DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS

THE

BULLETIN

OF THE

BEACH EROSION BOARD

OFFICE, CHIEF OF ENGINEERS
WASHINGTON, D.C.

SPECIAL ISSUE NO. 2

SHORE PROTECTION PLANNING AND DESIGN

(PRELIMINARY, SUBJECT TO REVISION)

MARCH 1953

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PREFACE

This report was prepared by the Engineering Division of the Beach Erosion Board under direct supervision of J. V. Hall, Jr. and under general supervision of Colonel E. E. Gesler, President of the Board and R. O. Eaton, Chief Technical Assistant. The group assigned to preparation of the report was headed by K. P. Peel, temporarily assigned to the Board for this purpose from the Los Angeles District (now assigned to the South Pacific Division) and Kenneth Kaplan of the Board staff. Other staff personnel, in this task group were W. J. Herron, R. L. Harris, W. H. Vesper and R. H. Allen. The report was edited for publication by A. C. Rayner. Drafting was done by V. E. Dahlin and W. E. Reece.

Views and conclusions stated in the report are not necessarily those of the Beach Erosion Board. The report is being published in preliminary form at this time in order to permit review and constructive criticism by interested technicians. It is planned to publish the report in final form in about one year, thereafter to serve as a guide in the planning and design of shore protection measures.

Information and data used in preparation of the report were secured from many sources, including prior reports on generally related subjects, by the Corps of Engineers and by other Federal, State and local agencies and individuals. Information was also obtained from private corporations engaged in manufacturing materials used in shore protection structures. Published and unpublished experimental data developed at various educational institutions, the Corps of Engineers' Waterways Experiment Station, and the laboratory of the Beach Erosion Board, were drawn upon freely.

This report is published under authority of Public Law 166, 79th Congress, approved July 31, 1945.

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PART I
FUNCTIONAL PLANNING

INTRODUCTION

1. Shore protection can be provided by three general methods: (a) offshore breakwaters, (b) protective beaches, and (c) seawalls, revetments or bulkheads. Offshore breakwaters provide protection by intercepting waves before they reach the shore. Protective beaches dissipate wave energy before the waves reach an erodible upland and may be employed either alone or in conjunction with groins. Seawalls, revetments and bulkheads are used to prevent damage to upland areas by interposing a wave resisting structure approximately parallel to the shore line at a location landward of the low water line. Sand dunes are a natural form of shore protection between the beach and the upland. As such they should be preserved. They also provide a reservoir of beach material which helps reduce beach recession at times of excessive loss of beach material.

2. In selecting the best method for remedying an erosion problem or improving the beach in a specific area, all methods believed capable of producing the desired end result should be considered and compared economically. The effect of each method upon shores beyond the limits of the problem area must be evaluated, in terms of public interest if the work is planned by a governmental agency and with a view to possible liability if the works are to be privately owned.

PHYSICAL FACTORS AFFECTING FUNCTIONAL PLANNING

3. The basic physical factors which establish the criteria for functional planning, in addition to characteristics of the shore itself in the problem area, are: wind and wave action; changes in water level; and littoral drift. Though ice may be a major factor, especially on the Great Lakes, it should primarily be considered in its structural aspects. Functionally only the fact that ice may reduce wave action at certain locations need be considered.

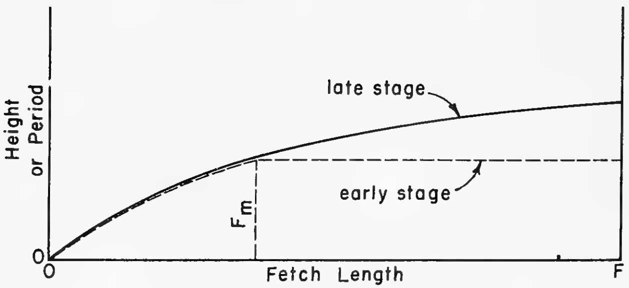
WAVE ACTION

4. Waves in Deep Water. - The vertical distance between a trough of a wave and the following crest, called the wave height (H); the time taken for identical points on two successive crests (or troughs) to pass a point, known as the period (T); and the distance between identical points on successive crests, called the wave length (L) are the parameters used to define a wave's

characteristics. A group of waves, all of the same period, will travel in deep water with a velocity known as the group velocity (C_g = 1.52T knots), while any one wave in the group will travel with a velocity C = 3.04T knots. (Deep water waves are those that are moving in depths greater than one-half their wave length).

5. Growth and Decay of Wind Waves. - An area of sea over which wind has been blowing and generating waves is known as a fetch. The growth of waves within a fetch is governed by three factors: the speed of the wind (U), the length of time this wind has been blowing, known as the duration (t), and the length of the fetch (f). In a nongrowing fetch, a constant wind speed will develop waves limited in height and period at first by the duration of the wind, then by the length of the fetch.

6. At an early stage of development of the waves, their heights or periods at various points within the fetch will be distributed as shown by the dashed line of Figure 1. At a later stage of development the distribution of wave heights or periods will be as given by the solid line of Figure 1.



WAVE DEVELOPMENT WITHIN A FETCH

FIGURE 1

The solid line represents a steady state condition. That is, at any later time, if neither the wind speed nor the fetch length changes, the same distribution of wave heights and periods will be found. A length of fetch for which a given wind speed will develop this steady-state condition regardless of how long the wind has been blowing is known as a minimum fetch (F_m). Similarly the length of time it takes a given wind blowing over a definite fetch length to cause a steady state condition is known as the minimum duration (t_m).

7. In Figure 1 the dashed line distribution shows that the portion of the fetch from 0 to F_m has reached a steady state condition. If the wind has been blowing for t hours, the minimum duration for that portion of the fetch from 0 to F_m is t hours. Conversely the minimum fetch for a minimum duration of t hours would be F_m in length. Note, in this case, the measured fetch (F) is longer than the minimum fetch (F_m) corresponding to the minimum duration t.

8. The least time taken to develop the distribution shown by the solid line, would be the minimum duration corresponding to a minimum fetch of length F. Note that the measured duration of wind may be longer than this minimum duration.

9. In actuality, the distributions shown in Figure 1 are simplified distributions, since a spectrum of wave heights and periods is generated in a fetch. These simplified distributions refer to what are known as significant waves, a statistical term which is used to describe the average of the heights and periods of the highest one-third of the waves in a group.

10. After leaving the fetch waves travel to a point some distance away--a coast for example -- with speeds proportional to their periods, (1.52 T knots, where T is in seconds). In this decay distance D (nautical miles), the longer period waves will move faster than, and consequently will arrive at the end of the decay distance before those with shorter periods. An observer at this point would see a group of waves whose significant period, called T_D, is longer than T_F, therefore the significant period of waves at the head of the fetch seems to increase.

11. The heights of waves do decrease after leaving a fetch, and at the end of the decay distance, the observed significant wave height H_D will be smaller than H_F, the significant wave height at the head of the fetch.

12. The time for the group of waves to travel over the decay distance is approximately the ratio of D to the group velocity of waves with a period T_D, and is known as the travel time t_D (t_D in hours = $D/1.52T_D$).

13. Forecasting Methods and Procedures. - The basic data to be taken from synoptic charts in making forecasts consist of two measurements of distance--decay distance and fetch length--and one of wind speed. Although waves travel along great circles, the distances to be measured are usually small enough to be adequately represented by straight lines. Therefore the

length measurements present no problem. However delineating the fetch and determining the wind speed are somewhat more complex.

14. The following facts and "rules of thumb" are used to determine the boundaries of the fetch:

a. Wind blows counter-clockwise about low pressure areas and clockwise about high pressure areas in the northern hemisphere; the reverse is true in the southern hemisphere;

b. If there is no spreading of isobars, the fetch front is located by rotating a straight edge about the forecasting area until it cuts the various isobars at a storm front at about 10 to 15 degrees;

c. Also, if there is no isobar spread, the fetch rear is located by rotating a straight edge in a similar manner till it cuts the isobars at the rear of a storm at about 45 to 60 degrees; and

d. If isobars spread apart at either the front or rear of a storm area, the fetch front or rear is located at the point of spreading. (Note: often a land mass, especially in the Great Lakes, or a meteorological front defines the fetch boundaries).

15. The charts reproduced for the typical forecast (Figures 8 through 10) show examples of all the above rules for fetch location.

16. To determine the wind speed from the individual isobar spacing in degrees latitude within the fetch, average them, and with the average value, read off the geostrophic wind speed from Figure 2. Determine from Figure 3 (or a similar chart) the sea temperature; from ship reports the air temperature; and, estimating the curvature of isobars, use Figure 4 to determine the actual wind speed, U.

17. Wind speeds may also be determined from either ship or land meteorological stations' reports if there are many such reports in the fetch area. On the Great Lakes especially, this is often the only means available for this determination.

18. Figures 5, 6 and 7 are the forecasting and decay curves. To apply them, the following simplified step by step procedure may be used:

a. Delineate a fetch, measure and tabulate the actual wind velocity U_1, the fetch length F_1, the decay distance D_1, and the estimated duration t_1 of the wind. (When a fetch first appears on a synoptic chart, the duration time may be taken as one-half the time Z between this chart and the one immediately preceding).

b. Enter Figure 5 with the wind velocity U_1 and follow U_1 across to its intersection with the fetch length F_1 or the duration t_1, whichever comes first. At that point read off the period T_F and the wave height H_F at the head of the fetch.

GEOSTROPHIC WIND SCALE

$$V_g = \frac{1}{2\Omega\rho\sin\phi}\frac{\Delta p}{\Delta n}$$

FOR Δp = 5mb & 3mb
Δn = Degrees Latitude
p = 1013.3 mb
T = 10° C
ρ = 1.26 gm.cm.$^{-3}$

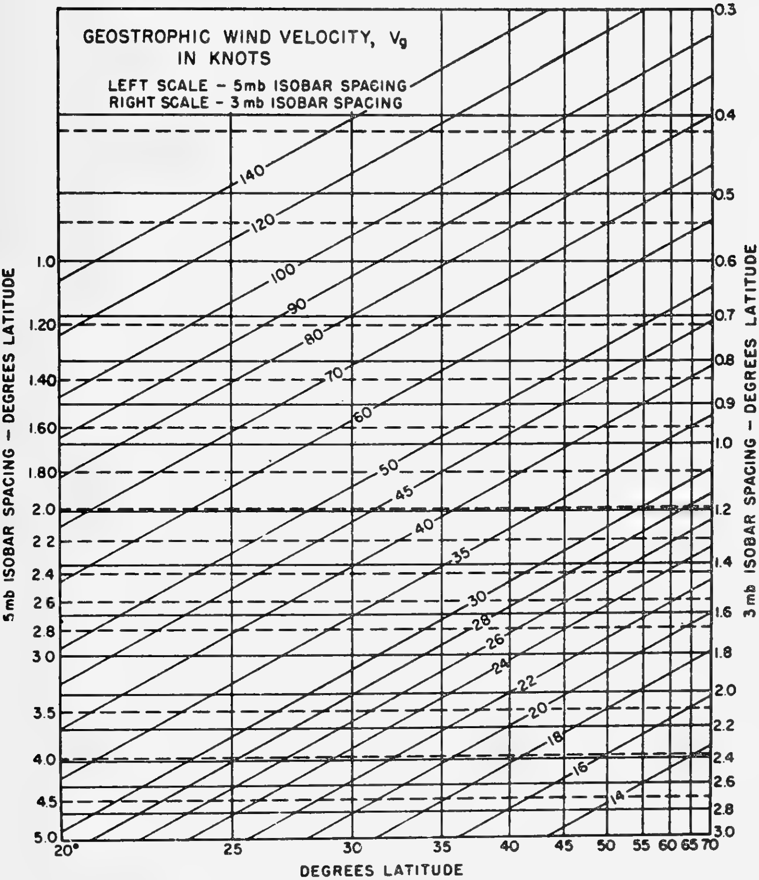


FIGURE 2

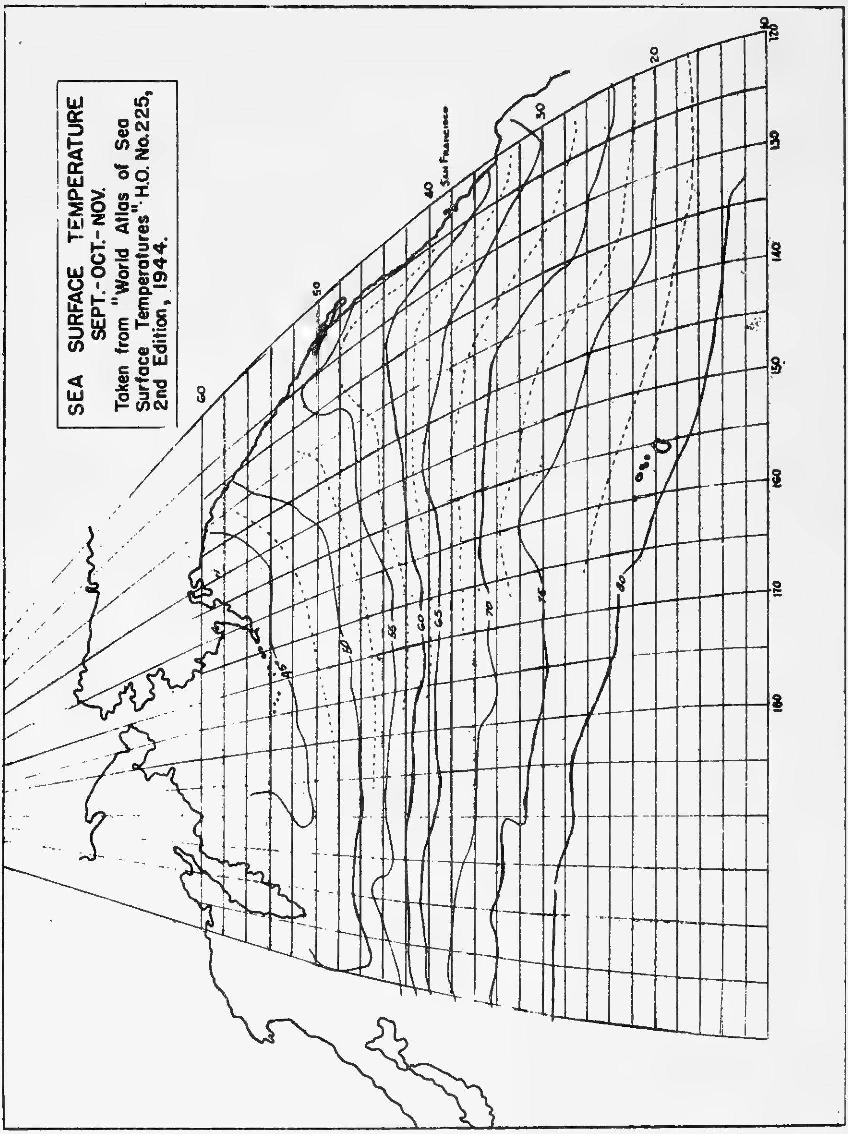
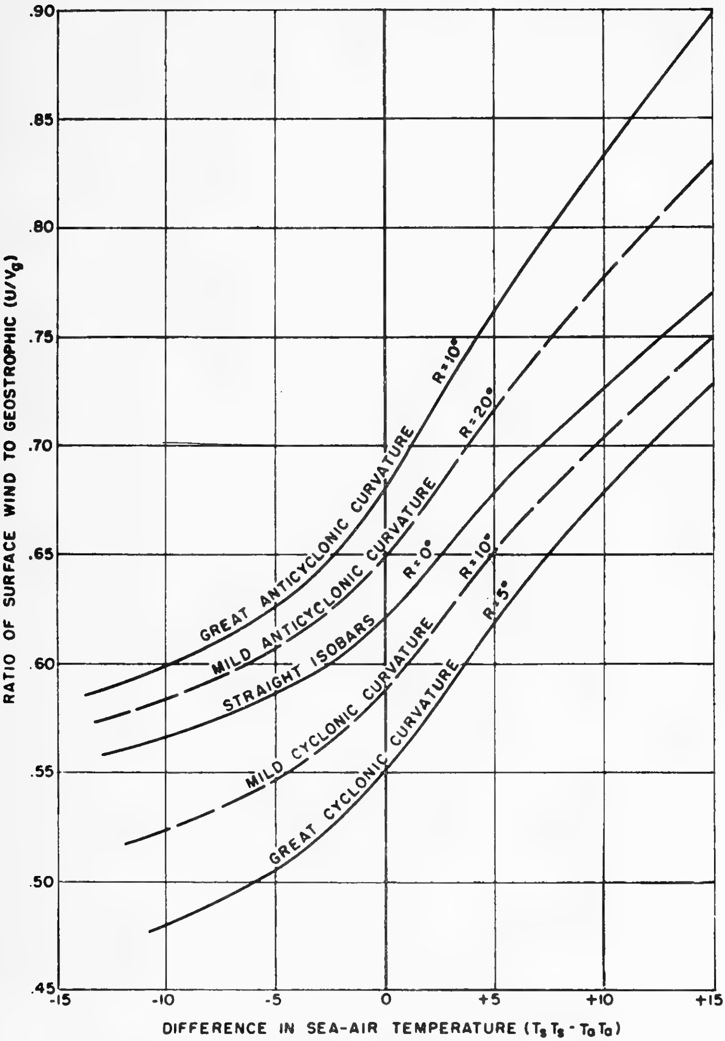

SEA SURFACE TEMPERATURE
SEPT.-OCT.-NOV.
Taken from "World Atlas of Sea Surface Temperatures", H.O. No. 225, 2nd Edition, 1944.
San Francisco

FIGURE 3



SURFACE WIND SCALE

FIGURE 4

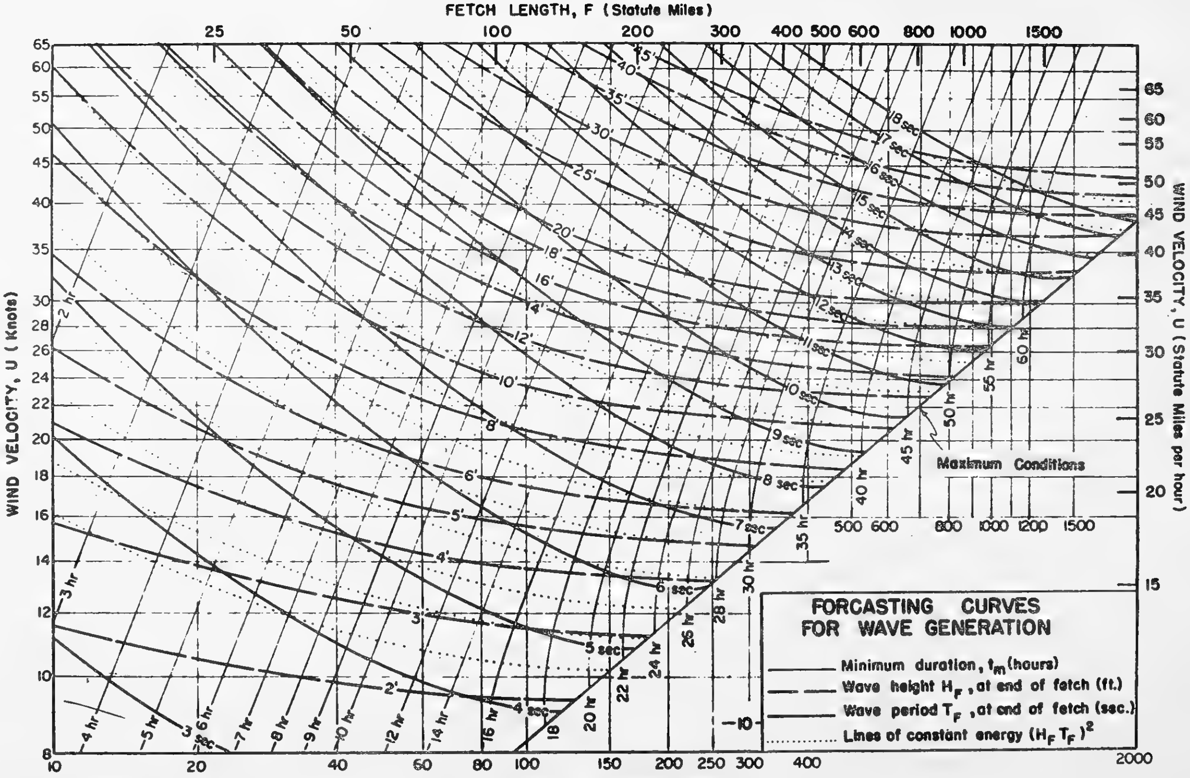

FETCH LENGTH, F (Statute Miles)
FETCH LENGTH, F (Nautical Miles)
WIND VELOCITY, U (Knots)
WIND VELOCITY, U (Statute Miles per hour)
Maximum Conditions
FORCASTING CURVES
FOR WAVE GENERATION
Minimum duration, t_m (hours)
Wave height H_F, at end of fetch (ft.)
Wave period T_F, at end of fetch (sec.)
Lines of constant energy $(H_F T_F)^2$

FIGURE 5

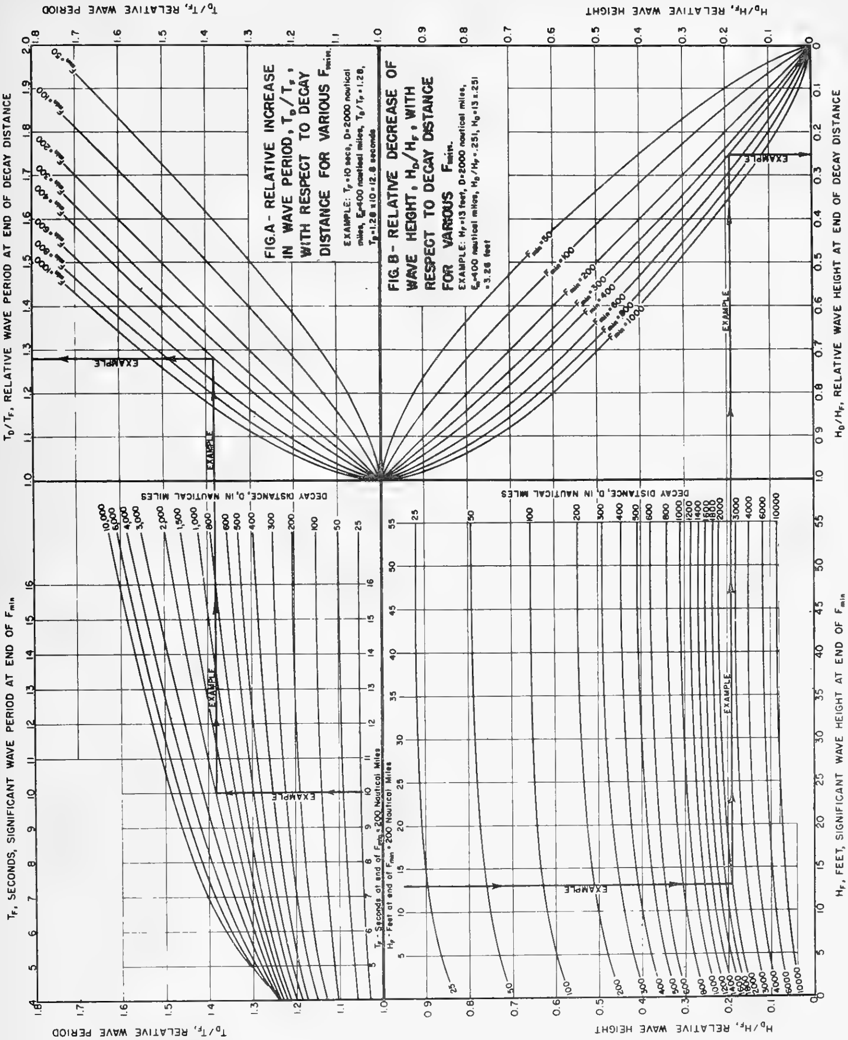
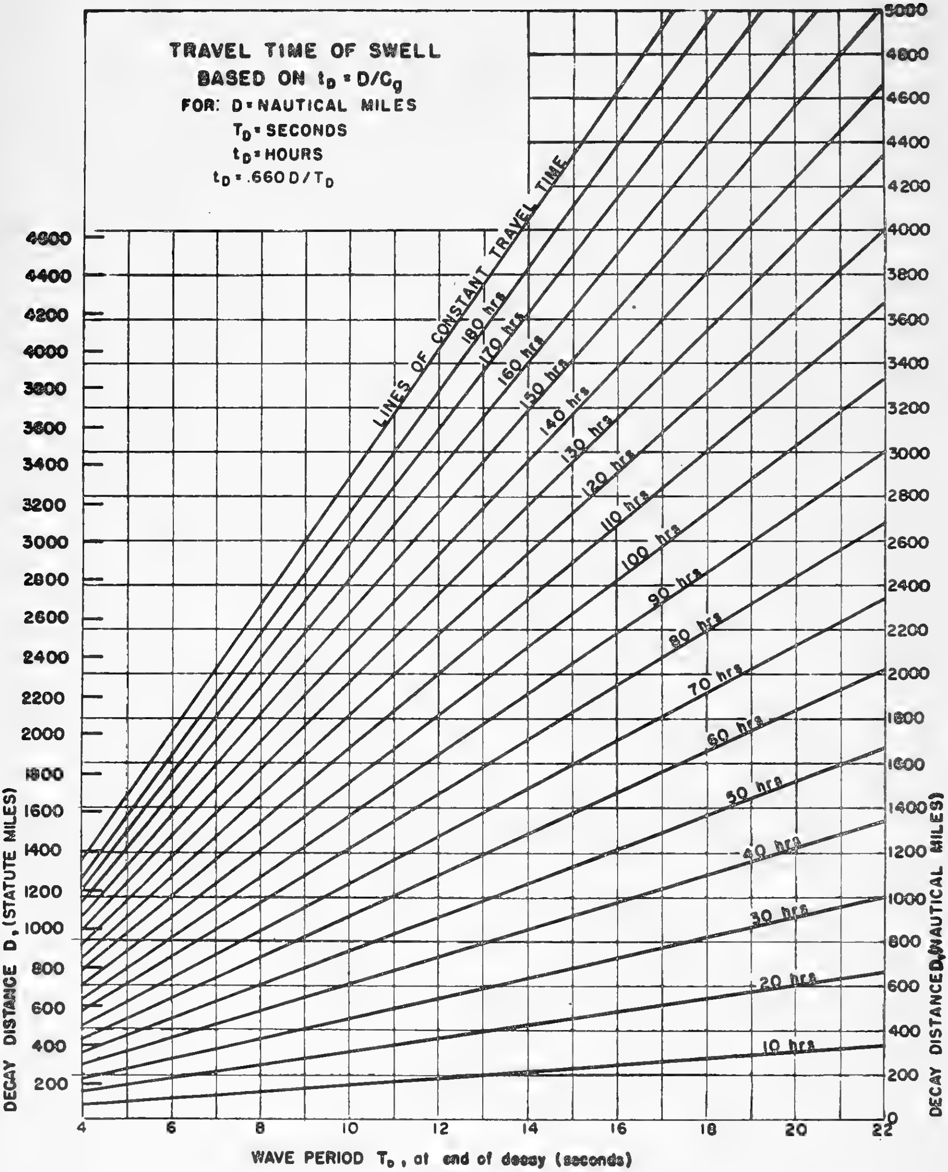


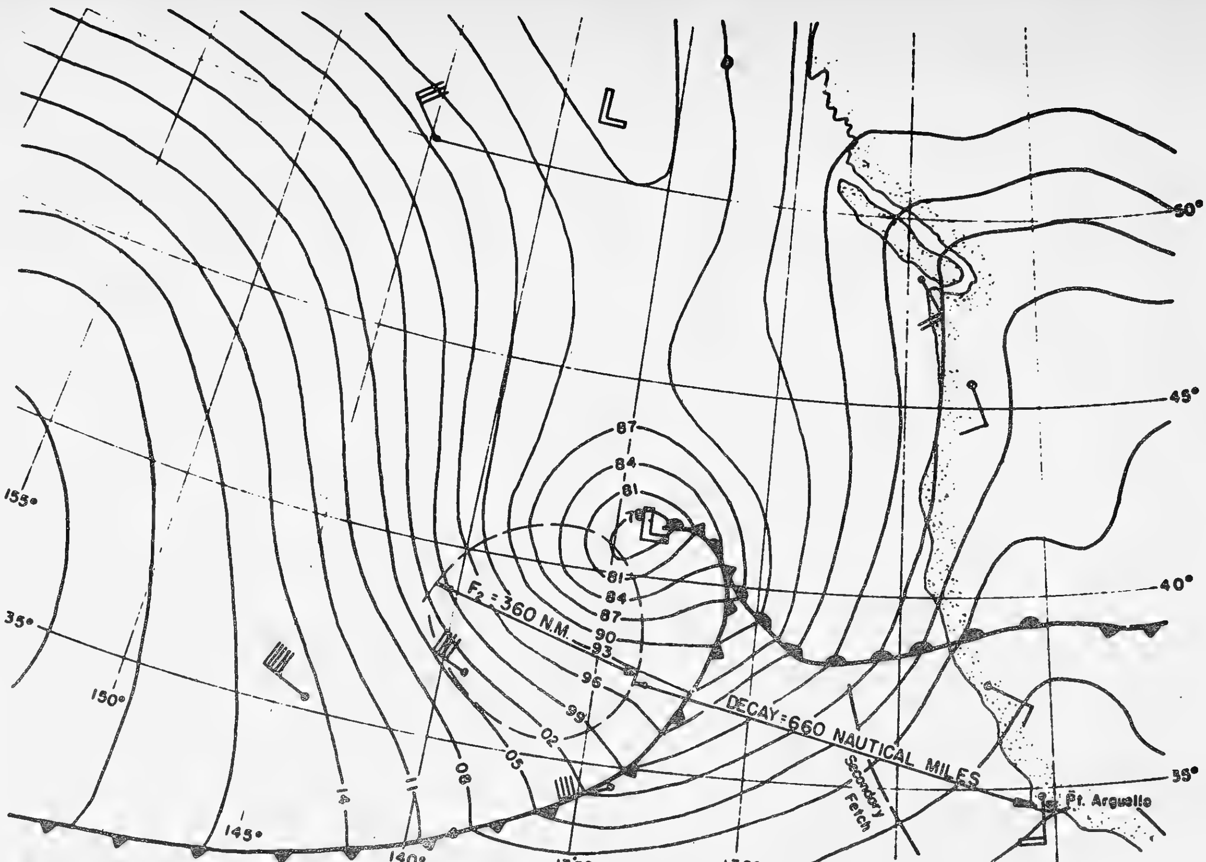
FIG. A - RELATIVE INCREASE IN WAVE PERIOD, T_D/T_F, WITH RESPECT TO DECAY DISTANCE FOR VARIOUS F_{min}.
EXAMPLE: T_F=10 secs, D=2000 nautical miles, F_{min}=400 nautical miles, T_D/T_F=1.28, T_D=1.28 x 10=12.8 seconds
FIG. B - RELATIVE DECREASE OF WAVE HEIGHT, H_D/H_F, WITH RESPECT TO DECAY DISTANCE FOR VARIOUS F_{min}.
EXAMPLE: H_F=13 feet, D=2000 nautical miles, F_{min}=400 nautical miles, H_D/H_F=.251, H_D=13 x .251 =3.26 feet
T_D/T_F, RELATIVE WAVE PERIOD AT END OF DECAY DISTANCE
H_D/H_F, RELATIVE WAVE HEIGHT AT END OF DECAY DISTANCE
T_F, SECONDS, SIGNIFICANT WAVE PERIOD AT END OF F_{min}
H_F, FEET, SIGNIFICANT WAVE HEIGHT AT END OF F_{min}
DECAY DISTANCE, D, IN NAUTICAL MILES
T_D/T_F, RELATIVE WAVE PERIOD
H_D/H_F, RELATIVE WAVE HEIGHT
T_F - Seconds at end of F_{min} = 200 Nautical Miles
H_F - Feet at end of F_{min} = 200 Nautical Miles
EXAMPLE

FIGURE 6

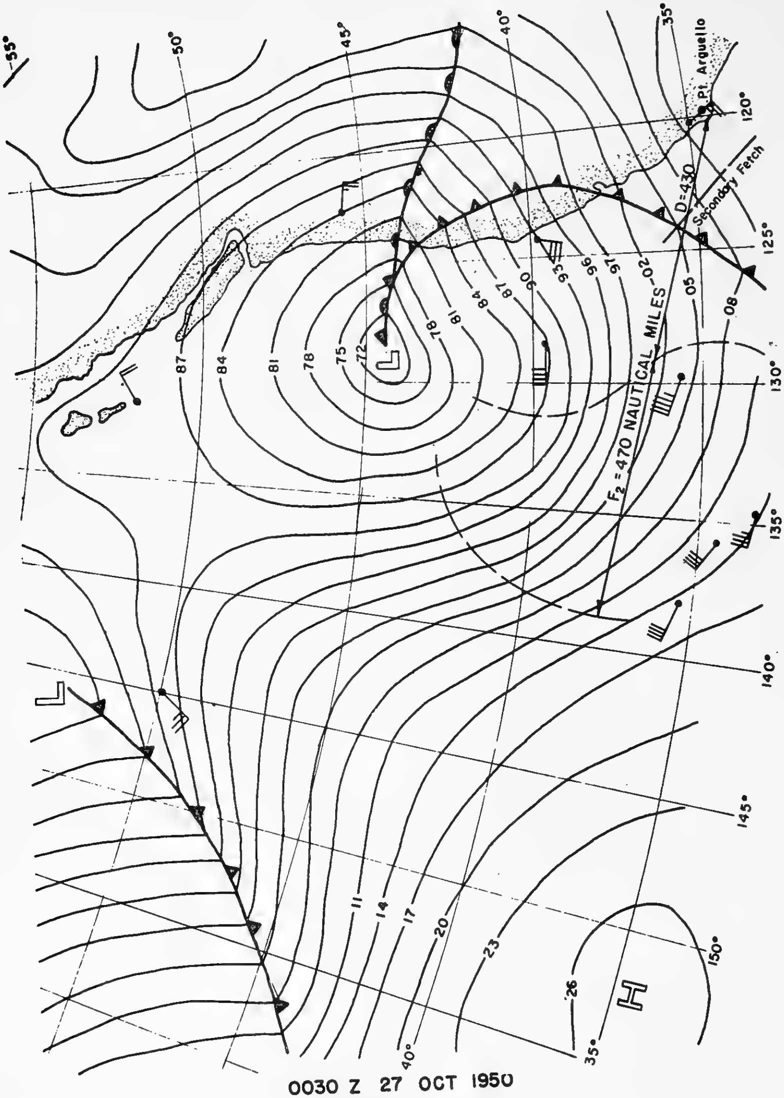


TRAVEL TIME OF SWELL
BASED ON $t_D = D/C_g$
FOR: D = NAUTICAL MILES
T_D = SECONDS
t_D = HOURS
$t_D = .660\,D/T_D$
LINES OF CONSTANT TRAVEL TIME
180 hrs
170 hrs
160 hrs
150 hrs
140 hrs
130 hrs
120 hrs
110 hrs
100 hrs
90 hrs
80 hrs
70 hrs
60 hrs
50 hrs
40 hrs
30 hrs
20 hrs
10 hrs
DECAY DISTANCE D, (STATUTE MILES)
DECAY DISTANCE D, (NAUTICAL MILES)
WAVE PERIOD T_D, at end of decay (seconds)

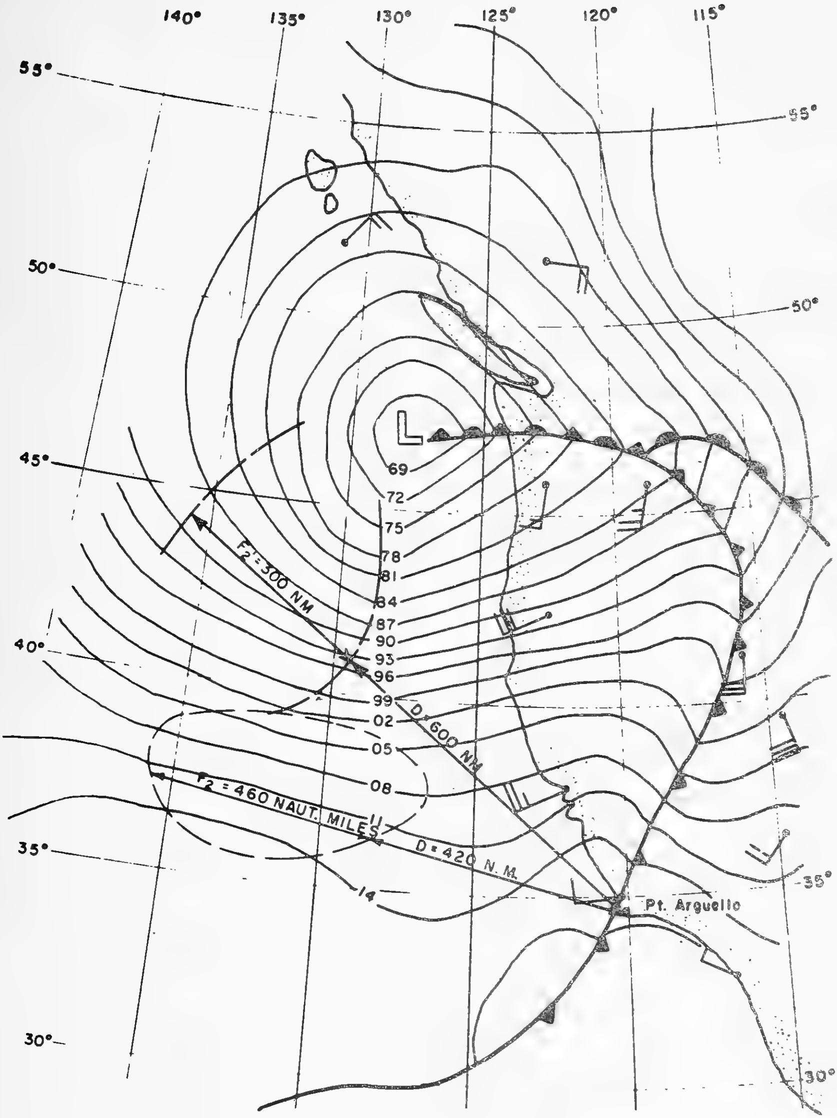
FIGURE 7



1230 Z 26 OCT 1950
FIGURE 8



0030 Z 27 OCT 1950
FIGURE 9



0630Z 27 OCT 1950

FIGURE 10

c. With these values of T_F and H_F and the tabulated value of the decay distance D_1, from Figure 6 find T_D/T_F and H_D/H_F and calculate the period T_D and the height H_D at the end of the decay.

d. From Figure 7, with T_D and D_1 find the travel time to the area under consideration.

The values of period and height T_D and H_D from (c) above are the significant deep water period and height respectively, for waves from this fetch being analyzed.

19. On a later chart, drawn for a time Z after the first chart for the fetch corresponding to the one chosen previously:

a. Measure and tabulate U_2, F_2 and D_2

b. Enter Figure 5 with the "U_1" from the preceding chart and move along the "U" line to its intersection with the $t_1 + Z/2$ duration line or the duration corresponding to $F_1 + Z/2$, whichever comes first.

c. From this point follow one of the dotted lines plotted or interpolated, to either

(1) the wind velocity line U_2 or

(2) the fetch line F_2 whichever comes first. If F_2 comes first, drop down along the F_2 line, to its intersection with U_2. If U_2 comes first, move along U_2 to its intersection either with duration line $t_1 + Z$ (or the duration corresponding to $F_1 + Z$) or F_2 whichever comes first.

(3) If the dotted line does not meet U_2 or F_2, follow it to the diagonal line labelled "maximum conditions" at the right of the graph and follow this line to its intersection with U_2.

d. At any of these final points along U_2, read off T_F and H_F.

e. and f. Same as steps c and d, paragraph 18.

20. To illustrate the preceding steps, the following short forecast is made. Synoptic conditions are shown on Figures 8, 9, and 10.

