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OF THE
BEACH EROSION BOARD

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VOL. 4

NO. 1
Figure 1 Long Beach breakwater damaged by severe wave action in April, 1930.
WAVE REFRACTION AT LONG BEACH AND SANTA BARBARA, CALIFORNIA

The following paper was first published in limited issue as Technical Report HE-116-246, Fluid Mechanics Laboratory, University of California. It is re-produced here to bring the utilization of wave refraction analyses and wave forecasting principles in shoreline improvement investigations to the attention of coastal engineers and others interested in the action of waves and surf. The paper was prepared by Dean M. P. O'Brien, University of California in May 1947.

Wave refraction analyses are rapidly approaching the status of a standard component of shoreline investigations (1). Results obtained from these analyses are frequently of only a qualitative character but even so, they provide a rational basis for planning shore protection and improvement. This report presents two examples of wave refraction problems prepared in 1936 which illustrated the way in which this technique may be used to supplement and explain fragmentary observations.

HEAVY SWELLS AT LONG BEACH

During the period April 20 to 24, 1930, large breakers damaged the outer portion of the Long Beach harbor breakwater (see figure 1). Damage to breakwaters by heavy wave action is not noteworthy but the circumstances at Long Beach were unusual because it was only at and near the end of the breakwater that large breakers were observed. There was only a light wind and the sea offshore was calm. At the gambling ships anchored in deep water, no unusual wave action was observed. At the San Pedro breakwater, only a few miles away there were no breakers and along the shore downcoast from the Long Beach breakwater, life guards noticed no wave action of unusual intensity. The breakwater and dislodged stones ranging in weight from 4 to 20 tons. In the Long Beach channel, just off the end of the breakwater, the swells peaking up but apparently did not break. Several small craft are reported to have "surf-boarded" in through the channel.

These circumstances indicate that, during this period, waves must have approached this shoreline from such direction, and with such period, that they were focused on the end of the Long Beach breakwater but were elsewhere of negligible size. The wave period was not measured but from the statements of eyewitnesses, it is believed to have been between 20 and 30 seconds.

In 1930, the Los Angeles-Long Beach outer breakwater had not been constructed and the Long Beach harbor breakwater was exposed to ocean swells over limited sectors from south and west (figure 2). In order to locate the possible sources of waves during this interval, the Northern Hemisphere
### Table 1

**Summary of the Characteristics of Two Storms in the Eastern Pacific Ocean in April, 1920**

#### Storm No. 1

<table>
<thead>
<tr>
<th>Day</th>
<th>Time (PWT)</th>
<th>Velocity (knots)</th>
<th>Direction</th>
<th>Fetch (nautical miles)</th>
<th>Decay (nautical miles)</th>
<th>Date Time (PWT)</th>
<th>Deep-water wave height (ft)</th>
<th>Period (sec)</th>
<th>Direction</th>
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<td>700</td>
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<td>240900</td>
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<td>12.3</td>
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</tbody>
</table>
FIGURE 2 - ISLANDS IN SANTA BARBARA CHANNEL AND THEIR INFLUENCE ON THE EXPOSURE OF SAN PEDRO BAY TO WAVE ACTION.
Historical weather maps were examined and it was found that there existed two storm centers capable of producing long-period waves, one to the northwest and the other approximately west. The characteristics of the waves to be expected from these storms were "hindcast" by Lt. Robert Stump (Air) following standard forecasting principles and procedures (2) (see Beach Erosion Board Bulletin, Special Issue No. 1, July 1, 1948). The only exception to standard procedure being that the time of arrival was adjusted so that the average velocity in the decay area equaled half the wave velocity in deep water just offshore. The computed characteristics of these waves are presented in table 1. Waves from storm No. 1 must have undergone refraction and diffraction around Santa Rosa Island before approaching the Long Beach breakwater through the westerly sector. Waves from storm No. 2 would approach the breakwater directly through this sector.

In order to analyze the possible effect of these storms, a refraction diagram was drawn for waves of 20-second period approaching from due west (figure 3). It appears that refraction of these waves does not produce the observed effect, namely, concentration of wave energy at the Long Beach breakwater and inappreciable wave action at adjacent points. It seemed safe to conclude on this basis that neither of these storms generated the waves which damaged the breakwater and that their source must have been outside of the area covered by the weather maps.

Almost due south of Long Beach there is a submerged ridge which might force waves from this direction on the breakwater. Being deeply submerged, it would affect only waves of unusually long period and thus would, in this respect at least, satisfy the requirement that the phenomenon be of rare occurrence. Figure 4 is a refraction diagram for waves of 20-second period approaching along the axis of the ridge (line a-a). The convergence of the orthogonals a-a and e-e shows that the energy of these waves would be focused on the breakwater. The multiplying factor for the increase in wave height exclusive of the effect of the shoaling bottom, is approximately three. At all adjacent points along the shoreline and at the San Pedro breakwater, refraction would reduce rather than increase wave height.

The shoaling bottom would increase the wave height over that in deep water. Using figure 1 (Beach Erosion Board Bulletin Vol. 3, No. 4, p. 31) and assuming that the breakers were of 20-second period and broke in 20 feet of water,

\[
\frac{d_b}{T^2} = \frac{20}{400} = 0.05
\]

\[
\frac{H_0}{T^2} = 0.017; \quad H_0 = 0.017 \times 400 = 6.8 \text{ ft.}
\]

\[
\frac{H_b}{H_0} = 1.85 \quad \text{H}_b = 12.2 \text{ ft.}
\]

Here, \(H\) is the height after refraction. Dividing by a factor of 3, to account for the convergence over the submerged ridge as indicated by the
FIG. 3 - REFRACTION DIAGRAM FOR SAN PEDRO BAY FOR WAVES FROM THE WEST
refraction diagram, figure 4, the deep-water height would be 2.3 feet.

Waves of 20-second period are about 2,000 feet long in deep water and it is consequently understandable that waves of this length and a height of only 2 feet would not be observed at sea. It is also understandable why waves of this height in deep water would not be noticeable at the shore when their energy was further distributed lengthwise by refraction.

The wave period was not measured but it was certainly greater than 20 seconds, the period used for the refraction diagram. A longer period would not change the character of the refraction diagram but would have produced slightly greater focusing by refraction and a greater ratio, $H_d/H_0$.

Thus, it appears probable that the larger breakers which damaged the Long Beach breakwater during the interval April 20 to 24, 1930 were generated by a storm almost due south of Long Beach, in the area not covered by available weather maps.

LITTORAL DRIFT AT SANTA BARBARA

The shoreline phenomena effective at Santa Barbara have been the subject of extensive study and one can now speak more positively about them than was possible when erosion first appeared. Refraction diagrams played an important part in providing a rational basis for reasoning about the causes of erosion and the means of improvement.

When this study was started, there were almost as many theories about the cause of erosion as there were inhabitants of Santa Barbara County and it was necessary to develop an explanation satisfactory not only to engineers but to public officials as well.

When refraction diagrams were prepared, they corresponded so obviously with what anyone observed from the end of the breakwater or from vantage points along the shore that the remainder of the reasoning about the direction of the littoral drift and the causes of erosion was generally accepted.

