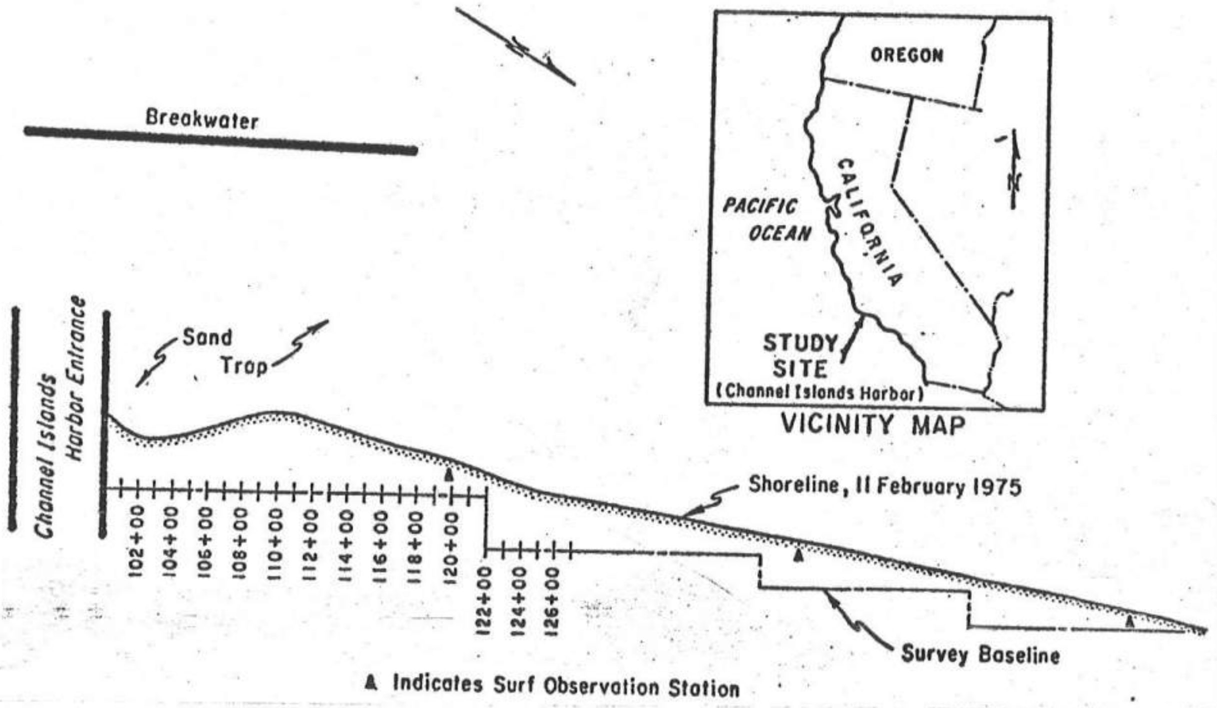


Figure 15. Locations of LEO Stations at Channel Islands Harbor, California.

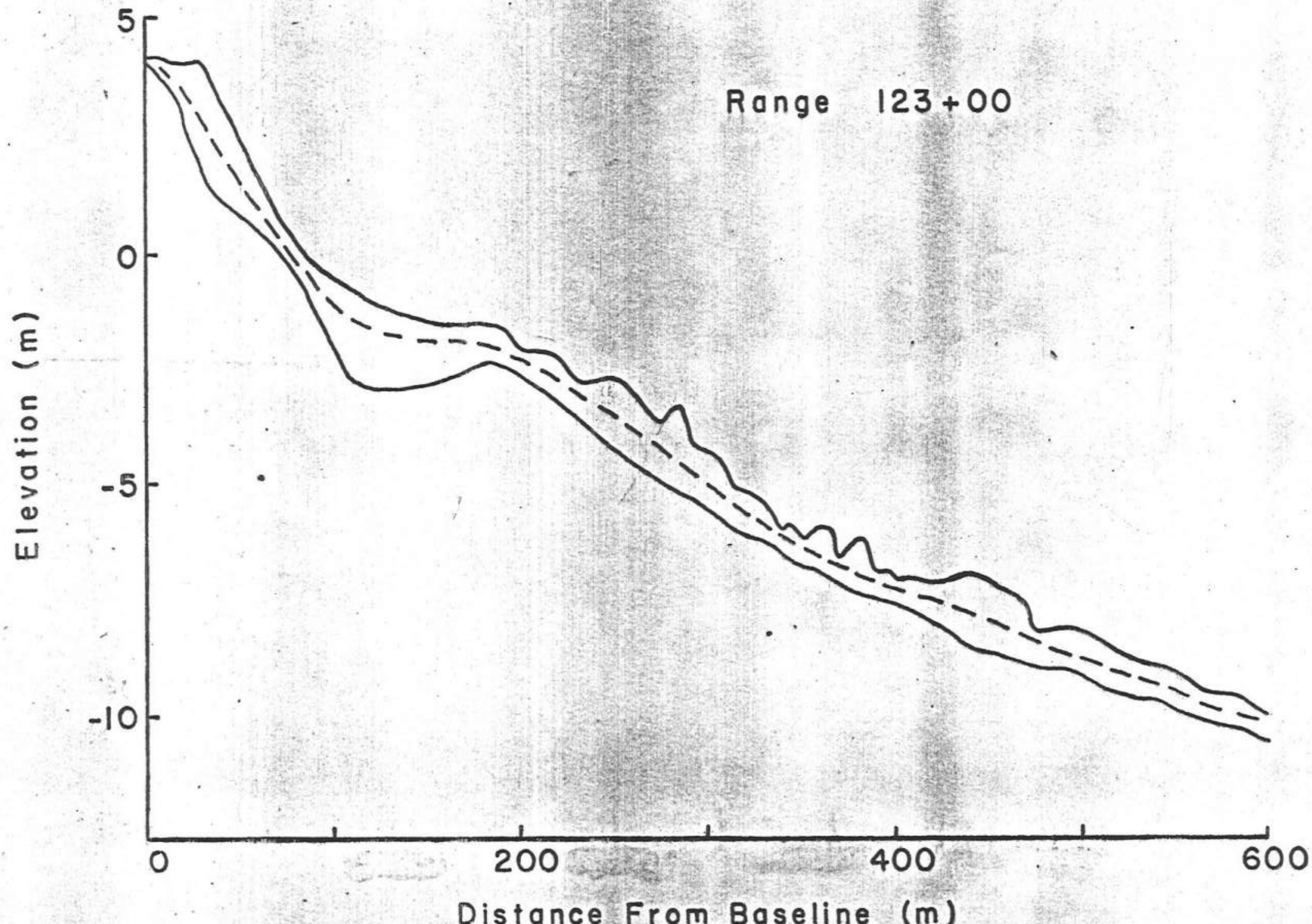


Figure 16a. Average and Extreme Profiles Measured Updrift of Trap at Channel Islands Harbor, California.