Table 1 - Forecast for synoptic charts of 26 October 1230Z 27 October 0030Z and 0630Z. (Z indicates Greenwich Mean Time, 8 hours earlier than Pacific Standard Time)

Chart date	26 Oct	27 Oct	27 Oct
Chart time	1230Z	0030Z	0630Z
Fetch No.	2	2	2
Isobar spacing (degrees latitude) (degrees longitude)	0.65°	0.70°	1.0°
Latitude (degrees)	38°	37°	37°
Geostrophic wind speed V_g (knots)	70(1)	67(1)	47(1)
Longitude (degrees)	137°	133°	132°
Sea temperature Θ_s (degrees F)	66°(2)	65°(2)	64°(2)
Air temperature Θ_a (degrees F)	65°	65°	64°
Difference, sea-air temperature (degrees F)	+1°	0	0
Isobar curvature	Mild Great Cyclonic	Mild Cyclonic	Straight
U/V_g	0.59(3)	0.59(3)	0.62(3)
Actual wind velocity U (knots)	41	40	29
Fetch length (naut. mile)	360	470	460
Duration t (hours)	6 hrs(4)		
Decay distance (naut. mile)	660	430	420(11)
T_F (seconds)	8.9(5)	12.9(8)	12.7(11)
H_F (feet)	17(5)	28(8)	22(11)
Min. duration t_m (hours)	6	19(8)	33
Min. fetch	58	250	460
T_D/T_F	1.48(6)	1.38(9)	1.18
H_D/H_F	0.22(6)	0.47(9)	0.52(9)
T_D (T_o) (seconds)	13 sec	16.5 sec	15 sec
H_D (H_o) (feet)	4 ft	13.0 ft	11 ft

Table 1 - Continued

Travel time (hours)	33[7]	18[10]	20[10]
Arrival time	27 Oct 2130Z	27 Oct 1830Z	28 Oct 0330Z
	27 Oct (1230 PST)	27 Oct (1030 PST)	27 Oct (1830 PST)

(1) From Figure 2
(2) From Figure 3
(3) From Figure 4
(4) Estimated
(5) From Figure 5 with U = 41 knots and t_{min} = 6 hrs
(6) From Figure 6 with T_F = -8.9 sec and D = 660 nautical miles
(7) From Figure 7 with T_D = 10 sec and D = 660 nautical miles
(8) From Figure 5 in the following manner. Follow the line U = 41 knots to the duration line of t + Z/2 = 6 + 6 = 12 hours; follow an imaginary dotted line from that point to its intersection with the U = 40 knots line which occurs at a point where the duration = 13 hours. Add Z/2 to this duration (13 + 6 = 19), and follow along U = 40 knots to the point where t_m = 19 hours. Read off T_F and H_F.
(9) From Figure 6
(10) From Figure 7
(11) From Figure 5 in the following manner. Follow the line U = 40 knots to the duration line of t_m + Z/2 = 19 + 3 = 22; follow an imaginary dotted line from that point to its intersection with the fetch line F = 420 miles (which occurs at a point where the duration = 33 hours) and drop down along this line to its intersection with the wind velocity line of 29 knots. Read off T_F and H_F.

---

21. Synoptic weather charts have been drawn and compiled for many years by the United States Weather Bureau for Northern Hemisphere regions. By applying the preceding forecasting methods to these charts, statistical studies may be made for any coastal locality by "hindcasting" the characteristics of waves that occurred in deep water. From these statistical studies the predominant deep-water wave directions, heights, and lengths can be determined. Wave characteristics in transitional and shallow water can then be determined by construction of refraction diagrams.

22. Waves in Transitional and Shallow Water. - In moving from deep water toward a coast, a wave's characteristics change. A wave travelling shoreward over depths of less than one-half its deep water wave length moves more slowly than it would in deeper water, its length shortens and its height is changed. If the wave is approaching a coast at an angle to the underwater contours, the part of the wave which first passes over a depth less than half its deep-water wave length will move more slowly than the rest of the wave. This will cause the wave crest to bend toward alinement

with the underwater contours. These changes in wave characteristics are known as wave refraction. (See Figure 11).

23. The velocity of a wave C in any depth of water d is given by

$$C = \sqrt{\frac{gL}{2\pi} \tanh \frac{2\pi d}{L}} \tag{1}$$

In deep water, $2\pi d/L$ becomes very large, the hyperbolic tangent of $2\pi d/L$ approaches unity, and the wave velocity approaches:

$$C = \sqrt{\frac{gL}{2\pi}} \tag{2}$$

In shallow water, $2\pi d/L$ becomes very small, $\tanh 2\pi d/L$ approaches $2\pi d/L$ and the wave velocity approaches:

$$C = \sqrt{gd} \tag{3}$$

Those depths over which equation (1) applies unchanged are known as transitional water.

24. The total energy per unit crest width in a wave is

$$E_T = (1/8)\rho\, gH^2L \tag{4}$$

where ρ is the water density and $\rho g = w$, the weight of water per cubic foot. Of this, only a part is transmitted forward with the wave form. This transmitted part may be determined as

$$E = n\, E_t \tag{5}$$

where

$$n = (1/2)\left[1 + \frac{\frac{4\pi d}{L}}{\sinh 4\pi\, d/L}\right] \tag{6}$$

In deep water $\sinh 4\pi d/L$ becomes very large and the expression for n becomes $n_o = 1/2$. (The wave length in deep water L_o may be found from equation (2) by substituting the general relationship $L = CT$, and in feet is given by $L_o = 5.12T^2$. Table 1 of Appendix D lists values of $2\pi d/L$, $4\pi d/L$, $\tanh 2\pi d/L$ and $\sinh 4\pi d/L$ as functions of the ratio d/L_o).

25. Refraction of Waves. - When wave refraction is considered, it is assumed that in a wave advancing toward shore, no energy flows laterally along a wave crest. That is, between two lines drawn perpendicular to a wave crest as it passes over changing topography, the transmitted energy remains constant. The wave energy transmitted forward between any two such lines in deep water is $E_o = 1/2\ b_o E_{T_o}$, where b_o is the spacing of the two lines in deep water. (The subscript o always refers to deep water conditions). It may therefore be equated to the energy transmitted forward between the same two lines in shallow water ($E = n\,b\,E_t$), where b is the

WAVE REFRACTION AT WEST HAMPTON BEACH L. I., N.Y. DECEMBER, 1938
FIGURE 11

spacing between the two lines in shallow water. Therefore $1/2\ b_o\ E_{T_o} = n\ b\ E_T$ or

$$\frac{E_T}{E_{T_o}} = \frac{1}{2} \times \frac{1}{n} \times \frac{b_o}{b} \qquad (7)$$

From equation (4) $H/H_o = \sqrt{\dfrac{E_T}{E_{T_o}} \times \dfrac{L_o}{L}}$, therefore equation (7) may be

written

$$H/H_o = \sqrt{\frac{1}{2} \times \frac{1}{n} \times \frac{L_o}{L}} \quad (\sqrt{b_o/b}\ )$$

The term $\sqrt{1/2 \times 1/n \times L_o/L}$ is known as the shoaling coefficient (H/H_o'). It is a function of d/L_o and may be found in Table 1 of Appendix D.

26. Equation (8) shows that wave heights in transitional or shallow water may be found, knowing deep water wave heights, if the relative spacing between lines drawn perpendicular to wave crests can be determined. The square root of this relative spacing (b_o/b) is known as the refraction coefficient. It should also be noted that these perpendicular lines, when constructed, will show the direction of movement of the waves to which they are drawn perpendicular.

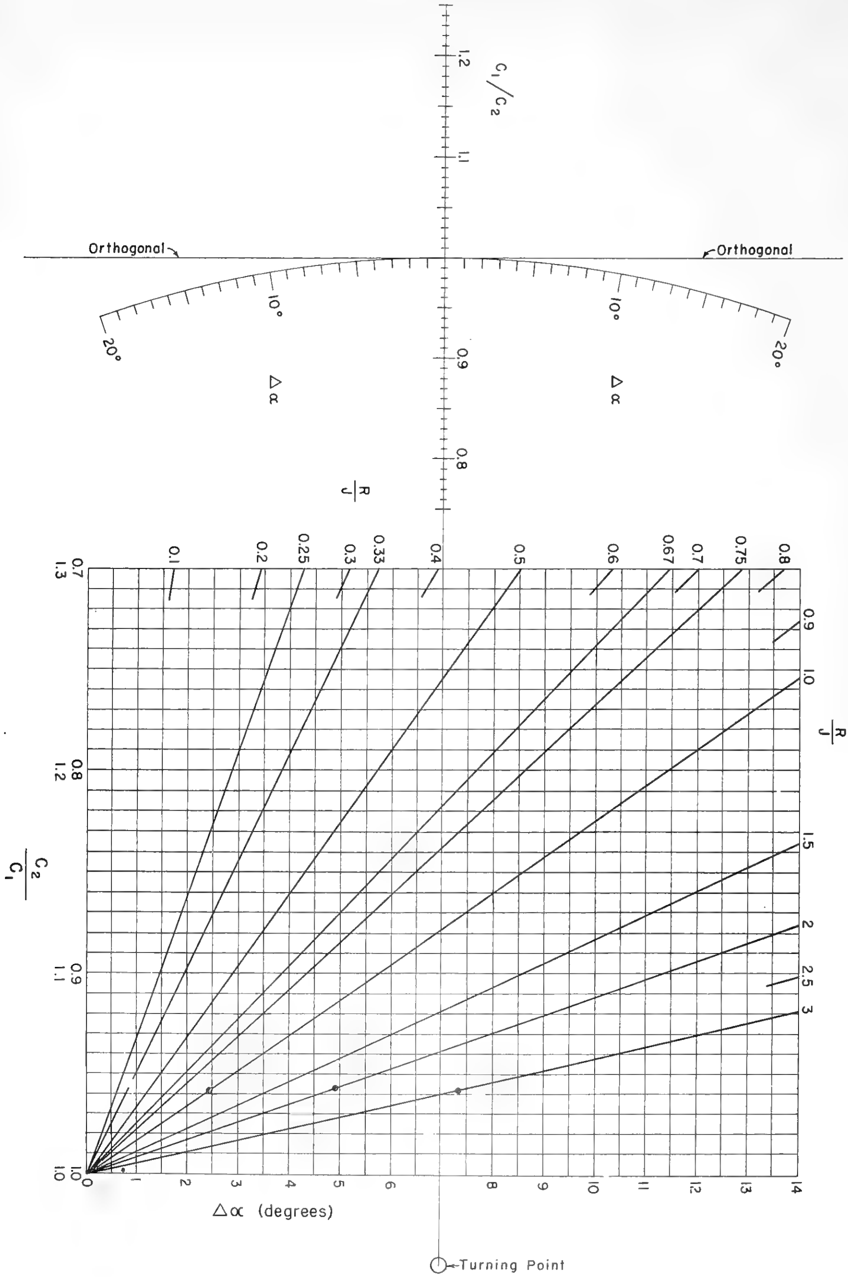
27. The lines drawn perpendicular to the wave crests are known as <u>orthogonals.</u> Various methods have been proposed for constructing these lines. The earlier approaches required drawing positions of wave crests, then erecting perpendiculars to them; later approaches eliminate the intermediate wave crest step, permitting the immediate construction of orthogonals themselves.

28. It can be shown (Appendix E) that the change of direction of an orthogonal as it passes over relatively simple underwater topography is

$$\sin\alpha_2 = C_2/C_1\ \sin\alpha_1 \qquad (9)$$

where α_1 is the angle a normal to an orthogonal makes with a contour the orthogonal is passing over,
- α_2 is a similar angle measured as the orthogonal passes over the next contour
- C_1 is the wave velocity (equation 1) at the depth of the first contour, and
- C_2 is the wave velocity at the depth of the second contour.

From this equation, a template may be constructed with which the angular change in α as an orthogonal passes over a definite contour interval may be found, and the changed-direction-orthogonal may be constructed. (see Appendix E). Such a template is shown on Figure 12.



C_1/C_2
1.2
1.1
0.9
0.8
Orthogonal
Orthogonal
10°
20°
10°
20°
$\Delta\alpha$
$\Delta\alpha$
$\frac{R}{J}$
0.1
0.2
0.25
0.3
0.33
0.4
0.5
0.6
0.67
0.7
0.75
0.8
0.9
1.0
1.5
2
2.5
3
$\frac{R}{J}$
$\frac{C_2}{C_1}$
0.7 1.3
0.8 1.2
0.9 1.1
1.0 1.0
0
1
2
3
4
5
6
8
9
10
11
12
13
14
$\Delta\alpha$ (degrees)
Turning Point

FIGURE 12

29. Procedures in Refraction Diagram Construction. - For a chosen shore location hydrographic charts showing bottom topography of the area are obtained. Two or more charts may be necessary, of differing scale, but the procedures are identical for charts of any scale. Next, underwater contours are drawn on the chart, or on a tracing paper overlay, for various depths, depending on the diagram accuracy desired. If overlays are used (this will preserve the charts) the shore line should be traced in for reference. In tracing the contours, judgment must be used in "smoothing out" small irregularities, since bottom features which are comparatively small in respect to the wave length do not affect the wave appreciably.

30. The range of wave periods to be used is determined by a hindcasting study of historical weather charts or from other historical data relating to wave periods. With the wave period so determined, C_1/C_2 values for each contour interval should be marked between the contours. The method of computing C_1/C_2 is illustrated by Table 2. A tabulation of C_1/C_2 and C_2/C_1, for various contour intervals and wave periods is given in Table 8 of Appendix D.

Table 2 - Computations for values of C_1/C_2, T = 10 sec.

d	d	$\frac{d}{L_o}$	$\tanh\frac{2\pi d}{L}$	$5.12\ T^2$	C	C_1/C_2 or	C_2/C_1
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(7)
(fathom)	feet						
1	6	0.0117	0.26	512	133		
						1.42	0.704
2	12	0.0235	0.37	512	189		
						1.22	0.82
3	18	0.0352	0.45	512	230		
						1.16	0.87
4	24	0.0469	0.52	512	266		

Columns 1 and 2 are depths corresponding to each chart contour. These would extend from 1 fathom to a depth equal to $L_o/2$.

Column 3 is column 2 divided by L_o corresponding to the determined period.

Column 4; these values may be found in Table 1 of Appendix D, as a function of d/L_o.

Column 6 is the product of columns 4 and 5.

Column 7 is the quotient of successive terms in column 6...

C_1/C_2 should be calculated for orthogonal construction from deep to shallow water.

C_2/C_1 should be calculated for orthogonal construction from shallow to deep water.

31. To construct an orthogonal from deep to shallow water, a deep water direction of wave approach is first selected by a hindcasting study of historical weather charts, by fan diagram analysis (see paragraph 38) or by direct observation. A deep water wave front (crest) is drawn as a straight line perpendicular to this wave direction and suitably spaced parallel lines (orthogonals) are drawn from this wave front in the chosen direction of wave approach. These lines are extended to the first depth contour shoaler than $1/2\ L_o$ where $L_o = 5.12T^2$.

32. Procedures for α less than 80°. - Starting with any one orthogonal, the following steps should be taken:

(a) Sketch in a mid-contour between the first two contours to be crossed, extend the orthogonal to this mid-contour, and construct a tangent to the mid-contour at this point;

(b) Lay the line labelled "orthogonal" along the incoming orthogonal with the point marked 1.0 at the intersection of orthogonal and mid-contour. (see Figure 13a);

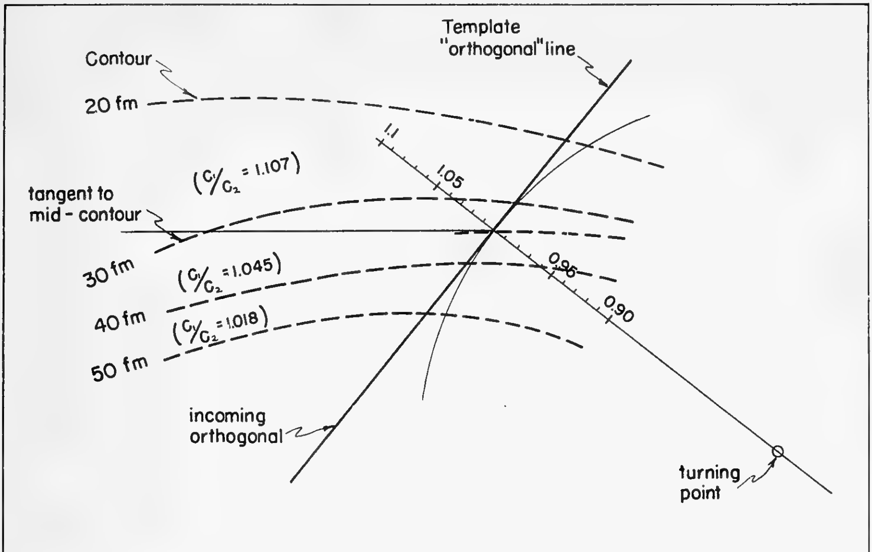
(c) Rotate the template about the "turning point" until the C_1/C_2 values corresponding to the contour interval being crossed intersects the tangent to the mid-contour. The "orthogonal" line now lies in the direction of the turned orthogonal. (See Figure 13b);

(d) Place a triangle along the base of the plotter and erect a perpendicular to it so that the intersection of this perpendicular with the incoming orthogonal is equidistant from the two contours measured along the incoming orthogonal and this perpendicular. This line represents the turned orthogonal;

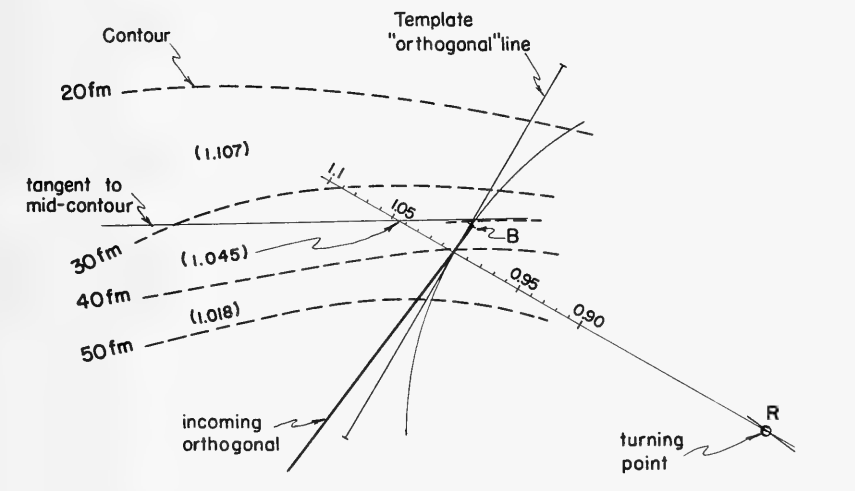
(e) Repeat the process for the next contour intervals.

33. If the orthogonal is being constructed from shallow to deep water, the same procedure is followed, except that C_2/C_1 values are used instead of C_1/C_2.

34. Procedures for α greater than 80°. - In any depth of water, when α becomes greater than 80°, the above procedures may not be used. The orthogonal no longer appears to cross the contours, but tends to run parallel between them. In this case the contour interval is crossed in a series of steps. In essence, the whole interval is divided into a series of smaller intervals, at the mid-points of which, orthogonal angle turnings are made.



13 a



Note: The template has been turned about R until the value $C_1/C_2 = 1.045$ intersects the tangent to the mid-contour. The template "orthogonal" line lies in the direction of the turned orthogonal. This direction is to be laid off at some point "B" on the incoming orthogonal which is equidistant from the two contours along the incoming and outgoing orthogonals.

13 b

USE OF THE REFRACTION TEMPLATE

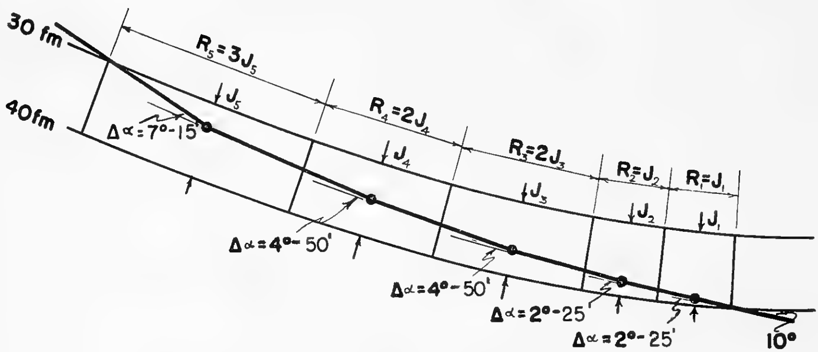
FIGURE 13

35. Referring to Figure 14, an interval to be crossed is divided into segments or boxes, by transverse lines. The spacing, R, of the lines is arbitrarily set as a ratio of the distance, J, between the contours. For the complete interval to be crossed, C_2/C_1 is computed or found from Table 8 of Appendix D (Note: C_2/C_1 not C_1/C_2).

36. On the template (Figure 12) is a graph showing orthogonal angle turnings ($\Delta\alpha$) at the center of a box, plotted as a function of the C_2/C_1 value of any contour interval for various values of the R/J ratio, which may be chosen.

37. The orthogonal is brought into the middle of the box, $\Delta\alpha$ is read from the graph, and the orthogonal turned by that angle. The procedure is repeated for every box until α at a plotted or interpolated contour becomes less than 80°. At this point this method of orthogonal construction must be stopped or error will result. The dots on the graph of Figure 12 are those used to determine values for the example on Figure 14.

J = Distance between contours at turning points, ●
R = Distance along orthogonal
T = 12 seconds
L_o = 737 ft.
For contour interval from 40 fm to 30 fm C_1/C_2 = 1.045, C_2/C_1 = 0.957



REFRACTION DIAGRAM USING $\frac{R}{J}$ METHOD

FIGURE 14

38. Refraction Fan Diagrams. - It is often convenient, especially where a coastal area is shielded by land features from waves approaching in certain directions, to construct refraction diagrams from shallow toward deep water. In such cases, a sheaf or fan of orthogonals may be projected seaward in direction some 5 or 10 degrees apart. (See Figure 15a). With the deep water directions determined by the individual orthogonals, companion orthogonals may be projected shoreward on either side of the seaward projected ones in order to determine the refraction coefficient for the various directions of wave approach. (See Figure 15b).

39. Refraction Diagram Limitations. - In many cases refraction diagrams provide a reasonably accurate measure of the changes waves undergo on approaching a coast. Quite often they provide the only measure of these changes available. However, the data determined from refraction diagrams are only as valid as the theory of their construction is accurate. The orthogonal direction change equation (9) is derived for the simplest case of straight parallel contours, and although little error is introduced by bringing orthogonals over relatively simple hydrography, it is difficult to carry an orthogonal accurately into shore over complex bottom features. Moreover, the equation is derived for small waves moving over relatively flat slopes. Although no strict limits have been set, strict accuracy cannot be expected where bottom slopes are steeper than 1 on 10. A third limitation is inherent in the assumption that no energy travels laterally along a wave crest. No strict limits have been set, but the accuracy of wave heights derived from orthogonals which bend sharply, is questionable.

40. Significant and Higher Waves. - It was noted in paragraph 9 that the wave height determined from forecasting or hindcasting procedures is the so-called significant wave height, the average of the one-third higher heights of a given wave group. If this wave height is used as H_o in determining the inshore wave height H through refraction analysis, H will also be the significant wave height in the transitional and shallow water zones.

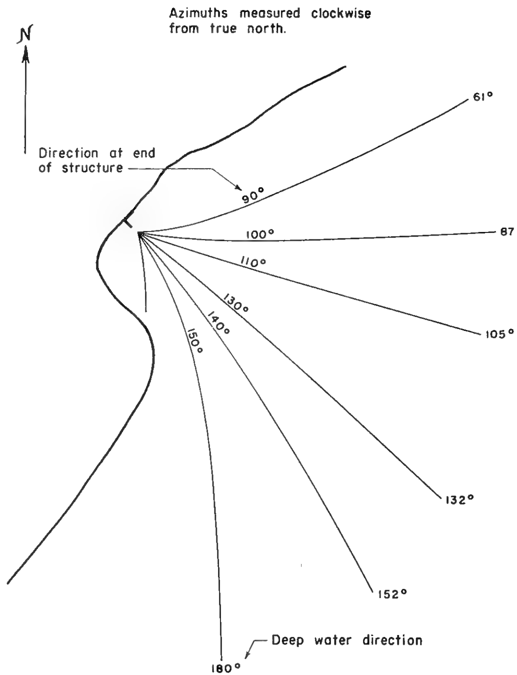
41. It has been found that a linear relationship exists between the significant wave height and the mean wave height of a group. This relationship is

$$H_m = 0.624\, H_s - 0.015 \tag{10}$$

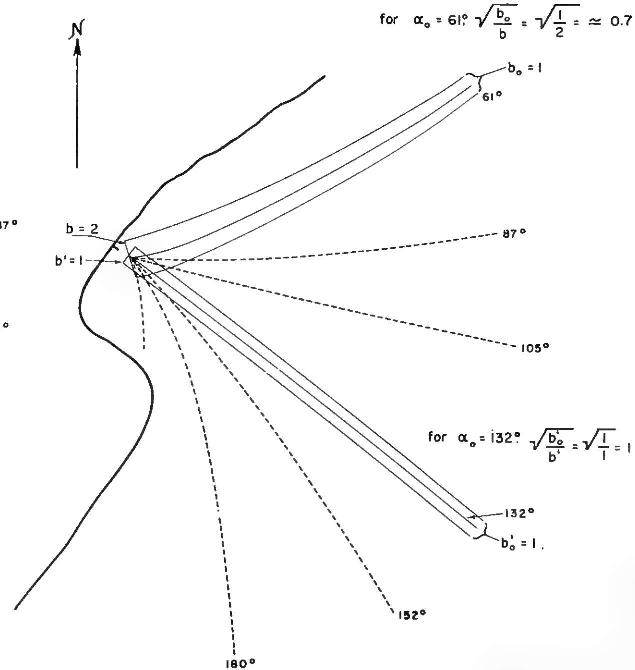
where H_s = the significant wave height of a wave group (in feet) and

H_m = the mean wave height of the group (in feet)

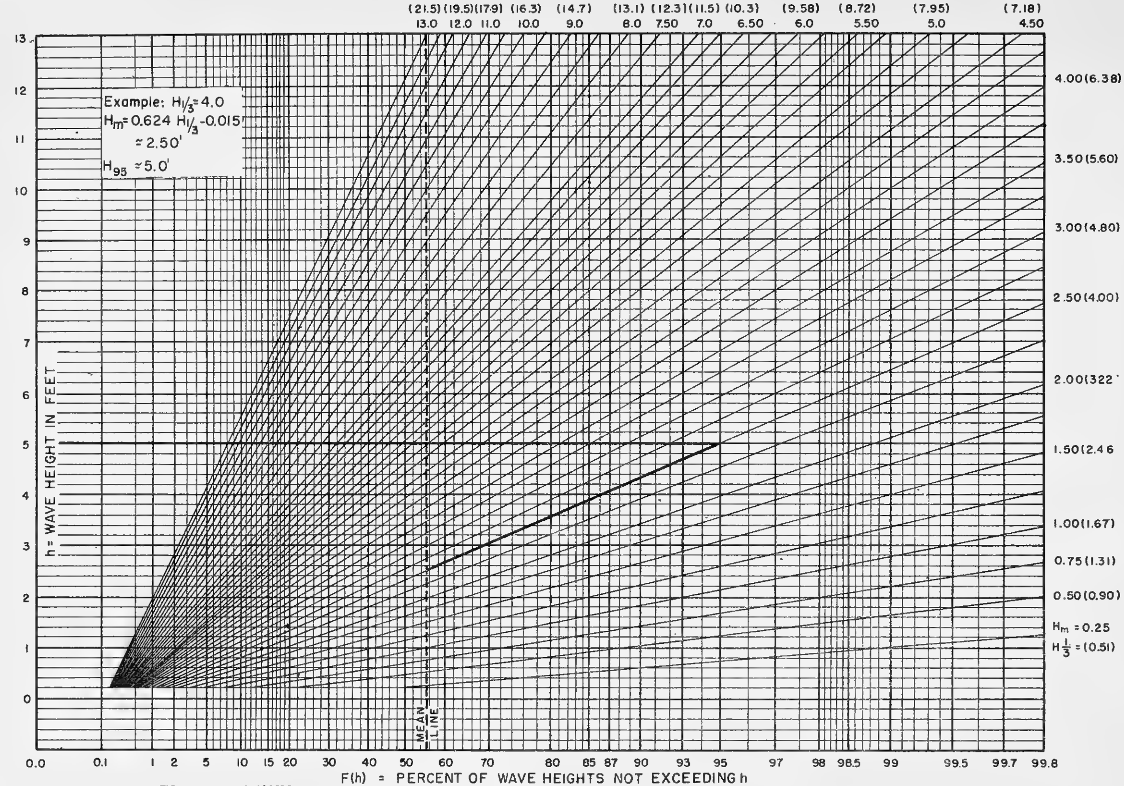
It was further found that this mean wave height could be related statistically to any other height which may occur more or less frequently. This relationship is represented graphically on Figure 16. The graph can be used to find a wave height more nearly the maximum of a group of waves, say one which will not be exceeded by 95 percent of the waves.



FAN — TYPE REFRACTION DIAGRAM
FIGURE 15 (a)



COMPANION ORTHOGONALS FOR DETERMINING
REFRACTION COEFFICIENTS
FIGURE 15 (b)



PREDICTION LINES FOR WAVE HEIGHT DISTRIBUTION FUNCTIONS FOR VARIOUS MEAN WAVE HEIGHTS

(SIGNIFICANT WAVE HEIGHT $H_{1/3}$ SHOWN IN PARENTHESES)

FIGURE – 16

42. In using Figure 16, the mean wave height of a group of waves should first be found by use of equation 10. The diagonal line corresponding to H_m is followed to its intersection with the vertical line labeled (say) 95 percent. The height which will not be exceeded by 95 percent of the waves in a group whose mean height is H_m is read on the vertical scale. In the example on the graph this height is 5 feet where H_m is 2.5 feet.

43. Breaking Waves. - At a certain point in its advance toward shore a wave becomes unstable, peaks up, and breaks. The water motion changes from a laminar orbital flow to turbulent, white-water conditions. The determination of point of breaking and breaking wave heights is of major importance in the planning and designing of shore protection measures.

44. Three different sets of curves are currently used to determine breaking depths and wave heights on breaking; one theoretical, and two empirical. The theoretical curve is derived from the analysis of a so-called solitary wave and results in the following equations:

$$\frac{H_b}{H_o'} = \frac{1}{3.3\sqrt[3]{H_o'/L_o}} \qquad (11)$$

$$d_b/H_b = 1.28 \qquad (12)$$

$$d_b/H_o = \frac{1.28}{3.3\sqrt[3]{H_o'/L_o}} \qquad (13)$$

Graphs drawn from these relations as well as those from the two other studies are presented in Figure 17 and 18. All three curve sets relate the breaker depths d_b and the breaker wave height H_b to deep water wave length L_o and deep water wave height, H_o', which would exist if refraction were ignored. (Knowing the deep water wave height H_o and the refraction coefficient K, H_o' may be determined from $H_o' = K H_o$). The solid line curves include the additional parameter of bottom slope , the effect of which was adequately verified under controlled laboratory conditions.

45. To use the curves on Figures 17 and 18, H_o' is determined as above, L_o from the known relationship L_o (in feet) $\doteq 5.12\ T^2$, and the beach slope from hydrographic charts. By computing the ratio H_o'/L_o the ratios d_b/H_o and H_b/H_o' may be picked off the graph and, from these, d_b and H_b determined by multipication by H_o' .

46. Diffraction of Waves. - Diffraction in water waves is that phenomenon whereby energy is transferred laterally along a wave crest. It is most noticeable where an otherwise regular train of waves is interrupted by a barrier such as a breakwater.

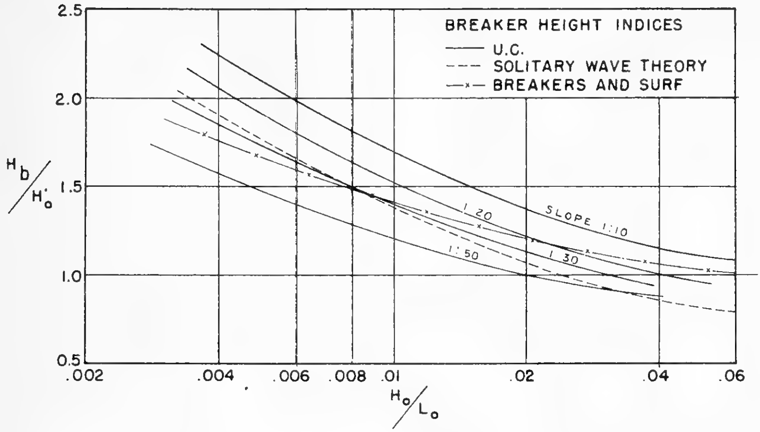


FIGURE 17

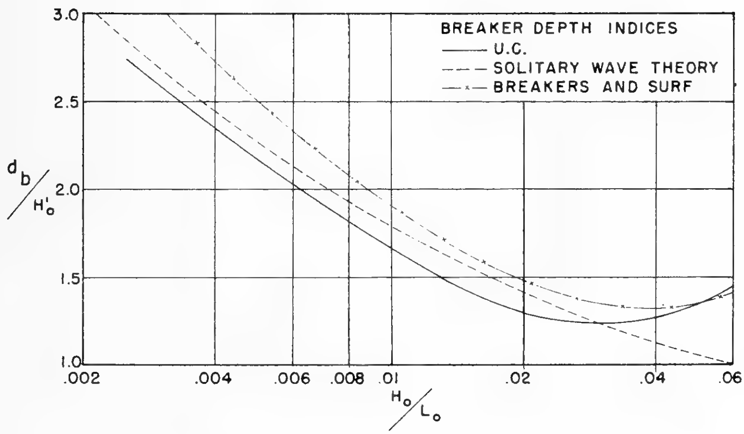


FIGURE 18

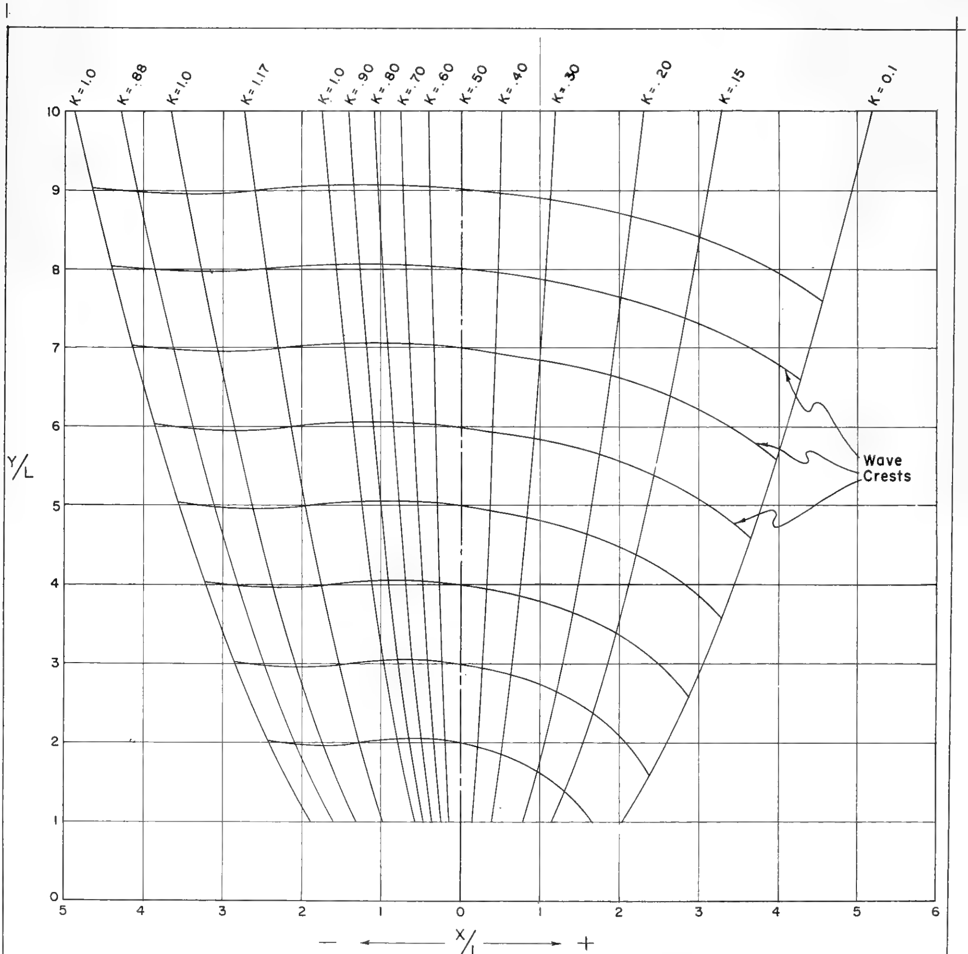
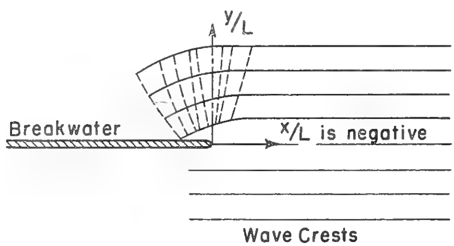


FIGURE 19 - GENERALIZED DIFFRACTION DIAGRAM

47. Putnam and Arthur (69) presented experimental data verifying a method of solution proposed by Penny and Price (66) for the behavior of waves after passing a single breakwater. Blue and Johnson (12) have dealt with the problem of the behavior of waves after passing through a gap, as between two breakwater arms.

48. Waves Passing a Single Breakwater. - The solution to this problem is presented in Appendix E. From this solution an overlay has been prepared (Figure 19) which for the case of uniform depth shoreward of the breakwater, shows positions of diffracted wave crests and lines of equal wave height reduction.

49. The diagram of Figure 19 is presented in dimensionless form and can therefore be used for any condition of wave period and water depth by scaling the entire figure up or down. The manner of use of the overlay is illustrated in Figure 20.



DIFFRACTION FOR A SINGLE BREAKWATER — NORMAL INCIDENCE
FIGURE 20

The wave length L at the depth, d, of the breakwater tip must be found by computing the ratio d/L_o (L_o = the deep water wave length), by referring to Table 1 of Appendix D for the corresponding value of d/L and dividing d by this ratio. The diagram itself must then be scaled up or down so that the distance from y/L = 0 to y/L = 1 corresponds to one wave length on the scale of the chart on which the diagram is to be drawn.

50. The diffraction diagram is then placed so that the x/L = 0 ordinate lies in the direction of wave approach, with x/L positive values on the sides of the breakwater toward the protected area in the breakwater's lee. The lines labelled "wave crests" then represent positions of first, second, etc. diffracted wave crests from the breakwater. The lines labelled K' are lines of equal decreased wave height. That is, along the K' = 0.20 line for example wave heights are two-tenths of their values outside the breakwater.

51. Waves Passing a Gap of Width Less than Five Wave Lengths. - The solution for this problem (Appendix E) is more complex, and it is not possible to construct one diagram for all conditions. A new diagram must be drawn for each different ratio of gap width to wave length. One for a gap width to wave length ratio of 2.00 (see Appendix E) is shown in Figure 21, which figure also illustrates its use. Figures 22 through 31 show lines of equal diffraction coefficient for gap width (B) to wave length (L) ratios of B/L of 0.50, 1.00, 1.41, 1.64, 1.78, 2.00, 2.50, 2.95, 3.82 and 5.00 drawn for a somewhat more complex solution of the diffraction problem than that in Appendix E. In all but Figures 27, for B/L = 2.00, the wave crest lines have been eliminated. Wave crest lines are usually only of pictorial use. One-half the diffraction coefficient lines have been eliminated from these figures but the diffraction coefficients are symmetrical about the x/L = 0 ordinate, thus the diagrams may be completed by folding the diagram about that ordinate.