Briefly, the littoral drift at Santa Barbara runs almost continuously from west to east and the volume transported is estimated to be around 1,000 cubic yards per day past each point on the shoreline. The beach consists of a relatively thin layer of sand overlying solid upland material and erosion quickly denudes the shore of sand. In 1929 construction was started on a breakwater at Santa Barbara harbor (3). Immediately, sand accumulated on the westerly side and erosion developed progressively eastward, becoming markedly evident as much as 5 miles east of the breakwater about 4 years later. As soon as the effect of this breakwater became evident, steps were taken to discover its cause.

The cause of the easterly littoral drift becomes evident when one examines the exposure of the shoreline between Point Conception and Mugu Lagoon, and the prevailing winds and waves. A line of islands, broken only by relatively restricted channels, parallels the shore and about 20 miles seaward. Prevailing winds and waves are from the west, only occasional periods of southerly
waves and winds occurring during winter storms. It is not the purpose of this memorandum to give more than a brief background of the problem in order to indicate the part which refraction plays in the observed phenomena. Space will not be taken to recount the many local theories, mostly unsound, but frequently difficult to refute, which effectively prevented remedial measures until a satisfactory explanation was developed.

Wave action at Santa Barbara is generally light, the breakers seldom exceeding 3 feet. The wave period appears to be greater than at points on the exposed coast north of Point Conception but there are no measurements to prove this. The waves are unusually regular in height and period, and are generally long-crested, the breaking of a single crest being frequently evident for a mile or more. The crests are always at a slight angle with the shore, the breaker appearing first at the westerly end of the crest and developing eastward. A current runs steadily from west to east in the surf zone and for a considerable distance seaward of the breaker line.

There are several features of the situation at Santa Barbara which seemed peculiar. In the first place, the rate of accumulation at the breakwater seemed large in view of the relatively light wave action observed. Few accurate determinations of rates of drift were available for comparison, but the few approximate determinations previously made, such as the average growth of the westerly tip of Long Island, were of the same magnitude and every circumstance of the Santa Barbara surf and waves suggested that the littoral drift there should be much smaller than on Long Island. The answer to this problem was that at Santa Barbara there was every reason to expect the sand movement to be easterly almost continuously throughout the year. A second unusual feature was that the waves always seemed to break at the same angle in spite of the fact that they obviously must approach from many different directions, as determined by winds. The refraction diagrams showed that from any westerly bearing between north and south the waves would be refracted in such manner that the crests would occupy the same position at the end of the breakwater, within the accuracy of visual observation. The only effect of variations in initial wave direction would be a variation in height after refraction. The third effect demonstrated by the refraction diagrams was that there would be filtering effect, the long-period waves being refracted more than those of short-period. This effect probably accounts for the apparent increase of wave period over that north of Point Conception, and the relative regularity in height and period.

Figure 5 shows a 10-second wave approaching from the southwest. Orthogonals to the wave fronts on this diagram have been drawn to show the extent of the spreading of the wave energy. This wave system shows an increase in crest length off Santa Barbara for waves passing the west end of San Miguel Island in the ratio 10 to 1 over the length in the open ocean, and the length of crest of breaker is more nearly 20 to 1 (i.e., k_d = 0.22). Waves passing between San Miguel and Santa Rosa Islands show so great an increase in crest length that their effect could not be noticed in the larger waves passing west of San Miguel Island. A portion of the same system would pass around the other islands but the increase in length would be even greater.
FIGURE 6 - REFRACTION DIAGRAM AT SANTA BARBARA AND VICINITY FOR 10-SEC. PERIOD WAVES FROM VARIOUS DIRECTIONS.

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HYD 2172-116-00
Diagrams, similar to figure 5, have been drawn for the same wave period but different directions with a slightly different refraction effect. Figure 6 shows in detail the refractive effect near shore of waves which, in the open ocean, approached from the west, southwest, and west southwest. In figure 6, which was prepared by transferring wave fronts from smaller scaled diagrams as illustrated by figure 5, waves from these three directions have practically the same wave front orientation in the vicinity of Santa Barbara. Figure 7 shows waves approaching the channel islands from the south and southeast. Wave fronts from figure 7 have been transferred to a larger scale chart and then carried into shore as shown in figure 8. Examination of figure 7 shows that waves from the southeast and those from the south, which are refracted around the east end of Anacapa Island, both approach the shoreline in the vicinity of Santa Barbara with practically the same alignment; hence, the wave fronts in figure 8 are indicated as being for both of these initial wave directions. Waves passing between Santa Cruz and Anacapa Islands show so great an increase in crest length that their effect near shore could not be noticed in comparison with waves passing to the east of Anacapa Island.

The wave and swell diagrams of the Hydrographic Office (2) had not been prepared at the time this study was made and conclusions regarding the sources of waves were based upon wind data, from which it appeared certain that waves from the south or southeast would be infrequent and that waves approaching in the sector between west and north would occur during most of the year. The limited fetch between Santa Barbara and the Channel Islands and the moderate winds precluded the possibility of local wind waves playing an appreciable part in the shoreline changes.

The refraction diagrams and supporting wind data seemed to justify the following conclusions:

1. Waves from westerly directions will break obliquely and generate a littoral current during most of the year.

2. The intensity of wave action at Santa Barbara will be materially less than at points on adjacent exposed coasts for any wave direction in deep water, including southeast—the direction of greatest exposure.

3. Conclusions regarding the effect of long waves arrived at by observations on exposed coasts are not directly applicable at Santa Barbara.

It might be said that these conclusions could be reached merely by inspection without drawing refraction diagrams and this comment would be valid in a formal sense. However, one is merely substituting a mental construction of the diagrams for the graphical development and this is readily done after some experience. Usually, only a few periods and directions need be drawn even for quantitative use, the remaining conditions being supplied by inspection and interpolation.
REFERENCES


* * * *

ANNOUNCEMENT OF PUBLICATION

The Beach Erosion Board announces the publication of its Technical Memorandum No. 11 "Reflection of Solitary Waves." Copies are being mailed to those individuals and institutions on the mailing list for technical publications.

The report is the result of experimental study of the reflection of solitary waves conducted in the Board's laboratory.

Solitary wave crests were made to impinge upon various types of structures. The amount of energy absorbed from an incident wave by the structures was determined. Fundamental equations defining the absorption of wave energy are developed and the various absorption coefficients are evaluated.

A limited number of copies are available for distribution upon request to the President, Beach Erosion Board, Corps of Engineers, 5201 Little Falls Road, N. W., Washington 16, D. C.
FIG. 7 - REFRACTION DIAGRAM FOR SANTA BARBARA AND VICINITY FOR WAVES FROM THE SOUTH AND SOUTHEAST.