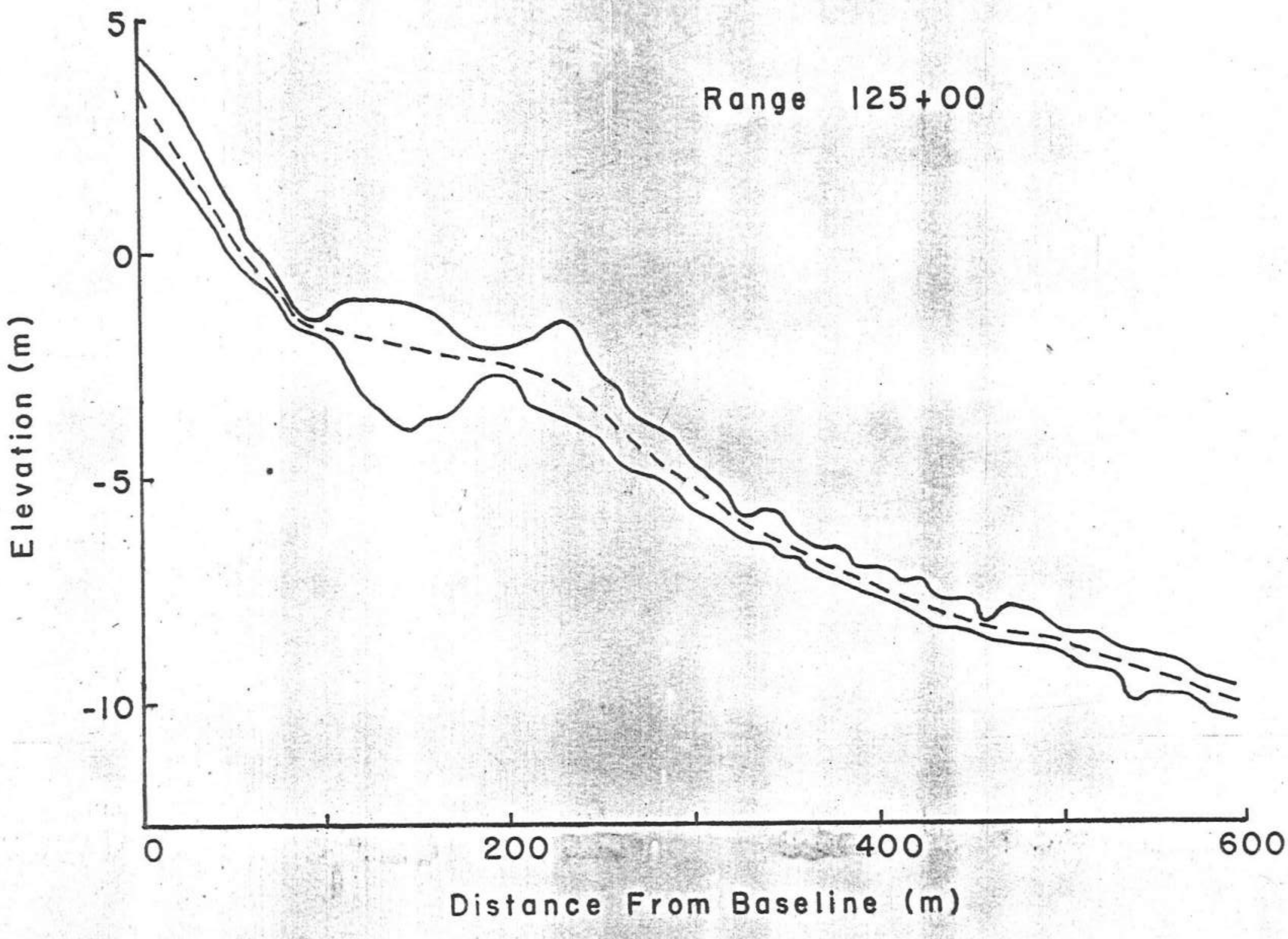


Figure 16b. Average and Extreme Profiles Measured Updrift of Trap at Channel Islands Harbor, California.

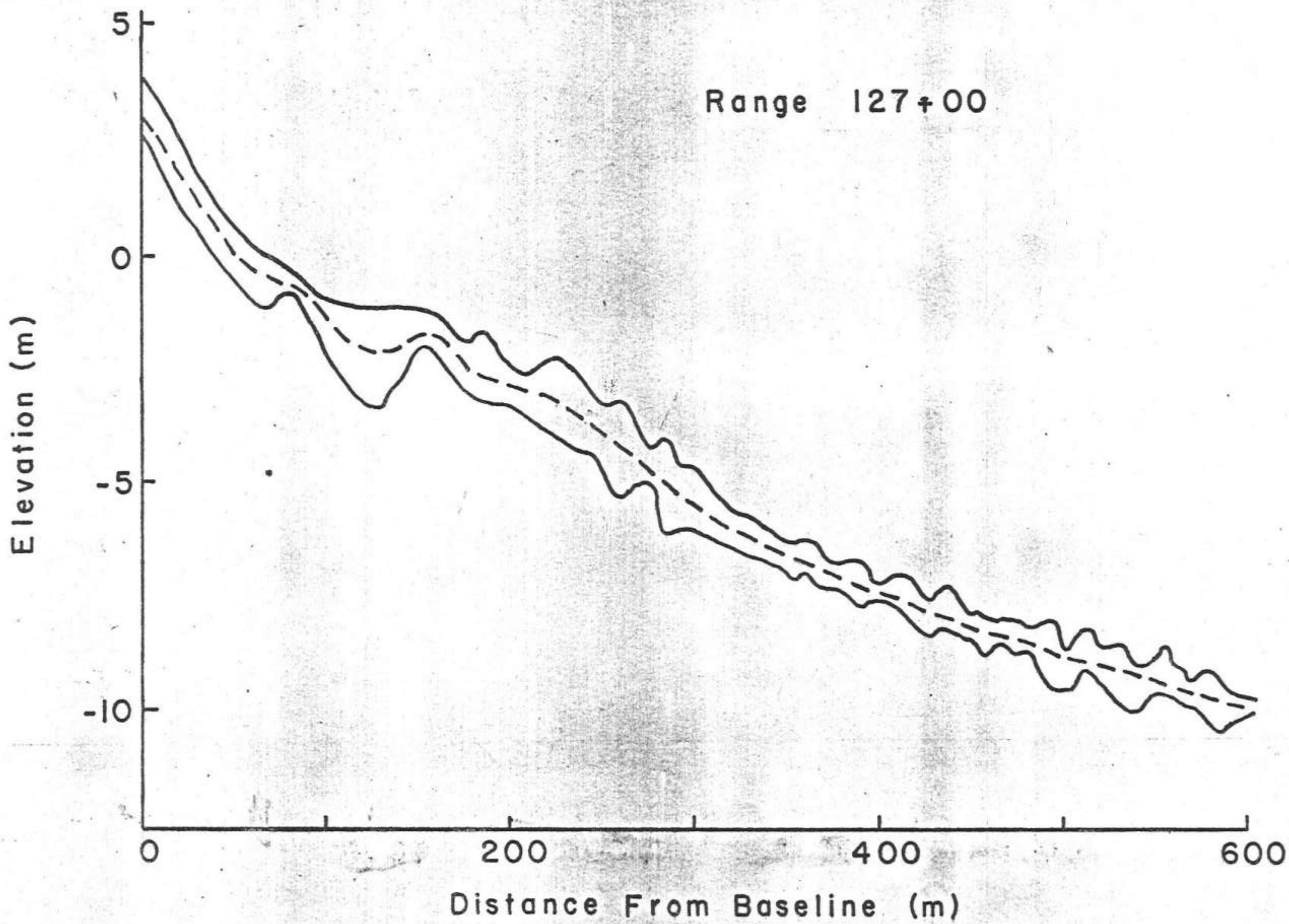


Figure 16c. Average and Extreme Profiles Measured Updrift of Trap at Channel Islands Harbor, California.

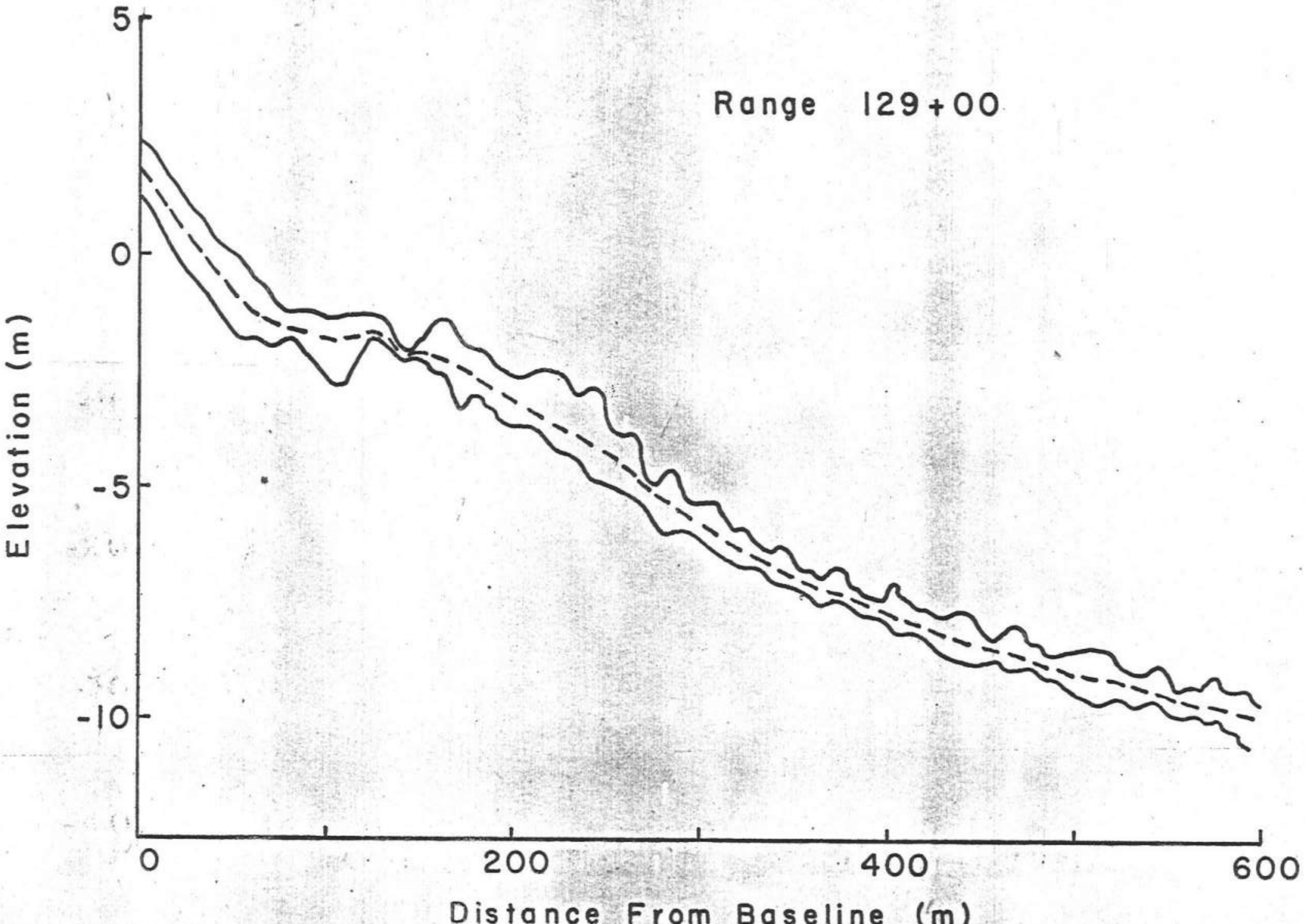


Figure 16d. Average and Extreme Profiles Measured Updrift of Trap at Channel Islands Harbor, California.