52. Waves Passing a Gap of Width Greater Than Five Wave Lengths. - Where the breakwater gap width is greater than five wave lengths, the diffraction effects of each wing are essentially independent, and the diagram (Figure 19) for a single breakwater may be used to define the diffraction characteristics in the lee of both wings. (See Figure 32)

53. Diffraction at a Gap - Oblique Incidence. - When the waves approach at an angle to the breakwater gap centerline, an approximate appraisal of diffracted wave characteristics may be made by considering the gap to be as wide as its projection in the direction of incident wave travel. Diffraction diagrams should be drawn in the manner of the preceding sections but using this effective gap instead of the true gap.

54. Refraction and Diffraction Combined. - If the bottom seaward and shoreward of the breakwater is not flat - the usual case - refraction as well as diffraction will take place. Though a unified theory of the two has not yet been devised, an approximate picture of wave changes may be drawn by: (1) constructing a refraction diagram to the breakwater,

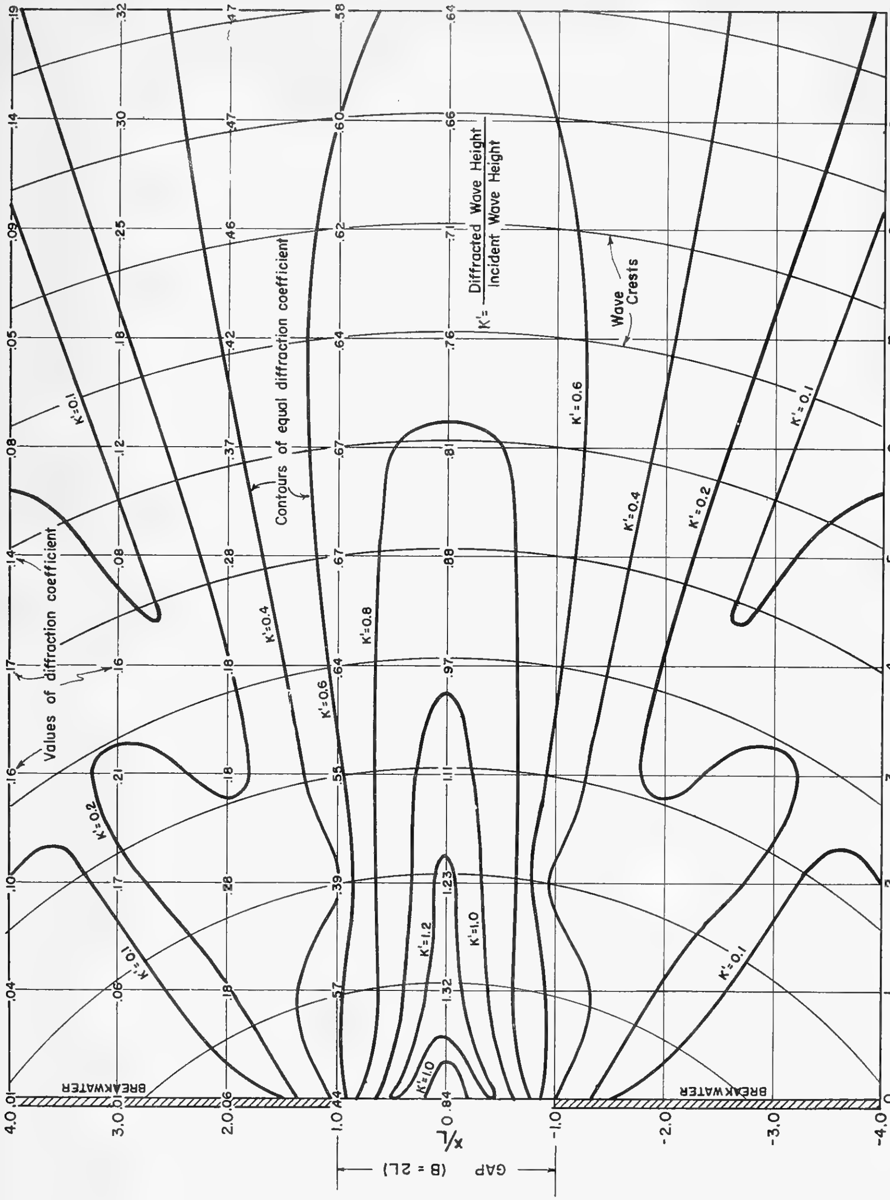


Fig. 21 GENERALIZED DIFFRACTION DIAGRAM FOR A BREAKWATER GAP WIDTH OF TWO WAVE LENGTHS (B/L = 2)

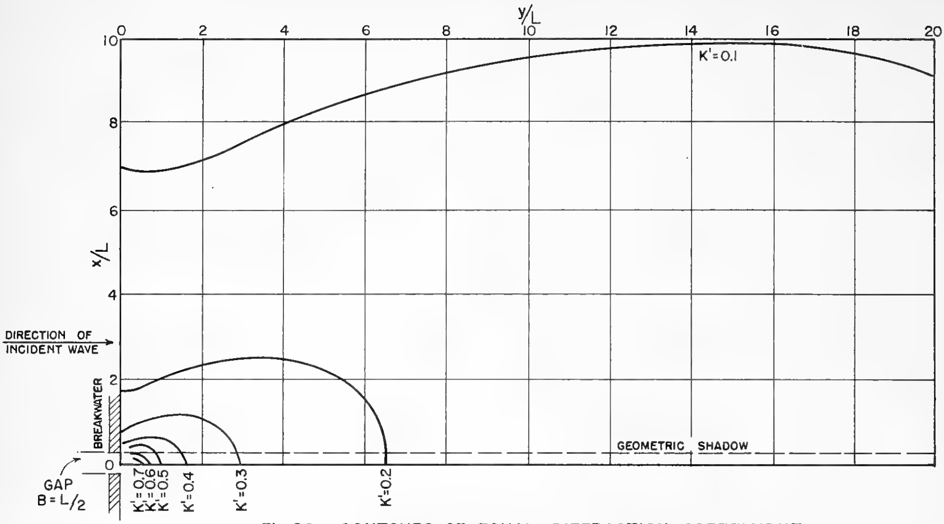


Fig. 22 CONTOURS OF EQUAL DIFFRACTION COEFFICIENT
GAP WIDTH = 0.5 WAVE LENGTH (B/L = 0.5)

$$K' = \frac{\text{Diffracted Wave Height}}{\text{Incident Wave Height}}$$

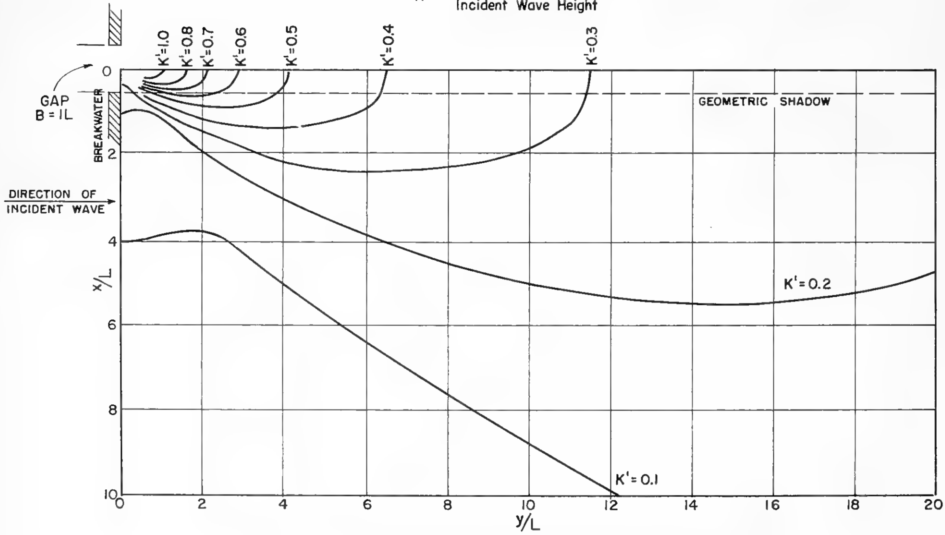


Fig. 23 CONTOURS OF EQUAL DIFFRACTION COEFFICIENT
GAP WIDTH = 1 WAVE LENGTH (B/L = 1)

DIFFRACTION OF WAVES AT A BREAKWATER GAP

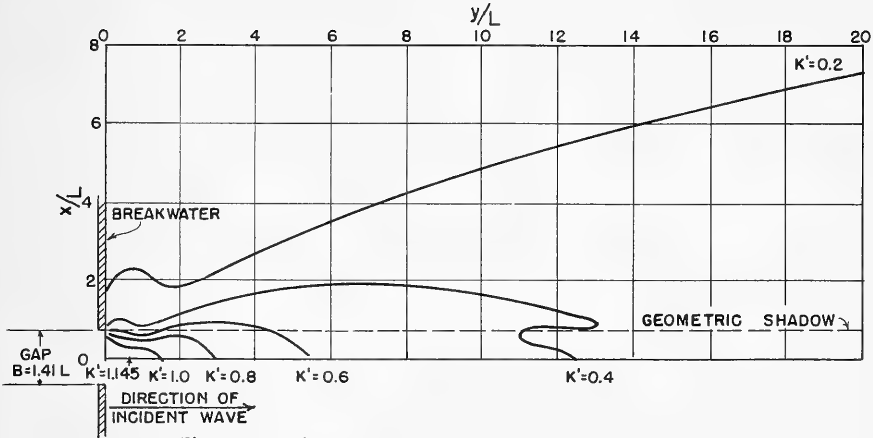


Fig.24 CONTOURS OF EQUAL DIFFRACTION COEFFICIENT
GAP WIDTH=1.41 WAVE LENGTHS (B/L=1.41)

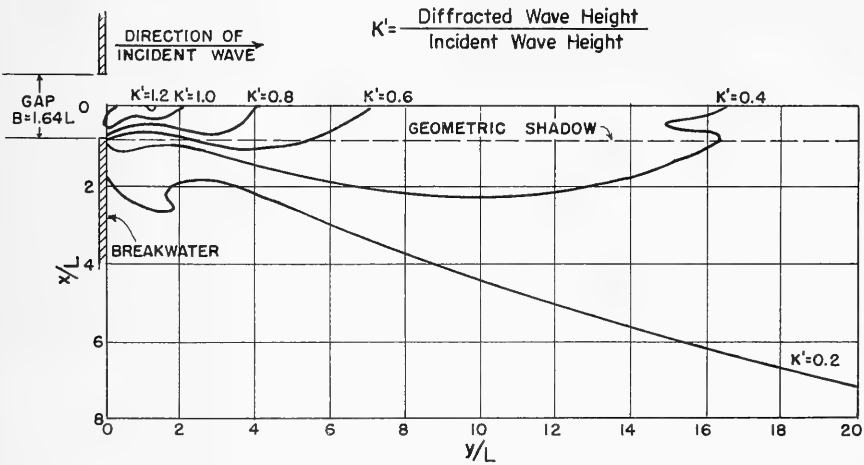


Fig.25 CONTOURS OF EQUAL DIFFRACTION COEFFICIENT
GAP WIDTH = 1.64 WAVE LENGTHS (B/L.=1.64)

DIFFRACTION OF WAVES AT A BREAKWATER GAP

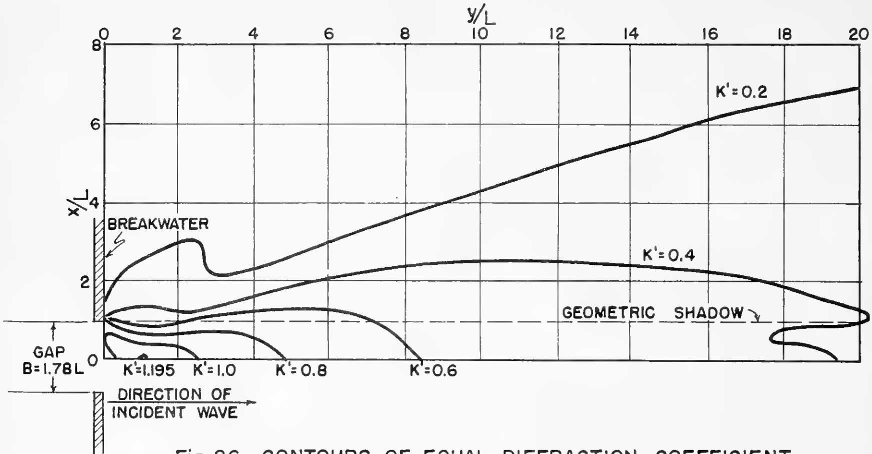


Fig. 26 CONTOURS OF EQUAL DIFFRACTION COEFFICIENT
GAP WIDTH = 1.78 WAVE LENGTHS (B/L = 1.78)

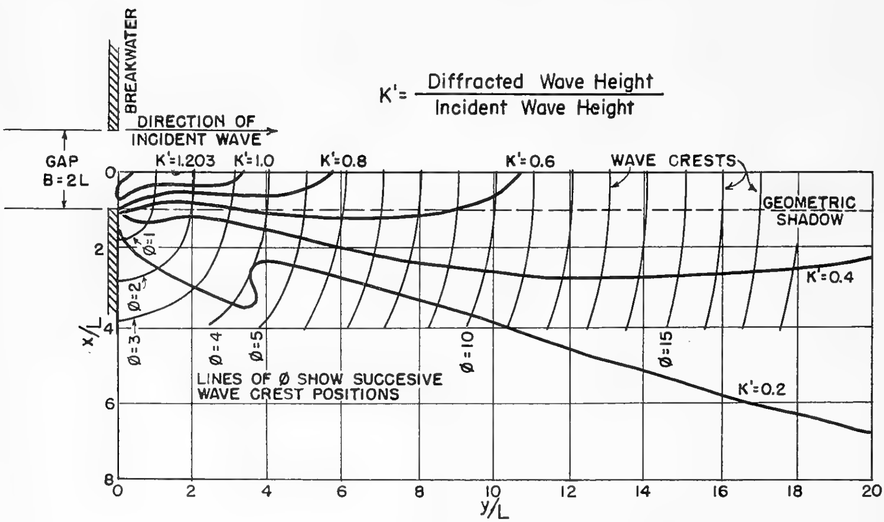


Fig. 27 CONTOURS OF EQUAL DIFFRACTION COEFFICIENT
GAP WIDTH = 2 WAVE LENGTHS (B/L = 2)

DIFFRACTION OF WAVES AT A BREAKWATER GAP

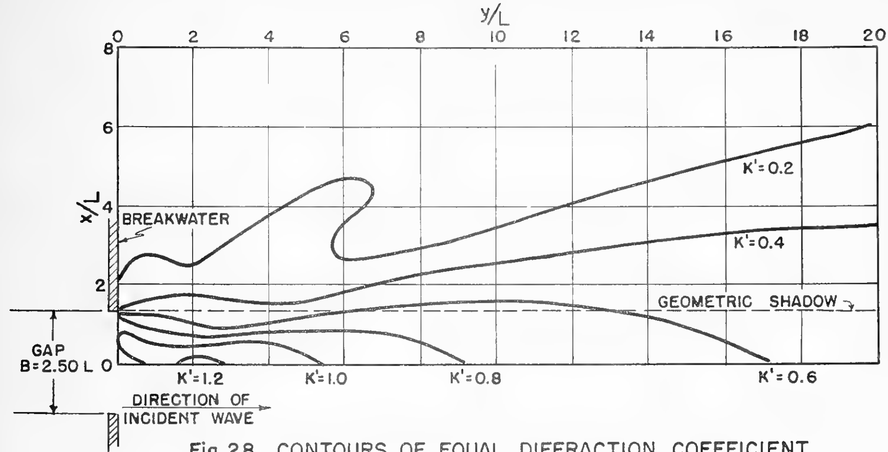


Fig.28 CONTOURS OF EQUAL DIFFRACTION COEFFICIENT
GAP WIDTH = 2.50 WAVE LENGTHS (B/L = 2.50)

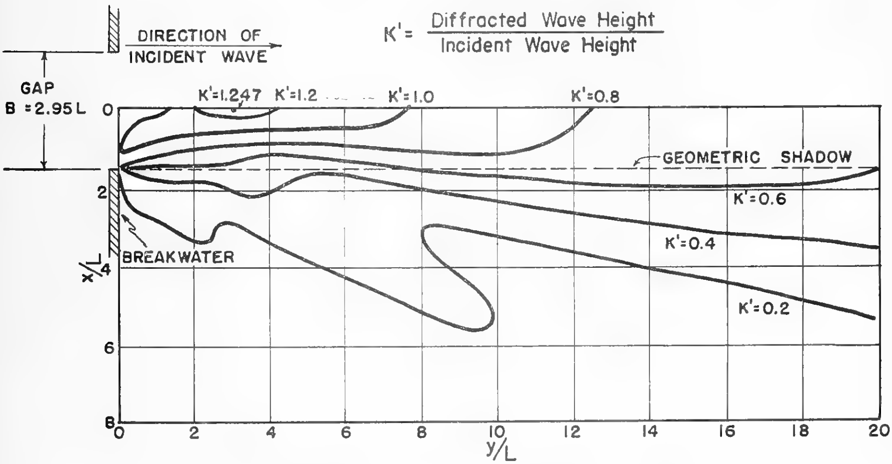


Fig. 29 CONTOURS OF EQUAL DIFFRACTION COEFFICIENT
GAP WIDTH = 2.95 WAVE LENGTHS (B/L = 2.95)

DIFFRACTION OF WAVES AT A BREAKWATER GAP

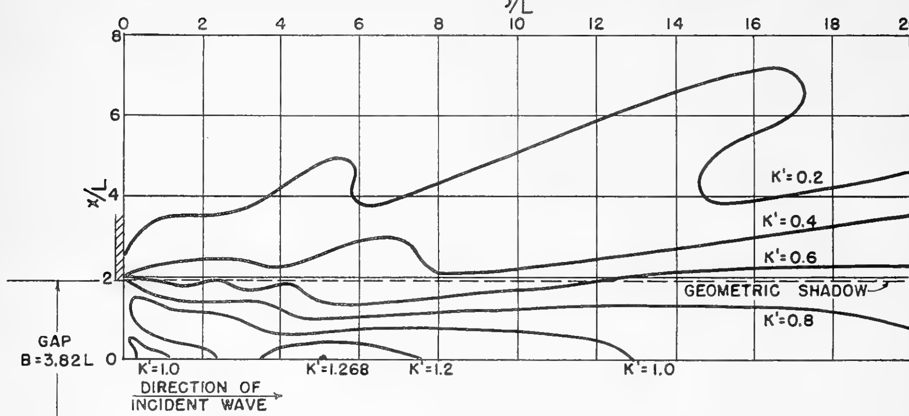


Fig. 30 CONTOURS OF EQUAL DIFFRACTION COEFFICIENT
GAP WIDTH = 3.82 WAVE LENGTHS (B/L = 3.82)

BREAKWATER

$$K' = \frac{\text{Diffracted Wave Height}}{\text{Incident Wave Height}}$$

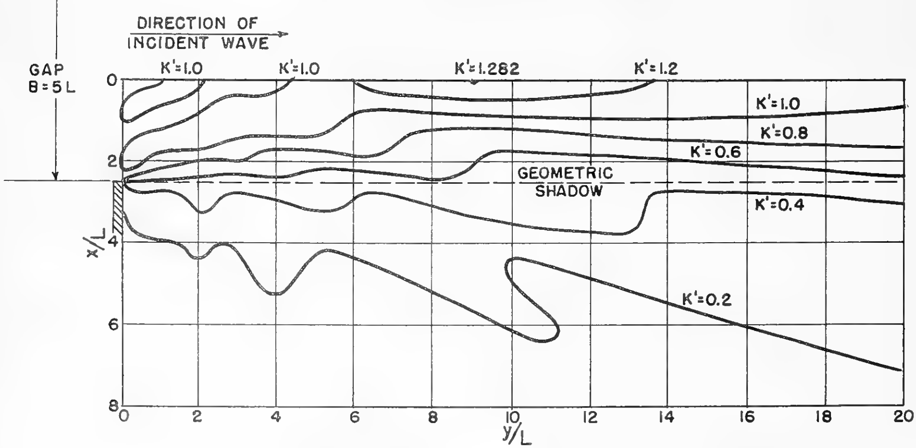


Fig. 31 CONTOURS OF EQUAL DIFFRACTION COEFFICIENT
GAP WIDTH = 5 WAVE LENGTHS (B/L = 5)

DIFFRACTION OF WAVES AT A BREAKWATER GAP

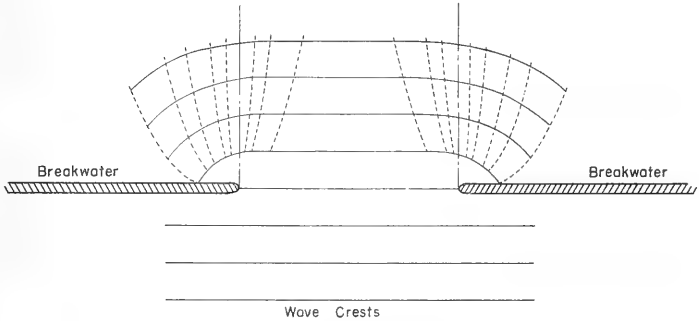


FIG. 32 — DIFFRACTION FOR A BREAKWATER GAP OF WIDTH > 5L (B/L > 5)

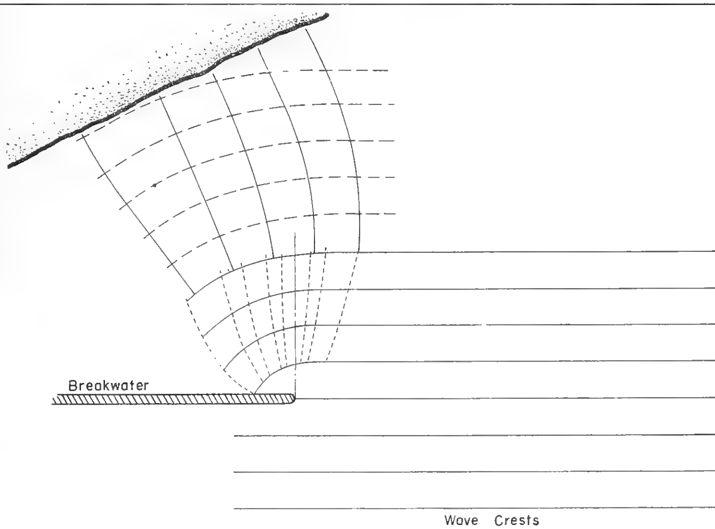


FIG. 33 — SINGLE BREAKWATER — REFRACTION-DIFFRACTION COMBINED

(2) at this point, constructing a diffraction diagram carrying successive crests 3 or 4 wave lengths shoreward, and (3) with the wave crest and wave direction indicated by the last shoreward wave crest determined from the diffraction diagram, constructing a new refraction diagram to the breaker line. A typical refraction-diffraction diagram is shown on Figure 33.

CHANGES IN WATER LEVEL

55. Tides. - There are usually two high and two low waters in a tidal or lunar day. Tides follow the moon more closely than they do the sun. As the lunar day is about 50 minutes longer than the solar day, the tides occur later each day. Because of the varying effects of the sun and moon, a diurnal inequality in tides occurs in which, at certain places, there may be a difference of only a few tenths of a foot between one high water and the succeeding low water of a day but a marked difference in height between the other high water and its succeeding low water. Along the Atantic coast the two tides each day are of nearly the same height. On the Gulf coast the tides are very low but in some instances have a pronounced diurnal inequality. Pacific coast tides compare in height with those on the Atlantic coast but have a decided diurnal inequality (see Figure 5 Appendix A).

56. Pertinent data concerning tidal ranges along the sea coasts of the United States are given in the following tables. Spring ranges are shown for areas having approximately equal daily tides and diurnal ranges are shown for areas having a pronounced inequality. Detailed data concerning tidal ranges are given in Tide Tables, U. S. Department of Commerce, Coast and Geodetic Survey.

Table 3 - Tidal Ranges - Atlantic Coast

Reference Station	Coastal Area From	Coastal Area To	Approximate *Tidal Ranges(ft.)
Eastport, Maine	Calais	W. Quoddy Head	18-22
Eastport, Maine	W. Quoddy Head	Englishman Bay	14-18
Portland, Maine	Englishman Bay	Belfast, Maine	11-14
Portland, Maine	Belfast	Rockport, Mass.	8-11
Boston, Mass.	Rockport, Mass.	Provincetown	10-11
Boston, Mass.	Provincetown	Chatham	5-10
Boston, Mass.	Chatham	Cuttyhunk Island	2-4
Boston & New London	Cuttyhunk Isd.	Connecticut River	3-6
Bridgeport & Willets Pt	Connecticut R.	Truman Beach	4-8
New London & Sandy Hook	Truman Beach	Hempstead Bay	1-3
Sandy Hook & New York	Hempstead Bay	Cape May	4-6
- - - - - -	Delaware Bay		5-7
Sandy Hook	Rehoboth	Smith Island	4-5
Hampton Roads - Charleston	Cape Henry	South Santee River	3-5
Charleston	S. Santee R.	N. Edisto River	5-7
Charleston-Savannah River Entrance	N. Edisto R.	Nassau Sound	7-9
Mayport	Nassau Sound	Daytona Beach	4-6
Miami & Key West	Daytona Beach	Tarpon Belly Keys	1-3

*Spring range

Table 4 - Tidal Ranges - Gulf of Mexico

Reference Station	Coastal Area From	Coastal Area To	Approximate Tidal Ranges (ft.)
Key West, St. Marks R., Tampa Bay	Tarpon Belly Keys, Fla.	St. George Sound, Fla.	*2-4
Pensacola, Mobile, Galveston	St. George Sound, Fla.	Port Isabel, Texas	**1-2

*Spring range
** Diurnal Range

Table 5 - Tidal Ranges - Pacific Coast

Reference Station	Coastal Area From	Coastal Area To	Approximate Tidal Ranges (ft.)
San Diego, Los Angeles San Francisco	Point Loma, Calif.	Cape Mendocino Calif.	5-6
San Francisco, Humbolt Bay	Cape Mendocino	Siuslaw River, Oregon	6-7
Humboldt Bay, Astoria	Siuslaw River	Columbia River	7-8
Aberdeen, Wash.	Columbia River	Port Townsend, Wash.	8-9
Seattle, Wash.	Puget Sound, Wash.		10-14

57. Wind Set-Up. - In most coastal locations, severe storms cause an increase in water level above that due to tidal action, if the wind drives water ashore more rapidly than subsurface return flows can carry it to sea. This effect is increased in gradually shoaling, narrowing, open mouth bays. Set-up due to wind occurring at the same time as high tides causes what are known as storm tides, which, especially in hurricane regions, may be significantly higher than normal tidal action. For example, Galveston, Texas, where the normal tidal range is 1 to 2 feet, experienced a hurricane in 1900 which caused storm water levels up to 15 feet, and in 1915 one which caused a storm tide height of 12.5 feet. Water levels may also be reduced by wind action in the opposite direction.

58. Seiches. - Every land-locked or semi-land-locked body of water may be subject to regular short period oscillations which are a function of the harmonic characteristics of the basin. Any change in water surface elevation may start these oscillations which are known as seiches. There is one natural period for longitudinal oscillations and a second for transverse oscillations. Depending on the dimensions of the body of water and the mean depth along the axis of oscillation, the period of the sieche may be either the natural period or some harmonic of the period. The natural period, or time of oscillation, is independent of the amplitude of the seiche. This time of oscillation may be calculated by the formula

$$t = \frac{2x}{\sqrt{\text{[illegible]}}} \qquad (14)$$

in which t = time of oscillation (seconds)
x = length of the axis of oscillation (feet)
d = mean depth along axis of oscillation (feet)
g = acceleration of gravity (feet/second^2)

59. Lake Levels. - The Great Lakes have only insignificant tidal variations but are subject to seasonal and annual changes in water level and to changes in water level caused by wind set-up, barometric pressure variations, and by seiches. The average or normal elevations of the lakes surfaces vary irregularly from year to year. During the course of each year the surfaces are subject to consistent seasonal rises and falls, reaching their lowest stages during the winter months and attaining their maximum stages during the summer months. A hydrograph of monthly lake levels from the year 1860 to the present is shown in Figure 34. Table 6 summarizes certain lake level data.

60. In addition to the seasonal and annual fluctuations, the lakes are subject to occasional seiches of irregular amount and duration. Sometimes these result from variations in barometric pressure, which may produce changes in water surface elevation ranging from a few inches to several feet. At other times the lakes are affected by wind set-up which raise the level at one end and lowers it at the other end of a lake.

61. In general, the maximum amounts of these irregular changes in lake level must be determined for each location under consideration. Some idea as to the extent of fluctuations which may be expected is given by the following. On Lake Superior a barometric storm in June 1939 caused an oscillation at Marquette with a maximum range in surface elevation of 7.4 feet. The storm of 28 November 1905 raised the water level at Duluth Harbor 2.3 feet above normal water stage. The largest flucutations of any of the lakes occur on Lake Erie because of its shallow depth. (see Figure A-11, Appendix A). The largest fluctations on this lake occur at Sandusky, Toledo, and the mouth of Detroit River at the western end of the lake, and at Buffalo Harbor at the eastern end of the lake. At Buffalo Harbor the extreme range is 13.7 feet, the highest level on record being 9.5 feet above low-water datum on 1 April 1929 and the lowest being 4.2 feet below that datum on 30 January 1939. The greatest range for any one year was 11.6 feet in 1927, with a high stage of 9.3 feet and a low stage of minus 2.3 feet. The least range for any one year was 6.5 feet above and 1.4 feet below datum. The foregoing are based on records for the past 50 years.

LITTORAL DRIFT

62. General Characteristics. - Littoral drift may be defined as the material that moves generally parallel to the shore under the influence of waves and currents. It is sometimes considered as the movement of the material as well as the material itself. Evaluation of littoral drift characteristics is the fundamental basis for functional design of shore protection measures.

TABLE 6 - Fluctuations in Water Level - The Great Lakes

Lake	*Low Water Datum	*Mean Surface Elevation	** Range Between Monthly Mean Stages: Maximum From	Maximum To	Minimum From	Minimum To	Maximum Range	Maximum Annual Fluctuation	Minimum Annual Fluctuation
	(feet)	(feet)	(feet)	(feet)	(feet)	(feet)	(feet)	(feet)	(feet)
Superior	601.60	602.27	0.94 above	2.03 above	0.06 below	0.41 above	3.76	1.83	0.45
Michigan	578.50	579.79	1.24 above	2.61 above	0.26 below	1.09 above	4.97	2.21	0.36
Huron	578.50	579.79	1.19 above	2.61 above	0.25 below	1.09 above	4.97	2.21	0.36
Erie	570.50	571.95	2.19 above	3.58 above	0.53 above	1.44 above	4.90	2.75	0.90
Ontario	244.00	245.77	2.96 above	4.97 above	0.61 above	1.91 above	6.29	3.20	0.69

* Referred to mean tide at New York Harbor

** Refers to low water datum

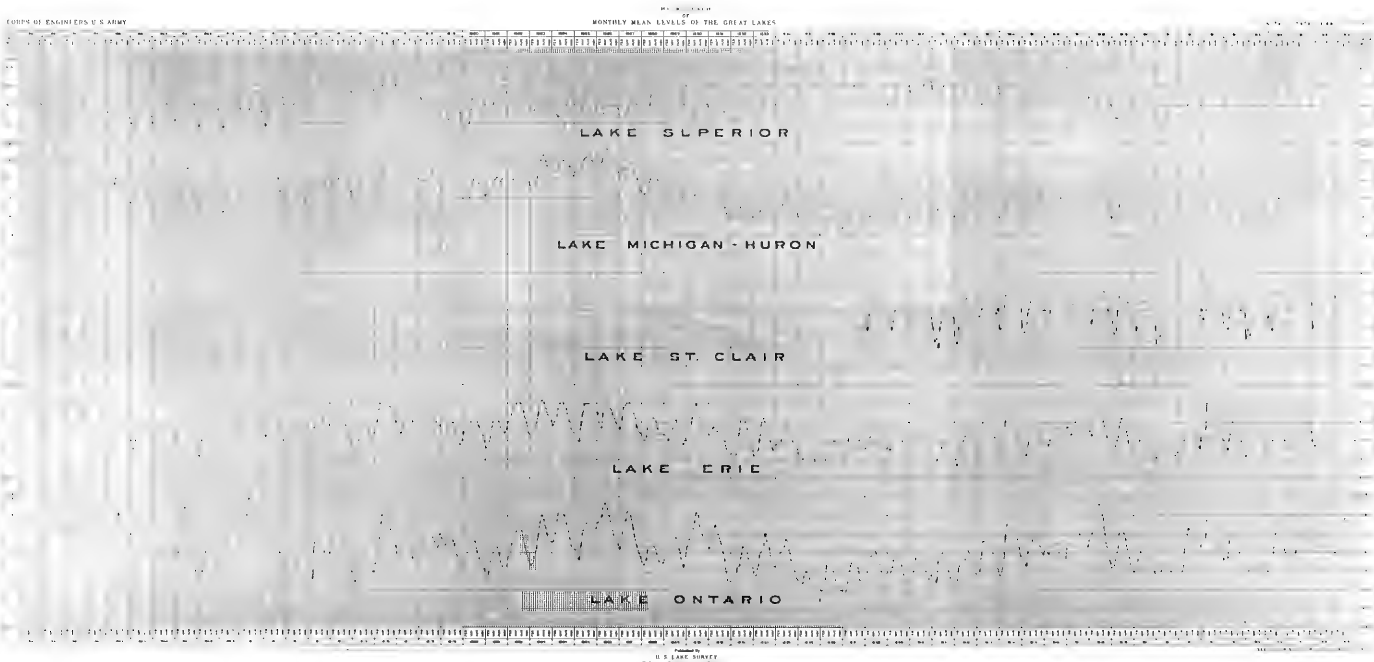


Fig. 34a

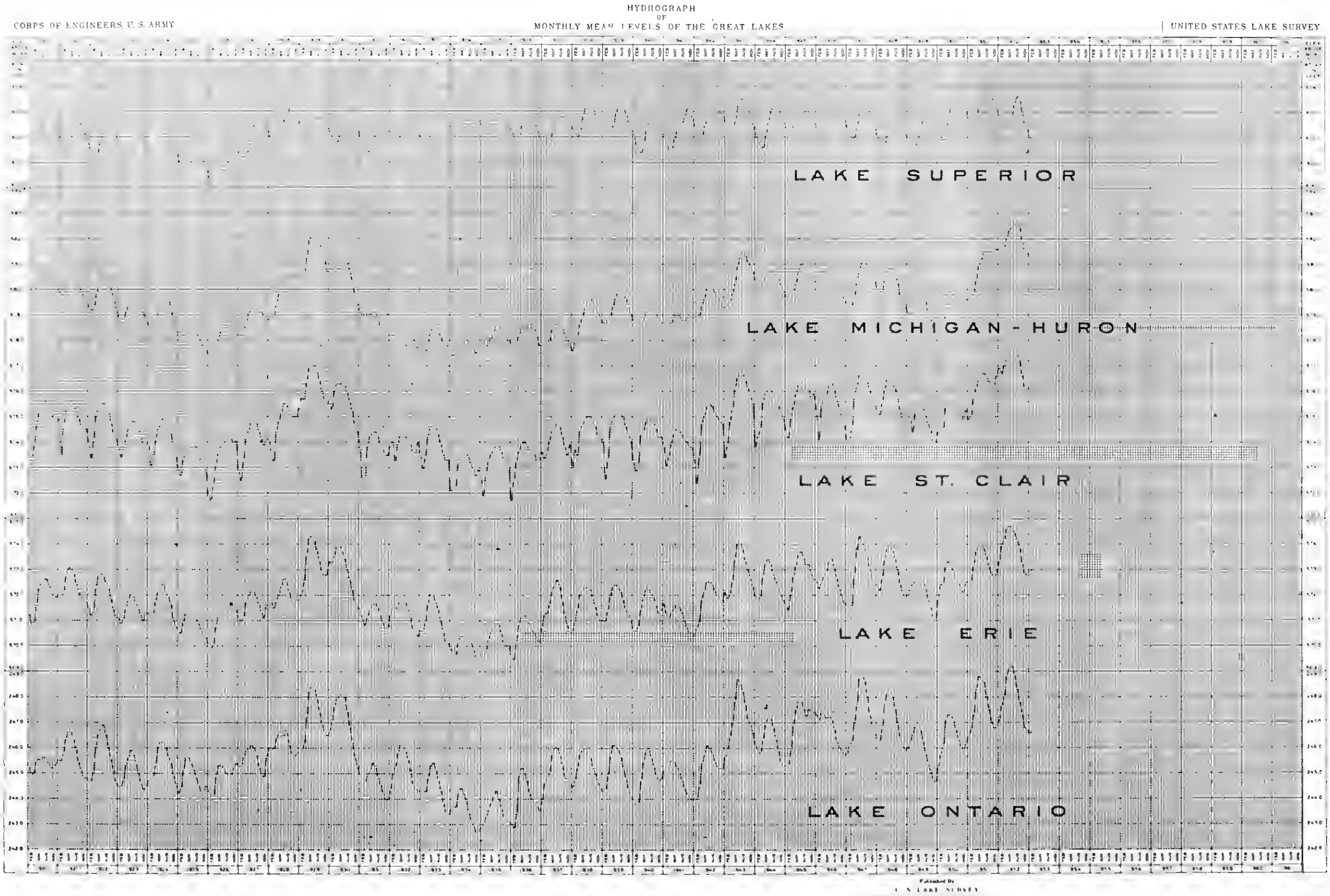

CORPS OF ENGINEERS U. S. ARMY
HYDROGRAPH
OF
MONTHLY MEAN LEVELS OF THE GREAT LAKES
UNITED STATES LAKE SURVEY
LAKE SUPERIOR
LAKE MICHIGAN-HURON
LAKE ST. CLAIR
LAKE ERIE
LAKE ONTARIO

Fig. 34b

63. Waves and currents provide the forces which move the material. The mechanics of transport are not precisely known, but it may be generally stated that there are three basic modes of transport; that which occurs on the foreshore in a more or less scalloped path due to uprush and backwash of obliquely approaching waves known as "beach drift"; that which is moved principally in suspension in the surf zone by littoral currents and the turbulence of breaking waves; and that which is moved close to the bed in depths beyond the surf zone by the oscillating currents of passing waves. Significant bottom movement has been observed in depths exceeding 100 feet on exposed sea coasts. Figure 35 illustrates the three basic modes of transport, and Figure 36 depicts trends of proportionate bed load transport related to wave energy and steepness from laboratory experiments.

64. It has been observed that changes in the shore profile occur with changes in water depth and with changes in wave characteristics. Profile adjustment due to change in water depth is relatively slow, and therefore minor with respect to single tidal cycles. Measurable change has been detected when comparisons are made with water level expressed as daily mean sea level. The profile adjustment due to change in wave characteristics is rapid, and a single storm of a few hours duration may cause a major change in profile. Profile changes may be ascribed primarily to the onshore or offshore shifting of beach and bottom material. Generally, the shift is offshore as the water level rises or the waves steepen, and onshore as the water level lowers or the waves are flatter. Laboratory studies indicate that the critical wave steepness defining the boundary between onshore and offshore movement is in the order of $H_o/L_o = 0.025$. This has not yet been confirmed in nature.

65. Onshore and offshore shifting of material results in longshore movement at a sluggish rate and may be classed as a mode of transport, the relative importance of which is unknown at this time. Suspended material moving with the littoral current in the surf zone, beach drift and wave current bed load transport seaward of the surf zone all represent more rapid transport, and are thus presently considered as primary factors in resultant littoral drift.

66. Regardless of the mode of transport, the direction and rate of littoral drift depend primarily upon the direction and energy of waves approaching the shore. Exceptions exist on short reaches of shore adjoining tidal inlets where the tidal current pattern may be dominant.