WAVE PERIOD = 10 SECONDS

--- DEPTHS IN FATHOMS

SCALE IN NAUTICAL MILES

HYDRO. 2179-116-00

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A FORMULA FOR THE CALCULATION OF THE TIDAL DISCHARGE THROUGH AN INLET

Dr. Garbis H. Keulegan, National Hydraulic Laboratory, National Bureau of Standards, and Jay V. Hall, Jr., Chief, Field Research Section, Beach Erosion Board

INTRODUCTION

During the course of an investigation of Santa Rosa Island, Florida, a number of engineering problems arose which required special study because of the lack of field data. The most interesting of these problems involved the advisability of cutting an inlet through the island. The specific problem, which required special mathematical treatment, was brought about through the determination of the division of flow between Pensacola Inlet and the relatively small cut through Santa Rosa Island. Inasmuch as adequate field data were available in the vicinity of the proposed cut, this locality presented no problem. However, the only information available for Pensacola Inlet was the tidal prism of Pensacola Bay. With this tidal prism quantity in mind as the only available working data, certain basic assumptions were made and a formula was derived for the determination of the maximum discharge that could be expected through the inlet at any tidal cycle. Since the formula as derived checked remarkably well when applied to a number of inlets that had been gaged, it was thought that the information would be of value to engineers. Hence it is presented in this monograph as a matter of record. In the following pages there is a complete mathematical analysis of the problem including the basic assumptions, the development of the formula, the test cases as applied to four inlets on the Atlantic coast, the development of a correction factor introduced to adjust the basic assumptions, and a demonstration of the practical application of the formula.

MATHEMATICAL TREATMENT OF PROBLEM

The following mathematical analysis presents the development of the equations used for the computation of the maximum quantity (discharge) of water which may be expected through a tidal inlet at any tidal cycle.
In figure 1, $T$ is the period of the tide. For any time $t$, the discharge of the tide is $q$. Hence, the total discharge (volume of the tidal prism) by definition is

$$V = \int_0^T q \, dt$$

(1)

As a first assumption that $q$ is a sine function it follows that

$$q = Q_m \sin \frac{2\pi t}{T}$$

(2)

where $Q_m$ is the maximum value of the tidal discharge, which may be expected at any tidal cycle.

Substituting the value of $q$ from equation 2 in equation 1

$$V = Q_m \int_0^T \sin \frac{2\pi t}{T} \, dt$$

(3)

Integrating the expression

$$\int_0^T \sin \frac{2\pi t}{T} \, dt = \frac{T}{\pi}$$

(4)

Substituting the result in equation 3

$$V = Q_m \frac{T}{\pi}$$

or

$$Q_m = \frac{\pi V}{T}$$

(5)

CHECK ON FORMULA BASED ON FIRST ASSUMPTION

In order to ascertain the correctness of the formula as derived, the maximum quantity of water which might be expected to pass through an inlet is computed and compared with actual field data of measured flows through certain inlets. The inlets to be discussed, which included Nantucket Inlet, Nantucket Island, Massachusetts; Manasquan Inlet, New Jersey; Beaufort Inlet, North Carolina; and Baker's Haulover Inlet, Florida; were not chosen because of their adaptability to the formula, but because they were the only inlets for which field data were available.

Nantucket Inlet, the entrance to Nantucket Harbor, is situated on the western shore of Nantucket Island, which lies off the coast of Massachusetts (figure 4). The harbor is a land locked body of water and has but one entrance, the inlet under discussion. The tidal prism of the harbor as derived from gaging data taken on 29 December 1938 is given as 431,730,000 cu. ft. With this value known, the maximum quantity of water which might be expected
through the inlet can be computed.

\[ Q_m = \frac{\pi V}{T} = \frac{3.14 \times 431,730,000}{12.42 \times 3,600} \]

\[ Q_m = 30,300 \text{ c.f.s.} \]

The computed \( Q_m \) when compared in the following manner

\[ \frac{Q_m \text{ (computed)} - Q_m \text{(observed)}}{Q_m \text{ (computed)}} \]

with the observed \( Q_m \) taken from the average discharge curve on figure 5 shows that the computed value is 12.5% high. This is considered to be in reasonable agreement with the gaged data.

Manasquan Inlet (figure 6) located on the New Jersey Coast about 25 miles south of Sandy Hook, is a long tidal estuary which derives the maximum portion of its tidal prism from the tidal flow through the inlet. The tidal prism as derived from gaging data taken in June 1935 is given as 174,500,000 cu. ft. With this value known, the maximum quantity of water, \( Q_m \), was computed to be 12,250 c.f.s. The computed \( Q_m \) when compared with the observed \( Q_m \) taken from the average discharge curve on figure 7 shows the computed value to be 15.1% high.

Beaufort Inlet (figure 8) is situated on the coast of North Carolina about 10 miles west of Cape Lookout. The inlet connects the Atlantic Ocean with Bogue Sound, Back Sound, and the tidal estuaries of the North and Newport Rivers. When gaged on 15 March 1927 the tidal prism was observed to be 4,250,000,000 cu. ft. Substituting this value in the derived formula, the theoretical \( Q_m \) is found to be 299,000 c.f.s. This computed \( Q_m \) when compared with the observed \( Q_m \) read from the average discharge curve in figure 9 discloses that the former value is 15.5% high.

Baker's Haulover Inlet (figure 10) lies on the eastern coast of Florida about 10 miles north of the entrance to Miami Harbor. The inlet connects the Atlantic Ocean with the upper reaches of Biscayne Bay which widens and opens southward. The tidal prism as determined from the gaging data observed 28 October 1943 was 360,100,000 cu. ft. Hence the computed \( Q_m \) is found to be 25,300 c.f.s. Then this value is compared with the observed \( Q_m \) from the average discharge curve on figure 11 it is noted that the value is 13.0% high.

The results derived from the above examples are listed in table 1.

<table>
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<th>Inlet</th>
<th>( Q_m \text{ (c.f.s.)} )</th>
<th>( Q_m \text{(observed) (c.f.s.)} )</th>
<th>Deviation</th>
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<td>+13.0</td>
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<tr>
<td>Average Deviation</td>
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<td>+14.0</td>
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CORRECTION OF FORMULA

The reason for the difference between the computed and observed maximum discharges lies in the fact that the curve for the observed discharge is not a true sinusoidal curve as it was considered to be in the first assumption. Since the hydraulic flow through an inlet is due to turbulent forces, it is expected that the true curve of the discharge has other harmonics. This is known to be true since the simple sine curve used as the first assumption applies only when the flow through an inlet is viscous. This can be shown very easily.