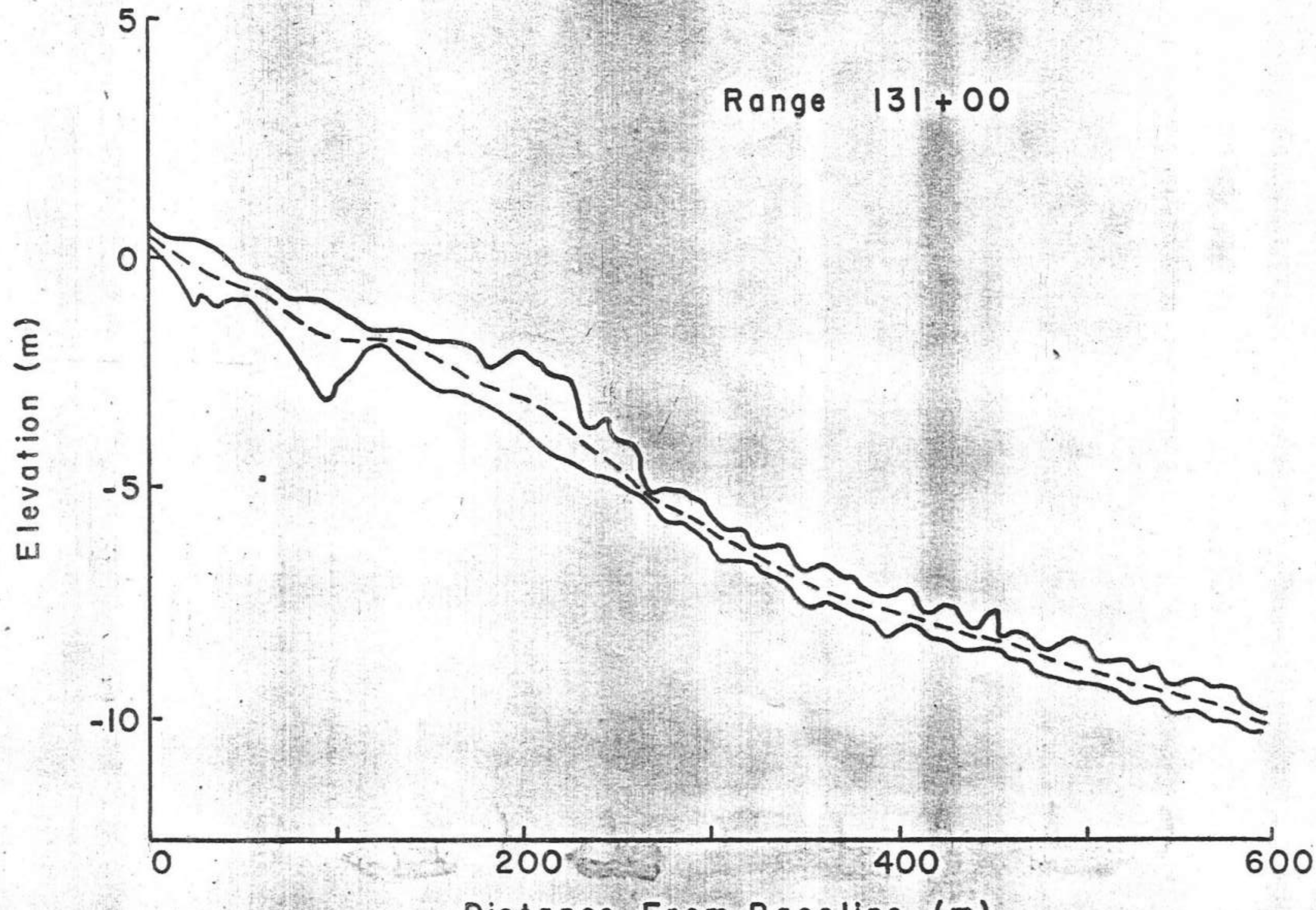


Figure 16e. Average and Extreme Profiles Measured Updrift of Trap at Channel Islands Harbor, California.

TABLE 4

AVERAGE LONGSHORE CURRENT AT CHANNEL ISLANDS HARBOR, CALIFORNIA
THE FLOW IS IN A SOUTHEASTERLY DIRECTION
(After Bruno and Gable, 1976)

	SURVEY PERIODS	LONGSHORE CURRENT (cm/sec)
1974	April 17 - May 7	45
	May 7 - July 30	36
	July 30 - August 20	27
	August 20 - September 24	22
	September 24 - December 6	36
	December 6 - January 7	34
1975	January 7 - February 11	44
	February 11 - March 4	36
	March 4 - April 14	66

WAVE DATA ANALYSIS

Introduction

Two sources of data were available for analysis; the National Oceanic and Atmospheric Administration's (NOAA) Tape Data Family-11 (TDF-11), and statistics based on wave hindcasts published by the Department of Navigation and Ocean Development (DNOD) of the State of California.

Due to the form of the data of each source, a different analysis method for each was adopted. Details of the form of the data and the method of analysis are presented in the following sections.

Analysis of the NOAA/TDF-11 Data

The Data Source - The sources for TDF-11 are basically surface marine observations compiled from ship logs, ship weather reporting forms, published ship observations, automatic observing buoys, teletype reports and data from foreign meteorological services.

In general the reliability of the data are non-uniform in that the instrumentation varies from rather primitive to the most sophisticated and the observers vary from deckhands to trained meteorologists.

The records contain information about the meteorology and atmosphere (wind speed and direction, temperature, pressure, visibility, cloud cover, etc.) and the sea (sea and swell wave heights, directions and periods, water temperature, presence and description of ice, etc.) giving location and time for each observation. A preliminary scan through the data revealed that although observations dating back to the second half of the 19th century exist, observations for sea and swell wave parameters were not recorded prior to 1950. The most recent records were dated 1974, and in general an increase in the quantity of records is observed from 1965 onwards.

All data are reported in metric units; therefore, the analysis will be carried out using the same units.

Preliminary Data Sorting - In order to determine the quantity of data available for the sites under consideration a preliminary data sorting was undertaken.

For Santa Barbara Harbor and for Channel Islands Harbor the areas scanned for data can be seen in Figure 17. A relatively narrow strip extending in the E-W direction was chosen for Santa Barbara so as to minimize the local effects of the Channel Islands.

The compatibility of wave direction with an approximate bottom contour orientation was also checked as a part of the preliminary sorting. The results of this procedure are given in Table 5. An average orientation of the shoreline was assumed and any wave whose orthogonal made an angle greater than or equal to $\pm 90^{\circ}$ with the normal of the shoreline was excluded from the analysis.