67. Slope Sorting of Littoral Material. - Wherever sandy beaches exist or where the surf zone and nearshore bottom is composed of sand, the grain size of material along a shore profile decreases generally as the water depth increases until depths are reached where wave currents are incapable of moving bed material. The coarsest material is usually found in the surf zone in the vicinity of the plunge point of waves, though in protracted periods of mild wave action the foreshore and surf zone material may be nearly equal in size. Foreshore and nearshore slopes are related to the grain size of material of which they are formed, but the

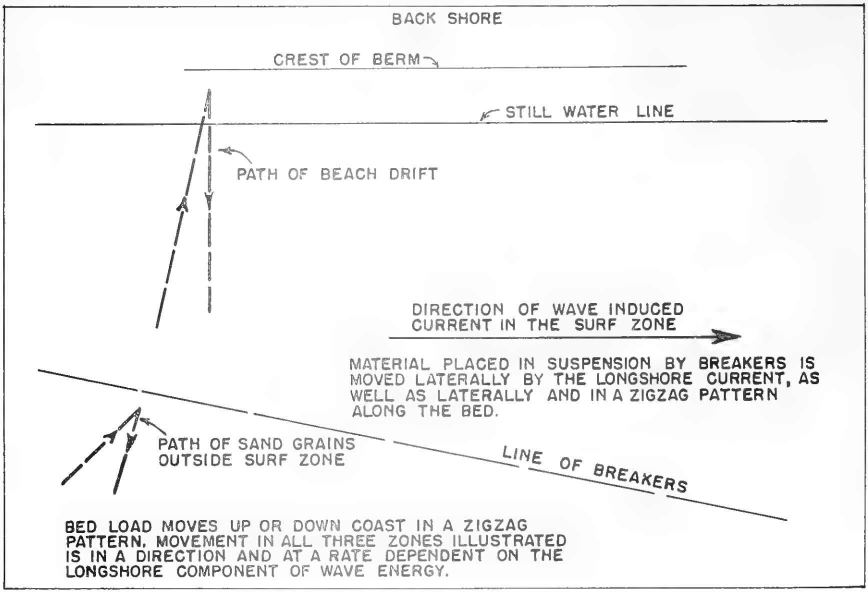


Fig. 35 LATERAL MOVEMENT OF LITTORAL DRIFT

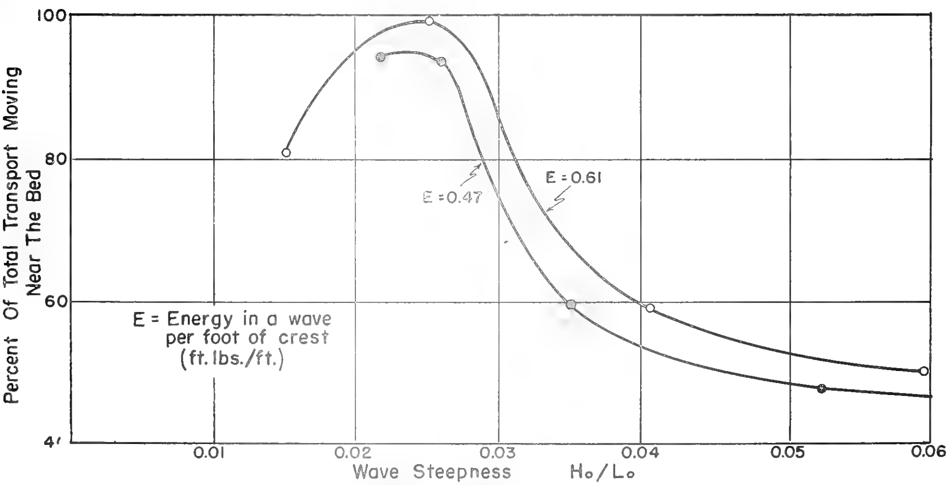


Fig. 36 LABORATORY RESULTS INDICATING TRENDS OF PROPORTIONATE BED LOAD AND SUSPENDED LOAD TRANSPORT RELATED TO WAVE ENERGY AND WAVE STEEPNESS. (SAVILLE 1950)

relationship is not the same at all localities since it is also influenced by water level variability and wave exposure.

68. Evidence of the slope sorting of beach materials is illustrated on Figures 37 and 38. The slope sorting is ascribed generally to the differential velocity in the oscillating wave currents in shallow water, whereas the velocity of water particles is theoretically uniform about the orbital path of the particles in deep water, moving in the direction of wave propagation while the crest is passing and in the opposite direction with passage of the trough, the uniformity ceasing when the wave is deformed by diminishing depth. Deformation takes the form of steepening the crests, shortening them in relation to lengths of the troughs, so that the particles move forward with the wave crest in less time than they return with the trough. Figure 39 illustrates laboratory measurements of this differential velocity. The effect of this phenomenon is to transport coarser particles of bed material shoreward. Where the slope becomes sufficiently steep, gravity counterbalances the current effect and an equilibrium condition results. Currents introduced by reflected waves likewise affect the balance of forces in this region near the shore.

69. Because of slope sorting, it may be stated that material in littoral transport moves generally within a depth range compatible with its size or resistance to transport. The actual path or rate of transport of individual particles or groups of particles cannot be stated from present knowledge. While density and particle shape are factors in transportability, median grain size is a satisfactory parameter for evaluating generally the transportability of littoral material.

70. Littoral characteristics. - The existence of an erosion problem is prima facie evidence that one of two conditions exist. The first is that the water level and the land have not become adjusted in terms of shore slope, as in the case of exceptionally high lake levels or storm tides, and upland material is being eroded to establish equilibrium slopes. The second and more common condition, which may exist concurrently with that cited above, is that material being removed from the area exceeds the rate of supply. Normal supply and loss occurs through the process of littoral drift. Analysis of an erosion problem preparatory to functional planning of remedial measures thus requires that conclusions be reached concerning the characteristics of littoral drift. These characteristics as ideally defined for a particular shore segment are: (a) the predominant drift direction; (b) direction variability; (c) character of littoral materials and depths in which they are transported; (d) rate of supply to the area and source of material supplied; and (e) rate of loss from the area and the manner in which loss occurs. It will not always be possible to reach well supported conclusions for these five features, nevertheless careful investigation will provide knowledge in place of opinion in many cases.

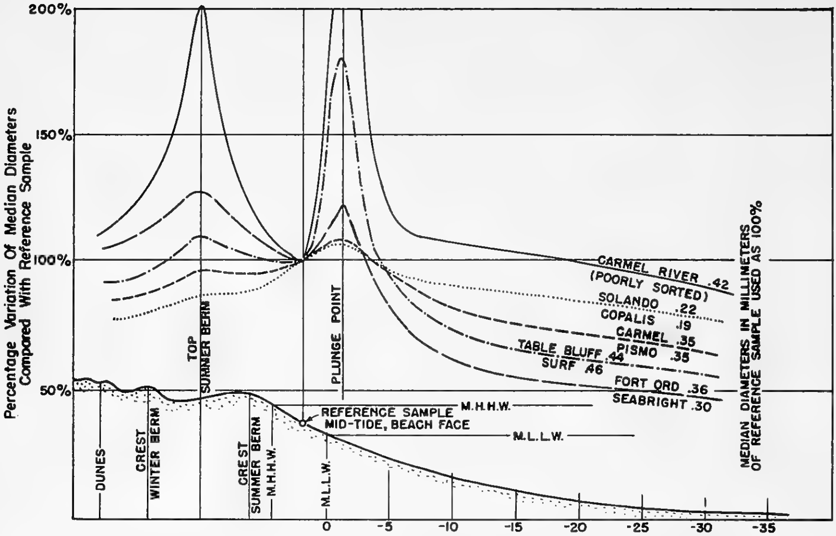


Fig. 37 SIZE DISTRIBUTION ACROSS A BEACH (BASCOM 1951)

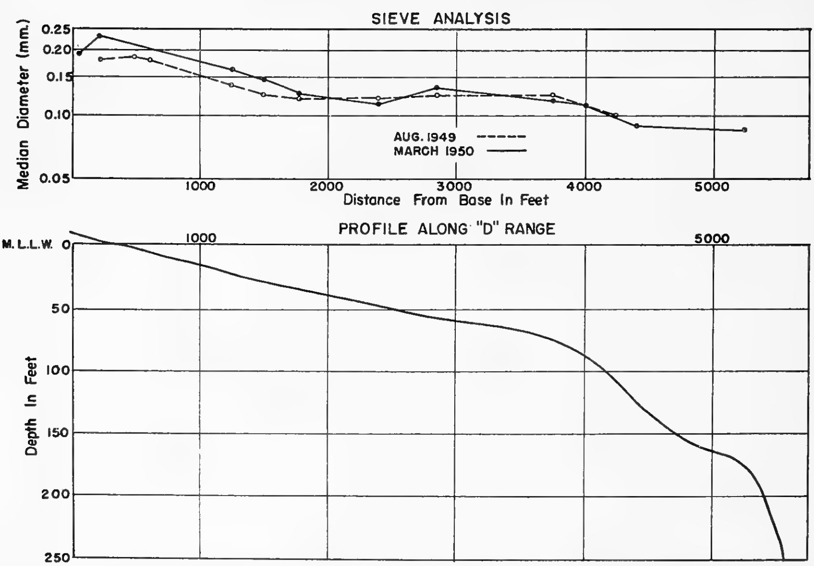
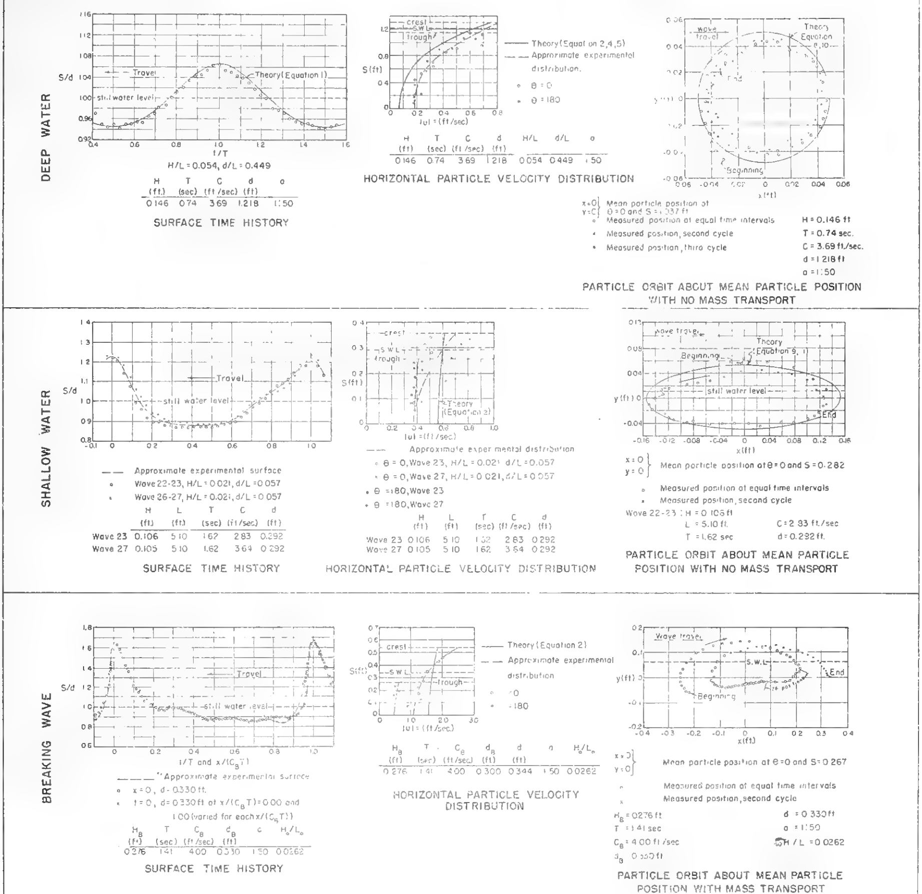


Fig. 38 SEASONAL VARIATION OF SIZE DISTRIBUTION IN SEDIMENTS ALONG "D" RANGE OFF LA JOLLA, CALIFORNIA (INMAN 1953)



Legend

- S - elevation of the particle above the bottom (ft); in this case the surface particle.
- d - still water depth (ft)
- H - wave height (ft)
- L - wave length (ft)
- t - time (sec)
- T - wave period (sec)
- u - horizontal particle velocity, (ft. per sec.)
- θ - angular position of particle in its orbit measured counterclockwise and where there is no mass transport, (degrees)
- x - horizontal position of particle in its mean horizontal position. (ft)
- C - wave velocity of propagation (ft./sec)
- y - verticle position of particle in its mean horizontal position
- α - beach slope
- subscript B refers to breaking conditions

Equations 1, 4, 5, 8 and 10 are from Stokes' second approximation.
Equation 2 is from Stokes' first approximation.
Equations 9 and 11 are reduced forms of Stokes' second approximation.

TIME HISTORIES, PARTICLE VELOCITY DISTRIBUTIONS AND ORBITAL PATHS, OF WAVES IN DEEP WATER, IN SHALLOW WATER, AND AT BREAKING

FIGURE 39

(From: Morison and Crooke, 1953)

71. Determination of Direction of Littoral Drift. - It is not only necessary to know the direction of littoral drift at any one time -- which can generally be determined by observation of existing structures -- but the predominant direction of littoral drift over a normal climatic cycle must be established. This may also involve locating the position of natural and unnatural littoral barriers and those areas called nodal zones in which the net littoral transport changes direction. In these zones the net littoral drift is zero or, in other words, the downdrift components of littoral drift are equal to the updrift compenents. Although the methods used in determining the direction of littoral drift may vary in each section of the country, determination of the instantaneous and predominant directions of littoral drift and the location of littoral barriers and nodal zones may ordinarily be accomplished by analysis of such of the following factors as may be required to reach conclusions:

Accretion or erosion effects of existing structures;

Shore patterns in the vicinity of headlands;

The configuration of the banks and beds of inlets;

Statistical analysis of wave energy;

Characteristics of beach and bed materials;

Current measurements, (in the vicinity of inlets).

72. Shore Effects of Existing Structures. - This provides the most reliable means of determining littoral characteristics, and will ordinarily outweigh all other evidence. The use of existing structures to determine the direction of littoral drift is illustrated in the series of sketches, Figures 40 to 45 inclusive. Considering the evidence presented by groins, Figures 40 and 41, the condition of the beach at the time of inspection probably indicates the direction of drift during the immediately preceding period. To determine the predominant direction of littoral drift requires observations at regular intervals over a period of at least one year to avoid misinterpretation due to seasonal effect.

73. Considering the evidence presented by breakwaters and entrance jetties, Figures 42, 43, and 44, the quantities involved are generally large enough so that the condition observed at any one particular time probably is indicative of the predominant direction of littoral drift, with incremental changes showing the short term direction variability.

74. Evidence of Headlands. - The evidence presented by headlands as to the direction of littoral drift is not ordinarily as clear as that presented by structures because of the frequency of rocky shores on both sides of headlands. In some instances the headland is so oriented as to cause a reversal of direction of littoral drift under all wave conditions, thus compartmenting the coast line. Figures 46 and 47 show two types

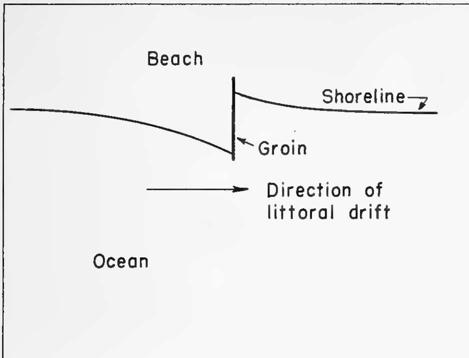


FIG. 40. EFFECT OF SINGLE GROIN

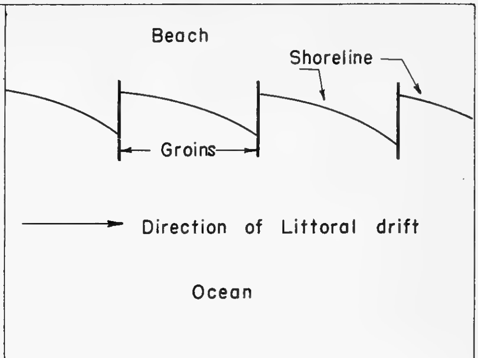


FIG. 41. EFFECT OF A SERIES OF GROINS

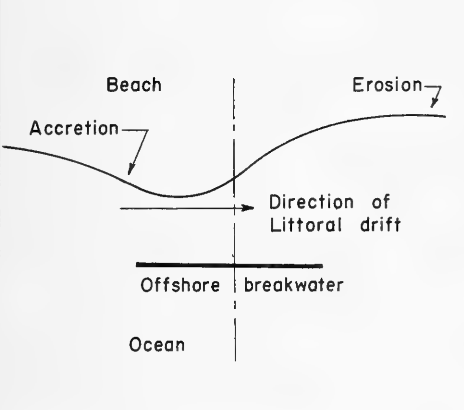


FIG. 42 EFFECT OF OFFSHORE BREAKWATER

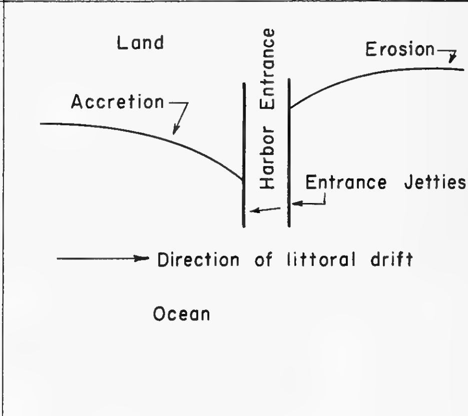


FIG. 43. EFFECT OF ENTRANCE JETTIES

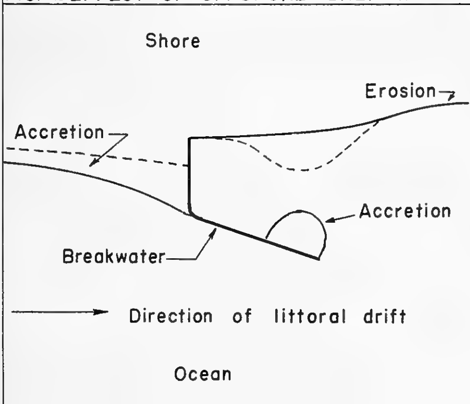


FIG. 44. EFFECT OF SHORE — CONNECTED BREAKWATER

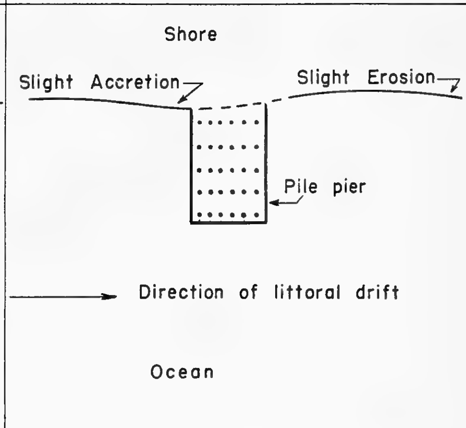


FIG. 45. EFFECT OF PILE CLUSTERS

DIRECTION OF LITTORAL DRIFT DETERMINED BY EXISTING STRUCTURES

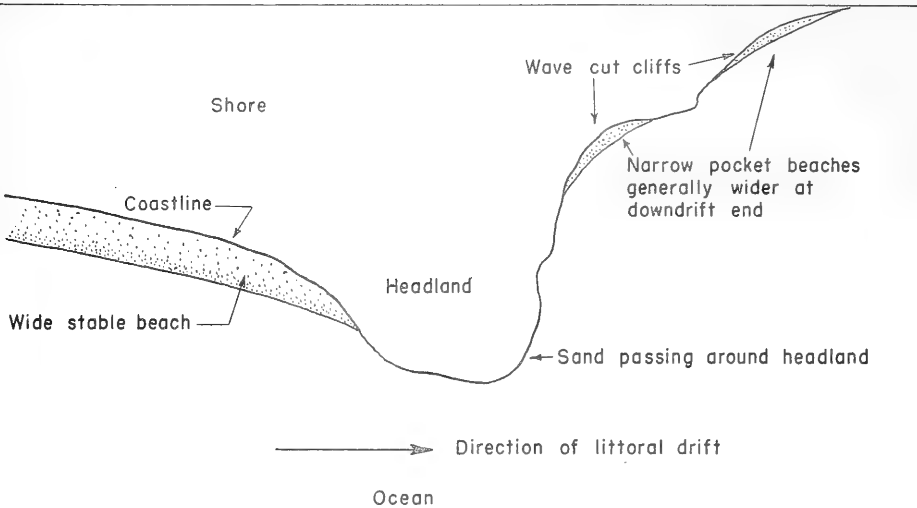


FIG. 46. DIRECTION OF LITTORAL DRIFT BY HEADLANDS

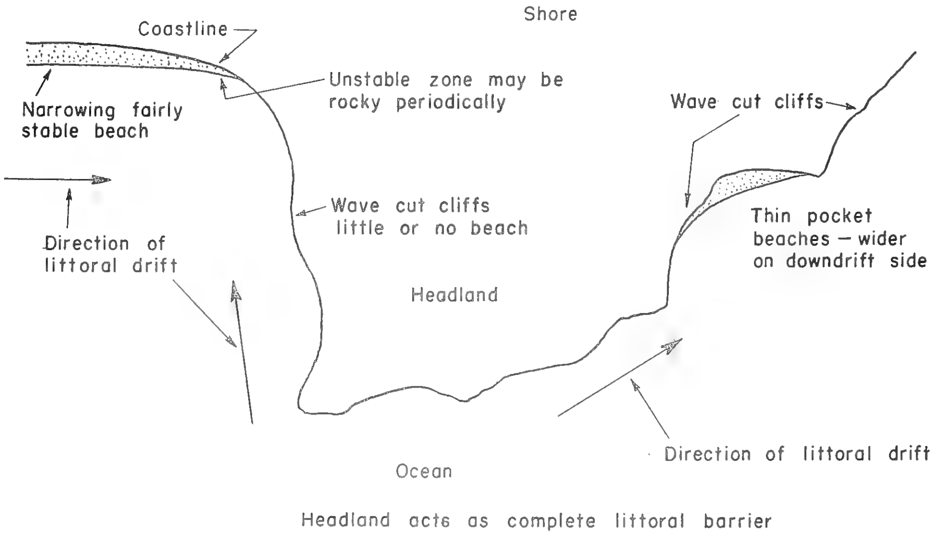


FIG. 47. DIRECTION OF LITTORAL DRIFT BY HEADLANDS

DIRECTION OF LITTORAL INDICATED BY HEADLANDS of headlands, the latter of which acts as a littoral barrier, whereas the former permits passage of drift even though no beach exists on the headland itself. Features to look for are wave cut cliffs and on which side of the headland they are located, usually marking the downdrift shore, and the existence of relatively wide stable beaches commonly found on the updrift shore.

75. Evidence At Tidal Inlets. - The location and formation of tidal inlets may also be indicative of the direction of littoral drift. In general, over a long period, such inlets tend to migrate in the direction in which littoral drift is moving. Brief reversals, associated with the shifting of the bar channel, are often observed. Natural closure and break-through at an updrift location may confuse the evidence. Figures 48 and 49 show typical inlet formations and the manner in which they indicate generally the direction of littoral drift.

76. Wave Analysis. - A complete knowledge of the directional components of wave energy acting upon the littoral zone will permit deduction of the direction and energy value of the longshore component of the primary force responsible for littoral drift. No satisfactory instrument for measuring wave direction has yet been devised, thus there are no continuing instrumental records which can be used for this purpose. Two methods have been employed for developing statistical wave data, the first being reports over a long period by ships at sea, compiled by the U.S. Navy Hydrographic Office and published as "Sea and Swell Charts." The second method, applied first by Scripps Institution of Oceanography on the California coast and later by the Beach Erosion Board for the Great Lakes and a portion of the north Atlantic coast, involves applying wave forecast techniques to produce wave statistics from historical synoptic weather maps. (See paragraph 13 to 21 on Wave Action). Both methods provide statistical data on wave heights and directions in deep water. The latter method also provides wave periods associated with height and direction, enabling energy evaluation.

77. When other reliable evidence as to predominant littoral drift direction is lacking, the longshore wave energy component has been employed for that purpose. There are two general methods for making this computation, the first by simple vector force diagrams to determine the resultant force in deep water parallel to the general shore alinement. The second method is a refinement of the first, involving projecting the deep water wave energy to a position near the shore by refraction analysis, and computing the longshore component at that point. For comparatively regular shore alinement and bottom topography, see Figure 50, there will be little difference in results from the two methods. For more complex topography, local inconsistencies are usually found. There is at present no proven technique for employing refraction analysis to determine drift direction, and as results of this method are sometimes in conflict with other more reliable evidence, its use for general practice does not appear warranted.

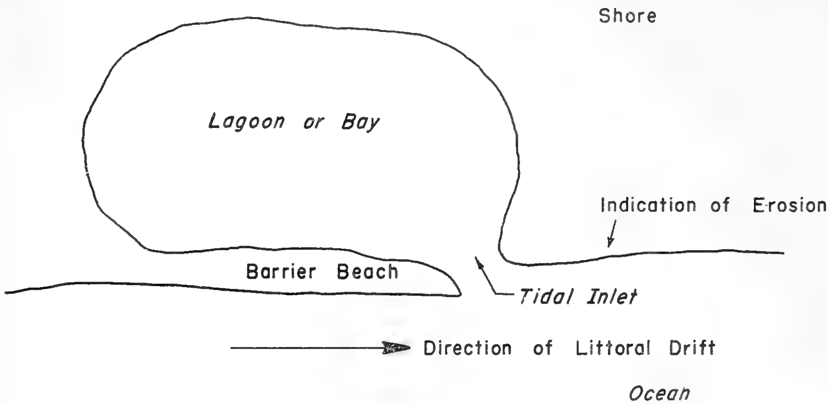


FIG. 48. TIDAL INLET TO BAY OR LAGOON

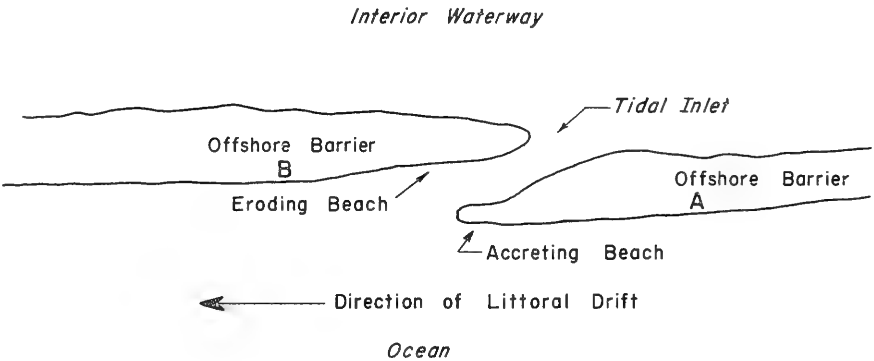


FIG. 49. TIDAL INLET THROUGH A BARRIER BEACH

DIRECTION OF LITTORAL DRIFT INDICATED BY INLETS

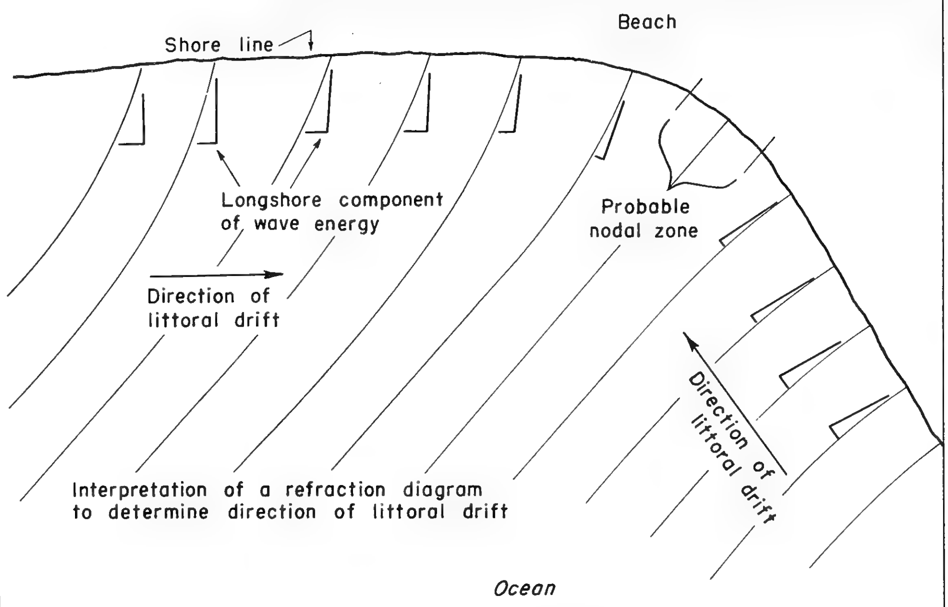


FIG. 50 — REFRACTION DIAGRAM-WAVE CONDITIONS BY ORTHOGONALS

78. Character of Materials in the Littoral Zone. - The beach, the nearshore and in some cases the offshore area to considerable depth is included in this general category. Samples of surface material in the littoral zone must be obtained in order to define its composition. Customary procedure is to obtain samples of the top 2 inches of material at the mean water line and along the profile at 5-foot depth intervals seaward as far as a regular sorting pattern along the profile is indicated. Any sampling device that will secure a representative sample without loss of fines is suitable. For sandy material, $\frac{1}{2}$-pint to 1-pint samples are usually adequate. Spacing of sampling profiles depends upon regularity of the shore and slopes, and on long straight beaches profiles quite widely spaced will usually provide adequate definition of the areal distribution of materials in the littoral zone.

79. Since the size distribution is of primary concern, all samples are subjected to mechanical analysis. Sieving or settling velocity methods are equally acceptable for this purpose. Uniform size classifications (based on Casagrande classification) approved by the Corps of Engineers, January 1952 are given in the following table.

Table 7 - Standard Size Classifications*

Name	Grain size limits (diameters)
Boulders	Above 12 inches (305 millimeters)
Cobbles	3 inches to 12 inches (76 mm to 305 mm)
Coarse gravel	3/4 inch to 3 inches (19 mm to 76 mm)
Fine gravel	** No. 4 sieve to 3/4 inch (4.7 mm to 19 mm)
Coarse sand	No. 10 sieve to No. 4 sieve (1.9 mm to 4.7 mm)
Medium sand	No. 40 sieve to No. 10 sieve (0.42 mm to 1.9 mm)
Fine sand	No. 200 sieve to No. 40 sieve (0.074mm to 0.42 mm)
Silt or clay	Passing No. 200 sieve (0.001 mm to 0.074 mm)

* Corps of Engineers Uniform Soil Classification

** U. S. standard sieve size

80. Preparatory to mechanical analysis, the samples are washed free of salt, dried, and quartered down to about 50 grams. They are then sieved or placed in settling velocity tubes and cumulative gradation curves are constructed. From these curves the grain size at the first and third quartiles and the median are read. Three general comparisons are ordinarily made, (1) the median diameter, (2) the sorting factor, and (3) the skewness. The sorting factor is defined as $\sqrt{Q_1/Q_3}$. The skewness is $(Q_1 \times Q_3)/M^2$ where Q_1 and Q_3 are the grain size at the 25 and 75 percent finer points, respectively, and M is the mean diameter, all in millimeters. Beach studies have indicated (Schalk, 1946) that as the grain size increases or decreases with seasonal changes at one location on the beach, the grain size at other locations show a corresponding increase or decrease. As the littoral zone sorting is very good on most shores the median diameter is generally an acceptable parameter.

81. If the source of littoral material is uncertain, petrographic analysis may provide evidence by comparison of mineral content on the beach and at possible sources. These sources would include beds of streams entering the ocean, friable cliffs subject to wave action, and the beaches themselves. Samples should be about 250 cc. in size or about $\frac{1}{2}$ pint. This size sample will permit both a sieve analysis and microscopic examination for mineral content.

82. The first step in the microscopic examination for mineral content is to separate the heavy minerals from the quartz and feldspar. This is accomplished by panning, or by use of heavy liquids, the electromagnet, or some special method or device. The minor accessories, or so-called "heavy minerals", even though present in very small amounts (tenth of 1 percent or less) are the important elements in determining source sands and from those an indication of the direction of littoral drift. The next step involves the identification of minerals and determination of the frequency of their occurrence. In this the percentage of heavy minerals in the sample is computed. The heavy minerals are then subdivided down to a quantity sufficient for mounting on a microscopic slide. After identification, the frequencies are determined by actual count or estimation and recorded for comparative purposes. A list of detrital and assocated minerals with comments as to occurrence and stability follows. The direction of littoral drift as indicated by petrographic analysis is determined through comparison of the mineral content and their frequency of occurrence in the beach samples with those in the samples taken from the various sources of beach material.

Table 8 - Detrital and Associated Minerals

*** Anatase (c)	*** Garnet (c)	* Olivine (r)
** Andalusite (l)	* Glauconite (c)	** Orthoclase (c)
* Apatite (r)	* Glaucophane (r)	** Plagioclase(c)
** Augite (r)	*** Gold (vr)	* Pyrite (c)
*** Barite (r)	** Gypsum (l) or (a)	** Pyrolusite (r)
** Biotite (l)	+ Hematite (a)	* Pyrrhotite (l)
*** Brookite (c)	** Hornblende (c)	*** Quartz (c)
+ Calcite (l) or (a)	** Hypersthene (r)	*** Rutile (c)
*** Cassiterite (l)	** Ilmenite (c)	+ Siderite (l) or (a)
*** Chalcedony (c)	+ Kaolinite (a)	*** Sillimanite (l)
+ Chlorite (c)	*** Kyanite (c)	*** Spinel (r)
*** Chromite (r)	*** Leucoxene (c)	*** Staurolite (c)
*** Columbite (vr)	+ Limonite (a)	*** Titanite (r)
* Cordierite (l)	** Marcasite (l)	*** Topaz (c)
*** Diamond (vr)	*** Magnetite (c)	*** Tourmaline (c)
** Epidote (c) or (a)	** Microcline (l)	*** Wolframite (l)
*** Fluorite (l)	*** Monazite (r)	*** Xenotime (vr)
	*** Muscorite (c)	*** Zircon (c)

*** Stable	(c) Common
** Moderately stable	(l) Local
* Unstable	(r) Rare
+ Stable as a secondary product	(vr) Very rare
	(a) Alternation product (secondary)

83. <u>Current Measurements</u>. - Measurements of littoral current may sometimes give an indication of the direction of littoral drift, but they require much time and are frequently unreliable. They must be made at frequent intervals over a full year to be of value, and if reversals in direction and wide velocity variations are observed, they cannot be evaluated in terms of littoral drift. The most common methods used to obtain the direction and velocity of the currents are by use of floats outside the breaker zone and fluorescein inside the breaker zone.

84. Fluorescein is a yellowish-red crystalline compound which receives its name from the brilliant yellowish-green fluorescence of its alkaline solutions. Fluorescein can be purchased at moderate cost from most chemical firms in quantities of one pound or more. A common method of employing fluorescein is to place a handful of dry sand in a paper towel, or other substance, which would disintegrate readily, with a heaping teaspoon of fluorescein crystals. The entire mass is twisted within the towel and tossed seaward into the breaker zone. As the towel disintegrates and the crystals disolve, a small patch of water is dyed an easily distinguishable brilliant yellowish-green. The movement of this patch of colored water in a longshore direction can be traced from the beach, noting the distance traveled and the time required for the travel. All measurements should be to the center of the colored area, which will gradually disperse until no longer distinguishable. From the distance moved and time for movement, the velocity of the current can be computed. Time permitting, 3 readings should be taken at each station and averaged. For the best effect, these readings should be taken twice a day at regular intervals along the beach under study, with concurrent measurements of wave height, period and direction.

85. Current measurements seaward of the breaker zone are generally made with floats. In general, these react still more erratically than does the fluorescein. In the low current velocities common to this area, wind conditions have considerable bearing on the floats unless great care is taken. Although many types of floats have been used, most of them follow the same general pattern of movement. Floats should be low in the water and designed to offer the least wind resistance and the maximum water resistance possible. The floats should be released in sequence along the entire length of a profile, spaced at regular intervals from 200 to 400 feet. Each float should have a distinctive color flag or marker to permit its identification. Locations of the floats at regular time intervals may be made by transit intersection. These measurements are generally continued through one or more tidal cycles. Because of the short time interval, this type of current measurement does not represent the seasonal changes and can be used only in connection with other observations.

86. Two general types of subsurface floats are used, the rod float and the vaned float. The rod float, of uniform dimensions, gives an approximation of the integrated values over the depth covered by the float. The vaned float, which is the type generally recommended, gives an

approximation of the current velocities at the depth of the vanes. Floats of too great length will drag on the bottom and give improper readings. Floats of comparatively short length record only surface currents. The float lengths should be varied by regular increments to permit selection of the proper length float for the varying depths along a profile. This selection of proper float lengths is based on experience at the site after a trial run. One design for vaned float is shown in Figure 51. Pegram current meters are used on extensive studies to measure bottom currents. These are not generally used in the determination of direction of littoral drift because the information gained is seldom commensurate with the cost of the operation. In some instances balls of slightly greater density than the water have been used to roll along the bottom dragging a very light line and float to mark their location.

87. Rate of Littoral Drift. - The rate of littoral drift is as important as the direction of littoral drift in the functional and structural design of shore protective structures. The rate of littoral drift can only be measured accurately at a substantially complete artificial littoral barrier. At such barriers this rate can be computed by measuring either the accretion at the updrift side of the barrier or the erosion at the downdrift side. Accretions can be measured at partial barriers, but no methods have been devised to determine what proportion of the total littoral drift is trapped by each partial barrier. Until some such method has been devised, the measurement of material trapped by groins or short jetties is a most inadequate way of determining the rate of drift. Natural littoral barriers are of little use in determining the rate of littoral drift because over geologic time the beaches either updrift or downdrift from these barriers tend to reach a condition of stability where the sand supply equals the sand losses.

88. Typical examples of essentially complete, substantial, and temporary artificial barriers are shown in Figures 52, 53, and 54. In these examples, and in all similar cases, the rate of littoral drift is determined by measuring the amount of accretion or erosion occurring during a known period of time. To compensate for seasonal changes, surveys should be taken at about the same time each year. To compensate for annual fluctuations, the period of time between surveys should be extended as conditions permit. The rate of drift should be expressed in amount of drift per unit time, usually a year.