Suppose that a small reservoir is connected with a large reservoir through a capillary connection.

\[ q = a(h_2 - h_1) \]  

where 
\[ a = \frac{\pi r^4 \cdot \rho g}{\mu l} \]

in which
- \( r \) = the radius of the capillary tube
- \( \mu \) = the viscosity of the fluid
- \( l \) = the length of the capillary tube
- \( \rho \) = the density of the fluid

Letting the surface area of the small reservoir equal \( A \)

\[ q = A \frac{dh_1}{dt} \]

Equating the right hand members of equations 6 and 7

\[ A \frac{dh_1}{dt} = a(h_2 - h_1) \] 

This equation may be written

\[ \frac{dh_1}{dt} = Ch_2 - Ch_1 \]

\[ h_1 \]

\[ \text{MEAN WATER LEVEL} \]

\[ h_2 \]

FIGURE 2

Let \( h_1 \) and \( h_2 \) be the fluctuations of the surface elevations above and below the mean water level in the small and large reservoirs respectively (figure 2). Inasmuch as the flow in this case is due to viscous forces the discharge \( q \) is proportional to the difference, \( h_2 - h_1 \), that is:

\[ \frac{dh_1}{dt} = C(h_2 - h_1) \]
where

\[ C = \frac{a}{A} \]

Equation 9 is satisfied when

\[ h_1 = \frac{TQm}{2\pi A} \cos \frac{2\pi t}{T} \]  \hfill (10)

and

\[ h_2 = H \cos \left( \frac{2\pi t}{T} + \epsilon \right) \]  \hfill (11)

where

\[ H = \frac{Qm}{A} \sqrt{\frac{T^2}{4\pi^2} + \frac{1}{C^2}} \]  \hfill (12)

and

\[ \tan \epsilon = \frac{2\pi}{C T} \]  \hfill (13)

These expressions given by equations 10 and 11 show that the fluctuations of the water surfaces in the two reservoirs are sinusoidal with a phase difference \( \epsilon \).

Substituting the value of \( h_1 \) from equation 10 in equation 7 it is seen that

\[ q = Q_m \sin \frac{2\pi t}{T} \]  \hfill (14)

which was the first assumption.

Since the results of the test cases show that the values computed on the first assumption deviated slightly from the actual gaged data, it is reasonable to assume as a second assumption that the introduction of a third harmonic into the basic equation would produce more accurate results.

![Figure 3](image-url)

Hence

\[ q = K_1 \sin \frac{2\pi t}{T} + K_2 \sin \frac{6\pi t}{T} \]  \hfill (15)

Since

\[ V = \int_0^T q dt \]  \hfill (16)

Substituting the value of \( q \) from equation 15 and integrating

\[ V = \left( K + \frac{K_2}{3} \right) \frac{T}{\pi} \]  \hfill (17)
By definition \( q_m \) is the maximum value of \( q \), then

\[
Q_m = K_1 - K_2
\]  

(18)

Therefore

\[
K_1 = Q_m (1 + n)
\]

\[
K_2 = nQ_m
\]  

(19)

Substituting these values in equation 17

\[
V = Q_m (1 + n + \frac{n}{3}) \frac{T}{\pi}
\]

or

\[
V = Q_m (1 + \frac{4}{3}n) \frac{T}{\pi}
\]

or

\[
Q_m = \frac{\pi V}{T (1 + \frac{4}{3}n)}
\]  

(20)

In order to compute the value of "n", the formula derived on the first assumption is arranged in the same form as that which was used to compute the percent deviation.

Hence

\[
\frac{Q_m \text{(computed)} - Q_m \text{(observed)}}{Q_m \text{(computed)}} = \frac{\pi V}{T} - Q_m
\]

Substituting the value of \( Q_m \) found in equation 20 we have

\[
\frac{\pi V}{T} - \frac{\pi V}{T (1 + \frac{4}{3}n)} = \frac{4n}{3 + 4n}
\]

Inasmuch as table 1 shows that the average deviation based on the first assumption is 14\% high, then

\[
\frac{4n}{3 + 4n} = 0.14
\]

\[
n = 0.122
\]

Substituting this value in equation 20 the corrected formula is

\[
Q_m = 0.86 \frac{\pi V}{T}
\]  

(21)

Recomputing the values of \( Q_m \) given in table 1 with the corrected formula the results are shown in table 2.

<table>
<thead>
<tr>
<th>Inlet</th>
<th>Computed ( Q_m ) (c.f.s.)</th>
<th>Observed ( Q_m ) (c.f.s.)</th>
<th>Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nantucket</td>
<td>26,050</td>
<td>26,500</td>
<td>-1.70%</td>
</tr>
<tr>
<td>Manasquan</td>
<td>10,500</td>
<td>10,400</td>
<td>+0.95%</td>
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<td>Beaufort</td>
<td>257,000</td>
<td>252,500</td>
<td>+1.75%</td>
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<tr>
<td>Baker's Haulover</td>
<td>21,750</td>
<td>22,000</td>
<td>-1.13%</td>
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</table>
Thus it can be seen that the formula as corrected by the introduction of the third harmonic in not deviating more than plus or minus 2% is sufficiently accurate to be used in tidal work where a like accuracy is seldom approached.

APPLICATION OF FORMULA

On reviewing a new formula, the first thought which comes to one's mind is the question of its adaptability to current engineering problems. Consequently an effort will be made to show a method of application and comparison of the results with actual gaged data.

First, let it be assumed that Nantucket Inlet is being studied and that it is desirable to know the maximum quantity of water which may be expected to pass through the inlet during the afternoon of 29 December 1933. Inasmuch as the tidal prism of the inner basin is equal to the area of the basin multiplied by the range in tide, the area of the basin was planimetered from U. S. C. & G. S. Chart No. 1209 and the tidal range for the afternoon of 29 December 1933 was taken from the predicted tides in the U. S. C. & G. S. tide tables for 1933. The area as measured from the chart was found to be 190,000,000 square feet, while the tidal range was found to be 2.3 feet. Consequently the tidal prism as computed is 437,000,000 cu. ft. Substituting this value of $V$ in the formula, $Q_m = 0.86 \sqrt{V}$ it is found that the maximum quantity of water that may be expected to flow through the inlet on the afternoon of 29 December 1933 is 26,300 cu. ft. per sec. This discharge when compared with the gaged value of $Q_m$ given in table 2 shows a deviation of only 0.75%. The remarkable agreement of the observed and computed maximum discharges should not be construed to mean that the formula will work equally well for any inlet under all conditions. In the first place the determination of the area of the inner basin can only be determined accurately when the inner basin is well defined such as Nantucket Harbor. Consequently, as the inner basins become more complex such as that at Beaufort Inlet, the accuracy of the application of the formula depends wholly on one's ability to choose the tidal points in the various bodies of water which make up the inner basin. In other words, the value of the application of the formula decreases as the inner basin increases in complexity. In addition to the limitations of accurately determining the area of the inner basin there is always the possibility that the predicted range of the tide for a specific day as read from the U. S. C. & G. S. tide tables will be altered by strong persistent winds. It can be seen, however, that a few tide gages advantageously placed in the inner basin will assist in making a more accurate estimate of the limits of the inner basin and also give accurate tidal ranges.

CONCLUSION

In presenting this monograph no effort has been made to show that the formula developed will necessarily eliminate inlet gaging, but it has merely been presented as an additional working tool to be used by engineers when applicable.
NANTUCKET HARBOR ENTRANCE, MASS.
LOCATION MAP

Fig. 4
NANTUCKET HARBOR ENTRANCE, MASS.

29 DECEMBER 1938

Fig. 5

Discharge - C.F.S.
Fig. 7

MANASQUAN INLET, N.J.
JUNE 1935

Discharge - C.F.S.

Hourly Interval

Av. Q_m. - 10,400 c.f.s.
BAKER'S HAULOVER INLET, FLA.

28 OCTOBER 1943

Discharge - C.F.S.