TABLE 5

PRELIMINARY DATA SORTING RESULTS

Site	Total Data	Compatible Data	% Compatible
Santa Barbara Harbor	1875	162	9
Channel Islands Harbor	19196	3333	17

Areas Scanned For Data

	N	S	E	W
—— Santa Barbara Harbor	34°30'	34°12'	119°18'	120°18'
---- Channel Islands Harbor	34°30'	34°12'	119°18'	120°
---- Channel Islands Harbor	34°18'	33°18'	118°30'	119°18'

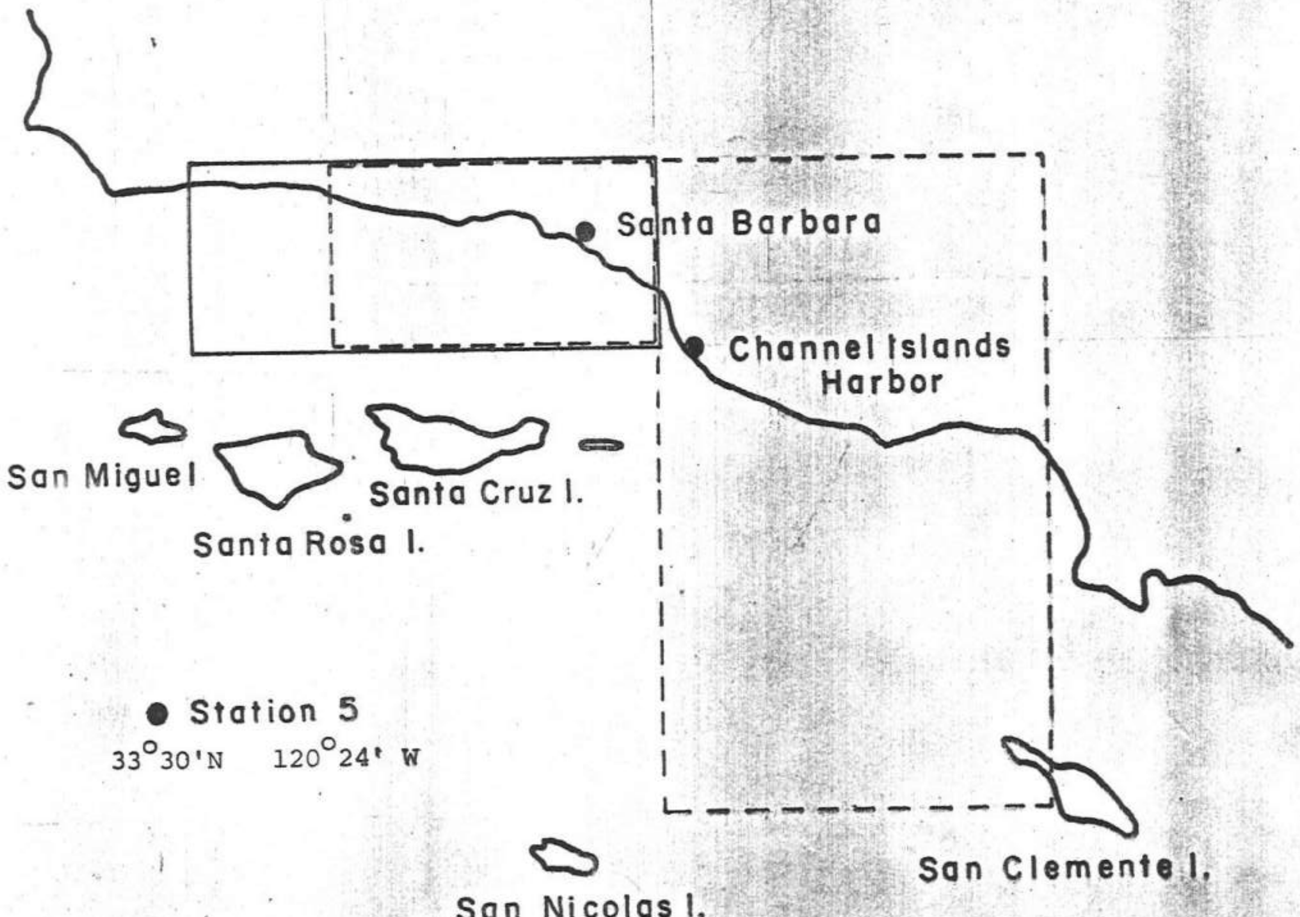


Figure 17. Sketch of Areas Scanned for NOAA/TDF-11 Data.

These results, when recalled that the data spans 25 years, indicate that any further analysis will be based on roughly one-half data point/month for Santa Barbara and eleven data points/month for Channel Islands Harbor. It should be noted here that in the case where any one record contained both sea and swell data it was assumed that no coupling existed between the two and that particular record was considered as two independent observations.

Method of Analysis

General - The analysis undertaken can be considered to constitute two parts. The first deals with the physical aspects of accounting for the transformation of a wave from deep water to the breaker line. This procedure accounts for shoaling, refraction, bottom friction and percolation effects. At the breaker line, the net, gross, positive and negative values of the longshore sediment transport rate Q are computed. The second part is an investigation into the variability of the breaker heights, and the net value of Q.

Computation Method - The transformation of a wave was assumed to be governed by:

$$H = H_o K_s K_r K_{fp} \tag{3}$$

where H and H_o are the local and deep water wave heights, K_s and K_r are the shoaling and refraction coefficients, respectively and K_{fp} is the combined friction-percolation coefficient. The phenomena associated with these coefficients were assumed to be uncoupled.

K_s may be evaluated through linear wave theory and is given by,

$$K_s = \left\{\frac{1}{\left(1 + \frac{2kh}{\sinh 2kh}\right)\tanh 2kh}\right\}^{1/2} \tag{4}$$

where k is the wave number, and h is the local depth.

Since K_r is dependent upon the angle between the wave direction and the depth contours, an approximate contour orientation was assumed based upon the shoreline orientation at the site under consideration. Locally the contours were assumed to be parallel to one another; K_r is then given by,

$$K_r = \sqrt{\frac{\cos \alpha_o}{\cos \alpha}} \tag{5}$$

where α_o and α are, respectively, the angles between the wave crest and the contours in deep water and at the depth under consideration, α may be obtained through Snell's law as,

$$\alpha = \sin^{-1}\left(\frac{C \sin \alpha_o}{C_o}\right) \tag{6}$$

where C_o and C are the deep water and local values of the wave celerity.

To obtain the combined friction-percolation coefficient, first the separate effects of each in wave damping are evaluated and then the two are combined.

The time averaged energy loss due to bottom friction may be evaluated from,

$$\overline{\tau u_b} = \frac{4}{T}\int_0^{T/4} \frac{\rho f}{8}\, u_b^3 \cos^3 \sigma t dt \tag{7}$$

where τ is the bottom shear stress, T is the wave period, ρ is the mass density of water, f is the bottom friction coefficient (taken to be 0.08), σ is the wave angular frequency, and u_b is the magnitude of the horizontal particle velocity evaluated at the bottom and is given by,

$$u_b = \frac{H\sigma}{2} \cdot \frac{1}{\sinh kh} \tag{8}$$

for a locally horizontal bottom. Under the assumption of a steady conservative system, the energy losses at the bottom will be compensated for by a gradient of the energy flux. This consideration leads to the conservation equation,

$$\frac{\partial}{\partial x}(EC_g) + \overline{\tau u_b} = 0 \tag{9}$$

where x is the spatial variable in the direction of wave advance. E is the wave energy per unit surface area and is given by,

$$E = \frac{1}{8}\rho g H^2 \tag{10}$$

and g is the gravitational acceleration. C_g is the group velocity defined as

$$C_g = C\,\frac{1}{2}\left(1 + \frac{2kh}{\sinh 2kh}\right) \tag{11}$$

Evaluation of Eq. (7) yields an expression for the wave height damping due to bottom friction

$$\left(\frac{\partial H}{\partial x}\right)_f = -BH^2 \tag{12}$$

where

$$B = \frac{2}{3\pi}\frac{f}{gC_g}\left(\frac{\sigma}{2\sinh kh}\right)^3 \quad (13)$$

In order to evaluate the percolation damping effects, the formulation introduced by Hunt (1959) was employed. It can be shown that the wave height undergoes an exponential decay due to percolation losses. This variation of wave height per unit distance in the direction of wave advance may be given by,

$$\left(\frac{\partial H}{\partial x}\right)_p = -k_i e^{-k_i x} H \quad (14)$$

where k_i is the imaginary part of the wave number given in complex form as $k = k_r + ik_i$. k_i may be obtained from the dispersion relation for the case of wave propagation over an infinitely thick sand layer;

$$\frac{\sigma^2}{gk\tanh kh} = 1 + \frac{i\sigma K}{g}\left(\text{cotanh}\, kh - \frac{\sigma^2}{gk}\right) \quad (15)$$

in which K is the permeability coefficient, and has a value of 1×10^{-7} m/sec for very fine sands. Eq. (15) leads to two transcendental equations in k_r and k_i, which may be solved simultaneously by numerical means. It can be shown that $O(k_i) << 1$.

Assuming that Eqs. (12) and (14) may be combined linearly to yield the total damping of wave height per unit distance of wave travel,

$$\frac{\partial H}{\partial x} = -BH^2 - k_i e^{-k_i x} H \quad (16)$$

Making use of the smallness of k_i, a perturbation solution may be devised for Eq. (16). With the appropriate boundary conditions the solution of Eq. (16) up to $0(k_i^2)$ is,

$$\frac{H(x)}{H_o} = \frac{1}{1 + BH_o x} - k_i \frac{\left(x + BH_o \frac{x^2}{2}\right)}{\left(1 + BH_o x\right)^2} \tag{17}$$

where x is the distance over which the wave height decays from H_o to H. The combined friction-percolation coefficient is defined as,

$$K_{fp} = \frac{H(x)}{H_o} \tag{18}$$

In the computations, it was assumed that the orientation of the wave orthogonal changed only at the midpoint between contours so that the travel distance could be calculated easily for use in Eq. (17).