89. Where the rate of littoral drift is to be established at a littoral barrier, the base surveys should be extended a sufficient distance updrift and downdrift from the barrier to include the entire accretion and erosion zones at the end of the study period. Where erosion is anticipated, the base line should be referenced to points at a considerable distance from the ocean as recessions of the shore line of 1,000 feet or more are not uncommon. Profiles are run from the base line seaward at least to the 30-foot depth contour, although extension of the profiles to greater depths may be required in areas of severe exposure or in the vicinity of submarine canyons. Profiles are seldom spaced in conformity

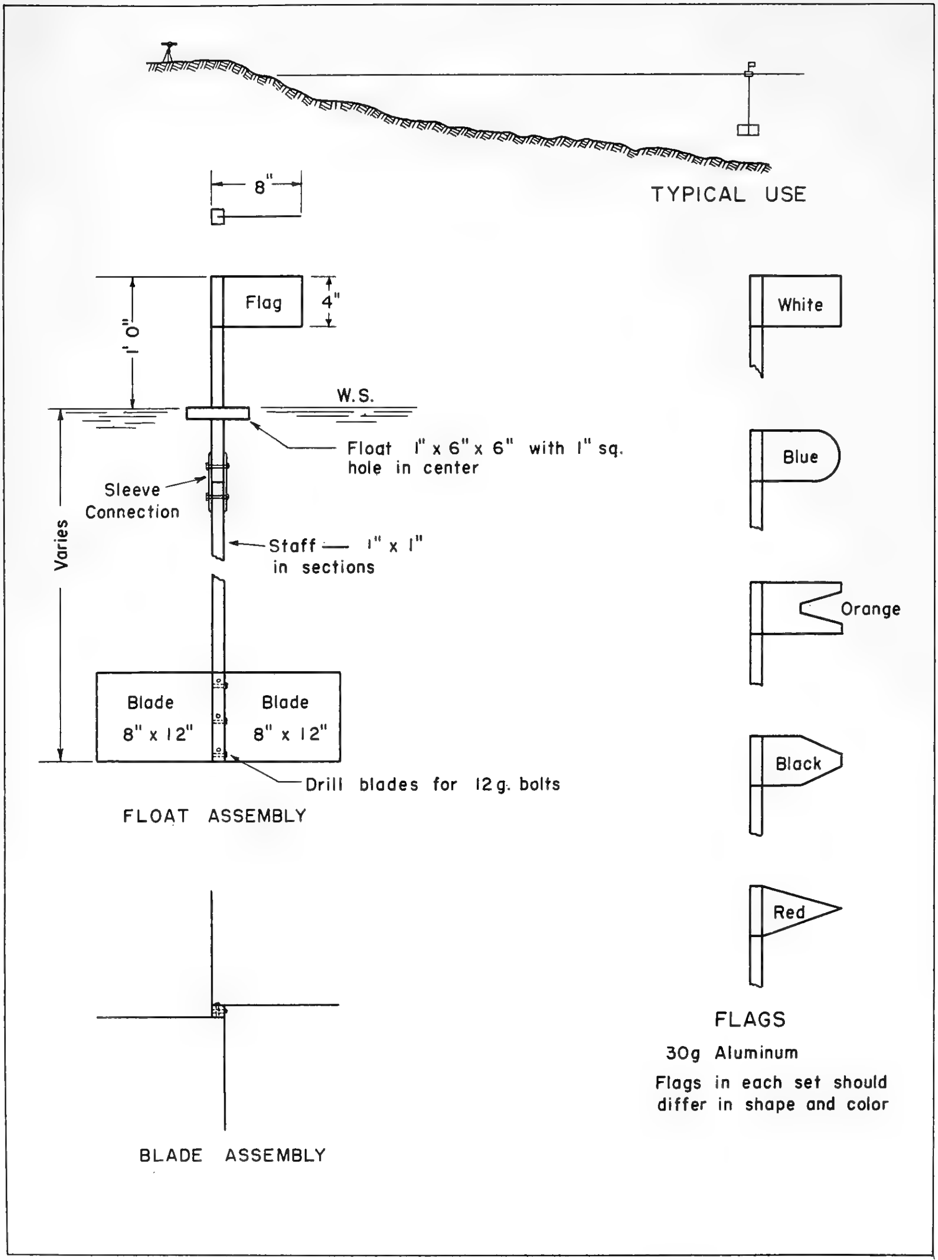


FIG. 51. TYPICAL VANE FLOAT ASSEMBLY

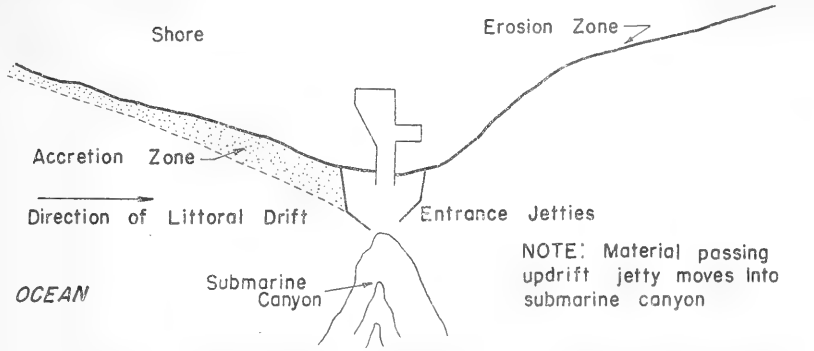


FIGURE 52. ESSENTIALLY COMPLETE LITTORAL BARRIER

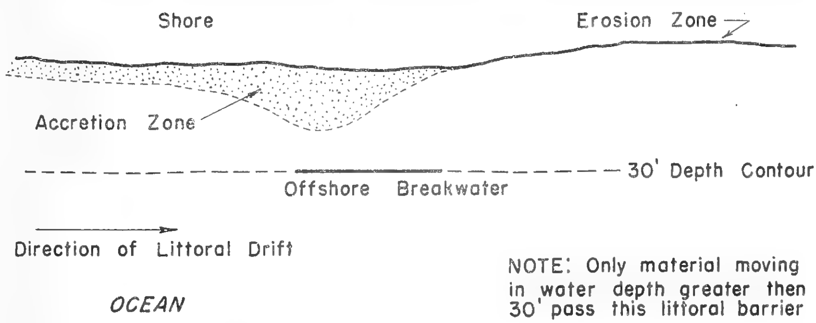


FIGURE 53. SUBSTANTIAL LITTORAL BARRIER

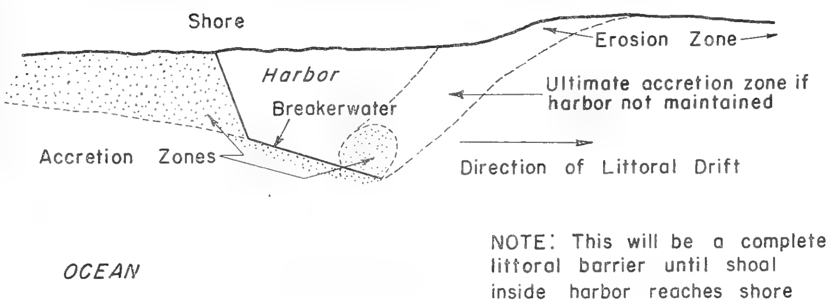


FIGURE 54. TEMPORARY LITTORAL BARRIER

with the accuracy desired and the degree of regularity of the area. (see Beach Erosion Board Technical Memorandum No. 32). Profiles may be run by any standard hydrographic method. Where the amount of such work to be done is large and continuing, sonic equipment and DUKWS may be used to advantage. Care should be taken to insure accurate vertical control and measurement as small vertical errors result in large quantity errors.

90. Measurement of accumulations on the updrift side of jetties or long groins provides a good basis for estimating the rate of littoral drift. Depth of water to which the structures extend, and the character of material trapped, must be considered in evaluating the impoundment rate in comparison with total littoral drift. Short groins provide a poor means for measuring the rate of littoral drift because the amount of material trapped is usually a small and indeterminable part of the entire quantity of littoral drift. Shoaling in entrance channels may provide a measurement of littoral drift in locations where maintenance dredging is done frequently. However, this method can seldom be used because of the difficulty of separating shoaling caused by reversals in the direction of littoral drift from that caused by the predominant drift.

91. Sources of Littoral Material. - A stable shore line is one in which the supply of littoral material to the area under consideration is approximately equal to losses of littoral material from the area. On an accreting shore line the supply of material exceeds the losses, and the reverse is true of an eroding shore line. Accordingly, the need for protective works and the choice of type of protective works to be provided are dependent on the net balance between supply and loss of littoral material.

92. The three main natural sources of littoral material to any beach segment are: (a) material moving into the area by natural littoral drift from adjacent beach areas; (b) contributions by streams; and (c) contributions by bluffs as the result of natural bluff erosion. In addition, there may occasionally be some long range net movement of material onshore apart from normal seasonal or other periodic fluctuations. These might occur, for example, with permanent or semipermanent changes in lake level. Considering coasts as a whole, maintenance of beaches must be attained at the expense of erosion of the land mass. For any individual segment of beach, the largest source of material moving into the area is generally littoral drift eroded from the adjoining updrift segment unless some major sediment bearing stream enters the segment in question or cliff or dune erosion is sufficiently rapid to provide appreciable supply.

93. Contributions by Streams. - The amounts of the various contributions to the littoral supply by sand carrying streams can be determined approximately by these general methods; (1) direct measurements, (2) studies of terrestrial sedimentation, and (3) computation of the sediment carrying capacity of the streams. To date, the only method upon which any great degree of reliance can be placed is that of direct measurement.

94. Direct measurements may be made with considerable accuracy under certain conditions. Delta measurements by successive hydrographic surveys are adequate to determine the amount contributed by those streams which carry sediment to the ocean or lake only during flash floods lasting a relative short period of time. Similar comparative surveys are adequate to determine the amount contributed by streams which bring material to the shore continuously, or over an extended period of time, where those streams terminate in navigable channels or other natural settling basins. Some correction may be necessary to account for sediment deposited outside of the channels or basin area, for material moving out of the area between surveys by natural littoral drift, and for material removed from the area artificially as maintenance of navigation channels and basins.

95. Should investigation show that the principal sources of beach-building materials are the drainage basins tributary to the shore under consideration, a detailed study of the continental geology of these basins may be required if direct measurements are impracticable. Such a study would include hydrology, physiography, stratigraphy, and sediment supply deduced from measured or estimated rates of terrestrial sedimentation.

96. For mountainous watersheds, the Forest Service has developed empirical methods for estimating the sedimentation rate (Anderson 1949). The determination was for a specific area of known geologic characteristics, but the results are valid for other areas if corrections are made for several variables; vegetation, hydrology, rock type, etc.

97. Even if there are excellent data on terrestrial sedimentation rates, it may be very difficult to estimate how much material reaches the shore. The measurement of losses must be indirect. If the streams are degrading or appear to be at grade, it can be assumed that all source material ultimately reaches the shore. But if the streams are aggrading deposition rates along the channel must be estimated and losses subtracted from the total sedimentation to determine the net sediment supply to the beaches.

98. The most accepted approach in the computation of the sediment carrying capacity of a stream is the direct measurement of the wash-load of a river by suspended-load sampling (Iowa University 1948). This method is expensive and time consuming, taking from 1 to 10 years of continuous observation to predict the wash-load of a river, depending on the regularity of its flows. Accordingly, a different method of approach has been developed based on grain size of the bed materials (Einstein 1950, 1951). Its basic concept is that bed material always moves according to the capacity of the stream. The capacity rates for bed material may be computed by formulas which were developed to permit the prediction of the individual bed-load rates of the different bed components in terms of stream discharge. The solutions are long and laborious but are not difficult to follow. However, the method would probably be used only if a determination of the stream carrying capacity

was of great importance and could not be determined by direct measurements or historic records.

99. Contributions by Bluff Erosion. - Eroding bluffs are the last major source of beach material. Along the Great Lakes this is a major source, whereas, along most of the sea coasts it is of comparative unimportance. As long as a beach berm is maintained between the bluff and the action of the waves, the bluff contributes negligibly to the littoral supply. At some locations, littoral drift has been interrupted by artificial barriers and the ocean has turned to the upland for its supply, causing serious recessions of the coast line. The amount of such contribution can only be estimated through comparative surveys, sub-division plot maps, property surveys, and statements of long-time residents of an area. The bluffs frequently contain much material too fine to remain on the beach. The proportion of beach material supplied out of total material eroded may be determined by mechanical analysis of a composite sample. Each stratum should be represented in proportion to its thickness. In the Great Lakes area, rises in lake level may allow waves to attack the bluffs, which are generally of a very friable material. This causes recession of the shore line and contributes to the supply of beach material. Where bluff erosion is important, a geological study may be required. The extent of field work and investigation will depend on the importance of bluff erosion as a source of littoral material.

100. Natural Losses of Littoral Material. - Principal avenues of loss of littoral material from a specific beach area include (a) drift of material laterally out of the area; (b) movement of material offshore into water of sufficient depth that it is lost to the littoral supply; (c) loss of material into submarine canyons; and (d) loss of material inland. Loss of material by abrasion of sand has been found of slight importance (Mason 1942).

101. Losses by Littoral Drift. - The drift of materials out of the area is measured by the net rate of drift at the downdrift end of the beach segment under study. It may be that this loss can be measured directly as outlined under rates of littoral drift. If it cannot be measured directly at a particular location, it may be possible to estimate it by considering the rates of drift at the two closest known points above and below where the rate has been established or can be measured directly. At best this is a rough estimate as the unknown factors of added supply and losses throughout the area must also be taken into consideration.

102. Movement of Material Offshore. - The quantity of material lost to the offshore depths cannot in itself be determined in the light of present knowledge. It is possible that as information on material sorting with respect to slope and wave characteristics is developed, equations may be evolved by which this important avenue of material loss may be evaluated. At present it can only be assumed as

the amount of loss remaining after all known losses have been subtracted. As the rate of material supply into an area is increased to exceed the transport capacity out of the area, or as the transport capacity along a beach decreases, either sediments accumulate along the coast line or losses occur to the offshore depths. As these deposits reduce the depth, the beach slope assumes a profile governed by the littoral forces and the beach material. Assuming material characteristics to remain constant in gradation, the profile of equilibrium would be reached when all of the beach material has been sorted roughly. Each size gradation assumes its characteristic slope, depending on the wave competence, between minimum and maximum depth limits governed by the material size gradation. Continued excess supply would advance the berm seaward without appreciable change in profile, causing deposit of sediments in greater depths.

103. Losses in Submarine Canyons. - The existence of a submarine canyon near the littoral berm provides a repository for important losses of material into the offshore depths. When combined with a jetty or breakwater, the submarine canyon may constitute an essentially complete littoral barrier by drawing off all material passing around the jetty or breakwater. Comparative surveys have been in insufficient detail to enable determination of the extent of the losses of littoral material into a submarine canyon.

104. Losses by Deflation. - As a beach widens and the expanse of permanently dry sand increases, the losses by wind deflation increase, generally resulting in the development of a dune belt immediately behind the beach. Losses by wind deflation are generally difficult to determine. In some instances this can be done by measurement of the increased dune size between successive surveys. This would generally be more costly than the information would be worth unless the problem of dune control had to be considered as well as the losses of material from the beach. In general, losses by wind deflation are not an important factor in the design of shore structures. However, because of the aspects of dune control, some attempts have been made to devise means of measuring the amount of deflation and to relate the quantities of beach sand moved to the wind velocities.

105. Experiments at the mouth of the Columbia River, (O'Brien 1936) give an indication of the order of sand losses by wind deflation. According to typical sieve analyses, these beach sands had a median diameter of 0.19 millimeter. Three types of sand traps were tried, two of which were in good agreement as to the measured rate of sand movement. Wind velocities were measured during each run at points ranging between 0.25 and 12 feet above ground. Sieve analyses of the sand caught in the traps and of the material from the surface of the beach showed variations in median diameter between 0.165 millimeter and 0.216 millimeter. The specific gravity of the sand was 2.65 and the grains were well rounded. The rate of sand movement was related to the wind velocity 5 feet above the beach. The measurements showed that when the velocity at this elevation was less than 13.5 feet per second (9 miles per hour) no movement of sand occurred,

but movement was general at this velocity and above. Figure 55 shows wind velocity gradients taken during typical runs. Figure 56 shows the relation between wind velocity and rate of sand movement. The rate of movement is in terms of the number of pounds per linear foot of beach passing a given line in one day.

SUMMARY OF FACTORS RELATED TO FUNCTIONAL PLANNING

106. As a basis for the functional planning of coastal works to attain a desired result, consideration of the foregoing factors will determine for each pertinent locality these significant facts in greater or less detail:

a. Direction and character of waves reaching locality;
b. Storms, storm waves and their effects;
c. Changes in water level;
d. The rates of littoral drift to various depths offshore;
e. Sources and losses of materials;
f. Grain size of materials along profiles;
g. Location of aggrading and eroding areas;
h. Seasonal variations related to above factors;
i. Conclusions as to dominant characteristics of shore processes.

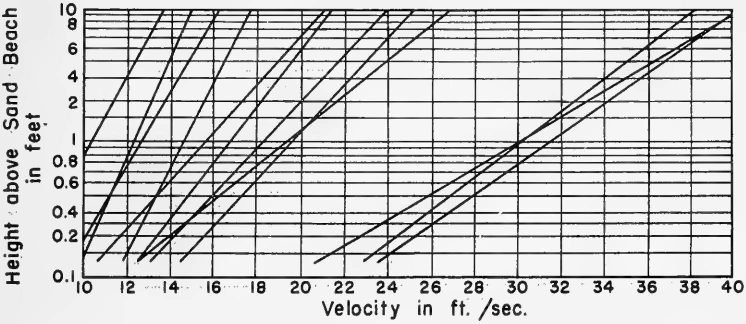
107. Many of the above factors may be expressed in a table to show relative importance at various points along the shore being studied.

FUNCTIONAL PLANNING OF PROTECTIVE MEASURES

108. Introduction. - In selecting the shape, size, and location of works the objective should be to design an engineering work which will accomplish the desired result most economically and with the least damaging effect on adjacent shore lines. The following paragraphs describe the most common engineering solutions now used to meet functional requirements, and give guides for their application.

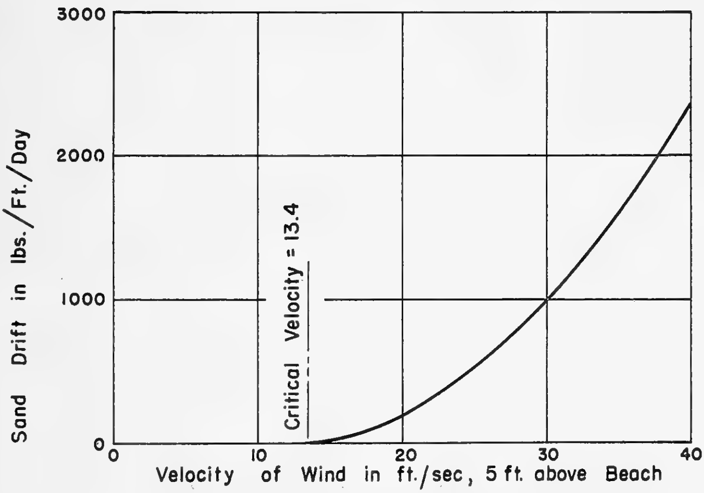
SEAWALLS, REVETMENTS AND BULKHEADS

109. Functions. - Bulkheads, revetments, and seawalls are structures placed parallel or nearly parallel to a shore line. They both separate a land area from a water area, but differ functionally in the type of protection afforded. The primary purpose of a bulkhead is to retain or prevent sliding of the land, with the secondary purpose of affording protection to the fill against damage by wave action. The primary purpose of a seawall or revetment is to protect the land and upland structures from damage by wave forces, with incidental functions as a retaining wall or bulkhead. Because of the similarity of the respective functions, the two types of structures are entirely similar in design. For example, a design for a bulkhead to resist high earth pressures at one locality could well be used for a seawall at another locale. However, for a given area, a seawall necessarily would be of heavier construction than a bulkhead



WIND VELOCITY GRADIENTS

FIGURE – 55



RELATION BETWEEN WIND VELOCITY AND RATE OF SAND MOVEMENT

FIGURE – 56

because it would have to withstand wave forces as well as static pressure forces.

110. Seawalls and bulkheads are used where there is little littoral drift along the shore and little or no protective beach, as along an eroding bluff, or where it is desired to maintain a depth of water along the shore line as for a quay or pier line.

111. Limitations. - In general, the seawall designed to withstand the forces of high waves of the open ocean, or of the Great Lakes, is a very costly structure. Because of its high cost, its use ordinarily is limited to those areas where, because of the value of the upland and improvements thereon, it is essential to maintain and protect the shore line in a fixed position. In many instances, the construction of a wall may be accompanied by loss of beach since no seawall is a cure for the cause of erosion but rather a defensive work to mitigate certain resulting effects without regard for other effects. Where the beach is of prime importance, the seawall must be augmented by companion works, and may even be undesirable.

112. At those localities where minor fluctuation of the shore line would not be harmful, the economic aspects of the high cost seawall should be studied with a view to determining the desirability of some less costly means of providing necessary protection. Further, the seawall provides protection only to the area immediately behind it. On an eroding shore, its construction tends to shift the focus of erosion to the downdrift end of the seawall. Extensions of the seawall have the same effect. Where this condition arises, consideration must be given to other methods to provide the protection throughout the entire littoral segment.

113. Functional Planning of the Structure. - The planning of seawalls and bulkheads is a relatively simple processes, since their functions are restricted to the maintenance of fixed boundaries. The features which must be analyzed in adequately planning such a structure are: the use of the structure and its overall shape, its location with respect to the shore line, its length, its height, and often the ground level in front of the wall.

114. Use of the Structure. - The shape to be chosen for a seawall must be determined by consideration of collateral uses of the wall. Wall profile shapes may roughly be classed as: vertical or nearly vertical face, sloping or curved convex seaward face, concave curved seaward and re-entrant face, or stepped face. Each silhouette has certain functional applications and so may be used in combination with any other if diverse functional criteria are to be met.

115. A vertical or nearly vertical face wall lends itself to use as a quay wall or landing place, where other wall shapes need to be provided with additional work to be so adapted. In addition, especially with the lighter types of walls, a vertical face wall (sheet pile, for example) may with one exception be constructed more quickly and often

more cheaply than any other type. This may be an important consideration where emergency protection is needed. Against wave attack, and specifically in regard to reduction of overtopping, a vertical face wall is more effective than any but the concave curved and re-entrant face walls.

116. A backward sloping or convex curved face wall, or revetment is the least effective of all types against wave attack for a given height of wall. It is, however, more adaptable to use as emergency protection, (sand bag, or dumped stone mounds, for example) than the other types. Actually the use of such a wall type should be restricted to those areas in which wave overtopping is not a problem, or where esthetic, emergency, or structural considerations prohibit the use of other wall shapes.

117. The concave curved or re-entrant faced walls are the most effective in reducing wave overtopping to a minimum. Where the wall crest is to be used (for a roadway or promenade for example), a wall so designed will be of the most desirable shape for protecting the crest. This is especially true if the beach is narrow or entirely absent, and the water level is over the base of the wall during high or storm tides.

118. Stepped face walls provide the most ready access to beach areas from protected areas and in addition act to disrupt the scouring action of the wave backwash.

119. Location of Structure With Respect to Shore Line. - Ordinarily the location of a seawall or bulkhead is determined more by the needs of the locale than by any consideration of possible wave attack. In general, a seawall would be constructed along that line landward of which further recession of the shore line cannot be permitted. Where an area is to be reclaimed, the seawall would be constructed along the seaward edge of the reclaimed area. A seawall constructed in the water, isolated from shore, becomes an offshore breakwater.

120. Length. - A seawall, revetment or bulkhead protects no more than the land and improvements immediately behind it. No protection is afforded either to upcoast or downcoast areas as is the case with beach fills. However, it must be emphasized that where erosion may be expected to occur at either end of a structure, wing walls or tie-ins to adjacent land features must be provided to prevent flanking and possible progressive failure of the structure from the ends. Short term beach changes due to storms, as well as seasonal and annual changes must be accounted for. It must be remembered that changes updrift from a seawall will continue unabated after the wall is built, and that downdrift, these changes will be, if anything, intensified.

121. Height of Structure. - Seawalls can be designed to be of such height that no water would overtop the wall regardless of wave attack. However, it is not ordinarily economically feasible to do so, and certain lesser criteria must be adopted. For example, if it is desired to prevent overtopping water which has damaging horizontal momentum, the wall crest

height of vertical, concave curved, or re-entrant face walls above the low water datum may be set at

$$h_c = h_t + 0.7\ H_b \tag{15}$$

if the wall is located in or landward of the breaker zone, and

$$h_c = h_t + 0.6\ H \tag{16}$$

if the wall is located seaward of the breaker zone

where h_c is the wall crest elevation above mean low water;
h_t is the highest still water level above mean low water expected at the structure; (this may be due to tides, seiches, wind set-up; etc.);
H is the wave before breaking (in the absence of any structure);
H_b is the wave height at breaking (in the absence of any structure).

122. Stepped or sloping faced seawalls should have a somewhat higher crest elevation as their shape is conducive to permitting water to overtop them with an effective horizontal momentum. For these, the wall crest elevation may be set at

$$h_c = h_t + 1.3\ H_b \tag{17}$$

123. Relative effectiveness of vertical and curved re-entrant face walls of heights less than required in equations (15) and (16) may be found by reference to Figure 57. Considering a wall designed as in equation (15) and (16) as being 100 percent efficient in preventing overtopping, the percent of incident wave height transmitted by walls of lesser height is shown by the solid lines of figure (57). It must be noted that these curves are approximate only.

124. Bulkheads so located as to have a permanent beach berm to protect them from the direct impact of the waves may have their crest height reduced to a minimum of 2 feet above the height of maximum wave uprush or to the height of fill the bulkhead is designed to retain.

125. <u>Determination of Ground Elevation Before the Wall</u>. - Seawalls are usually built to protect the upland area from the effects of continuing erosion as well as to protect shore improvements from damage by wave attack. Although the exact nature of the effect of such a wall on the processes of erosion cannot be determined (in certain instances these processes seem to have been halted or reversed), for safety in design they must be considered to continue. A rigorous determination of the beach profile that will obtain after the construction of the wall being impossible, approximate methods must be relied upon.

126. One such method is to assume that a wall would have no effect

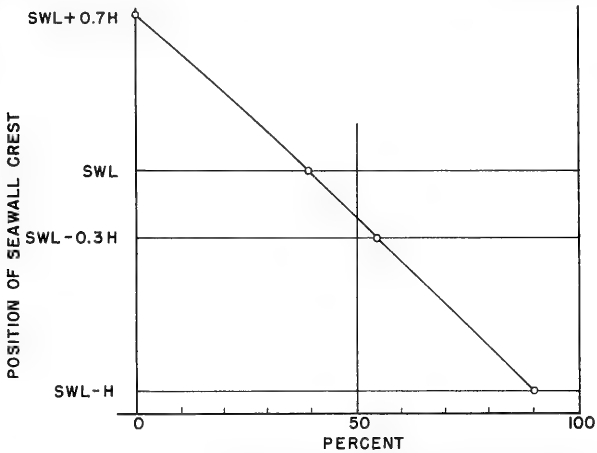


FIGURE 57 A. PERCENT WAVE HEIGHT TRANSMISSION OF SEAWALLS IN THE BREAKER ZONE

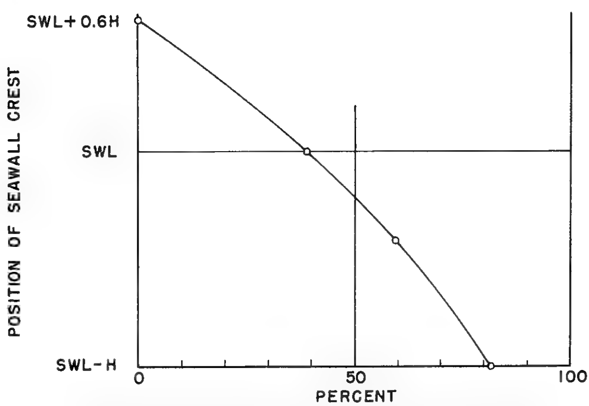


FIGURE 57 B. PERCENT WAVE HEIGHT TRANSMISSION OF SEAWALLS SEAWARD OF THE BREAKER ZONE

FIGURE 57

on the erosion regime before it. In other words, the beach seaward of the wall would erode in the same manner as if the wall were not there. Since the determination can only be very approximate, rules of thumb may be adopted.

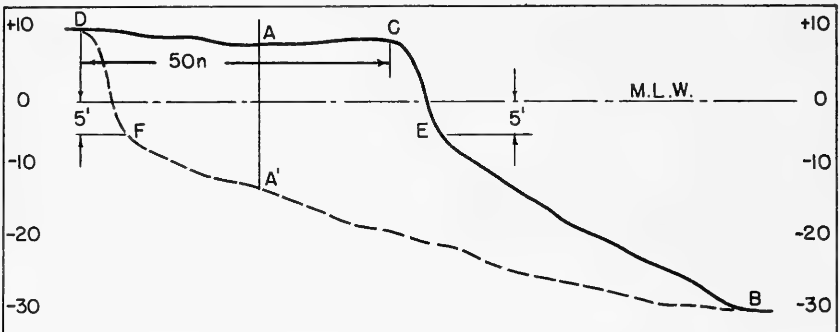


FIGURE 58 — Effects of erosion

127. Consider a beach as shown in Figure 58 where the line DACEB represents an average profile. It is desired to place a seawall at point A as shown. From prior records, either the loss of beach width per year, or the annual loss of material over an area which includes the profile is known. In the latter case, the annual quantitative loss may be converted to an annual loss of beach width by the rule of thumb "loss of 1 cubic yard of beach material is equivalent to loss of 1 square foot of beach area at the berm."

128. In general, beach slopes are fairly steep shoreward of a depth of 0.to 10 feet, and fairly flat seaward of that depth. Analysis of beach profiles on eroding beaches would indicate that it may reasonably be assumed that the beach seaward of a depth of 30 feet will remain essentially unchanged, that the point of slope break will remain at about the same elevation, and that the profile of the beach shoreward of the point of break in slope will remain essentially unchanged, accordingly the ultimate depth at the wall may be estimated as follows:

a. In Figure 58 let B represent the beach at a water depth of 30 feet, E the beach at the point of slope break (say) at a depth of 5 feet, and C the present position of the berm crest. If at A it is desired to built a structure whose economic life is (say) 50 years, and it is found that n is the annual loss of beach width at the berm, then in 50 years without the wall this berm will retreat a distance 50 n to point D;

b. From D to the elevation of point E draw a profile DF parallel to CE, and connect points B and F. This line DFB will represent the approximate profile of beach after 50 years, without the presence of the wall. The new beach elevation at the wall's location will be approximated by point A'. Similar calculations may be made for anticipated short time beach depredations caused, for example, by storms.

PROTECTIVE BEACHES

129. Functions. - Beaches are the most effective means of dissipating wave energy. They provide protection to upland property, while maintaining full recreational use of the shore area. They may be placed artificially or may be accumulated by groins. Direct placement of a beach fill involves the artificial deposition of a sand fill directly in front of the specific reach of upland the fill is designed to protect, with secondary consideration being given to the beneficial effects of the deposit on adjacent areas. Conversely, artificial nourishment involves provision of a sand supply to augment the natural supply of littoral material. Protection is thereby provided to long reaches of upland through maintenance of protective beaches by natural forces, with secondary consideration being given to the beneficial use of the sand supply itself.

130. For artificial nourishment, the supply is ordinarily in the form of a feeder beach, which, in eroding through the action of littoral forces, increases the quantity of material in littoral transport. Attempts have been made at Long Branch and Atlantic City in New Jersey and at Santa Barbara in California to nourish an eroding beach by depositing material in relatively deep water, depending on the action of natural wave forces to move the material shoreward to the beach. Although observations made in these tests indicate that this method will not provide nourishment at a suitable rate to justify its general use, observations over a longer period of time may indicate a benefit to some portion of the shore, possibly at some distance from the location of deposit.

131. Limitations. - The provision of beaches for protection of upland areas and for beneficial use has two distinct limitations; (1) beach material must be available for initial construction and subsequent maintanance, and (2) the annual cost of maintenance is generally high, unless a considerable length of shore is involved.

132. The efficacy and comparatively low total cost of shore protection through placing and maintaining beach is predicated on the availability of adequate supplies of beach material within economic pumping distance of the beach. If suitable beach material is not readily available, some other manner of protection may be required. Where projects are planned for long periods, (say) 50 years, the total quantity of material, which would have to be readily available, could conceivably approach 50,000,000 cubic yards, depending on the rate of erosion or loss of material from the area.

133. Protection by provision of widened beaches generally results in low initial costs but comparatively high maintenance or replacement costs, especially in the case of limited frontages. Other methods of protection generally have higher initial costs, but often have lower apparent annual charges. The choice must be made between high debt charges and high maintenance costs. In many instances a community may be willing to bond itself for a high first cost, but the community's governing body cannot commit succeeding bodies to continuing maintenance costs. Where maintenance is discontinued, either by choice or necessity, residual protection after this time cannot be expected. Where annual costs of replenishment may be excessive, the economic justification of groins to reduce the rate of loss of fill should be investigated. Since maintenance material placed at the updrift limit of an uninterrupted segment may maintain the entire segment, the cost per unit length is usually comparatively low for long reaches of shore. The method is therefore, most suitable for large scale operations, rarely so for small scale operations.

134. Tidal Zone Slopes. - It is desirable to estimate in advance of placement of a fill, the slope the fill may be expected to assume. Field observations of existing beaches in the area or in nearby areas should always be made if possible; although complete data on offshore slopes are lacking, some information is available on slopes in the vicinity of the mid-tide line. Figure 59 is a plot of slopes at approximately the mid-tide line versus median grain diameter of the material at this point. The median grain diameter of material which has been exposed to wave action will ordinarily be coarser (due to selective transport by wave action) than the median diameter of the material before placement. However, slopes deduced on the basis of this last diameter may not be expected to differ widely from those based on the first diameter. As seasonal, storm or other short term changes have been neglected in Figure 59, large but temporary slope changes may be anticipated. To use the curves of Figure 59, the approximate foreshore slope may be read directly from the graph based on the median diameter of the material available for placement.

135. Size of Beach Fills. - Basically, the two primary functions of any placed beach fill are (1) protection of upland areas and (2) beneficial use of the beach fill. A beach deposit must be planned with a residual width sufficient for recreational or other uses for which the fill may be intended. This width will generally be established by the city planner or other agency planning the development of the beach and upland. If this width must be maintained, the protective fill must have

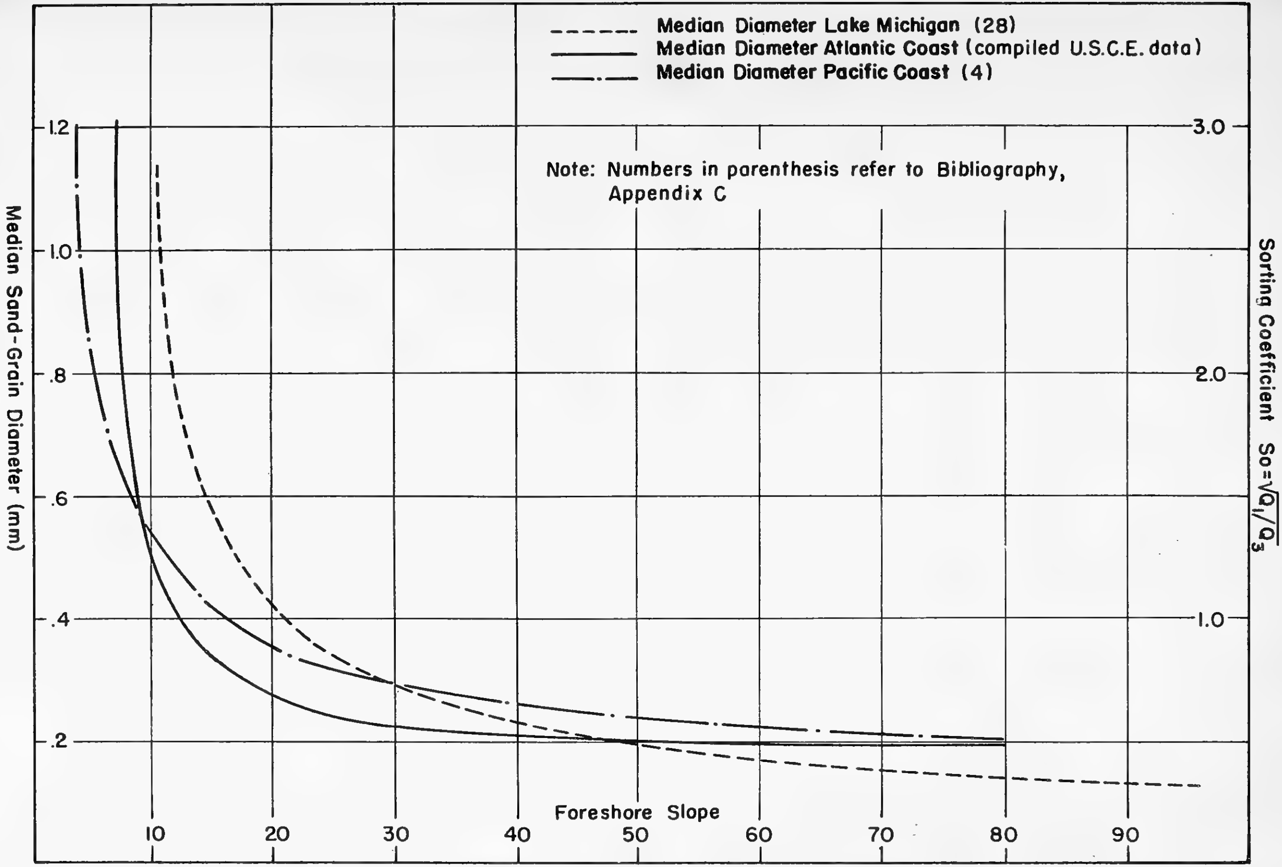


FIG. 59. RELATIONSHIP BETWEEN GRAIN SIZE AND FORESHORE SLOPE

sufficient excess width to provide for net losses of material from the area between maintenance periods. Still additional width should be provided for such seasonal changes as offshore and onshore movement of material.

136. The width of beach required to provide for net losses out of an area can be estimated after the annual rate of recession has been determined from comparative beach profiles. (A comparison of rates of drift into and out of an area can be used but is not as reliable unless all other losses and gains are accounted for.) The annual loss of material is computed either from the profiles or directly from contours. The total quantity of material required may be computed as the annual loss times the number of years between maintenance periods. Protective beach width required can then be roughly estimated from the equation

$$W = Q/L \tag{18}$$

where W = required width of beach, in feet

Q = total quantity of material lost from the area in the period between maintenance operations in cubic yards

L = length of beach to be filled, in feet.

137. It should be noted that this method takes no account of possible seasonal or other short term beach losses. Seasonal changes up to 150 feet are not uncommon. Accordingly the additional width of beach required for protection against short term losses must be based on experience at the site in question.

138. Size of Feeder Beaches. - A feeder beach is designed to provide for the net losses out of the feeder area only. The required size of a feeder beach may be determined by multiplying the annual net rate of littoral loss by the number of years between maintenance periods. Beach width may be determined in the same manner as for beach fills. By regular augmentation of supply of beach material to the extent of the wave competence to move it, erosion or other wave damage to the downdrift shore line may be prevented.

139. Sand Dunes. - Sand dunes perform the function of beach fills in preventing waves from reaching the upland, but are located landward of the beach itself. A belt of sand dunes will provide effective protection to improved upland property as long as the tops of the dunes remain above the limit of wave uprush. At those locations which have an adequate natural supply of sand and which are subject to innundation by storm tides and high seas, a belt of sand dunes may provide more effective protection, and at lower cost, than either a bulkhead or seawall.

GROINS

140. Definitions. - A groin is a shore protective structure devised to provide, build or widen a protective beach by trapping littoral drift or to retard loss of an existing beach. It is usually perpendicular to the shore, extending from a point landward of possible shore line recession into the water a sufficient distance to stabilize the shore line at a desired location. Groins are relatively narrow in width (measured parallel to the shore line) and may vary in length from less than 100 feet to several hundred feet.

141. Groins may be classified as permeable or impermeable, high or low, and fixed or adjustable. They may be constructed of timber, steel, stone, concrete, or other materials, or combinations thereof. Impermeable groins have a solid or nearly solid structure which prevents littoral drift passing through the structure. Permeable groins have openings through the structure of sufficient size to permit passage of appreciable quantities of littoral drift. Some permeable stone groins may become impermeable with heavy marine growth. A series of groins acting together to protect a long section of shore line is commonly called a groin field.

142. Groins differ from jetties in that jetties generally are larger with more massive component parts, and are used primarily to direct and confine the stream or tidal flow at the mouth of a river or entrance to a bay and prevent littoral drift from shoaling the channel. In some sections of the country groins are commonly referred to as jetties or piers.

143. Theory of Groin Operation. - The manner in which a groin operates to modify the rate of littoral drift is approximately the same whether it operates singly or as one of a field. As discussed under "Littoral Drift", the beach material so referred to moves approximately parallel to the shore line as a broad band extending from the limit of wave uprush to some depth of water greater than 30 feet. A groin interposes a total or partial barrier to littoral drift moving in that part of the band between the seaward end of the groin and the limit of wave uprush. The extent to which the littoral drift is so modified depends on the height, length, and permeability of the groin. Groins are most effective, economically, when the largest part of the littoral movement is close inshore.