Fig. 11
CHARACTERISTICS OF MEASURED WAVE ACTION ON THE BASIS OF THE FREQUENCY DISTRIBUTION OF WAVE LENGTH, WAVE HEIGHT, AND STEEPNESS

The following translated paper appeared in limited issue as Technical Report HE-116-105, Fluid Mechanics Laboratory, University of California. It is reproduced here to bring the concepts therein to the attention of research workers and others having an interest in ocean waves. The original paper "Kennzeichnung des gemessenen Seegangs auf Grund der Häufigkeitsverteilung von Wellenhöhe, Wellenlänge und Steilheit" was prepared by H. Ehring and was published in Technische Berichte 4, 1940.

Since 1932, a wave measuring instrument (1) has been used for the measurement of wave action and has permitted the continuous recording of a great amount of data. The results of numerous measurements that have been made are satisfactory, since the customary description of measured waves has been shown to be incomplete. For the designation of height, H, the measured wave has, in general, been taken as the height of the largest measured waves, or the height which was exceeded by 10 or 20 waves in the hour. The wave length, L (in meters), is obtained from the period, T (in seconds), which the instrument records, by the trochoidal wave theory relationship L = 1.56T^2. The wave length of the measured waves is designated by means of the mean length of the 10 or 20 highest waves, or by the mean length, which resulted from a number of large waves counted in a definite time (2). The steepness of the waves, that is, the ratio H/L, is seldom specified and is very inaccurate in the case of the correlation of a few chosen wave heights to one of the above mentioned mean wave lengths. Frequently the determined wave measurements have been correlated with a wave scale (3), and the measured wave action estimated by this wave scale. All of these methods do not describe the complete picture of wave action and exclude a numerical comparison of different wave actions.

Since it is not possible to characterize the wave action, which a varied collection of higher and longer waves represents, in its entirety by the specification of a quantity such as \( H_{\text{max}} \), or \( H_{\text{mean}} \) (which remains independent of the number and quantity of all occurring waves), only the evaluation of a great number of measured waves according to height and length can yield a satisfactory result. The results may be presented best in the form of a frequency curve, whereby the wave lengths and heights can be specified as known values which are prominent in relation to the frequency of their occurrence, either absolute or as percent in reference to the total number of measured waves.
The frequency value of different wave intensities (up to 3 meters in height) which have been recorded in the Baltic and North Seas has shown that a frequency distribution of wave heights and lengths characterizing the waves can be set up with the evaluation of 300 of the highest 800 waves according to the height of the waves from which can be selected the class distribution for the frequency value.

For the evaluation, a form sheet has been set up (figure 1) which permits a quick and easy tabulation of the successively evaluated waves and in which, by simple addition of the numerical values, the frequency distribution of the first and second kind can be determined for wave heights and lengths. With sufficient accuracy the frequency distribution for the steepness $H/L$ can be determined with the aid of the form sheet in the more predominant classes.

The procedure of a frequency valuation (4) follows: From a given wave condition, waves are taken which, according to their properties or measurements, will be assigned to definite classes (of equal size). The occurrence of the waves in a single class interval, expressed through a number, $H_1$, makes a frequency of the first kind, representing the class average. The sum of all the investigated waves is the total frequency $N$. By addition, one obtains the frequency of the second group ($H_2$) from the frequency of the first kind ($H_1$). The frequency of the second group indicates how often a class limit will be exceeded or how many values will be below a class limit when the characteristics are considered the class average. With a sufficient number of observations, the frequency of the second kind (cumulative frequency) may be produced in the form of a curve (summation curve). The preparation of a frequency distribution as a summation curve has the advantage that the collections which have been evaluated with different class distributions, may be compared with one another, since the class limit is worked out only from the number of determined points for the curve, in which process the principles are not changed. To make the frequency distribution independent of the time or the extent of the valuation, one specifies the frequency in percentage (percent frequency).

The cumulative frequency of wave steepness ($H/L$) for the data given and analyzed as in figure 1 resulted in the following:

<table>
<thead>
<tr>
<th>$H/L$</th>
<th>$H_1$</th>
<th>$H_2$</th>
<th>$H_2%$</th>
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<td>1/1 - 1/10</td>
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<td>161</td>
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### FORM SHEET - FREQUENCY EVALUATION

(Period T in seconds and wave length L in meters)

<table>
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<tr>
<th>T-IN CLASS AVERAGES</th>
<th>0.81</th>
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<th>1.89</th>
<th>2.43</th>
<th>2.97</th>
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| WAVE LENGTH-L        | 0.45 | 1.82 | 4.09 | 7.28 | 11.37| 16.38| 22.29| 29.17| 36.85| 45.49| 55.04| 65.50|

**GRADUATIONS**

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</table>

| H1     | 100 | 100 | 14.9 | 71 | 171 | 24.85 | 54 | 225 | 32.7 | 76 | 301 | 43.75 | 58 | 359 | 52.2 | 406 | 59.0 | 458 | 66.6 | 52 | 510 | 74.1 | 39 | 549 | 79.8 | 34 | 583 | 84.7 | 28 | 611 | 88.8 | 22 | 633 | 92.0 | 18 | 651 | 94.6 | 16 | 667 | 96.9 | 9 | 676 | 98.3 | 7 | 683 | 99.27| 3 | 686 | 99.71| 2 | 688 | 100 |

**FIGURE 1**
FREQUENCY CURVES OF WAVES
JUNE 23, 1937

INSTRUMENT LOWERED AT ABOUT 11:40 EVALUATED
FROM 12:50 TO 13:40 INSTRUMENT RAISED AT
ABOUT 13:40 TIME EVALUATED: 90 MINUTES

RESULTS MEDIAN DEVIATION FOR 50% OF WAVE

| WAVE HEIGHTS H | 36 CM | 15 — 60 CM |
| WAVELENGTHS L | 7.2 M | 3 — 15 M |
| STEEPNESS H/L | 1:23 | 1:18 — 1:32 |

a READ AT 50 VS H OF THE TOTAL FREQUENCY
b READ AT 25 AND 75 VS OF THE TOTAL FREQUENCY

FIGURE 2
In the light of these considerations, the percent cumulative frequency 
(H₂ vs H) is the most convenient form for the preparation of the frequency 
distribution of wave action. For the plotting of H₂ vs H, probability paper 
(5) has shown itself to be advantageous. The normal distribution, in accord-
ance with the Gaussian Probability Law, appears as a straight line (frequency 
distribution) on this paper. Since the investigated distribution of wave 
heights and periods on the probability paper likewise showed a greater range 
over rectilinear coordinates, it was natural that this method of preparation 
be referred to for the estimation of wave action. The reproduction of a fre-
quency distribution on probability paper (figure 2) replaces, in part, the 
otherwise customary mean values and deviations for the estimation of a frequency 
distribution (4).

From the stated mean values, according to Czuber (4), the median value 
C for a given distribution can be read immediately from the frequency curve 
(50% vs H) on probability paper.