The breaking criterion $H_b = 0.78\ h_b$ was adopted to locate the position of the breaker line.

For the computation of the sediment transport rate, Q, an immersed load formula was employed;

$$Q = \frac{I}{g(\rho_s - \rho)(1 - p)} \tag{19}$$

in which ρ_s(= 2650 kg/m^3) and ρ are the mass densities of the sediment and of the water, respectively, and p(= 0.35) is the porosity. I in Eq. (19) is given as follows,

$$I = K \frac{\rho g H_b^2}{16} \sqrt{g h_b} \sin 2\alpha_b \tag{20}$$

where K(= 0.77) (Komar and Inman, 1970) is a constant, and the subscript b is used to indicate the values of the variables at the breaker line.

Analysis of the DNOD Data

The Data Source - The DNOD publication (1977) reports data processed by the Fleet Numerical Weather Central (FNWC) Singular Sea/Swell Model. The basic approach utilized by this model is to convert actual barometric pressure observations (based on a 63 x 63 grid on a northern hemisphere polar-stereographic projection) to a pressure field, from this derive the wind field, and then to use the wind field to generate the wave field. The resulting wave field is reported in tabular form giving height and period class and directional information. Separate tabulations of the sea and swell wave fields as well as tabulations for the combined field based on once daily observations over a total period of 24 years for each month are presented in the publication.

The portion of the data analyzed is reported at Station 5 (located at 33.5° N and 120.4° W) which lies to the south of the Channel Islands, see Figure 17.

A detailed account of the limitations of the data may be found in the publication mentioned above.

Method of Analysis - Since Station 5 lies to the south of the Channel Islands a method which takes into account the sheltering effect of the islands was required. An approximate method to transform the available data to wave conditions at breaking may be given as follows,

$$H_b = H_o K_{sb} K_{rb} K_{fp} K_E \qquad (21)$$

in which K_E is an exposure coefficient accounting for the sheltering effect of the islands, and K_{rb} and K_{sb} are the values of the refraction coefficient and the shoaling coefficient at breaking. Since the DNOD deep water wave results are available in 10^o sectors an exposure coefficient, K_E, was developed for the wave data in each sector depending on the decimal percentage of geometric exposure at each site from that sector. As an example, it is seen that the eastern and western tips of the Anacapa Island group lie along azimuths directed 147^o and 153^o, respectively from Santa Barbara Harbor, Figure 14. The geometric exposure weighting factor and effective deep water direction for the wave data contained in the sector 145-155 would then be 0.4 and 150^o, respectively. In the computations it was assumed that $K_E = 1.0$ for sea waves since the local winds would probably regenerate those screened out by the islands in the approximate 20 mile fetch between the islands and the shore. For swell waves the values of the exposure coefficient and the corresponding effective directions for both sites are given in Table 6.

The dissipation factor, K_{fp}, was assumed to be equal to unity. Although this is not strictly correct, the effect should be small (due to the relatively narrow continental shelf) and approximately the same for all wave sectors.

TABLE 6

EFFECTIVE DEEP WATER WAVE DIRECTIONS AND EXPOSURE COEFFICIENTS
(BASED ON GEOMETRIC EXPOSURE)

	Santa Barbara Harbor		Channel Islands Harbor	
Direction Sector Class	α_E	K_E	α_E	K_E
115^o-125^o	124^o	0.2	SHADOW ZONE	
125^o-135^o	130^o	1.0	SHADOW ZONE	
135^o-145^o	140^o	1.0	SHADOW ZONE	
145^o-155^o	150^o	0.4	SHADOW ZONE	
155^o-165^o	157^o	0.4	160^o	1.0
165^o-175^o	NORMAL INCIDENCE		170^o	1.0
175^o-185^o	SHADOW ZONE		180^o	1.0
185^o-195^o	SHADOW ZONE		190^o	1.0
195^o-205^o	SHADOW ZONE		200^o	1.0
205^o-215^o	SHADOW ZONE		210^o	1.0
215^o-225^o	SHADOW ZONE		215.5^o	0.1
225^o-235^o	231^o	0.4	234.5^o	0.1
235^o-245^o	242.5^o	0.5	286.5^o	0.3
245^o-255^o	250^o	1.0	NORMAL INCIDENCE	
255^o-265^o	SHADOW ZONE		264.5^o	0.1
265^o-275^o	SHADOW ZONE		270^o	1.0
275^o-285^o	SHADOW ZONE		280^o	1.0

The refraction coefficient at breaking, K_{rb}, for straight and parallel bottom contours is

$$K_{rb} = \sqrt{\frac{\cos \alpha_o}{\cos \alpha_b}} \tag{22}$$

where the breaker angle, α_b, may be computed with the aid of Eq. (6).

In the following, the further simplifying assumption will be made that sea and swell can each be represented by an effective wave period.

The shoaling coefficient at breaking may be written as,

$$K_{sb} = \sqrt{\frac{C_{g_o}}{C_{g_n}}} \tag{23}$$

It can be assumed with reasonably good accuracy that the conditions at breaking may be considered to be shallow water (that is, the breaking depth is less than 0.05 to 0.1 of the wavelength at breaking). Thus, the group velocity at breaking is,

$$C_{g_b} = \sqrt{gh_b} \tag{24}$$

As before, it is assumed that the breaking depth, h_b, is related to the breaker height, H_b, by some constant, $\kappa(= 0.78)$,

$$H_b = \kappa h_b \tag{25}$$

Utilizing Eqs. (22), (23), (24) and (25), it can be shown that Eq. (21) can be expressed as,

$$H_b = H_o^{4/5} (g\kappa)^{1/5} \left(\frac{T}{4\pi}\right)^{2/5} \left(\frac{\cos \alpha_o}{\cos \alpha_b}\right)^{2/5} \tag{26}$$

where T is the effective wave period.

As before, to employ Eq. (18) for the computation of the sediment transport rate we assume

$$I = K\rho g \frac{H_b^2}{8} C_{g_b} \sin \alpha_b \cos \alpha_b \tag{27}$$

Using Eqs. (24) and (26) in Eq. (27) and noting that from Snell's Law

$$\sin \alpha_b = \sqrt{\frac{gH_b}{\kappa}} \frac{2\pi}{gT} \sin \alpha_o$$

we obtain

$$I = \frac{K\rho g^{1.6} H_o^{2.4} T^{0.2}}{4^{2.2} \kappa^{0.4} \pi^{0.2}} \frac{(\cos \alpha_o)^{1.2} \sin \alpha_o}{(\cos \alpha_b)^{0.2}} \tag{28}$$

Eq. (28) requires knowledge of the breaking angle, α_b. A further simplifying assumption, regarding α_b, will now be introduced. The breaking angle is usually less than 10^o and probably in 98% of the cases less than 20^o. The values of $(\cos \alpha_b)^{-0.2}$ in Eq. (28) for $\alpha_b = 10^o$ and 20^o are 1.003 and 1.013 respectively and it is therefore a very good approximation to set this factor equal to unity. The resulting simplified equation then depends only on deep water quantities. Moreover, in the same manner, it is readily shown that the relationship is not very sensitive to the wave period, T, and for present purposes only one period each will be used for sea and swell.

Finally noting that the significant wave height, H_s, given is related to the root mean square wave height, H_{rms}, by

$$H_s = 1.416\ H_{rms}$$

we may rewrite Eq. (28) as

$$I_j = \frac{K\rho g^{1.6} T^{0.2}}{4^{2.2} \kappa^{0.4} \pi^{0.2} (1.416)^{2.4}} \sin \alpha_{o_j} \left(\cos \alpha_{o_j}\right)^{1.2} \Sigma n_j K_{E_j}^{2.4} H_{s_j}^{2.4} \qquad (29)$$

in which the subscript j denotes a direction sector class (characterized by α_{o_j}) and n_j is the number of waves encountered in the significant wave height class H_s. In the computations for the longshore sediment transport rate, Q_j, Eq. (29) was utilized in Eq. (19) and an average was computed using the number of waves (sea and swell) in each particular month, and the results were added to determine the annual transport rate. The wave period T was taken as 5 sec. for sea and 13 sec. for swell waves.

RESULTS

The results of the analyses are summarized for Santa Barbara Harbor in Table 7 and Figure 18(a-b), and for Channel Islands Harbor in Table 8 and Figure 18(c-d). The convention used for transport signs is that a positive value corresponds to transport directed to the right as an observer looks offshore.