144. A typical groin is illustrated in Figure 60. In this figure, the groin extends from some distance landward of the top of berm to the 6-foot depth contour. The net direction of wave attack, as typified by the orthogonals shown, is such as to cause a net movement of littoral drift in a downcoast direction. The crest of berm and 6-foot depth contour are represented by e a i and g c h, respectively, occurring in a state of nature prior to the construction of the groin a b c. Prior to the construction of the groin the offshore beach slope had stabilized between a and c in a manner dependent on the median diameter of the beach material and the type of wave attack.

145. With the construction of the groin a b c, the tendency is for the updrift beach to accrete as littoral drift is intercepted, and for the downdrift beach to erode as the normal supply of material is withheld by the groin. Now at the seaward end of the groin, the sand elevation on either side of the groin will remain the same so that any advance on the updrift side must be accompanied by a steepening of the beach as between b and c. Conversely, any recession on the downdrift side of the groin must be accompanied by a flattening of the beach as between d and c.

146. The changes in beach slopes are directly related to two principles governing the movement of beach materials; (1) the beach slope is a function of grain size, e.g. for the same wave conditions, the coarser the beach material the steeper the slope on which it will stand; (2) the beach slope is a function of the wave characteristics, e.g. for the same beach material, high steep waves cause offshore movement of material and flatter slopes, whereas low long waves cause onshore movement of material and steeper slopes.

147. Referring to the updrift side of the groin, Figure 60 with the interception of littoral drift the shore line begins to advance from a to b resulting in an unstable slope. When the slope becomes unstable, the waves attempt to recreate a stable condition by moving material offshore to c whence it can move on downcoast or farther offshore. In this offshore movement, the coarser fractions of the sand are left behind resulting in a material having a larger median diameter and a steeper stable slope. This action tends to continue until all of the material in littoral drift is passing around or over the groin or until the updrift beach has reoriented itself to a position normal to the wave orthogonals and the downdrift movement of material shoreward of c comes to a halt for all practical purposes.

148. With the reorientation of the shore line, the waves cease to diverge as they approach shore which results in slightly higher wave action. This higher wave action tends to move additional material offshore thus slowing the advance of the shore line or accelerating the gradation of the beach material. This combined action continues until a balance has been reached between grain size and beach slope.

149. On the downdrift side of the groin, the shore line starts to recede from a towards d as the normal supply of material is interrupted. As the elevation remains constant at c, this results in a flatter slope and the beach is again unstable but in an opposite direction than on the updrift side. With the flatter slopes, the waves are competent to move material into the eroding area from seaward of c h. This action somewhat nullifies the tendency of the shore line to recede. Recession of the downdrift shore line will continue until sufficient material passes around or over the groin to stabilize the shore line.

150. Figure 61 shows the general position of the shore line to be expected for a field of 2 or more groins. The positions shown assumes

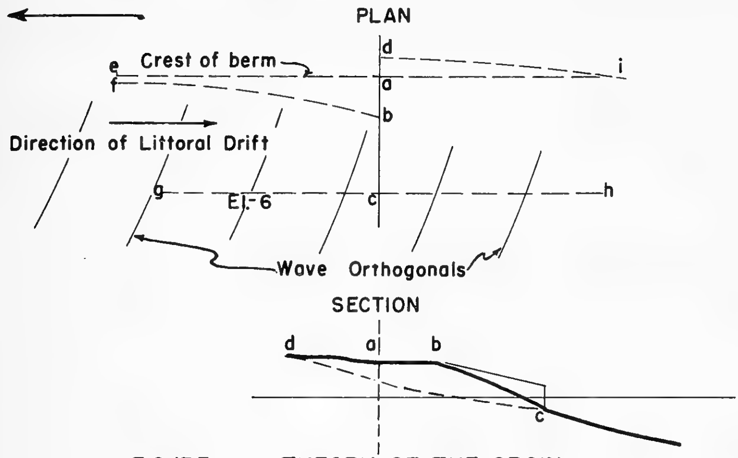


FIGURE 60. THEORY OF THE GROIN

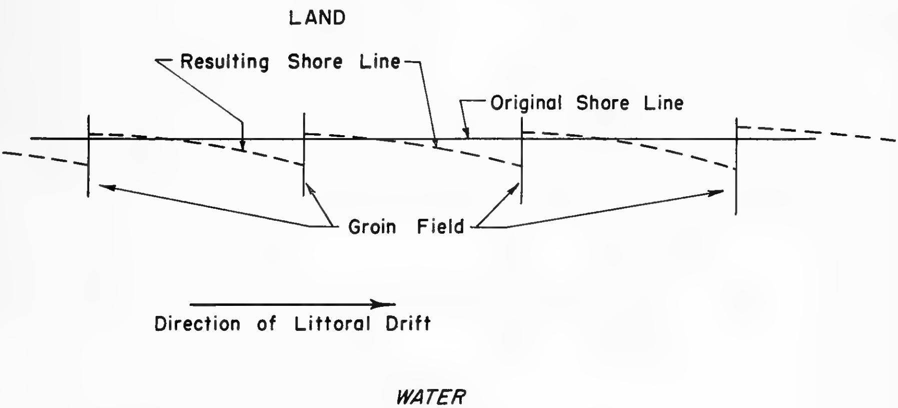


FIGURE 61. DESIRED EFFECT OF GROIN FIELD

a well established net littoral transport in one direction.

151. Purpose. - Under some conditions and subject to definite limitations imposed on their use, groins may be used to:

a. Stabilize a beach subject to intermittent periods of advance and recession;

b. Provide upland protection through prevention of removal of a protective beach;

c. Reduce the rate of littoral drift out of an area by re-orienting a section of the shore line to an alinement more parallel to the wave crests;

d. Build or widen a beach by trapping littoral material;

e. Prevent loss of material out of an area by compartmenting the beach;

f. Prevent advance of a downdrift area by acting as a littoral barrier; and

g. Stabilize a specific area by reducing the rate of loss from the area.

All of the foregoing ends are attained through modification of the rate of littoral drift. Also, all are forms of shore protection.

152. Limitations On The Use of Groins. - Because of their limitations, the use of the groins as a major protective feature should be decided upon only after careful consideration of the problem and the many factors involved. Principal limitations on the use of groins include:

a. Because groins function by modifying the rate of littoral drift, they serve no useful purpose in an area of no wave action, or where the waves are parallel to the beach because there will be little or no littoral drift;

b. Even where the wave action is such as to set up a strong littoral current, there must be a continuing supply of littoral material from updrift of the groin or groin field under consideration for it to function properly;

c. Where the quantity of material in the littoral stream is small and there is a delicate balance between stability and erosion, the interruption of this drift may cause damaging erosion to the shore line downdrift of the groin;

d. Caution must be used that a single groin, or the downcoast groin of a field of groins, is securely anchored at the shore end, so that it will not be flanked by downcoast erosion;

e. In providing protection by groins, it must be remembered that the alinement of the shore adjacent to a groin is dependent upon the direction of wave attack and will vary. If minor fluctuations of the shore cannot be allowed a solution other than the use of groins must be found; and

f. Where the supply of littoral material is insufficient to permit the withdrawal from the littoral stream of enough material to fill the groin or groin field without damage to downdrift areas, artificial fill may be required to fill the groin or groin field and thus permit the natural littoral supply to pass without interruption.

153. Permeability of Groins. - One criticism of the use of groins is that in withdrawing material from the littoral stream to build an accretion of sand on the updrift side of a groin, this amount of sand is lost from the downdrift side, often causing severe erosion. Many attempts have been made to design a groin with the proper degree of permeability to pass sufficient sand through the groin to maintain the alinement of the downdrift shore line or lessen the erosion, at the same time reducing the scouring effect around the end of the groin. The generally accepted method of designing a permeable groin is to increase the degree of permeability from the bottom to the top and from the shore to the seaward end of the groin. No agreement has been reached as to the amount of permeability needed. Although permeable groins have been built on the Great Lakes, at Rockaway Beach, New York, in Florida, and at other locations, their effectiveness as compared to impermeable groins has not been proven. At many locations they have failed to trap sand; at some other locations where accretions have occurred, it is problematical whether or not the accretions would not have occurred without the groins.

154. High and Low Groins. - The amount of sand passing a groin depends to some extent on the height of the groin. Groins based on a headland or reef, or at the entrance to a bay or inlet where it may be either unnecessary or undesirable to maintain a sand supply downdrift of the groin, may be built to such a height as to completely block the passage of all material moving in that part of the littoral zone covered by the groins. Where it is necessary to maintain a sand supply downdrift of a groin, it may be built to such a height as to allow overtopping by storm waves, or by waves at high tide. Such low groins serve the same purpose as that intended by designers of permeable groins, and are more positive in operation.

155. Adjustable and Fixed Groins. - The great majority of groins are fixed or permanently built structures. In England, the Case and Du-Plat-Taylor adjustable groins have been used with reported success. These groins are essentially adjustable batter boards between piles, with a raising and lowering device so that the groin can be maintained at a fixed height (usually one to two feet) above the sand level, allowing a considerable part of the sand to pass over the groin and maintain the downdrift

beach. Adjustable groins are reported to be particularly useful where an attempt is being made to widen a beach with a minimum of erosion damage to the downdrift area. However, they are effective only where there is an adequate supply of littoral material.

156. <u>Dimensions of Groins</u>. - The width and side slope of a groin depend on the wave forces to be withstood, the type of groin, the materials with which it is constructed, and the construction methods used. These features are considered under structural design. The length, profile and spacing are important considerations with respect to functional success.

157. The length of a groin is determined by the depths in the offshore area and the extent to which it is desired to intercept the littoral stream. The length should be such as to interrupt such a part of the littoral drift as will supply enough materials to create the desired stabilization of the shore line or the desired accretion of new beach areas. Care must be exercised that these ends are attained without a corresponding damage to downdrift areas. For functional design purposes, a groin may be considered in three sections:

a. The horizontal shore section;

b. The intermediate sloped section; and

c. The outer section.

158. <u>The Horizontal Shore Section</u>. - This section would extend from the desired location of the crest of berm as far landward as is required to anchor the groin to prevent flanking. The height of the shore section depends on the degree to which it is desirable for sand to overtop the groin and replenish the downdrift beach. The minimum height of the groin is the height of the desired berm, which is usually the height of maximum high water that occurs frequently plus the height of normal wave uprush. The maximum height of groin to retain all sand reaching the area (a high groin) is the height of maximum wave uprush during all but the least frequent storms. This section is nearly horizontal or sloped slightly seaward paralleling the existing beach profile or the desired slope in case a wider beach is desired or a new beach is to be built.

159. <u>The Intermediate Sloped Section</u>. - The intermediate section would extend between the shore section and the more or less level outer section. This part of the groin should approximately parallel the slope of the foreshore the groin is expected to maintain. The elevation at the lower end of the slope will usually be determined by the construction methods used, the degree to which it is desirable to obstruct the movement of the material, or the requirements of swimmers or navigation interests.

160. <u>The Outer Section</u>. - The outer section includes all of the groin extending seaward of the sloped section. With most types of groins, this

section is horizontal with a height determined by the method of construction. Otherwise, the height of this part of the groin also depends upon the extent to which it is desirable to withhold sand from the littoral stream. With this as a controlling factor, the outer section of the groin may be either level or parallel to the bottom, with a top elevation as low as 4 feet above the bottom. The length of the outer section will depend upon the slope of the beach and the extent to which it is desired to interrupt the littoral drift.

161. Spacing of Groins. - As a long section of shore line must be protected by more than one groin, a group, or field, of groins must be used. The spacing of groins is a function of the length of the groin and the wave direction which would cause the net rate and direction of littoral drift. The length and spacing of these groins must be so correlated that when the groin is filled to capacity the fillet of material on the updrift side of each groin will reach to the base of the adjacent updrift groin with sufficient margin of safety to maintain the minimum width beach desired or prevent flanking of the updrift groin. Figure 62 shows the desirable resulting shore line if groins are properly spaced. The solid line shows the shore line as it will become shortly after construction of the groin field, when erosion is a maximum at the toe of the updrift groins. This assumes both groins were constructed simultaneously. The erosion shown occurs before the updrift groin is full and material begins to pass around and over it into the area between groins. At the time of maximum recession, the solid line is nearly normal to the direction of net wave approach and the triangle of recession "a" is approximately equal to the triangle of accretion "b". In other words there has been a shifting of material between the groins. The dotted line m-n shows the stabilized shore line which will obtain after material passes the updrift groin to fill the area between groins and in turn commences to pass the downdrift groin. It will be noted that the fillet of sand between groins tends to become and remain perpendicular to the predominent direction of wave attack. This alinement may be quite stable after equilibrium is reached. However, if there is a marked seasonal variation in the direction and intensity of wave attack there will be a corresponding seasonal variation in the alinement and slope of the fillets between groins. In areas where there is a periodic reversal in the direction of littoral drift, an area of accretion may form on both sides of a groin as shown in Figure 63. Between groins the fillet may actually oscillate from one groin to the other as shown by the dotted lines or may form a U-shaped beach somewhere in between depending on the rate of supply of littoral material. With regular reversals in littoral drift, the maximum line of recession would probably be somewhat as shown by the solid line, with the triangular area (a) + triangular area (c) approximately equal to the circular segment (b). The extent of probable beach recession must be taken into account in establishing the length of the horizontal shore section of groin and in estimating the minimum width of beach that may be built by the groin field.

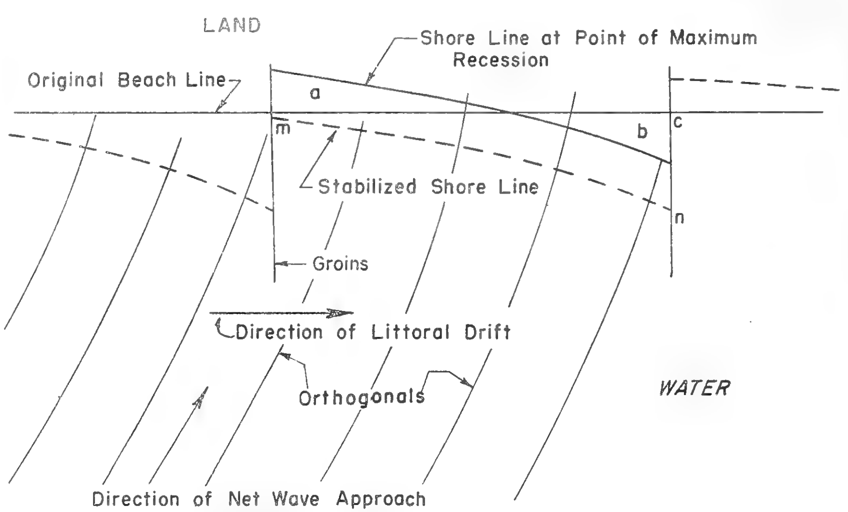


FIGURE 62. GROIN FIELD OPERATION

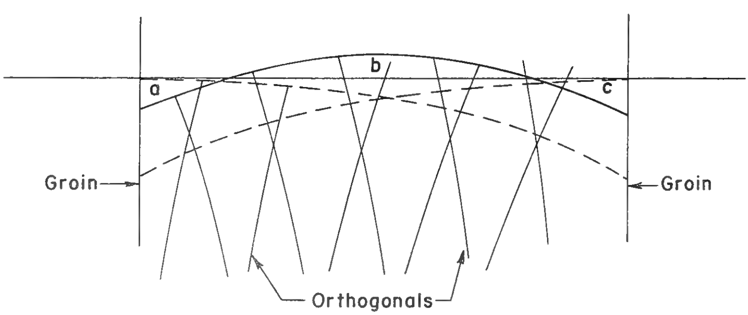


FIGURE 63. GROIN FIELD OPERATION WITH REVERSAL OF DRIFT

162. Length of Groin. Before the total length of a groin and the position of the shore line adjacent to a groin, can be determined, it is necessary to determine the location of the ground line, or stabilized beach profile, on each side of the groin. The two ground lines are determined separately and then combined to determine the relative positions of the shore line.

163. Determination of the ground line on the updrift side of a groin is the same whether there is one groin or several in a groin field. The steps involved for a typical groin are:

a. Determine the original beach profile in the vicinity of the groin;

b. Determine the conditions of littoral drift;

c. Plot a refraction diagram for the mean wave condition: i.e. the wave condition which would produce the predominant direction and net rate of littoral drift;

d. Determine the minimum width beach desired updrift of the groin. This may be a width desired to provide adequate recreational area, adequate protection to the upland area, or in the case of a groin field, adequate depth of beach at the next groin updrift to prevent flanking of this groin by wave action. The latter criterion is depicted at point m on Figure 62, if line m-n represents the berm crest of the beach.

e. The position and alinement of the beach to the groin under study is indicated by the line m-n, Figure 62 drawn approximately normal to the orthogonals based on mean wave conditions from m to n;

f. Apply the distance c-n from Figure 62 to Figure 64 and this distance, plus sufficient groin landward of c to prevent flanking, will represent the length of the horizontal shore section;

g. The slope of the ground line from the crest of the berm seaward to about the mean low water line will depend upon the gradation of the beach material and the character of the wave action. This section of groin , the intermediate sloped section, Figure 64, is usually designed parallel to the original beach profile. The ground line will assume the slope of the groin section n-p or, if the material trapped is coarser than the original beach material, will assume a steeper slope. The length of the outer section p-r depends upon the amount of littoral drift it is desired to intercept. It should extend to sufficient depth so that the new profile p-s will intercept the old profile c-d-s within the toe of the groin;

h. The final ground line on the updrift side of the typical groin shown in Figure 64 is indicated by the line c-n-p-s.

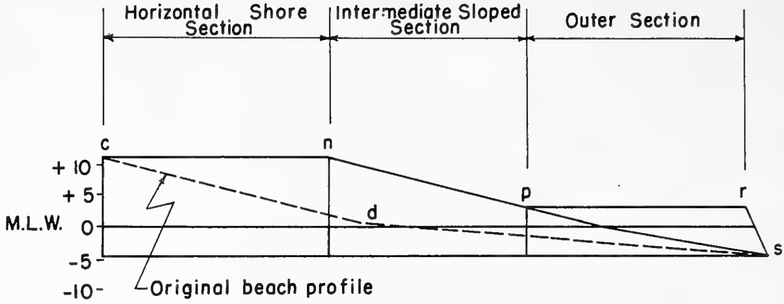
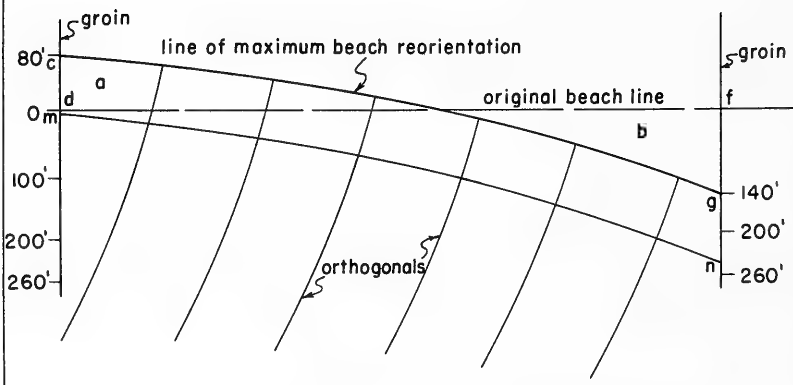


FIGURE 64



RECESSION AND ACCRETION BETWEEN GROINS

FIGURE 65

164. The ground line on the downdrift side of a groin will be different for an intermediate groin in a field than it will for a single groin or for the farthest downdrift groin in a field. If the field is properly planned and constructed, the ground lines would be about the same for the latter two.

165. Considering first an intermediate groin in a groin field, the maximum shore recession on the downdrift side of the groin would occur before the updrift groins were full, permitting the entire quantity of littoral material to move past the groins. During this time the maximum recession would occur when the shore line between the intermediate groin and the next downdrift groin has reoriented to a position normal to the net wave orthogonals such that area a = area b in Figure 65.

166. To determine the maximum recession of the downdrift ground line show the proposed groin on the original beach profile as in Figure 66. From the crest of berm at station 0, lay off distance cd taken from Figure 65. Draw the foreshore from crest of berm to datum plane parallel to the original beach line and connect that point of intersection with original profile at the seaward end of the groin.

167. After the position of maximum recession has been reached, as shown by c-g on Figure 65, the shore line will begin to advance seaward maintaining its alinement normal to the net wave orthogonals until sufficient material flows around or over the downdrift groin to produce a stabilized shore line as shown by the line m-n in Figure 65.

168. To determine the stabilized downdrift line, see Figure 66. From the crest of berm at station 0 lay off the distance dm taken from Figure 65. Draw the foreshore from the crest of berm to datum plane parallel to the original beach line and connect that point of intersection with the original profile at the seaward end of the groin.

169. Considering a single groin or the last groin of a field, the maximum recession that could occur would be to assume that the downdrift area loses an amount equal to the full annual rate of littoral drift for the period required for the groin to fill to capacity. It is known that some unknown percentage of the total littoral drift moves seaward of the seaward ends of the groins. Also, it is known that some additional percentage of the material moving shoreward of the seaward ends of the groins, will bypass the groin before it is completely filled. Accordingly, for approximating the position of the downdrift ground line, it is believed safe to reduce the net littoral drift by some amount depending on the type of groin constructed. Percentage of net littoral drift considered safe for computing downdrift losses due to certain groin types are given as follows:

a. For high groins extending to a depth of water 10 feet or more, use 100 per cent of the total littoral drift;

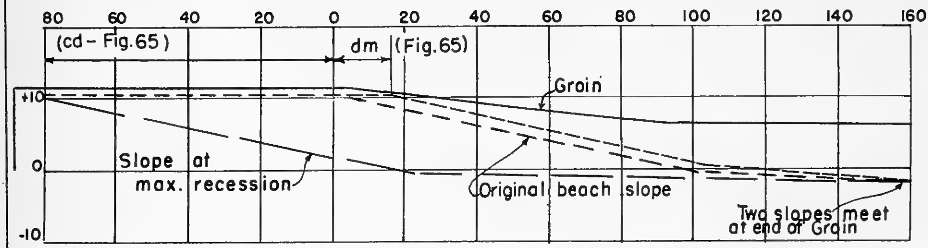


FIGURE 66. SLOPE ON DOWN DRIFT SIDE OF INTERMEDIATE GROIN

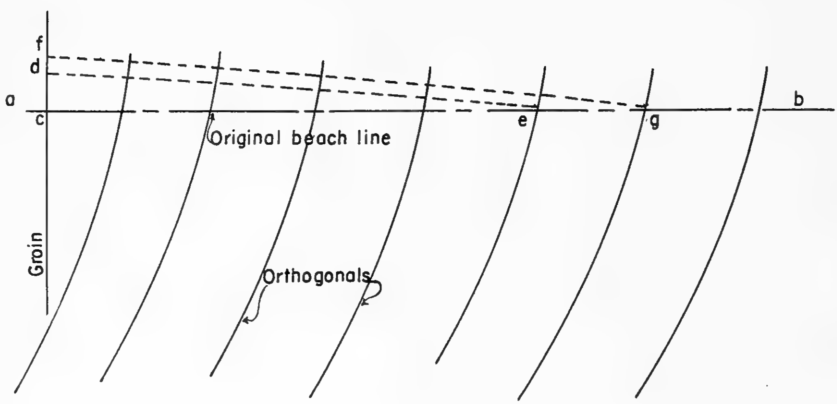


FIGURE 67. RECESSION-DOWN DRIFT OF A SINGLE GROIN

b. For high groins extending to a depth of 4 to 10 feet below mean low water (or mean lower low water), or for low groins extending to a depth greater than 10 feet, use 75 per cent of the total littoral drift;

c. For high groins extending from mean low water to 4 feet below mean low water (or mean lower low water), or for low groins extending to depth less than 10 feet below mean low water, use 50 per cent of the total annual rate of littoral drift.

170. The following steps can now be used to determine the position of the downdrift ground line.

a. Lay out the wave diagram as for the updrift side. See Figure 67;

b. Determine the time required for the groin to fill in years or fractions of a year;

c. Draw receded shore line, "de", normal to the orthogonals such that area dec in square feet is equal to the portion of the annual rate of littoral drift in cubic yards (reduced according to groin type) determined by the time for the groin to fill;

d. If more than one year is required, the recession during the second year is shown in the same way such that area dfge is equal to the adjusted annual rate of littoral drift;

e. Plot the original bottom profile and show the groin on this profile as in Figure 66. Plot cd as the maximum recession expected and proceed as in paragraph 166.

171. The foregoing assumes an erodible bottom and backshore. Naturally, wherever a non-erodible substance is encountered, recession would halt at that point. However, all other places would continue to recede to the ultimate plane predicted. This would also be true where the groins are tied to a seawall or bulkhead. In this case the expected ground line would be determined as if the seawall were not there or in a similar manner as for seawalls. The position of the ground line where it intersects the seawall would determine the approximate scour to be expected in front of the wall. The deficiency in material would tend to be made up by recession of the shore line at the downdrift end of the seawall.

172. Alinement of Groins. - For most effective operation with least tendency toward localized erosion, groins should be constructed parallel to the direction of resultant wave action; in other words, parallel to the direction of approach of the wave which will produce the predominant direction and net rate of littoral drift. This direction may be determined through the use of refraction diagrams combined with a statistical wave study over an extended period of time, as discussed previously under wave analysis (para. 76). However the accuracy of the determination

with present methods does not ordinarily warrant its use. For all practical purposes, short groins can be constructed normal to the beach, as this will approximate very closely the resultant direction of wave approach. Long groins though usually built normal to the shore may warrant an effort to determine the resultant wave direction.

173. Order of Groin Construction. - This applies only to sites where a groin field is under consideration. Here two conditions arise: (1) where the groin field will be filled and it is desired to stabilize the new beach in its advanced position; and (2) where littoral drift is depended upon to make the fill and it is desired to stabilize the existing beach or build additional beach with a minimum of detrimental effect on downdrift areas.

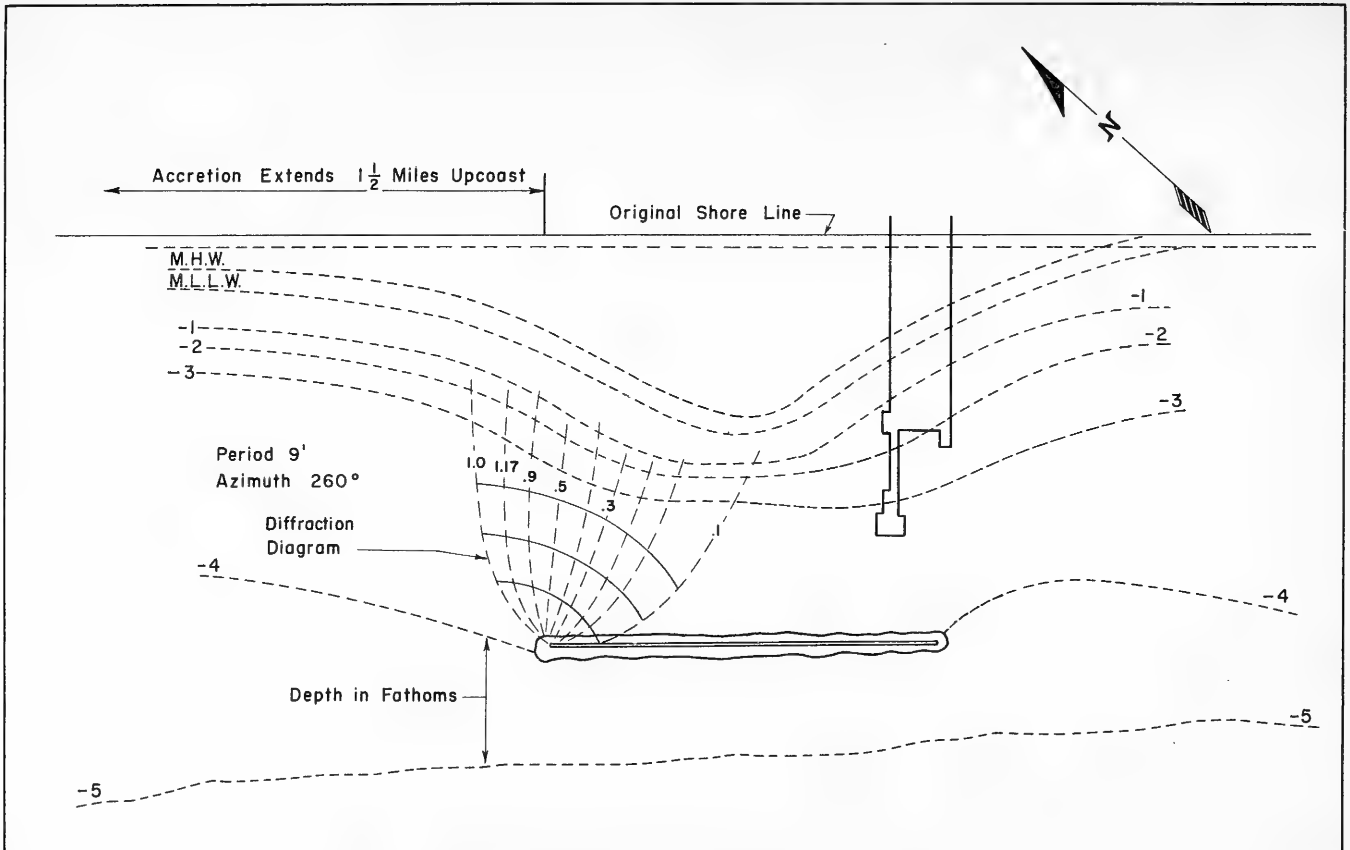
174. In the first instance the only interruption to littoral drift will be between the time the groin field is constructed and the time the artificial fill is made. In the interests of economy, the fill must be placed at one time, especially if it is being accomplished by hydraulic dredge. Accordingly to reduce the time interval between groin construction and deposition of fill, all groins should be constructed concurrently or as rapidly as practicable if constructed consecutively. Deposition of fill should commence as soon as the stage of groin construction will permit.

175. In the second instance no groin can fill until all of the preceding updrift groins have been filled. The time required for the entire field to fill and material to resume its unrestricted movement downdrift may be excessive so that severe damage may result. Accordingly only the groin or group of groins at the downdrift end should be constructed initially. The second groin, or group, should not be started until the first has filled and material passing around or over the groins has again stabilized the downdrift beach. Although this method may increase mobilization costs, it will not only aid in holding damage to a minimum but will provide a practicable guide to spacing of groins to verify the previously computed spacing.

OFFSHORE BREAKWATERS

176. Use. - An offshore breakwater differs from other breakwaters in that it is generally parallel to and is not connected with shore. This type of structure seldom has been constructed solely for shore protection because of its comparatively high first cost and the difficulty of minor maintenance, if required. However, under some specific conditions, this cost may be justified.

177. Effect On The Shore Line. - The effects of an offshore breakwater on the shore line are partially illustrated by Figure 66. An offshore breakwater has an effect on the shore regimen similar to that of any other structure, such as a groin, which modifies the rate of littoral drift. It is probably the most effective means of completely intercepting



EFFECT ON SHORELINE OF OFFSHORE BREAKWATER

Contours shown represent the shore line at the end of 11 years

FIGURE 68

littoral drift of all such modifying structures now in use, since, being usually in deeper water than the seaward end of a jetty or groin, it controls a wider portion of the band of littoral drift than do structures attached to shore. Because littoral drift is the direct result of wave action, the extent to which the breakwater intercepts the littoral drift is directly proportional to the extent of interception of wave action by the breakwater.

178. The effect on the shore line of an offshore breakwater is typified by the 2,000-foot structure at Santa Monica, California. This breakwater was constructed parallel to and 2,000 feet distant from shore in approximately 27 feet of water. Figure 68 shows in dashed lines contours of the accretion that existed behind the breakwater after 11 years.

179. Operation of An Offshore Breakwater. - An offshore breakwater at first tends to cause sand to deposit in its lee by slowing the wave generated littoral current in that area. Diffraction causes some wave action within the geometric shadow, but such wave action is much less than that which would exist in the area in the absence of the breakwater. The typical diffraction diagram drawn on Figure 68 shows that wave heights within the breakwater's geometric shadow are less than one-half the wave heights outside the breakwater.

180. As sand is deposited, a shore salient is formed in the still water behind the breakwater. This salient itself acts as a groin, which causes the updrift shore line to advance. Concomitant with the advancement of the shore line which brings the zone of littoral drift closer to the breakwater, is an increase of the efficiency of the breakwater in acting as a sand trap. The salient is more rapidly formed and thus becomes increasingly efficient as a groin.

181. If the breakwater is of sufficient length in relation to its distance from the shore to act as a complete littoral barrier, the sand depositing action may continue until a tombolo is formed with the breakwater at its apex. Such a tombolo accretion is shown on Figure 69, an aerial photograph of the offshore breakwater at Venice, California.

182. The precise shape of the deposit is difficult or impossible to determine. In general there will be accretion updrift from the breakwater and erosion downdrift. The area immediately behind the breakwater customarily assumes a form convex seaward. It has been found for complete barriers that a large percentage of the total accumulation collects in the breakwater lee during the first year and that the ratio of material in the lee of the structure to total material trapped decreases steadily until such time that the trap is filled and littoral drift begins moving around the structure. A shore line profile for a complete barrier may be roughly approximated by drawing the high water line so that the area in square feet between this line and the original high water line is equal to the anticipated accretion in cubic yards at the time for which the profile is to be drawn.

OFFSHORE BREAKWATER; VENICE, CALIFORNIA
FIGURE 69

183. Offshore Breakwaters in Series. - It is not necessary to build offshore breakwaters as a single unit. A series of relatively short structures will have the same general effect as a single one, but the efficiency of the series as a sand trap will be decreased; a condition which is sometimes desirable. The tendency for a tombolo to form will be decreased. Figure 70 is an aerial photograph taken in 1949 of the breakwater set off Winthrop Beach, Massachusetts constructed in 1931-1933. The characteristic convex accretion in the breakwater lee is evident, as is also the erosion zone downdrift. Note that here shoals formed from the breakwaters extending landward, indicating that these breakwaters are in the littoral movement band.

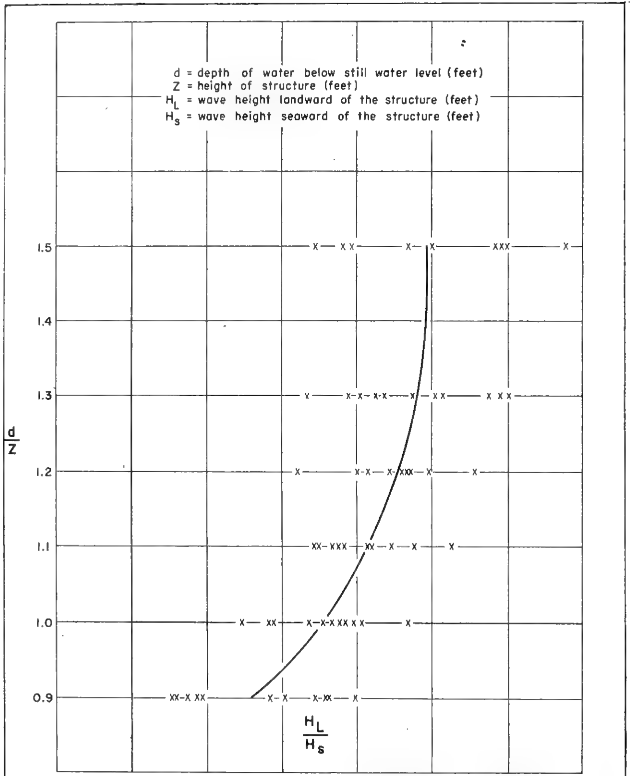
184. Height of Offshore Breakwater. - For the purpose of height determination, offshore breakwaters can be considered in two categories; (1) submerged breakwaters, and (2) exposed breakwaters.

185. Submerged Breakwaters. - Submerged breakwaters (also known as submerged sills or submerged barriers) are those with their tops below the low water line. These may be used to retain beach fill or to reduce the wave forces by causing the waves to break. Where used only to retain beach material, the top of the structure should be in line with the design slope of the beach fill. In theory (Tratman 1940), where a structure is designed to reduce the wave force, the sweep of the waves is broken by the obstruction, so that their destructive force is dissipated or expended in the semi-enclosed stretch of water between the barrier and the shore. In addition, the return currents flowing back along the bottom strike the inner side of the submerged barrier and are compelled to rise vertically. This upward flow assists in breaking up the flow of water shoreward over the barrier.

186. Model studies of the effects of submerged structures on wave heights and wave energy were made by the Beach Erosion Board and published in 1940. Other model studies were made by the University of California at Berkeley. In general, both studies indicated that an underwater structure parallel to a shore line would decrease wave height and wave energy on the shore. A vertical wall was found to be the most effective structure for decreasing landward wave action, with the effectiveness increasing with the width of the wall. However, the difference in effectiveness of variously shaped structures was sufficiently slight that construction costs and difficulties probably would govern the choice of structure.

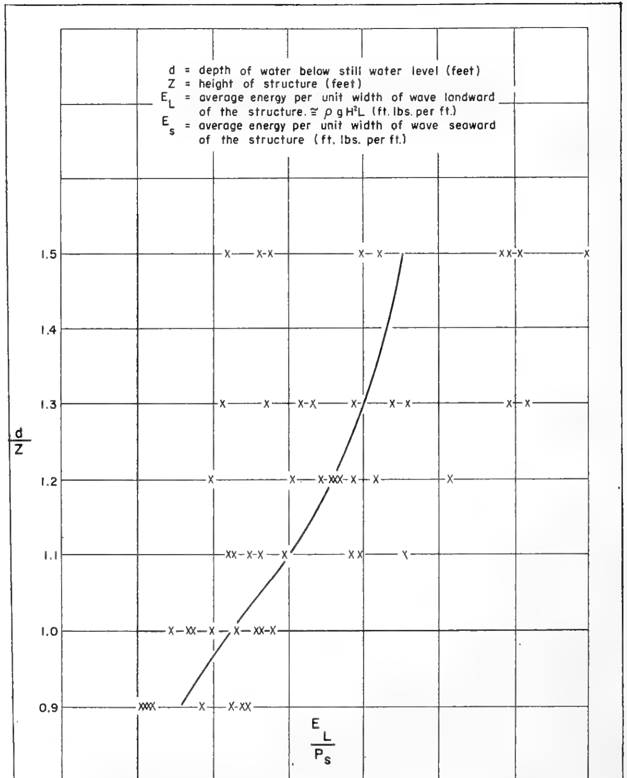
187. The extent to which wave action was reduced was found to be a function of the distance between the top of the structure and the surface of the water or, to use a dimensionless parameter, of the ratio of the depth of water over the structure to the depth of water at the site. The average effects of submerged structures on wave height and wave energy in terms of relative depth, found in these model studies, is shown on Figures 71 and 72. By making use of these curves, rough approximations of wave action and sand moving capabilities behind a submerged breakwater

OFFSHORE BREAKWATER; WINTHROP BEACH, MASSACHUSETTS
FIGURE 70



AVERAGE EFFECT OF STRUCTURE ON WAVE HEIGHT RATIO WITH SUBMERGENCE

FIGURE 71



RATIO OF ENERGY LANDWARD TO SEAWARD FOR VARIED SUBMERGENCE OF STRUCTURES

FIGURE 72

may be found since the ratio of transmitted to incident wave energy may be used as an indication of the relative wave competence to move sand. (The energy E, per unit length along the crest, in a wave of height H, length L, is given approximately by $E = wH^2L$ where w is the unit weight of the water).

188. Exposed Breakwaters. - Exposed breakwaters include those uncovered only at low tide stages, as well as those never completely submerged except perhaps by overtopping waves. Data are lacking on the effects of such structures on transmitted wave characteristics, however it has been standard practice to assume insignificant overtopping if the breakwater crest is $1\frac{1}{2}$ times the height of the design wave over the design water level. Under this assumption, it would seem conservative to extend the curves of Figure 71 and 72 by a linear relationship to H_L/H_S or $E_L/E_S = 0$ at the point of zero overtopping. For example, a structure founded in 20 feet of water at high tide, with a design wave of 10 feet should have its crest 15 feet above the high water line. In this case, $d = 20$ feet, $s = d + 15 = 35$ feet and H_L/H_S or $E_L/E_S = 0$ at $d/s = 20/35 = 0.57$, and by extending the curves of Figures 71 and 72 to this point, effectiveness of lesser height structures may be found.