The calculation of a deviation is not necessary, since the curve on 
the probability paper permits immediately the reading of the deviation which 
appears as a percentage of the total occurrences. Thus, for example, the 
deviation range of 50% of the distribution (known as quartile deviation) on 
either side of the median values is included between 25% N and 75% N. A 
进一步, not to be underrated, advantage of this method is its relationship 
to the probability of the occurrence of given wave sizes. The median value 
C indicates the value which will be exceeded (or not exceeded) with the proba-
bility of 0.5. Waves with measurements which fall within the specified limit-
ing value of those of the range of deviation, occur with the probability 0.5 
since the deviation embraces 50% vs H of all the measured waves. With the 
statement of this value, it still will not be possible to obtain the expression 
with what probability an aircraft taking off or landing will encounter a 
definite wave height. The probability of the single exceedance of a given 
wave height in a take-off or landing, depends not only upon the wave action, 
but also upon the number of waves which would be met in the take-off or land-
ing. Closer investigation of this still should be conducted.

For the evaluation of wave action, suitable key values of the median 
value C and the deviation are prepared for 50% vs H of all investigated waves. 
These key values as the example showed will indicate the wave length and the 
steepness H/L from the wave height, H₁ and the period, T, and thus the wave 
action can be determined numerically.

How completely the proposed key values of measured wave action by means 
of the median value and deviation can generally be employed is not to be 
overlooked. The use of these values is not necessarily unlimited since the 
frequency distribution as such, if it originates on the basis of uniform 
values and comes in uniform form for the "preparation", clearly determines 
the measured wave action. From this method, the corresponding quantities 
can be taken; for example, it showed with all values that the specified heights 
which would be exceeded by 10 or 20 waves in the hour fell approximately in 
1% vs H of the total frequency (figure 2). The somewhat troublesome deter-
mination of the frequency distribution of H/L can be eliminated if one is 
only concerned with the determination of the median value for H/L, which,
as the example in figure 2 showed, permits a sufficiently accurate determination by means of the combination of the median values of H and L.

The proposed frequency valuation for a determination of wave action requires from 2 to 4 hours of evaluation work or about one-half of the time required by other methods. This frequency valuation works out favorably for long continued measurements for statistical purposes because valuations carried out at intervals of 3 to 6 hours suffice for the description of the wave action over long periods. For the future, an instrument must be sought, which undertakes the location of the measurements of a number of wave heights and lengths into classes and as a result yields the frequency distribution of the first and second kind.

Schrifttum (References)


* * *
The Central Committee for Oceanography and Coastal Studies is a French Governmental agency associated with the Naval Hydrographic Service of the French Ministry of National Defense. In January 1949 this committee initiated the periodic issuance of an information bulletin. This bulletin contains articles of general information on committee activities and interests, technical communications in condensed form, and bibliographic notes. Distribution of this bulletin is limited and the Board takes this opportunity to again brief some of the articles contained in the bulletins to acquaint American students with the extent and character of the coastal problems in other areas.

* * *

REMARKS ON WAVE FORECASTING
by M. Gelci

This article reports the results of the comparison between the Sverdrup-Munk method of forecasting and the Jeffrey Suthons method of forecasting, both being applied to the forecast of waves on the Moroccan coast. As a result of several studies the following conclusions are reached: (1) the measured amplitudes of waves, are approximately ten times as large as those forecast; (2) the time of arrival of the waves varies from 2 to 4 days earlier than forecast; and (3) the wave period is forecast within an accuracy of 2 to 3 seconds.

In an attempt to explain the differences which have been noted in the comparison, the author notes that study of the weather charts indicates that the isobars of the charts are not sufficiently precise, especially over the ocean areas, to permit the prediction of the value of the wind velocity with accuracy.

He states further that it appears that in the actual state of African meteorology wind velocities derived from isobars are probably not useful because of their lack of precision for purposes of wave prediction.

* * *

RELATIONS BETWEEN MICROSEISMIC ACTIVITY AND WAVES
by J. Debrach

The study concerns principally the microseismic activity frequently noted on seismograph records in the absence of any seismic activity on land. The height of these waves is very small, being of the order of several microns. The periods of the waves vary from about one second to one minute but usually lie in the range, 3 to 10 seconds. It has been noted that these seismic waves occur simultaneously over very large areas, for example, occidental Europe frequently is the order of the magnitude of the area involved. Several hypotheses have been proposed by numerous
authors to explain the agitation. As a result of the study made by Mr. Debrach, he concludes that study of the microseismic waves, at this time does not offer possibility of the use of such microseismic activity as a means of forecasting the occurrence of oceanic waves. It does appear however that microseismic waves may be useful in the consideration of other data to be used in the forecasting of waves.

* * *

WAVE DIFFRACTION
by H. Lacombe

A comparison is made between the classical hydrodynamic methods of studying wave diffraction and the optical methods of Huyghens. The author cites a number of studies which have been made on the diffraction of waves and presents the results of a study which he has made comparing the Huyghens method with the classical hydrodynamic method developed by Sommerfeld. He concludes that while further experimental verification is necessary, the simple method of Huyghens gives results which are very similar to those of the classical hydrodynamic method; and that the Huyghens method appears to have certain advantages in that it is a much simpler method to use in cases where the hydrodynamic solution may be extremely difficult. He points again to the need for considerable further study which he anticipates will be very fruitful.

* * *

AN EXPLANATION OF CERTAIN MARINE CURRENTS
by M. P. Antoine

In the Mediterranean one often notes long tongues or bands of pale green or pale blue water occurring in an area of deep blue water. These occurrences are noted usually after several days of high winds. The explanation of these bands may be discovered by observing the water immediately adjacent to a beach. One notes first a yellow area which contains sand in suspension, followed to seaward by a green area which contains a suspension, probably of very fine mud particles, this latter area grading in color to the deep blue of the offshore waters. Coastal currents push this water along the coast. When it arrives at a cape or other headland, the currents continue their path at the surface essentially in a straight line and thus form in the deep blue offshore water a tongue of green water which often has advanced quite far from the coast. Similarly when the wind blows from the land and is canalized, for example by a valley descending to the sea, this land wind then pushes before it the green water of the coast which later becomes pale blue by dilution. The author does not believe that the change in color is due to a mixing of the blue water but rather states the following explanation. The action of wind is not transmitted to the deep water by reason of the viscosity of the water, therefore the currents must lie as a very thin layer on the surface of the sea. The coastal current and the suspended material entering the deep water therefore occurs solely on the surface of the water and as the

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coastal currents of the suspended material proceed farther from shore they lose a large part of the suspended material, causing their color to change by reason of the settling of the material. The author notes that these color bands may be found at very great distances from land and also states that they exist only in times of calm. He explains this by stating that whenever the wind is strong or irregular the tongues of water are mixed with the surrounding water and lose their identity.

* * * *
The expansion of the Beach Erosion Board research facilities by the construction of a large wave tank and a coast model test basin is in progress adjacent to the Board's offices and laboratory. A brief description of these facilities appeared in the Bulletin, Vol. 3, No. 4, October 1, 1949. Contract work is about 60 percent complete at the time this publication goes to press.

Excavation work for the wave tank and coast model basin has been completed and concrete work is now progressing slightly ahead of schedule. Progress photographs of the construction operations on the wave tank are shown in figures 1-5. Progress pictures of the basin construction will be published in a later issue of this publication.