Comparison of the annual transport rates obtained from the two data sources indicates that values obtained using the DNOD data are consistently higher than those resulting from the NOAA data. Although it is not possible to expect agreement in results obtained from two different data sources using two different methods of analysis, the

TABLE 7

TRANSPORT RATES FOR SANTA BARBARA HARBOR

	Data Source: NOAA/TDF-11				Data Source: DNOD			
Month	Positive	Negative	Gross	Net	Positive	Negative	Gross	Net
January	88,393	19,055	107,448	+ 69,338	26,479	78,923	105,402	- 52,444
February	3,132	6,073	9,205	- 2,941	37,510	47,298	84,808	- 9,788
March	362	27,991	28,353	- 27,629	23,728	66,125	89,853	- 42,397
April	19,903	33,915	53,818	- 20,012	0	92,841	92,841	- 92,841
May	4,264	11,454	15,718	- 7,190	0	8,805	8,805	- 8,805
June	81	121,231	121,312	-121,150	0	77,940	77,940	- 77,940
July	0	16,334	16,334	- 16,334	0	0	0	0
August	0	149,028	149,028	-149,028	0	0	0	0
September	15	8,888	8,903	- 8,873	790	16,158	16,948	- 15,368
October	81	5,496	5,577	- 5,415	0	20,659	20,659	- 20,659
November	5,732	30,425	36,157	- 24,693	54,256	69,401	123,657	- 15,145
December	20,023	1,379	21,402	+ 18,644	26,738	69,048	95,786	- 42,310
ANNUAL (m^3/yr.)	141,986	431,269	573,255	-289,283	169,501	547,198	696,040	-377,697
AVERAGE ANNUAL (m^3/mo.)	11,832	35,939	47,771	- 24,107	14,125	45,600	58,003	- 31,475

*Unless Specified Otherwise, Values are in m^3.

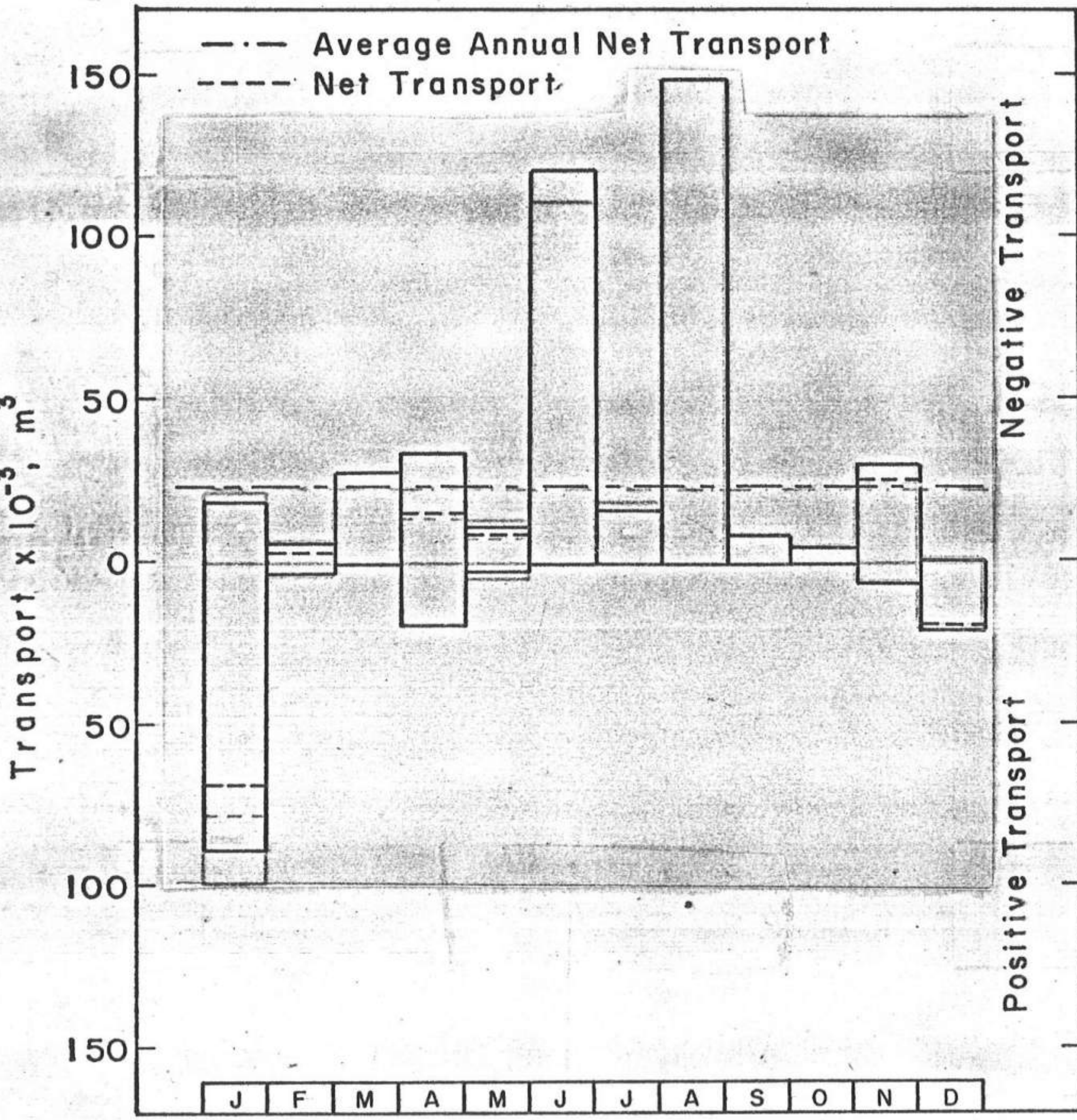


Figure 18a. Average Monthly Longshore Transport Characteristics for Santa Barbara Harbor as Determined from Analysis of NOAA's TDF-11 Data (Based on Surface Marine Observations, 1950 to 1974).

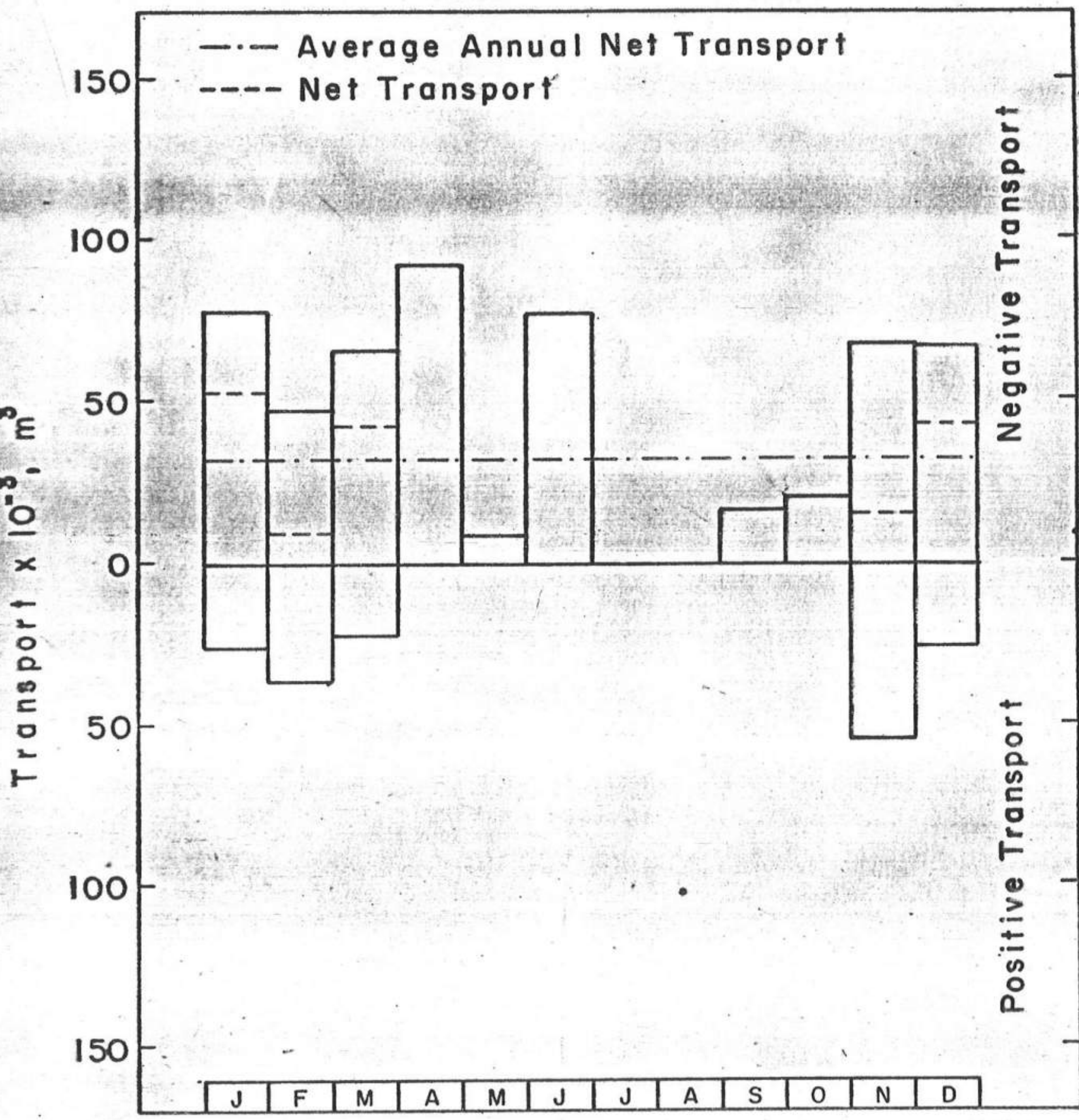


Figure 18b. Average Monthly Longshore Transport Characteristics for Santa Barbara Harbor Based on Analysis of DNOD Data (Based on Analysis of Upper Level Pressure Field to Predict Surface Wind and Wave Fields, 1951 to 1974).