189. Length and Distance From Shore. - No definite relationships have been established between the length of breakwater, its distance from shore, and the depth of water at the site. It is known that such relationships do exist. When the length of a breakwater is only a fraction of its distance offshore, diffracted and refracted wave crests have enough distance to turn through at least one quadrant, thereby propagating wave energy sufficient to maintain sand in suspension and transit behind the breakwater (Handin and Ludwick, 1950). At Santa Monica, a 2,000 foot breakwater, 2,000 feet offshore in 27 feet of water, acted as a complete sand barrier. About two miles away at Venice, a breakwater about 300 feet long and 1,200 feet from shore allowed sand to pass along the shore behind it with only a slight accretion upcoast and a corresponding erosion downcoast. When the beach was artificially widened by about one-half the distance, or 600 feet, a tombolo promptly formed between the breakwater and the new shore line, completely intercepting littoral drift as may be seen on Figure 70.

190. The principal lesson to be learned is that extreme care should be used in planning any offshore structure designed for the sole purpose of reducing the rate of littoral drift. It is far better to design too short a structure requiring later extension than one too long which creates a serious erosion problem.

191. Spacing of Offshore Breakwaters. - Again no definite relationships have been established to determine criteria for the spacing of off-shore breakwaters where it is desired to protect a reach of coast line yet not form a complete littoral barrier. Accordingly, extreme care must be used to avoid creating more damage than would have occurred without the structure. The following general rules may be observed:

a. Determine the direction and rate of littoral drift;

b. Determine whether or not a reduction in the rate of littoral drift over the area would result in damage to downdrift areas. The amount of this damage must be considered in determining the economic justification for the project;

c. Construct the first breakwater at or near the downdrift end of the area to be protected. It should be short and an adequate distance from shore;

d. Observe the action of this structure and when the shore line has reached an apparent stability, construct the second breakwater at the point farthest updrift from the first structure that any accretion is apparent;

e. Repeat the process until the entire reach has been protected.

192. An offshore breakwater will have the same effect on the shore line if it is not parallel to shore as it would have if it were parallel to shore, except that the effective length of the structure will be reduced to the projection of the structure normal to the resultant direction of wave propagation. See Figure 73.

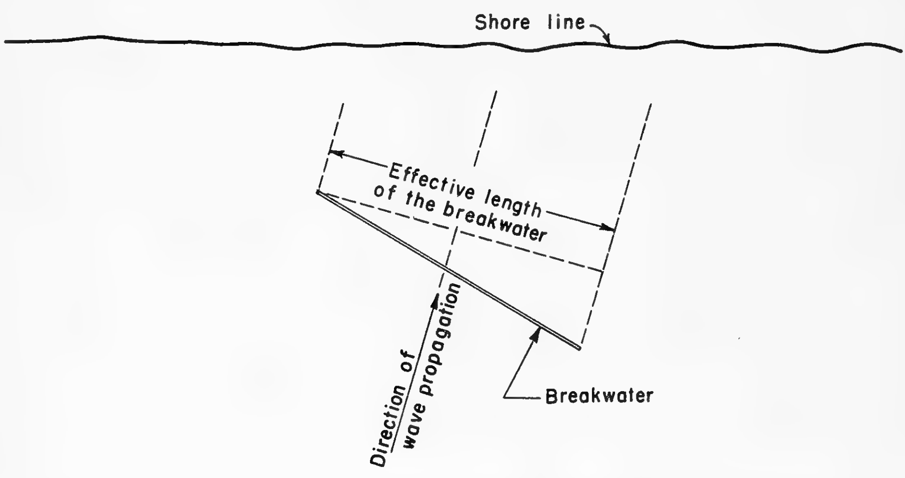


FIGURE 73. EFFECTIVE LENGTH OF AN OFFSHORE BREAKWATER

PART II
STRUCTURAL DESIGN

SCOPE

193. The first part of this section, PHYSICAL FACTORS, contains general data pertaining to design and includes: methods for determining wave heights and forces; earth and ice forces and their points of application; and certain considerations of materials of construction commonly used in shore protective works. The second part contains solutions of typical problems of design of protective measures.

PHYSICAL FACTORS

WAVE HEIGHTS

194. Determination of Design Wave Height and Direction. - If economically feasible, any structure exposed to wave action should be designed to withstand the effects of the highest wave to be expected at the structure's location. Visual observations of storm waves are usually unreliable. Direct measurement is costly and time consuming. General procedures for developing the height and direction of the design wave by use of refraction diagrams are described in the following paragraphs.

195. Significant Design Wave - Structure Outside the Breaker Zone. - From the structure's intended location, draw a set of "refraction fans", for various wave periods in increments of about 2 seconds that might be expected at the site, and determine refraction coefficients by the method given in Part I. Tabulate the refraction coefficients so determined for the various wave periods chosen and for each deep water direction of approach. The statistical wave data derived from synoptic weather charts or other sources should then be reviewed to determine if the directions and periods for which the refraction coefficients are large may be expected to recur with reasonable frequency.

196. That combination of deep water wave height and refraction coefficient which gives the highest wave height at the structure's location determines the design wave direction of approach and period. The inshore height so determined is the significant design wave height.

197. A typical example of such an analysis is shown in the following table.

Table 9 - Determination of Significant Design Wave Heights

Direction	Significant Deep Water Wave Height	Period Range	Refraction Coefficient*	Refracted Wave Height
(1)	(2) (feet)	(3) (seconds)	(4)	(5) (feet)
NW	15	8	0.10	1.5
		10	0.07	1.0
		12	0.04	0.5
WNW	12	8	0.15	2.0
		10	0.12	1.5
		12	0.09	1.0
W	10	10	0.30	3.0
		12	0.31	3.0
		14	0.20	2.0
		16	0.25	2.5
WSW	10	10	0.60	6.0**
		12	0.50	5.0
		14	0.35	3.5
		16	0.35	3.5
SW	8	12	0.72	6.0**
		14	0.59	5.0
		16	0.40	3.0

* This refraction coefficient is equal to $\sqrt{\frac{b_o}{b}}\,(H/H_o')$.

** Adopted as the significant design wave height.

Columns 1, 2, and 3 are taken from the statistical wave data as determined from synoptic weather charts.

Column 4 is determined from the relative distances between two adjacent orthogonals in deep water and in shallow water

Column 5 is the product of columns 2 and 4

198. In the preceding example, although the highest deep water waves approached from directions ranging from W to NW, the refraction study indicated that higher waves inshore may be expected from more southerly directions.

199. It must be remembered that the accuracy with which refraction procedure may be applied in the determination of design wave characteristics decreases as the underwater contours over which the refraction diagrams must be drawn become more complex. Under highly complex bottom conditions direct observations may be required.

WAVE FORCES

200. In an analysis of the forces exerted on structures by waves, a division should be made between the action of breaking waves, non-breaking waves, and broken waves. Pressures due to non-breaking waves will be essentially hydrostatic. Broken and breaking waves on the other hand, exert additional pressures due to the dynamic effects of the turbulent water in motion and the compression of entrapped air pockets. These pressures may be much greater than those due entirely to hydrostatic forces. Therefore structures located in an area in which storm waves may break, should be designed to withstand much greater forces and moments than those structures which would be attacked only by non-breaking waves.

201. Determination of Breaker Depths and Heights. - Depths at which waves may break, may be determined, given deep water wave conditions, from the curves of Figure 18. A note of caution is in order on the use of these curves. Rarely are storm waves so regular that the depth of breaking may be precisely determined. Storm wave heights and lengths are extremely variable if the generating area is not far removed from the point of observation.

202. The curve labelled U.C., because of the controlled nature of the model wave parameters used in the construction of the curve, probably represents a lower limit of the range of breaker depth to deep water height ratios which would be found in nature for any one wave condition. If d_b as found from this curve is less than the depth of water along a shore structure, it should not be immediately assumed that these waves would not break on the structure. For safety, waves must be assumed to be capable of breaking in a depth of water d, greater than the d_b found from the curve, or if the ratio $d/d_b \leq 1.5$. Breaker heights may be determined from the curves of Figure 17 labelled U.C. For structures in shallow water, when deep water conditions are not known or when the design wave determined from deep water conditions would break before reaching the structure, the height of the maximum wave which would break on the structure can be found approximately from the relationship $d_b = 1.3H$.

203. Non-breaking Waves. - Ordinarily, a shore structure would be so located that storm waves would break in the depth in which the structure is founded. However, in protected regions or in areas where the available fetch is limited, non-breaking wave conditions may occur. The most commonly used method for the determination of pressures due to these waves in that of M. Sainflou.

204. Sainflou Method: Forces Due to Non-breaking Waves. - If a wave of length L and height H strikes the vertical face of the wall AC, a standing wave or clapotis will be set up. (See Figure 74). The point A is the maximum elevation of the crest, and point G is the minimum elevation of the trough of the clapotis. The mean level or orbit center is above the still water level D a distance

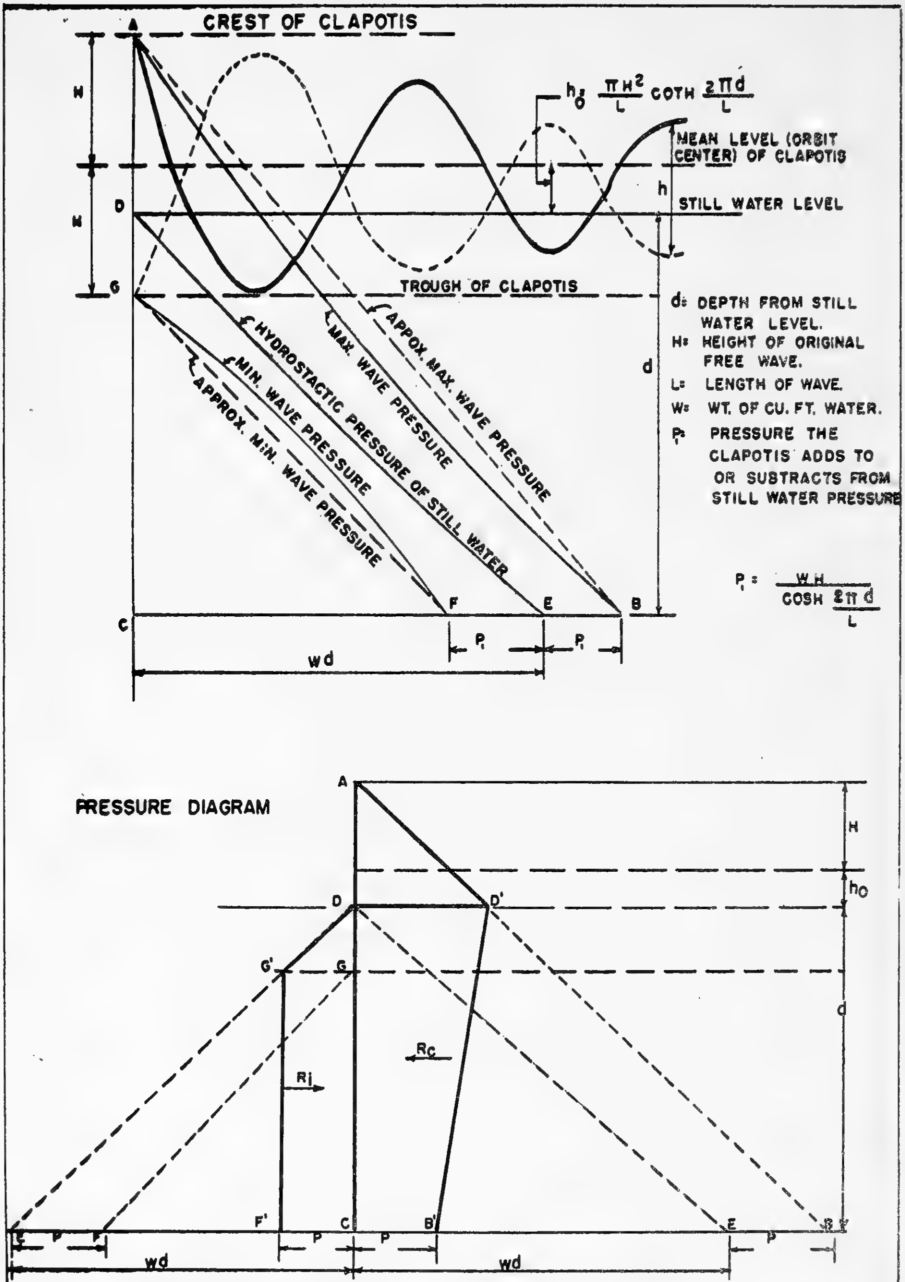


FIGURE 74. CLAPOTIS ON VERTICAL WALL

$$h_o = \frac{\pi H^2}{L} \operatorname{Coth} \frac{2\pi d}{L} \tag{19}$$

and DA is equal to $H + h_o$, while DG equals $H - h_o$. The hydrostatic pressure (wd) at the base C of the wall is scaled out from C and plotted as E. The triangle CDE is the hydrostatic pressure distribution against the wall due to water at still water level. As the surface of the clapotis moves above or below still water it will increase or decrease the hydrostatic pressure at the base of the wall by the amount P_1. This change in pressure is

$$P_1 = \frac{WH}{\operatorname{Cosh} \frac{2\pi d}{L}} \tag{20}$$

205. Plotting P_1 in both plus and minus directions from point E gives points B and F as the maximum and minimum pressures, respectively, at the base caused by the clapotis against the sea face of the wall. The solid curved lines labelled maximum wave pressure and minimum wave pressure denote the pressure distributions computed by theoretically exact formulas. These curved lines are so close to a straight line that it is permissible, and conservative, to approximate this distribution by use of straight dashed lines connecting A to B and G to F as shown in Figure 74. Figure 75 shows the equation for h_o reduced to graphical form, indicating values for Lh_o for different ratios of 2d/L and for wave heights at 5-foot intervals up to 40 feet. Figure 76 shows the equation for P_1 reduced to graphical form, indicating values of P_1 for different ratios of 2d/L for the same wave heights.

206. Assuming the same still water level on each side of the wall, an outward or seaward pressure exists which is equal to the hydrostatic pressure shown by the triangle CDE in Figure 74. As the two pressures at still water level balance each other, the resultant pressure on the wall when the crest of the clapotis is against it is toward the land and is shown by the area ABED or AD'B'C. When the trough of the clapotis is at the wall the resultant pressure is toward the sea and is represented by the area DEFG or DG'F'C. A diagram of the resultant pressures on a vertical wall is also shown in Figure 74. Should there be no water on the landward side of the wall, then the total resultant pressure would be represented by the triangle ACB when the clapotis crest is at A. If there were wave action on the landward side, then the condition of crest of clapotis on the sea side and trough of the wave on the harbor side would produce maximum pressure from the sea side. The maximum pressure from the harbor side would be produced when the trough of the clapotis on the sea side and the crest of the clapotis on the land side are at the structure.

207. For a unit length of wall, with h_o as the mean level of the clapotis above the still water level and P_1 the common length of the segments EB and EF_1 the resultant R_1 and the moment about the base, M_1 are given respectively, for the maximum crest level (subscript e) and the minimum trough level (Sbscpt.i) of the clapotis by the formulas:

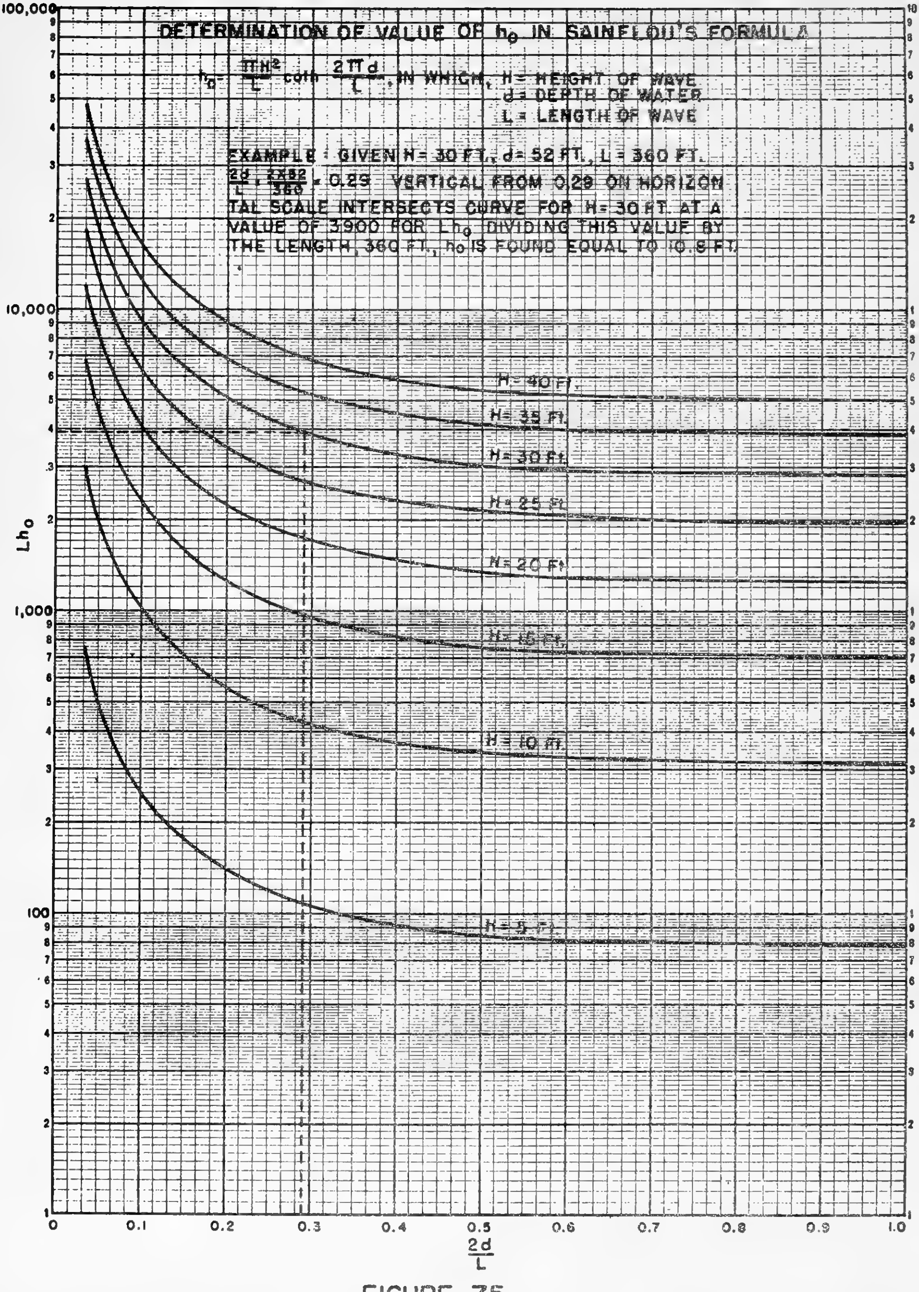

DETERMINATION OF VALUE OF h_0 IN SAINFLOU'S FORMULA
$h_0 = \frac{\pi H^2}{L} \coth \frac{2\pi d}{L}$, IN WHICH, H = HEIGHT OF WAVE
d = DEPTH OF WATER
L = LENGTH OF WAVE
EXAMPLE: GIVEN H = 30 FT., d = 52 FT., L = 360 FT.
$\frac{2d}{L} = \frac{2 \times 52}{360} = 0.29$ VERTICAL FROM 0.29 ON HORIZONTAL SCALE INTERSECTS CURVE FOR H = 30 FT. AT A VALUE OF 3900 FOR Lh_0 DIVIDING THIS VALUE BY THE LENGTH, 360 FT., h_0 IS FOUND EQUAL TO 10.8 FT.
H = 40 FT.
H = 35 FT.
H = 30 FT.
H = 25 FT.
H = 20 FT.
H = 15 FT.
H = 10 FT.
H = 5 FT.
Lh_0
$\frac{2d}{L}$
100,000
10,000
1,000
100
1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0

FIGURE 75

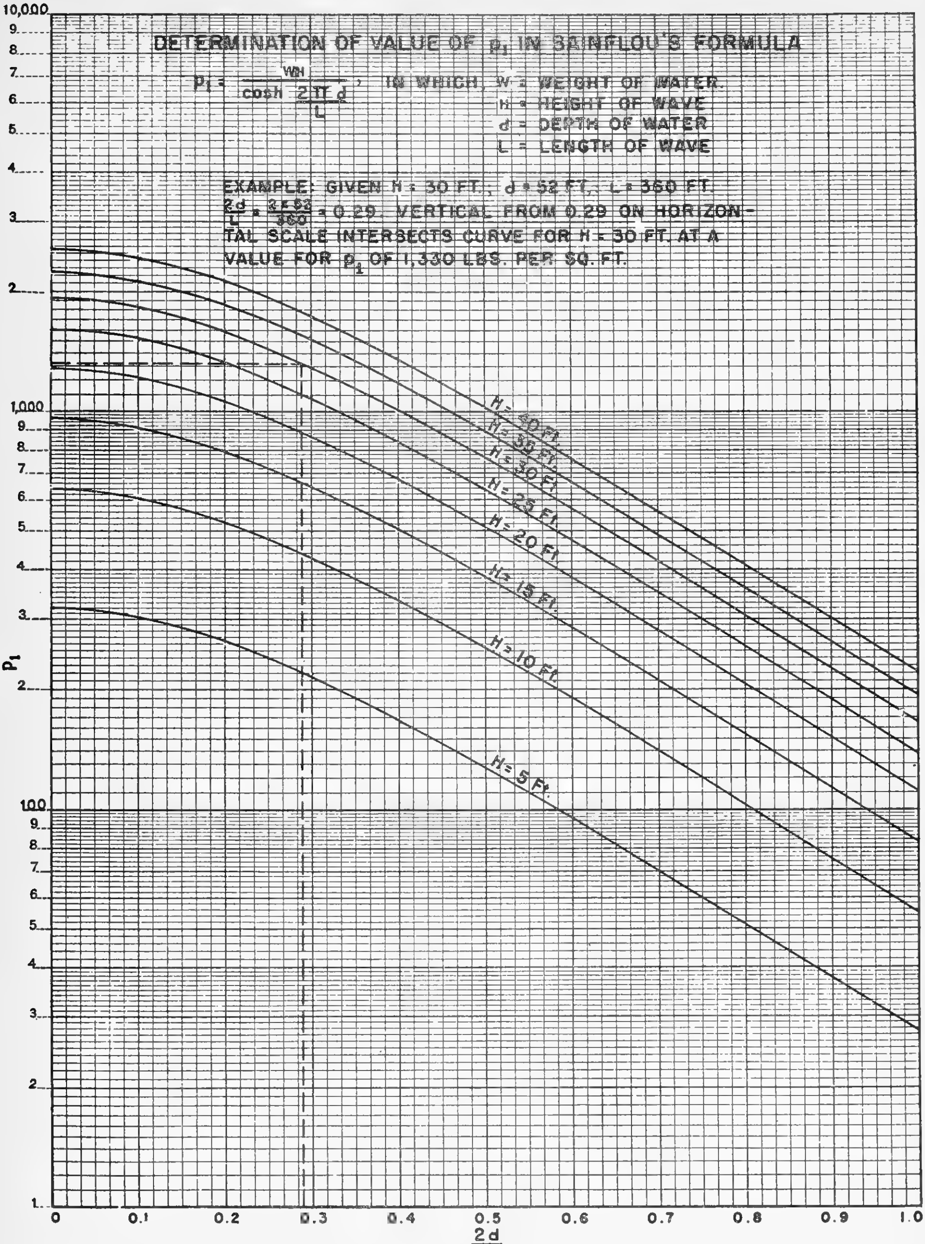

DETERMINATION OF VALUE OF p_1 IN SAINFLOU'S FORMULA
$p_1 = \frac{wH}{\cosh \frac{2\pi d}{L}}$, IN WHICH, w = WEIGHT OF WATER
H = HEIGHT OF WAVE
d = DEPTH OF WATER
L = LENGTH OF WAVE
EXAMPLE: GIVEN H = 30 FT., d = 52 FT., L = 360 FT.
$\frac{2d}{L} = \frac{2 \times 52}{360} = 0.29$. VERTICAL FROM 0.29 ON HORIZONTAL SCALE INTERSECTS CURVE FOR H = 30 FT. AT A VALUE FOR p_1 OF 1,330 LBS. PER SQ. FT.
H = 40 FT.
H = 35 FT.
H = 30 FT.
H = 25 FT.
H = 20 FT.
H = 15 FT.
H = 10 FT.
H = 5 FT.
p_1
$\frac{2d}{L}$

FIGURE 76

$$R_e = \frac{(d + H + h_o)(wd + P_1)}{2} - \frac{wd^2}{2} \tag{21}$$

$$M_e = \frac{(d + h_o + H)^2 (wd + P_1)}{6} - \frac{wd^3}{6} \tag{22}$$

$$R_i = \frac{wd^2}{2} - \frac{(d + h_o - H)(wd - P_1)}{2} \tag{23}$$

$$M_i = \frac{wd^3}{6} - \frac{(d + h_o - H)^2 (wd - P_1)}{6} \tag{24}$$

These formulas for pressures created by the clapotis are based upon the assumption that the vertical wall rests upon the natural bottom. If the vertical wall rests on a stone foundation, the action of the wave depends on the profile of the foundation structure.

208. Wall of Low Height. - If the height of the wall is less than the predicted wave height at the wall, forces may be approximated by drawing the force polygons as if the wall were higher than the impinging wave, then analyzing only that portion below the wall crest. Thus in Figure 77 the force due to a wave crest at the wall is computed from the area AFBSC.

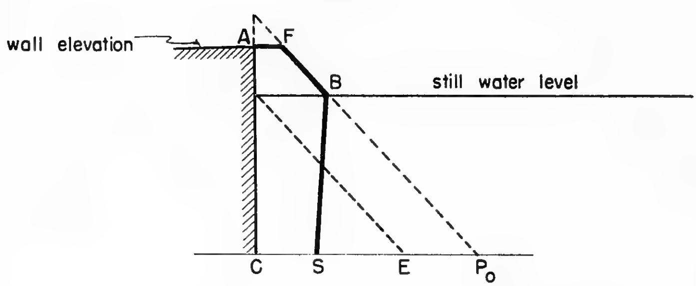


FIGURE 77. Pressure on walls of low height

209. Waves Breaking on a Structure. - Ordinarily bulkheads and seawalls are so located that storm waves may break directly on them. Even those structures which are located landward of the low water shore line may be exposed to the action of breaking waves at times of high water.

210. There have been three major attempts to correlate the high pressures known to exist with measurable wave parameters. In 1934, D.A. Molitor published a suggested solution of this problem, using a semi-theoretical approach and making use of the observations of D. D. Gaillard taken on Lake Superior in 1904 and spring dynamometer readings taken during a storm at Toronto in 1915. Unfortunately since publication of the paper, pressures have been observed far in excess of those predicted by the Molitor equations, and structures have failed which according to the Molitor equations were adequately designed.

211. Essentially the Molitor wave pressure solution formed an envelope of the dynamometer readings taken in 1915. The measurements were taken throughout the storm and the maxima at various elevations recorded. Thus, though his equations purport to give a pressure curve for a single impinging wave, they really give a curve representing the maximum pressure recorded from many waves. This would ordinarily lead to conservative results but the range of wave parameter variables was too restricted for the results of these measurements to be applied to general wave conditions.

212. In 1939, R. A. Bagnold reported on measurements of shock pressures due to breaking waves recorded under model conditions. Pressures so recorded were greatly in excess of any prior predicted ones. Bagnold found that for these "shock pressures", a correlation could be established between the magnitude of the peak pressure and the thickness of a cushion of air entrapped by waves breaking on a structure. Unfortunately these experiments were interrupted and no further relationship has been established between the thickness of the air cushion and various wave parameters.

213. The last approach to the determination of wave pressures was made in 1946 by R. R. Minikin. Although this method has some inconsistencies, it probably represents the closest approach to the actual pressures caused by a breaking wave. With the Minikin methods, failures of structures, otherwise unpredictable, may be explained.

214. Minikin Method: Forces Due to Breaking Waves. - According to Minikin, the total pressure caused by waves breaking on a structure is due to a combination of dynamic and hydrostatic pressures. The dynamic pressure is concentrated at still water level, and is equal to

$$P_m = \frac{101\ Hw}{L_D} \times \frac{d}{D}\,(D + d) \qquad (25)$$

where H = the height of wave just breaking on the structure (in feet)
w = the unit weight of the water (in pounds per cubic foot)
d = the depth of water at the structure (in feet)
D and L_D = deeper water depth and wave length respectively (in feet)

Based on descriptive passages in his paper, values found for D and L_D by the following method should approximate those he considered.

215. Given a wave period, the deep water wave length L_o (in feet) may be found from

$$L_o = 5.12\ T^2 \tag{26}$$

With the ratio d/L_o, the wave length at the structure L_d at depth d may be determined from the d/L value taken from Table 1 of Appendix D. Knowing the slope S before the structure, D may be determined from

$$D = d + L_d S \tag{27}$$

L_D may then be determined, again from Table 1 of Appendix D, by computing the ratio D/L_o, tabulating the corresponding value of d/L (which in this case is D/L_D), and dividing D by this ratio. The hydrostatic pressure on the seaward side at still water level (subscript "s"), and at the depth d (subscript "d") are given by

$$P_s = \frac{wH}{2} \quad \text{and} \quad P_d = w\ (d + H/2) \tag{28}$$

The pressure curve may be plotted by considering that the dynamic pressure is concentrated at still water level, and falls away rapidly along a parabolic curve to zero at a distance 1/2 H above and below still water level. To this is added the hydrostatic pressures on the seaward side plotted in a triangular area from 1/2 H above still water level to the bottom. Hydrostatic pressures on the landward side plotted in a triangular area from still water level to the bottom must be subtracted if such pressures obtain. This construction is shown in Figure 78, the dashed and dotted lines indicating the separate effects of the dynamic and hydrostatic pressures, and the solid line the combined pressures, for the case where hydrostatic pressures are on each side of the wall.

216. The resultant wave thrust or force per linear foot of structure may be determined from the area of this diagram and is given by

$$R = \frac{P_m H}{3} + P_s\ (d + \frac{H}{2}) \tag{29}$$

The resultant overturning moment about the ground line before the wall, is the sum of the moments of the individual areas and is given by

$$M = \frac{P_m H}{3}\, d + \frac{P_s\ d^2}{2} + \frac{P_s H}{4}\ (d + \frac{H}{6}) \tag{30}$$

Similar computations may be made if there is no water on the land side, in which case the thrust per linear foot of the structure is

$$R = \frac{P_m H}{3} + \frac{P_d}{2} \left(d + \frac{H}{2}\right) \tag{31}$$

and the moment about the ground line is

$$M = \frac{P_m H}{3} d + \frac{P_d}{6} + \left(d + \frac{H}{2}\right)^2 \tag{32}$$

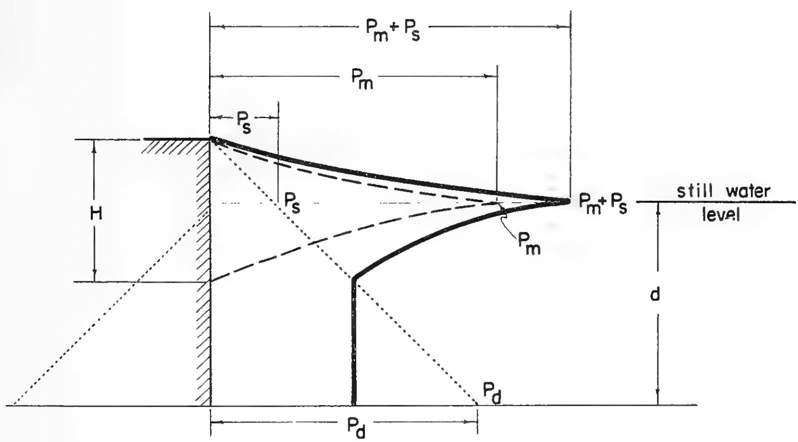


FIGURE 78. Minikin wave pressure diagram

217. <u>Waves Breaking Seaward of a Structure.</u> - Certain protective structures may be so located that even under severe storm and tide conditions waves will break before striking the structure. For example, bulkheads built landward of the high water shore line may have water against them only under extreme tidal and storm conditions and then only to a depth of one or two feet. Under these conditions, approximate design forces may be computed with the following assumptions:

(1) That after breaking a wave will run up a slope to an elevation no higher than the elevation of the crest of the wave on breaking.

(2) Immediately after breaking, the water mass moves forward with the velocity of wave propagation just before breaking; that is upon breaking the water particle motion changes from orbital to translatory.

Under these assumptions, the wave pressure or force on an obstruction may be calculated by assuming a uniform velocity decrease to zero from the point of breaking to the point of maximum uprush. It has been found from model tests, that upon breaking, approximately 70 percent of the full breaking wave height H_b is above the still water level. Then in Figure 79, the velocity v of wave uprush at the wall for a breaking wave velocity of $C = \sqrt{gd_b}$ is:

$$v = \left(\frac{x_2 - x_1}{x_2} \right) C = C\left(1 - \frac{x_1}{x_2}\right) \tag{33}$$

where
x_1 = distance from the breaking wave to the structure
x_2 = distance from the breaking wave to the limit of wave uprush
$= (d_b + h_c) \cot\alpha = (d_b + 0.7\ H_b) \cot\alpha$
α = the angle of the beach slope with the horizontal
d_b = breaking depth
h_c = the height of the breaking wave above still water level
H_b = the breaking wave height.

218. The pressure on the wall will be partly dynamic and partly static; the dynamic part being

$$P_m = \frac{wv^2}{2g} = \frac{wd}{2} \left(1 - \frac{x_1}{x_2}\right)^2 \tag{34}$$

and the static part varying from zero at a height h_c above still water level to a maximum $P_s = w(h_c - h_w)$ at the wall base (depth d_b) h_w being the

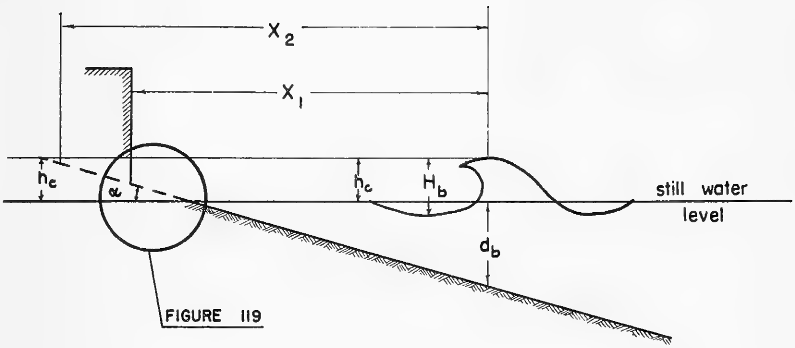


FIGURE 79. Waves breaking seaward of a wall

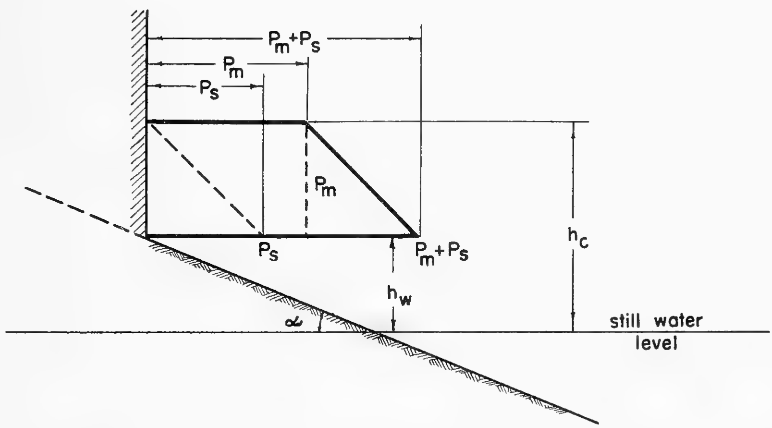


FIGURE 80. Pressure diagram for waves breaking seaward of a wall

elevation of the wall base above still water level. (Note h_W may be negative.)

219. From the pressure diagram of Figure 80, the dynamic thrust R_d is given by:

$$R_{d+} = P_m(h_c - h_W) \text{ if } h_W \text{ is positive,} \tag{35}$$

$$R_{d-} = P_m(h_c) \text{ if } h_W \text{ is negative}$$

The total thrust is then given by

$$R = R_d + P_s\left(\frac{h_c - h_W}{2}\right) \tag{36}$$

The overturning moment about the ground line at the seaward face of the structure is:

$$M = \left(R_{d+}\right) \frac{(h_c - h_W)}{2} + P_s \frac{(h_c - h_W)^2}{6} \text{ ; if } h_W \text{ is positive} \tag{37}$$

$$= \left(R_{d-}\right) \left(\frac{h_c}{2} + \left|h_W\right|\right) + P_s \frac{(h_c + \left|h_W\right|)^2}{6} \text{ if } h_W \text{ is negative.}$$

220. Effect of Face Slope on Wave Pressures. - The preceding formulas may be used for all cases of structures with essentially vertical faces. If the face is sloped backwards at an angle with the horizontal (Figure 81a) the horizontal component of the dynamic pressure due to waves breaking either on or seaward of the wall should be reduced to

$$P_m' = P_m \cos^2 \theta \tag{38}$$

The vertical component of this wave force may be neglected in stability computations. Forces on stepped face structures (Figure 81c) may, for design calculations, be computed as if the face were vertical, since it is probable that dynamic pressures of the same order as those computed for vertical walls would exist. Forces on curved non-re-entrant face structures (Figure 81b) may be calculated by using a line from the top to the bottom of the face to determine an average slope. Re-entrant curved face walls may be considered as vertical.

221. Wave Forces on Rubble Mound Structures. - Until recently, the design of rubble mound structures was largely based on experience and general knowledge of a particular site's conditions. Efforts to rationalize the design of these structures have been made. These entail the observation of failures and the determination of constants to be applied to various parameters in an attempt to explain the failures. Because of the empirical nature of the formulas developed, they are generally expressed in terms of the size stone required to withstand design wave condition. These formulas have been partially substantiated in model studies. However, they still are only a guide and cannot be blindly substituted for experience.

222. In 1938, Iribarren presented formulas for the design of rubble mound structures. These formulas permitted calculation of sea side slopes

and weights of individual stones above the water surface. His result for the calculation of the weight of cap rock, in wide use for some years, is

$$W = \frac{H^3 K S_r}{(\cos\alpha - \sin\alpha)^3 (S_r - 1)^3} \tag{39}$$

where
- S_r = specific gravity of the rock
- α = the angle the slope makes with the horizontal (A slope is usually referred to as $1/\cot\alpha$)
- H = wave height (in feet)
- W = weight of stone (in tons)
- K = an empirically determined coefficient for all unevaluated variables
  - = 4.68×10^{-4} for natural rubble
  - = 5.93×10^{-4} for artificial blocks

This equation however, is not dimensionally homogeneous and has been modified by Hudson, using the same assumptions and force diagram as Iribarren to obtain

$$W = \frac{K' \gamma_w S_f^3 S_r \mu^3 H^3}{(\mu\cos\alpha - \sin\alpha)^3 (S_r - S_f)^3} \tag{40}$$

in which the additional symbols are:

- γ_w = unit weight of fresh water
- S_f = specific gravity of the fluid in which the breakwater is located
- μ = effective coefficient or friction rock on rock, $\simeq 1.09$.
- K' = a variable dimensionless empirically determined coefficient, values of which as determined by Hudson are plotted in Figure 12, Appendix D;

the equivalent values from equation (39) are

$K' = 0.015$ for natural rubble
$K' = 0.019$ for artificial blocks.