WAVE TANK PROGRESS PHOTOGRAPHS

FIGURE 1- VIEW SHOWING EXCAVATION AND SUBGRADING THE FLOOR OF 6-FOOT WAVE TANK PRIOR TO PLACEMENT OF FORMS, REINFORCING AND CONCRETE OF THE FLOOR SLAB
WAVE TANK PROGRESS PHOTOGRAPHS

FIGURE 2-POURING THE INITIAL BUCKET OF CONCRETE FOR THE FLOOR SLAB

FIGURE 3-PROGRESS OF THE PLACEMENT OF FLOOR REINFORCING STEEL, FLOOR SLAB FINISHING, AND WALL CONSTRUCTION
WAVE TANK PROGRESS PHOTOGRAPHS

FIGURE 4 - WALL REINFORCEMENT IN FOREGROUND, WALL FORMS WITH CROSS BRACING IN BACKGROUND

FIGURE 5 - WAVE TANK WALL SECTIONS
BEACH EROSION STUDIES

The principal types of beach erosion reports of studies at specific localities are the following:

a. Cooperative studies (authorization by the Chief of Engineers in accordance with Section 2, River and Harbor Act approved on 3 July 1930).

b. Preliminary examinations and surveys (Congressional authorization by reference to locality by name).

c. Reports on shore line changes which may result from improvements of the entrances at the mouths of rivers and inlets (Section 5, Public Law No. 409, 74th Congress).

d. Reports on shore protection of Federal property (authorization by the Chief of Engineers).

Of these types of studies, cooperative beach erosion studies are the type most frequently made when a community desires investigation of its particular problem. As these studies have, consequently greater general interest, information concerning studies of specific localities contained in these quarterly bulletins will be confined to cooperative studies. Information about other types of studies can be obtained upon inquiry to this office.

Cooperative studies of beach erosion are studies made by the Corps of Engineers in cooperation with appropriate agencies of the various States by authority of Section 2, of the River and Harbor Act approved on 3 July 1930. By executive ruling the cost of these studies is divided equally between the United States and the cooperative agency. Information concerning the initiation of a cooperative study may be obtained from any District Engineer of the Corps of Engineers. After a report on a cooperative study has been transmitted to Congress, a summary thereof is included in the next issue of this bulletin. A list of cooperative studies now in progress follows the summary.

SUMMARY OF REPORT ON COLONIAL BEACH, VIRGINIA
TRANSMITTED TO CONGRESS

Colonial Beach is located on the west bank of the Potomac River 40 miles from its mouth at Chesapeake Bay and 69 miles downstream from Washington, D. C. The town occupies a low peninsula between the Potomac River and Monroe Creek. The frontage of the town along the river is about 2¼ miles. Colonial Beach is a popular summer resort and is served by good highways. Colonial Beach draws its summer population from Baltimore, Washington, Richmond, and intermediate population centers. In addition to its summer residences, hotels and recreational facilities are available. The small-boat harbor in Monroe Creek also attracts many pleasure craft to Colonial Beach in addition to providing a base for a considerable seafood fleet.
The shore in the vicinity of Colonial Beach consists of a narrow beach up to about 50 feet in width in front of a bank up to about 20 feet high. The bank is composed of silty sand with some gravel. Much of the bank material is too fine to remain on the beach when eroded from the bank. The beach consists principally of sand and gravel with little material finer than fine sand.

Wave action at times of high river level attacks the bank above beach level causing recession of the shoreline. Over the period of record the shoreline generally receded throughout the area studied, the maximum recession amounting to about 300 feet or nearly 4 feet a year. An exception was the shore at the south end of Colonial Beach where the shoreline advanced over the period of record. From this accretion it appears that the predominant littoral drift is southward although accretion on the south sides of groins during southerly storms indicates that reversals in direction of drift occur. The minor shoaling of the channel into Monroe Creek indicates that the volume of littoral drift is not large.

In attempts to arrest erosion of the bank and to stabilize the beach, local interests have constructed light wooden bulkheads and retaining walls totalling approximately 3,200 feet in length and have also built numerous timber groins. When in good condition these structures served their intended purpose. At present, parts of the bulkheads and walls and all of the groins are in poor condition.

The shore area north from Gun Bar Point lying between the State highway, Riverside Drive, the boardwalk and the Potomac River for a distance of about 10,000 feet is owned by the town of Colonial Beach and has been dedicated as public beach. Northerly thereof the beach is privately owned, except for the area covered by the 16-foot boardwalk but the public is admitted to the beach in front of the boardwalk without charge.

The District Engineer considered the desires of the cooperating agency, studied the changes in the shoreline, determined the most suitable methods of protecting the shore against erosion and made an economic analysis of proposed protective measures for the publicly-owned shore. The District Engineer found that the best method of preventing this erosion consists of a stone revetment. He considered that about one-half the frontage requires immediate protection and estimated the total cost of the work at $105,000. The District Engineer considered stabilization of a portion of the beach by a groin system, but found that the beach can be preserved at lower cost by artificial replenishment as required.

The District Engineer recommended that a project be adopted by the United States authorizing Federal participation, subject to certain conditions of local cooperation, in an amount equal to one-third the cost of the protection of the shore at Colonial Beach, Virginia, which comprises the construction of 7,350 linear feet of stone revetment along the State highway, about one-half of which he recommended for initial construction and the remainder as deferred construction. The Division Engineer concurred in the conclusions and recommendations of the District Engineer.

The Board carefully considered the reports of the reporting officers. It concurred in their views and recommendations. The shore along the State highway is receding. Part of this shore needs immediate protection. The remainder will probably need protection in the future. The most suitable
and economical type of protection for this bank of the estuary is stone revetment. A course of stone ranging in size from screenings to 100 pounds should be included next to the bank to prevent erosion by water passing through the voids of the larger stone. The initial construction should protect the portions of the shore that are now seriously eroding. The remaining portions of the shore should be included in the project for protection, but construction can be deferred until necessitated by further erosion.

The beaches appear to be reasonably stable at present. Protection of the banks will result in stoppage of the supply of beach material from that source. However, this supply is meager. The stability of the beach in spite of this meager supply indicates that relatively little littoral movement exists. Under this condition artificial replenishment of the beach as needed will be more economical than retardation of the erosion by a groin system. It is the opinion of the Board that such replenishment should be effected by local interests when required as maintenance without Federal aid.

The shore of the estuary along the State highway is publicly owned. Its protection against erosion by waves and currents is justified by prospective benefits. The public interest involved warrants Federal participation to the extent of one-third of the total cost, in accordance with the policy stated in Public Law 727, 79th Congress.

In accordance with existing statutory requirements, the Board stated its opinion that:

a. It is advisable for the United States to adopt a project authorizing the construction of stone revetment along the State highway;

b. The public interest involved in the proposed protective measures is substantial. It is associated with the direct damages to publicly-owned shores by waves and currents and the loss of tax revenue that will be prevented;

c. The share of the expense which should be borne by the United States is one-third of the first cost of the proposed protective measures. The estimated amount of this share is $35,000.