TABLE 8

TRANSPORT RATES FOR CHANNEL ISLANDS HARBOR

	Data Source: NOAA/TDF-11				Data Source: DNOD			
Month	Positive	Negative	Gross	Net	Positive	Negative	Gross	Net
January	5,318	48,726	54,044	- 43,408	110,155	63,207	173,362	+ 46,948
February	6,788	67,559	74,347	- 60,771	89,723	56,669	146,392	+ 33,054
March	5,071	92,214	97,285	- 87,143	44,948	97,445	142,393	- 52,497
April	1,013	78,820	79,833	- 77,807	28,990	140,998	169,988	-112,008
May	1,875	74,806	76,681	- 72,931	5,208	147,781	152,989	-142,573
June	3,967	45,541	49,508	- 41,574	681	175,883	176,564	-175,202
July	688	64,636	65,324	- 63,948	0	94,188	94,188	- 94,188
August	1,887	35,919	37,805	- 34,032	0	114,146	114,146	-114,146
September	1,275	30,983	32,258	- 29,708	966	203,548	204,514	-202,582
October	410	51,944	52,354	- 51,534	36,297	58,626	94,923	- 22,329
November	2,249	103,134	105,383	-100,885	70,865	49,149	120,014	+ 21,716
December	4,914	78,554	83,468	- 73,640	101,427	44,426	145,853	+ 57,001
ANNUAL (m^3/yr.)	35,455	772,836	808,291	737,381	489,260	1,246,066	1,735,326	756,806
AVERAGE ANNUAL (m^3/mo.)	2,955	64,403	67,358	61,448	40,772	103,839	144,611	63,067

*Unless Specified Otherwise, Values are in m^3.

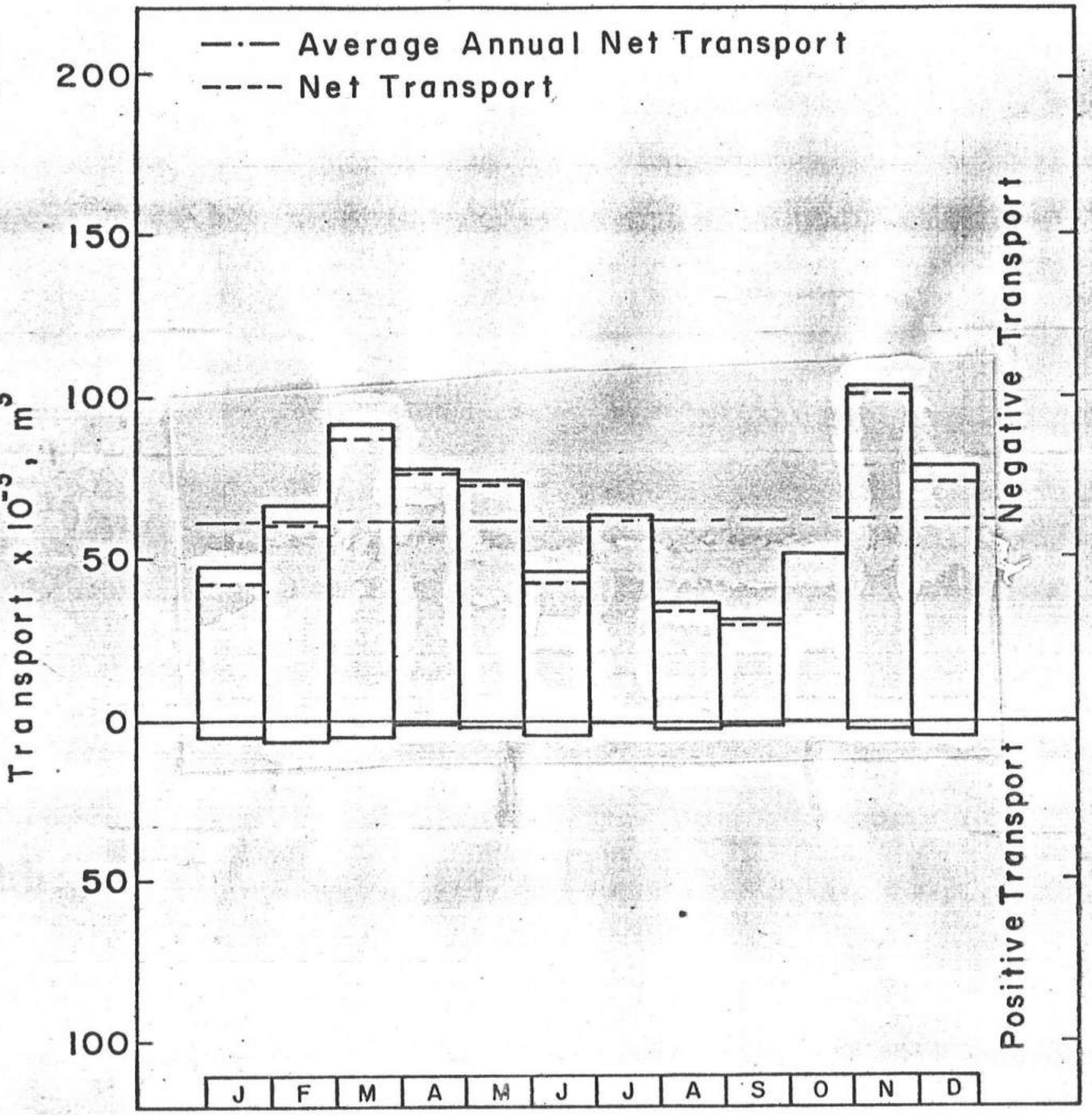


Figure 18c. Average Monthly Longshore Transport Characteristics for Channel Islands Harbor as Determined from Analysis of NOAA's TDF-11 Data (Based on Surface Marine Observations, 1950 to 1974).