The Board recommended that a project be adopted by the United States authorizing Federal participation by the contribution of Federal funds in an amount equal to one-third of the cost of the proposed protective measures at Colonial Beach which comprise construction of a stone revetment in sections totalling 7,350 feet in length along the State highway between Hawthorne Street and Castlewood Park, about one-half of which would be accomplished immediately and the remainder when required as a result of future erosion. Federal participation was recommended subject to the conditions that responsible local interests will: (1) adopt the plan of improvement described herein; (2) assure maintenance of the protective measures during their useful life, as may be required to serve their intended purpose; (3) provide, at their own expense, all necessary lands,
easements, and rights-of-way; (4) hold and save the United States free from all claims for damages that may arise either before, during or after prosecution of the work; (5) assure that water pollution from sources within the jurisdiction of local authorities that would endanger the health of bathers will not be permitted; (6) assure continued public ownership of the beach and its administration for public use only. The Board further recommends that the adequacy of work proposed by local authorities, detailed plans, specifications, assurances that the requirements of local cooperation will be met and arrangements for prosecuting the entire project be approved by the Chief of Engineers prior to commencement of work.

COOPERATIVE BEACH EROSION STUDIES IN PROGRESS

NEW HAMPSHIRE


Problem: To determine the best method of preventing further erosion and of stabilizing and restoring the beaches; also to determine the extent of silting and erosion in the harbor.

MASSACHUSETTS

METROPOLITAN DISTRICT BEACHES, BOSTON. Cooperating Agency: Metropolitan District Commission (for the Commonwealth of Massachusetts).

Problem: To determine the best methods of preventing further erosion, of stabilizing and improving the beaches, and of protecting the sea walls of Lynn Shore Reservation, Nahant Beach Parkway, Revere Beach, Quincy Shore, Nantasket Beach.

SALISBURY BEACH. Cooperating Agency: Department of Public Works (for the Commonwealth of Massachusetts).

Problem: To determine the best methods of preventing further beach erosion. This will be a final report to report dated 26 August 1941.

CONNECTICUT


Problem: To determine the most suitable methods of stabilizing and improving the shore line. Sections of the coast will be studied in order of priority as requested by the cooperating agency until the entire coast is included.
NEW JERSEY

OCEAN CITY. Cooperating Agency: City of Ocean City.

Problem: To determine the causes of erosion or accretion and the effect of previously constructed groins and structures, and to recommend remedial measures to prevent further erosion and to restore the beaches.

VIRGINIA

VIRGINIA BEACH. Cooperating Agency: Town of Virginia Beach.

Problem: To determine the methods for the improvement and protection of the beach and existing concrete sea wall.

SOUTH CAROLINA

STATE OF SOUTH CAROLINA. Cooperating Agency: State Highway Department.

Problem: To determine the best method of preventing erosion, stabilizing and improving the beaches.

LOUISIANA

LAKE PONTCHARTRAIN. Cooperating Agency: Board of Levee Commissioners, Orleans Levee District.

Problem: To determine the best method of effecting necessary repairs to the existing sea wall and the desirability of building an artificial beach to provide protection to the wall and also to provide additional recreational beach area.

TEXAS

GALVESTON COUNTY. Cooperating Agency: County Commissioners Court of Galveston County.

Problems: a To determine the best method of providing a permanent beach and the necessity for further protection or extending the sea wall within the area bounded by the Galveston South Jetty and Eight Mile Road.

b To determine the most practicable and economical method of preventing or retarding bank recession on the shore of Galveston Bay between April Fool Point and Kemah.

CALIFORNIA

STATE OF CALIFORNIA. Cooperating Agency: Division of Beaches and Parks, State of California.
Problem: To conduct a study of the problems of beach erosion and shore protection along the entire coast of California. The initial studies are to be made in the Ventura-Fort Hueneome area and the Santa Monica area.

WISCONSIN

RACINE COUNTY. Cooperating Agency: Racine County.

Problem: To prevent erosion by waves and currents, and to determine the most suitable methods for protection, restoration and development of beaches.

ILLINOIS

STATE OF ILLINOIS. Cooperating Agency: Department of Public Works and Buildings, Division of Waterways, State of Illinois.

Problem: To determine the best method of preventing further erosion and of protecting the Lake Michigan shore line within the Illinois boundaries.

OHIO


Problem: To determine the best method of preventing further erosion of and stabilizing existing beaches, of restoring and creating new beaches, and appropriate locations for the development of recreational facilities by the State along the Lake Erie shore line.

TERRITORY OF HAWAII

WAIKIKI BEACH. Cooperating Agency: Board of Harbor Commissioners, Territory of Hawaii.

Problem: To determine the most suitable method of preventing erosion, and of increasing the usable recreational beach area, and to determine the extent of Federal aid in effecting the desired improvement.

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BEACH EROSION LITERATURE

There are listed below some recent acquisitions to the Board's library which are considered to be of general interest. Copies of these publications can be obtained on 30-day loan by interested official agencies.


This article reports the results of experiments in echo sounding through ice conducted during Feb.-March, 1948, at Devils Lake and Buffalo Lake, Washington. The authors conclude that; (1) the experiments have demonstrated that with present portable depth recorder projectors and receiver units soundings can be successfully obtained to a depth of 80 feet when the equipment is stationary, but they are not sensitive enough to be used successfully when moving across the ice surface; (2) the greatest obstacle to overcome with moving equipment is the establishment of a proper bond between the units and the ice; (3) there is some indication that the double echoes recorded could possibly furnish a measurement of the ice thickness.


Mean sea level is the basic datum for elevations and must be carefully distinguished from half-tide level or mean tide level. A discussion of the daily, monthly, and yearly sea level variations is presented. Charts indicating monthly and yearly variations are shown for several points on the Atlantic and Pacific coasts. A method of the determination of mean sea level from a continuous series of observations over a fixed number of years and during a given epoch is analyzed.


A review of the principal projected tidal-power schemes in various parts of the world is presented. The paper gives consideration to the principles of single- and double-tide working and their application under the conditions prevailing in the Severn Estuary. The engineering features of the project are described and economic considerations are discussed. The author concludes that the future possibilities for tidal-power development are not unfavorable, and further investigation of economic factors, constructional methods and plant design are justified.

The author presents a mathematical approach, based on Holster's "Lines-of-Influence" method, to the predetermination of the tidal regime in the new proposed sea-level Panama Canal. His investigation is based on data by Meyers and Schultz, who treated the problem by means of a small-scale model. As far as could be ascertained, the agreement between experimental results and the authors' study may be regarded as a fair one. In conclusion, the author states that any important technical problem involving tidal phenomena should be submitted to both experimental and analytical treatment, and when the results of the two lines of approach agree, they may be regarded as reasonably accurate.


This paper is a review and discussion of research in sedimentary petrology in Germany during the years 1936-48 with particular emphasis to heavy mineral analysis. The author presents the methods of analysis, selection of grain sizes for analysis, and several examples of studies of the heavy mineral assemblages found in various geological horizons. Further investigation on heavy mineral resistance to transportation and the meaning of grain shape is necessary before our present knowledge of these problems can be considered satisfactory.