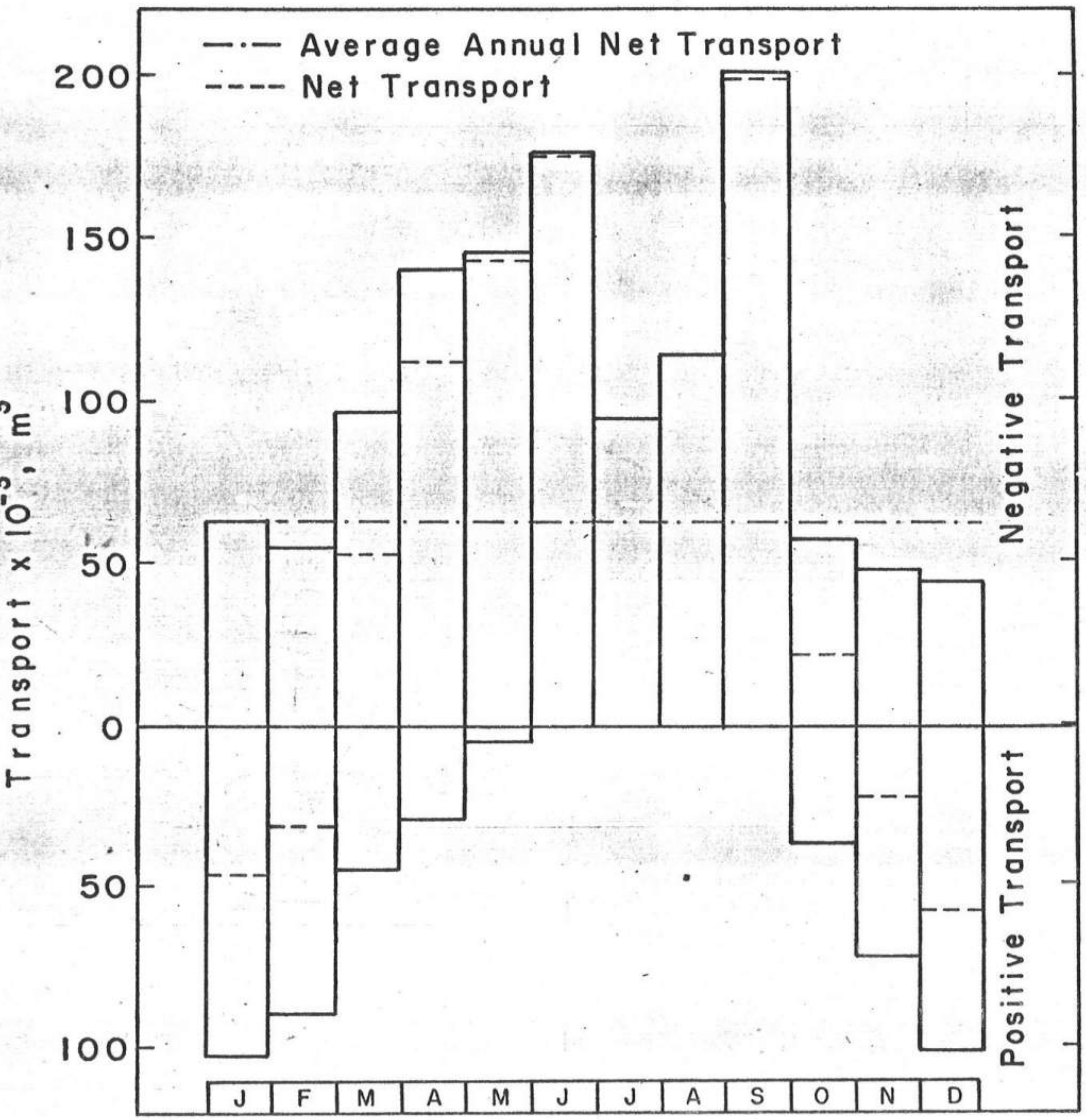


Figure 18d. Monthly Longshore Transport Characteristics for Channel Islands Harbor as Determined from Analysis of DNOD Data (Based on Analysis of Upper Level Pressure Field to Predict Surface Wind and Wave Fields, 1951 to 1974).

annual net transport rates obtained are within 27% of each other for Santa Barbara Harbor and 3% for Channel Islands Harbor.

In addition to the analyses explained in the preceding sections an attempt was made to determine the variability of the wave and transport conditions at the two sites. For this purpose, a normalized standard deviation as given below was employed for the NOAA data.

$$\varepsilon = \sqrt{\frac{\sum_i (\beta_i - \overline{\beta})^2}{\sum_i \beta_i^2}}$$

where β is the quantity whose variability is under question, the subscript i refers to the i^{th} observation, and an overbar is used to indicate the statistical mean of the quantity. From the equation above it can be seen that ε approaching unity corresponds to the case of extreme variability relative to the mean, and that if β is absolutely steady ($\beta_i = \overline{\beta}$ for all i), $\varepsilon = 0$.

Santa Barbara Harbor

In addition to the differences in the monthly transport rates obtained from the two sources, the NOAA data analysis indicates reversals in the transport directions for the months of December and January. Although reversals are possible due to waves originating from storms in the southeast, the wave pattern and therefore the direction of transport (easterly) at Santa Barbara Harbor is unvarying in general (Wiegel, 1964, p. 475). Considering the overall scarcity and unreliability of the NOAA data, such discrepancies in the fine structure of the results are possible and expected. In spite of these discrepancies, the annual net to gross transport ratios are 50% (NOAA) and 54% (DNOD). Table 7 (or Figure 18b) shows that the DNOD data indicates no transport for the months of July or August. An inspection of the data shows that this is due to the absence of waves from sectors that may cause transport rather than the screening effects of the islands. The NOAA data, on the other hand, yields unidirectional negative transport for these months which is more reasonable. The results indicate a fairly unidirectional transport period for the months of June-October with a positive to negative transport ratio less than 5% in comparison to a maximum annual ratio of 36%. The annual net transport rates are 289,283 m^3/yr. (NOAA) and 377,697 m^3/yr. (DNOD) and are within 27% of each other.

The variability of the longshore transport is, in general, high ($\varepsilon > 0.80$), but during June-October the values are relatively lower. This, however, does not warrant assuming unvarying transport rates for any period of time. The variability of the breaker heights on the other hand is found to be $\varepsilon < 0.6$. Especially during the period mentioned above, $\varepsilon < 0.4$ which indicates reasonably constant breaker heights.

Channel Islands Harbor

The calculated annual transport rates for this site are within 3% of each other (737,381 m^3/yr. (NOAA) and 756,806 m^3/yr. (DNOD) in a south-easterly direction). The NOAA data yields results which indicate unidirectional net transport for each month. The DNOD data analysis, on the other hand, shows reversals in the net transport direction for the months of November-February. The net to gross transport ratios for this site are 91% (NOAA) and 44% (DNOD). During the months of May-September relatively unidirectional transport conditions exist for this site with a positive to negative transport ratio less than 4% in comparison to an average annual ratio of 31% (DNOD).

The variability of transport rates is high for this site ($\varepsilon > 0.9$) for all months. The breaker heights, on the other hand, exhibit less variability ($\varepsilon < 0.5$).

SUMMARY AND RECOMMENDATIONS

Summary

Based on the desirability of selecting a site at which either natural or artificial features would facilitate entrapment and measurement of net longshore sediment transport, two west coast sites, Santa Barbara Harbor and Channel Islands Harbor, have been considered. The two sites are in reasonable proximity to each other and thus it is not surprising that many of the physical features at the two sites are similar. The tide and sand characteristics do not differ significantly and based on the most comprehensive (DNOD) wave data source, the ratios of calculated net to gross longshore sediment transport do not differ greatly at the two sites: Santa Barbara (0.54) versus Channel Islands Harbor (0.44). Due to less sheltering of deep water waves at Channel Islands Harbor, the magnitudes of longshore sediment transport are more than a factor of two greater than at Santa Barbara Harbor. The logistical considerations are favorable at both sites.

The principal relevant difference at the two sites is the type of trap. The trap at Channel Islands Harbor is an offshore breakwater and based on reasonable considerations of wave diffraction and longshore sediment transport mechanics, it appears that this system may result in an unquantified overtrapping of the longshore sediment transport and a perturbation which extends a considerable distance updrift. The breakwater (shore-connected) at Santa Barbara Harbor provides a sand flow control feature which does not have an undue influence on the updrift shoreline. The sediment transport past a point should be relatable to the wave conditions (including relative wave-beach orientation) and the volumes transported during intervals between surveys. The volumes measured should encompass the entire segment to the east of the location of wave measurement including a portion of Leadbetter Beach, along the Santa Barbara Harbor breakwater and in the spit inside the distal end of the breakwater.

Recommendation

Based on the factors presented in the Summary above, it is recommended that the first Field Trap Experiment be conducted at Santa Barbara Harbor.

REFERENCES

Bruno, R. O. and C. G. Gable, Longshore Transport at a Total Littoral Barrier, Proc., Fifteenth Coastal Eng. Conf., 1203-1222, 1976.

Hunt, J. N., On the Damping of Gravity Waves Propagated Over a Permeable Bottom, J. Geophys. Res., 64, 437-442, 1959.

Johnson, J. W., Sand Transport by Littoral Currents, Proc., Fifth Hydraulics Conference, Bulletin 34, State University of Iowa Studies in Engineering, 1953, p. 89-109.

Komar, P. D. and D. L. Inman, Longshore Sand Transport on Beaches, J. Geophys. Res., 75 (30), 5914-5927, 1970.

Marine Advisers, A Statistical Survey of Ocean Wave Characteristics in Southern California Waters, Prepared for the Los Angeles District, U. S. Army Corps of Engineers, 1961.

Meteorology International, Inc., Deep-Water Wave Statistics for the California Coast, Station 5, Prepared for the Department of Navigation and Ocean Development, State of California, February, 1977.

National Marine Consultants, Wave Statistics for Seven Deep Water Stations Along the California Coast, Prepared for the Los Angeles District, U. S. Army Corps of Engineers, December, 1960.

O'Brien, M. P., Wave Refraction at Long Beach and Santa Barbara, California, Bulletin, Beach Erosion Board, January, 1950, p. 1-12.

U. S. Army Coastal Engineering Research Center, Shore Protection Manual, 1977.

U. S. Congress, Beach Erosion at Santa Barbara, California, House Document No. 522, 75th Cong. 3d Sess. Mar. 18, 1938.

U. S. Congress, Santa Barbara, California, Beach Erosion Control Study, House Document No. 761, 80th Cong. 2d Sess. Dec. 22, 1948.

Wiegel, R. L., Oceanographical Engineering, Chapter 18 - A Case History, Prentice-Hall, p. 472-486, 1964.