223. <u>The Equations for the Weight of Above Surface Stones.</u> - Equation (40) may be reduced to

$$W = \frac{88.3\, K' S_r H^3}{(1.09\cos\alpha - \sin\alpha)^3 (S_r - 1.03)^3} \tag{41}$$

if the breakwater is founded in sea water; and to

$$W = \frac{70.7\ K'\ S_r\ H^3}{(1.09 \cos \alpha - \sin \alpha)^3\ (S_r - 1)^3} \tag{42}$$

if it is founded in fresh water. The wave height H to be used is that height which would exist in the absence of the breakwater. This wave height at the breakwater's position may be related to a deep water wave height H_o by

$$H = H_o\ (H/H_o')\ x\ K_r \tag{43}$$

where H_o = the deep water wave height
K_r = the refraction coefficient at the breakwater's depth (determined from refraction diagrams)
(H/H_o') = the shoaling coefficient, values of which are tabulated Appendix D (H_o' is the wave height which would exist in deep water if the wave at depth d is unaffected by refraction.)

224. The Equations for Weights of Sub-Surface Stones. - The only method presently available for the determination of sub-surface stone weights is that suggested by Iribarren and Nogales. In this method, a hypothetical wave height H' is determined, whose maximum orbital velocity is the same as that which exists at the depth d. This value is

$$H' = \frac{\pi H_s^{\ 2}}{L_o \sinh \frac{2\pi d}{L}} \tag{44}$$

where H_s, the height of a wave steepened by the breakwater, is determined by extending equation (43) to points over the breakwater slope. This hypothetical wave height is then substituted in equation (41) or (42) with K' = 0.015 or 0.019 (as determined from Iribarren).

225. Charts and Tables (Description). - The following charts and tables have been included to facilitate the solution of the basic equation

$$W = \frac{C\ K'\ S_r\ (H \text{ or } H')^3}{(1.09 \cos \alpha - \sin \alpha)^3\ (S_r - S_f)^3} \tag{45}$$

where C = 88.3 and S_f = 1.03 for sea water (46)
and C = 72.4 and S_f = 1.00 for fresh water (47)

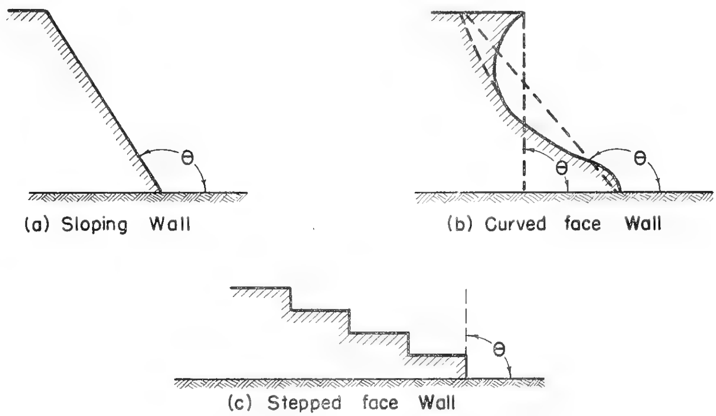


FIGURE 81. Wall shapes

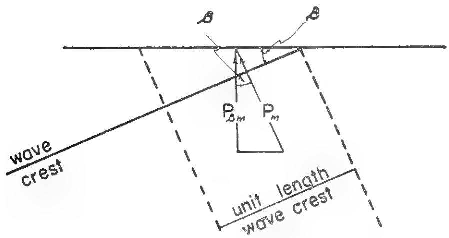
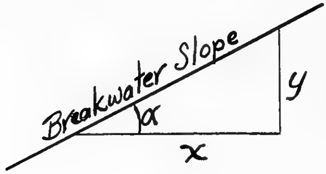


FIGURE 82. Wave approaching wall at angle β

Table 10 - Values of $(1.09 \cos \alpha - \sin \alpha)^3$ for Various Breakwater Slopes.



$$\text{Slope} = \frac{y}{x} = \frac{1}{\cot \alpha}, \quad \text{Slope ratio} = \cot \alpha$$

Slope Angle (α in degrees)	Slope Ratio (cot α)	Slope and Friction Function $(1.09 \cos \alpha - \sin \alpha)^3$
45°	1	2.57×10^{-4}
42° 16'	1.1	2.41×10^{-3}
39° 48'	1.2	7.68×10^{-3}
38° 40'	1.25	0.0116
37° 34'	1.3	0.0164
35° 32'	1.9	0.0286
33° 41'	1.5	0.0438
32° 00'	1.6	0.0613
29° 45'	1.75	0.0912
26° 34'	2.00	0.147
23° 58'	2.25	0.205
21° 48'	2.5	0.258
19° 59'	2.75	0.315
18° 26'	3.0	0.370
17° 06'	3.25	0.418
15° 57'	3.5	0.462
14° 56'	3.75	0.503
14° 02'	4.0	0.541
13° 14'	4.25	0.576
12° 32'	9.5	0.608
11° 53'	4.75	0.638
11° 19'	5.0	0.664
10° 18'	5.5	0.711
9° 28'	6.0	0.755
8° 45'	6.5	0.792
8° 08'	7.0	0.824
7° 36'	7.5	0.864
7° 08'	8.0	0.878
6° 43'	8.5	0.900
6° 20'	9.0	0.921
6° 01'	9.5	0.939
5° 43'	10.0	0.955

Table 10 - lists values of angle α, the slope ratio, and $(1.09 \cos \alpha - \sin \alpha)^3$ for slopes ranging from 1 on 1.1 to 1 on 10)

Plates 7-11, Appendix D - are curves showing the variation of W/K' from equation (46) for seawater, with wave height H (or H'). Values of the functions have been computed for slopes of 1 on 1 1/4 to 1 on 10 with stone density being used as a parameter.

Plate 12, Appendix D - shows the variation of K' with d/L for slopes ranging, 1 on 1 to 1 on 3.

226. Applications. - Since the equations for above surface and sub-surface stone weights are not entirely compatible, the determination of sub-surface weights as outlined above should be restricted to depths below a wave height from the surface, slopes for above surface stones being carried to that level. The crest height of a breakwater should be at least one and one-half wave heights above design water level, if relatively complete wave obstruction is desired.

227. Effect of Wave Angle of Approach. - If the waves approach a structure at an angle β (Figure 82), there is a chance that shock pressures will not be produced, but for design purposes, the possibility of their existence must be accepted. However, the pressures developed will be spread over a longer reach of wall than would be the case for normal incidence. Therefore, the horizontal dynamic pressure due to either waves breaking on or seaward of a structure may be considered to be reduced to

$$P_{\beta m} = P_m \sin^2 \beta \tag{48}$$

In the case of a rubble mound structure, the value of H may be considered to be reduced to

$$H_\beta = \sqrt{\sin \beta} \; (H) \tag{49}$$

228. In this case, it is assumed the energy between two orthogonals a unit distance apart is spread along the structure over a distance $1/\sin \beta$ or the energy per unit length of structure is $e \cdot \sin \beta$. Since wave height is proportional to $\sqrt{e}$, $H_\beta = \sqrt{\sin \beta}\,(H)$. No further reduction in wave height is made to compensate for angle of approach because of the irregularity of the structure surface.

EARTH FORCES

229. Active Forces. - The horizontal component of the active earth force is evaluated from the general wedge theory formula:

$$H = 1/2\ wh^2 \tan^2 1/2\ (90 - \phi) \tag{50}$$

where H = the horizontal component of the lateral force
w = the unit weight of fill
ϕ = the internal angle of friction of the material.

230. Equation (50) is used only for vertical walls with substantially horizontal backfill. The structure is assumed to be non-rigid to the extent that an extremely small rotational movement, necessary to produce the internal friction of the backfill, can occur. Table 11 gives values for $\tan^2 1/2\ (90 - \phi)$ for various values of ϕ.

Table 11 - Values of $\tan^2 1/2\ (90 - \phi)$.

ϕ	$\tan \phi$	$\tan^2 1/2\ (90 - \phi)$	ϕ	$\tan \phi$	$\tan^2 1/2\ (90 - \phi)$
0	0	1.00	45	1.000	0.17
10	0.176	0.70	50	1.192	0.13
20	0.364	0.49	60	1.732	0.07
25	0.466	0.41	70	2.748	0.03
30	0.577	0.33	80	5.671	0.01
35	0.700	0.27	90	∞	0
40	0.839	0.22			

231. If the wall is vertical but the fill slopes at an angle p to the horizontal, the complete equation is

$$P = \frac{wh^2}{2} \cos p \, \frac{\cos p - \sqrt{\cos^2 p - \cos^2 \phi}}{\cos p - \sqrt{\cos^2 p - \cos^2 \phi}} \tag{51}$$

232. For structures having a uniform back batter and fills with uniform slope, the general wedge theory, equation (52) may be used to evaluate the magnitude and direction of earth force. This formula takes into account the friction along the surface of the wall. (See Figure 83).

$$P = 1/2 \left[\frac{\sin (\theta - \phi)}{(1 + N) \sin \theta} \right]^2 \frac{wh^2}{\sin (\phi_1 + \theta)} \tag{52}$$

in which $$N = \sqrt{\frac{\sin (\phi + \phi_1) \sin (\phi - p)}{\sin (\phi_1 + \theta) \sin (\theta - p)}}$$

Where P = total force in pounds
h = vertical height of fill in feet
w = unit weight of fill material
ϕ = internal friction angle of fill material
ϕ_1 = friction angle between backfill and face of wall
p = angle between surface of backfill and horizontal plane
θ = angle between back of retaining wall and a horizontal plane.

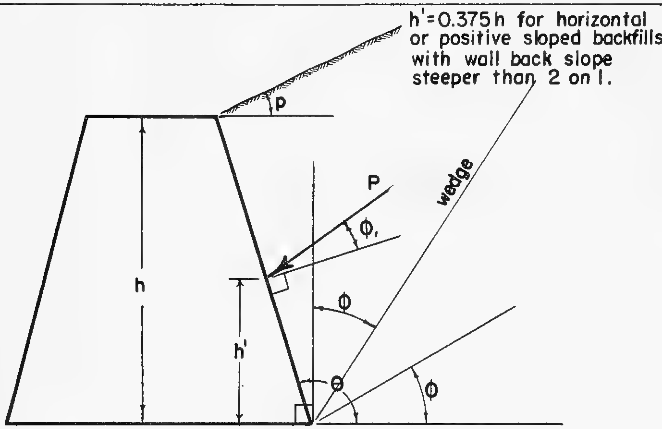
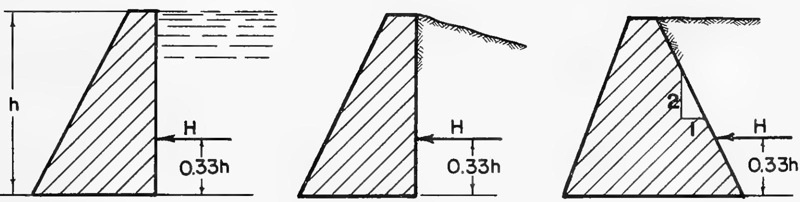


FIGURE 83. Earth pressure on an inclined wall



(a) Liquid pressure

(b) Negative slope backfill

(c) Back batter flatter than 2 on 1

FIGURE 84. Application of resultant of earth pressure

233. Unit Weights and Internal Friction Angles. - The unit weight (w) of typical materials and their internal friction angle (ϕ) are given in Table 12. These are average or normal values of w and ϕ.

Table 12 - Unit Weights and Internal Friction Angles.

Material	Weight Dry	Weight Natural Drained	Weight Submerged	Internal Friction Angle
	lbs. / cu.ft.	lbs. / cu.ft.	lbs. / cu.ft.	degrees
Clay, Soft	60 - 90	100 - 120	35 - 55	10
Clay, Compact	90 - 115	120 - 135	55 - 70	20 - 25
Gravel	105 - 120	115 - 135	65 - 75	40 - 50
Silt, Compact	85 - 105	115 - 130	55 - 65	25 - 40
Silt, Loose	65 - 85	105 - 115	40 - 50	20 - 30
Sand	90 - 105	105 - 120	55 - 65	25 - 40
Sand-Clay, Compact	115 - 130	135 - 140	70 - 80	40 - 50

234. The resultant pressure is inclined from the normal to the back of the wall by the angle of wall friction ϕ. Values for ϕ_1, can be taken from Table 13, but should never exceed the internal friction angle of the backfill material. The vertical component of the earth pressure (P) need not be considered in the stability analysis unless it has considerable effect on the structural design.

Table 13 - Coefficients and Angles of Friction.

Kind of Surface	Coefficient of Friction f	Angle of Friction ϕ_1
Granite, Limestone, Marble:		
Soft dressed on soft dressed	0.70	35° 00'
Hard dressed on hard dressed	0.55	28° 50'
Hard dressed on soft dressed	0.65	33° 00'
Stone, brick, or concrete:		
Masonry on masonry	0.65	33° 00'
Masonry on wood-with grain	0.60	31° 00'
Masonry on wood-cross grain	0.50	26° 40'
Masonry on dry clay	0.50	26° 40'
Masonry on wet or moist clay	0.33	18° 20'
Masonry on sand	0.40	21° 50'
Masonry on gravel	0.60	31° 00'
Soft stone on steel or iron	0.40	21° 50'
Hard stone on steel or iron	0.30	16° 40'

Table 13 cont'd

Note: Angles of friction should be reduced by about 5 degrees if the wall fill will support train or truck traffic. The coefficient of friction f would equal the tangent of the new angle ϕ_1.

235. Earth pressure against structures or irregular section such as stepped stone blocks or those having two or more back batters may be computed by equation 52 by substituting an approximate average back batter to determine the angle (θ).

236. Application of the Resultant of Earth Force. - For all structures with hydrostatic pressure or negative sloped backfills, the resultant of force should be considered to act at 0.33 of the height. For structures having batters on the back face equal to or greater than 2 on 1, the earth pressure should also be applied at 0.33 h. For structures with vertical back faces (or batters less than 2 on 1) and horizontal or positive sloped backfills the earth pressures should be applied at 0.375 h. (See Figures 83 and 84).

237. Earth Pressures - Passive Forces. - Though the passive resistance of an earth mass to movement is very much greater than the active pressure, to develop this passive resistance to movement, the wall must translate or rotate to a significant degree. Therefore, except for sheet pile structures, the passive resistance should not be considered in a stability analysis. For sheet pile structures the passive earth resistance may be computed as

$$Rp = wh^2 \tan^2 1/2\ (90 + \phi) \qquad (53)$$

238. Surcharge Loads. - For a uniform surcharge the density of backfill material may be assumed to be increased in the following manner:

Let W = the surcharge load in pounds per square foot
w = the unit weight of backfill material, either drained or saturated dependent on backfill condition
h = the vertical height of fill in feet
$h' = \frac{W}{w}$

then w' the new value of unit backfill weight is given by

$$w' = w \frac{(h + 2h')}{h} \qquad (54)$$

When this procedure for dealing with surcharge loads is adopted, the point of application of the active thrust for vertical walls with horizontal or positive sloped backfill should be raised from 0.375 h, at the rate of 0.01 h for each 10 percent increase in the artificial density, to a limit of 0.475 h.

239. Submerged Materials. - Pressures due to submerged fills may be calculated by substituting for w in the preceding equations the unit weight of the material reduced by buoyancy, and adding to the pressures so calculated, the full hydrostatic head of water. Note that for surcharge loads this buoyed unit weight of the material must be increased as shown in the preceding paragraph.

240. Uplift. - For design computations, uplift pressures should be considered as full hydrostatic pressure for walls whose bases are below sea level or for computations involving saturated backfill.

ICE FORCES

241. The common forms of ice are usually classified by the use of terms which indicate the manner of formation or the effects produced. Usual classifications include, sheet ice, shale ice, slush ice, frazil ice, anchor ice, and agglomerate ice.

242. The amount of expansion of water in cooling from 39° F. to 32° F. is 1.32 hundredths of 1 percent, whereas in changing from water at 32° F. to ice at 32° F. the amount of expansion is about 9.05 percent or 685 times as great. It has been found that a change of structure to denser form takes place in the ice when, with a temperature lower than -8° F., it is subjected to pressures greater than about 30,000 pounds per square inch. Excessive pressure, with temperature above -8° F., causes the ice to melt. With the temperature below -8° F., the change to a denser form at high pressure results in shrinkage which relieves pressure. Thus, the probable maximum pressure which can be produced by water freezing in an inclosed space is 30,000 pounds per square inch.

243. Designs for dams include allowances for ice pressures varying from no special allowance to as much as 45,000 to 50,000 pounds per linear foot. The crushing strength of ice has been found to be about 400 pounds per square inch and the thrust per linear foot for various thicknesses of ice as about 28,800 pounds for 6 inches, 57,600 pounds for 12 inches, etc. Structures subject to blows from floating ice should be capable of resisting from 10 to 12 tons per square foot (139 to 167 pounds per square inch) on the area exposed to the greatest thickness of floating ice. Ice also expands when warmed from temperatures below freezing to a temperature of 32° F. without melting.

244. Assuming a lake surface to be free from snow, with an average coefficient of expansion of ice between -20° F. and 32° F. equalling .0000284, the total expansion of a sheet of ice a mile long for a rise in temperature of 50° F. would be 3.75 feet. Normally, shore structures are subject to wave forces comparable in magnitude to the maximum probable pressure that might be developed by an ice sheet. As the maximum wave forces and ice thrust cannot occur at the same time, usually no special allowance for overturning stability to resist ice thrust is made. However,

where heavy ice, either in the form of solid ice sheet or floating ice fields may occur, adequate precautions must be observed to insure that the structure is secure against sliding on its base. Ice breakers may be required in relatively sheltered water where wave action does not require a heavy type structure.

245. Floating ice fields may exert a major pressure on structures, when driven by a strong wind or current, by piling up in large ice packs against the obstructions. This condition must be given special attention in the design of small isolated structures. However, because of the flexibility of the ice field, the pressures exerted probably are not as great as would be caused by a solid ice sheet in a confined area.

246. Ice formations may at times cause considerable damage to shore lines in local areas, but their net effects are largely beneficial. Spray thrown up by wind and wave action during the winter may freeze on the banks and structures along the shore, covering them with a protective layer of ice. Ice piled on shore by wind and wave action does not, in general, cause serious damage to beaches, bulkheads, or protective riprap, and generally provides additional protection against damage from the severe winter storm waves. Some abrasion of timber or concrete structures may be caused and individual members may be broken or bent due to the weight of the ice mass. Piling have been slowly pulled by the repeated lifting effect of ice freezing to the piles or attached obstructions such as wales, and then being forced upward by a rise in water stage or wave action.

MATERIALS

247. The structural design of shore protective works must take into account the effects on the materials used, of the environmental conditions peculiar to the shore line area. General modifying criteria which should be applied to materials commonly used are discussed in the following paragraphs.

248. Concrete. - Concrete exposed to sea water, freezing and thawing, or other destructive agents or conditions should have an ultimate compressive strength of 3,000 pounds per square inch. A rich, dense, stiff mix is to be preferred where placement is to be done underwater. Care should be taken, when reinforcing steel is to be used, to cover the steel adequately thereby minimizing possible spalling and exposure of the steel. Working stresses for those conditions may be found in the Corps of Engineers Engineering Manual for Civil Works, Construction, Part CXXI, Chapter 1, August 1947.

249. Steel. - Where exposed to weathering, allowable working stresses must be reduced to take into account corrosive action, abrasion, or the combination of both which would result in loss of effective steel area. Working stresses for reinforcing and structural steel may be found in the preceding reference.

250. Timber. - Allowable stresses for timber should be those for timbers more or less continuously damp or wet. These working streeses may be found in U. S. Department of Commerce publications dealing with American lumber standards.

251. Stone. - Usually the availability of stone sources determines the quality of stone used in waterfront structures. However, care should be taken to avoid use of stone which may decompose more or less rapidly under wave and water action. Where such stone has been used, the effective life of the structures was decreased considerably.

SEAWALLS, REVETMENTS AND BULKHEADS

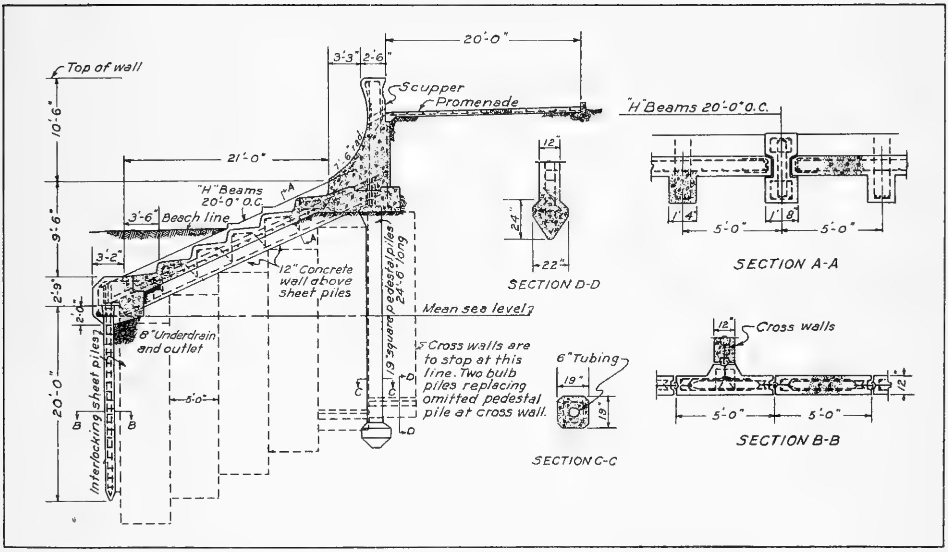
252. Types. - Since seawalls, bulkheads and revetments perform overlapping functions, the structural types will generally be similar. Actually the choice between these three kinds of shore structures is mainly one of nomenclature; overall design features are determined at the functional planning stage and the structure is named to suit its intended purpose. A seawall at one locality may be a bulkhead or revetment at another.

253. In general, however, seawalls tend to be the more massive of the three because consideration must be given to fill retention characteristics as well as to stability against wave forces. Bulkheads are ordinarily next in size for they must withstand wave forces in addition to retaining a fill. Revetments often are the lightest of the three because consideration in their design is given only to their effectiveness in reducing wave-caused erosion.

254. Illustrated in Figure 85 through 93 are various structural types which either have been used, or are typical of those which have been used. Of seawall types, there are four:

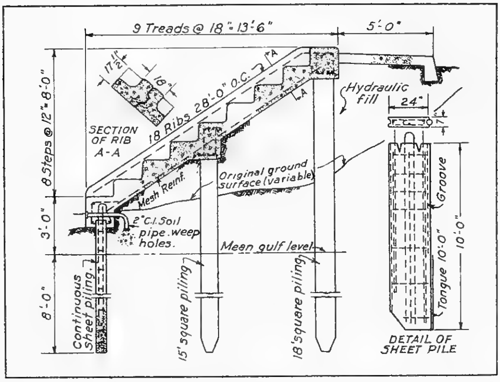
a. Combination stepped and curved face; concrete (San Francisco, California)	Figure 85
b. Stepped face; concrete (Harrison County, Mississippi)	Figure 86
c. Cellular, sheet pile; steel (Typical)	Figure 87
d. Stone; rubble mound (Typical)	Figure 88

The first of these, a combination stepped and curved face wall, is a relatively massive structure, built to resist high wave action and inhibit scour. The second, a stepped face wall, was designed for stability against more moderate wave conditions. Note in both of these the presence of a sheet pile cut-off wall to prevent leaching of the backfill. The typical cellular sheet pile wall is one type which may be used on a rocky bottom where pile penetration is not adequate for inherent pile stability. A concrete, rubble-mound, or cut-stone masonry (not illustrated) wall may also be used under these circumstances. A rubble-mound wall (Figure 88) is useful where bottom conditions may permit some settlement.



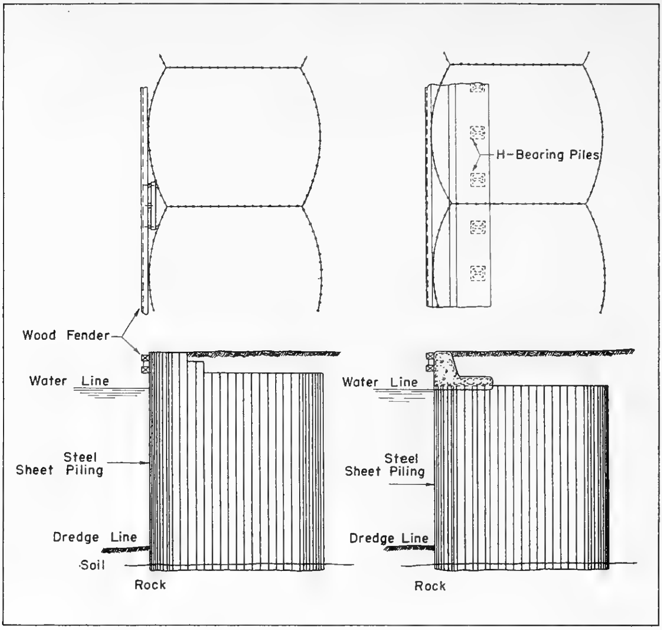
COMBINATION STEPPED AND CURVED FACE WALL; CONCRETE

FIGURE 85

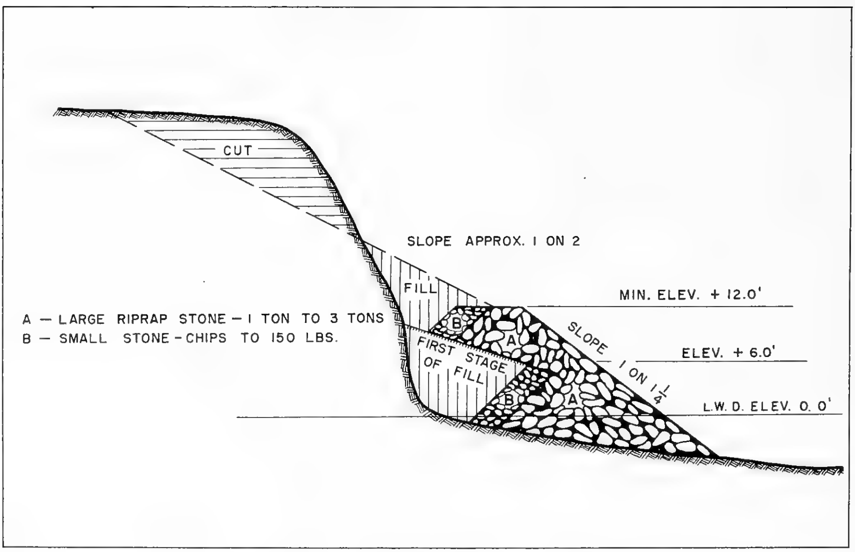


STEPPED FACE WALL; CONCRETE

FIGURE 86



CELLULAR SHEET PILE WALL; STEEL
FIGURE 87



STONE SEAWALL; STAGE PLACED RUBBLE-MOUND
FIGURE 88

255. The bulkheads shown are:

a. Slab and king pile; concrete (Virginia Beach, Virginia)	Figure 89
b. Sheet pile; steel (Typical)	Figure 90
c. Sheet pile; timber (Typical)	Figure 91

These three types are generally interchangeable, though concrete or steel are generally used for higher backfills.

The revetments shown are:

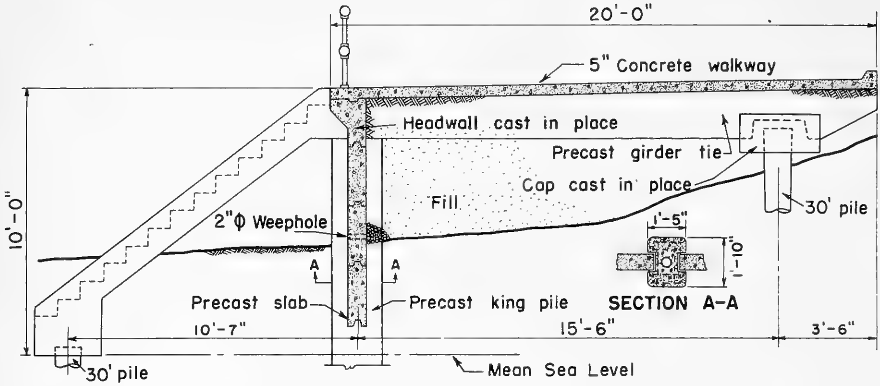
a. Stone (Typical)	Figure 92
b. Concrete (Pioneer Point, Chesapeake Bay, Maryland)	Figure 93

256. Selection of Type. - The major factors which enter the problem of selection of type are: (1) foundation conditions; (2) exposure to wave action; (3) availability of materials; and (4) costs. The following paragraphs illustrate the manner of review for these factors.

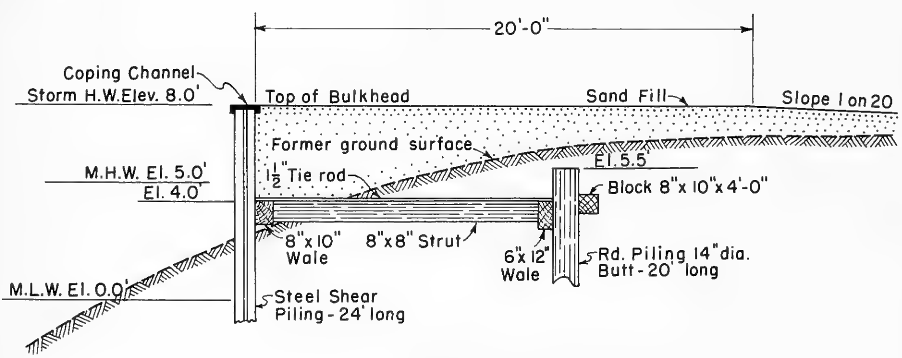
257. Foundation Conditions. - Foundation conditions have a profound effect on the selection of type of structure to construct, often leading to the adoption of a much more costly type than might be suitable under more favorable foundation conditions. Foundations must be considered from two general aspects. First there is the obvious consideration that the bottom as it exists must be suitable for the type of structure. A structure which depends on bottom penetration for stability could not be used on a rocky bottom. Generally, random stone or some type of flexible structure involving a stone mat would be used on soft bottom, though a cellular steel sheet pile structure might be used under these conditions. Second, it should be remembered that the presence of a seawall or bulkhead may change the foundation conditions so that, unless precautions are taken, a structure might fail. Because of induced bottom scour, a foundation otherwise stable could become unstable. For example, a masonry wall or mass concrete wall must be protected from the effects of settlement due to bottom scour induced by the wall itself (see Part I).

258. Exposure to Wave Action. - This factor is most important in the structural design of any one wall or bulkhead, and must also be considered in choosing between structural types. For example, in areas exposed to severe wave action, the lighter types of structures (timber crib, light riprap revetment, etc.) may not be used. Where waves are high, a curved re-entrant face wall might be considered over a stepped face wall.

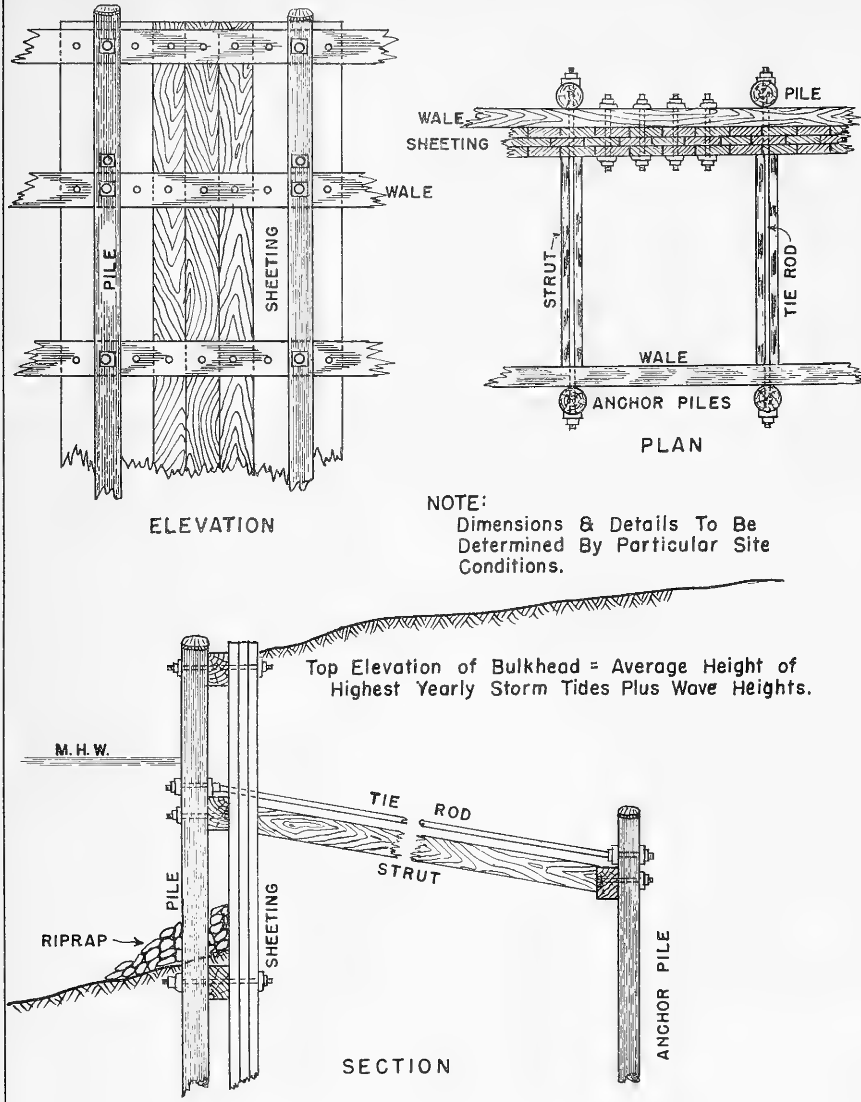
259. Availability of Materials. - This factor would normally be reflected in the cost, as generally, any kind of material can be made available at a price. In times of shortages and restrictions this does not always hold true and more costly structures have been constructed of



SLAB AND KING PILE BULKHEAD; CONCRETE
FIGURE 89

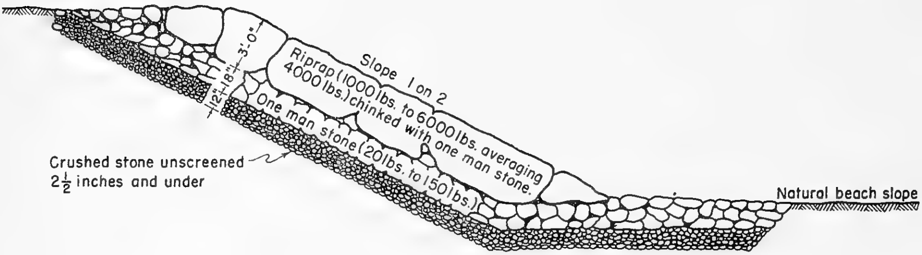


SHEET PILE BULKHEAD; STEEL
FIGURE 90



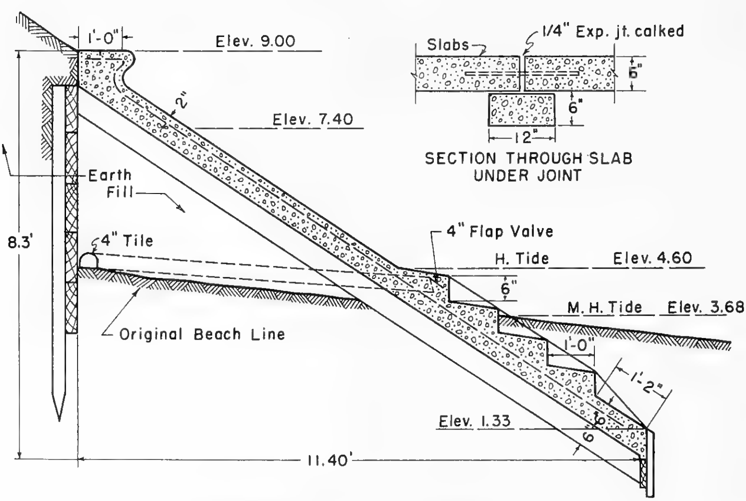
TIMBER SHEET PILE BULKHEAD

FIGURE 91



STONE REVETMENT

FIGURE 92



CONCRETE REVETMENT

FIGURE 93

stone, for example, because of shortages of steel. The price which must be paid for the constituent materials is a major item in first construction and maintenance costs. If these materials are not available near the site of construction, or are in short supply, a particular type of seawall or bulkhead may become economically infeasible to construct. In some instances a compromise may have to be made and a lesser degree of protection provided.

260. Costs. - The analysis of costs must include the first costs of construction, the estimated maintenance charges, and amortization of the investment, over the economic life of the structure. Other things being equal, that structure would be built which would provide the desired degree of protection at the lowest annual or total cost. Because of wide variations in first and maintenance costs, this comparison is usually made by reducing all costs to an annual basis.

TYPE PROBLEMS

261. High Semi-Gravity Type Concrete Wall. - It is desired to build a protective seawall in an area where there is a 6-foot tide and a possible additional 2-foot wind set-up. An erosion trend analysis indicates that the expected ultimate ground line at the wall's location will be 2 feet below mean low water, with a bottom slope before the wall of approximately 1 on 20.

262. Analysis of refraction diagrams indicates that waves from a certain direction may approach the area without decrease in height due to refraction while waves from other directions will be significantly decreased. Analysis of synoptic weather charts indicates that waves from this direction, under storm conditions, may have deep water heights up to 8 feet and periods from 7 to 11 seconds.

263. The backfill material has a unit weight of 120 pounds per cubic foot, and an angle of internal friction of 25°. The backfill will be subject to a uniform surcharge load of 250 pounds per square foot. The foundation material has a bearing capacity of 2,000 pounds per square foot.

264. Design. - General. - To reduce water overtopping, the face of the wall will have a re-entrant angle of 15° to the vertical. To reduce beach scour, the face of the wall will be stepped. The wall base is to be at least 2 feet below the ultimate ground line. A sheet pile cut-off wall is to be placed at the toe as a safety factor to prevent damage to the wall by undercutting of the foundation should the beach lower to a greater depth than estimated.

265. Computation of Wave Forces. - Deep water wave lengths may be computed from the relationship $L_o = 5.12\ T^2$. With these the following table should be drawn up to determine whether waves may break on the structure.

TABLE 14 - Breaker Position

(1) H_o	(2) T	(3) L_o	(4) H_o/L_o	(5) d_b/H_o(curves)	(6) d_b'/H_o (calculated)
(feet)	(seconds)	(feet)			
8	7	251	0.032	1.25	1.25
8	9	415	0.019	1.3	1.25
8	11	620	0.013	1.5	1.25

Column (1) H_o = deep water design wave height from synoptic charts

Column (2) T = wave period from synoptic charts

Column (3) L_o = wave period wave length from $L_o = 5.12\ T^2$

Column (4) H_o/L_o = column (1) divided by column (2)

Column (5) d_b/H_o (curve) = These values are tabulated from the curve of Figure 18

Column (6) d_b'/H_o (calculated) = The design depth at the structure divided by column (1) = 10/8 = 1.25.

266. For the 7-second period, the fact that columns (5) and (6) are the same, indicates that a 7-second wave may break directly on the wall. The values for the 9-second period are so close that breaking waves may again be assumed. For these two periods, then, wave forces must be determined by Minikin's criteria. For the 11-second wave, column (5) is the larger than column (6) indicating that waves would break seaward of the wall, and the force criteria for broken waves apply. If column (6) had been smaller than column (5) the Sainflou criteria would be used.

267. Breaking Waves. - The curve, U.C., of Figure 17 may be used to determine the breaking wave height H_b.

TABLE 15 - Breaking Wave Heights

Period T	H_o/L_o	H_b/H_o	H_b
(seconds)			
7	0.032	1.1	9
9	0.019	1.25	10
11	0.013	1.4	11

H_b = the design wave height at the structure.

268. The Minikin relationships for shock pressures, thrusts, and overturning moments due to wave action are:

$$P_m = 101w\, H_b/L_o \quad d(1+d/D) \text{ for maximum pressure} \tag{55}$$

$$R_m = P_m \times H_b/3 \quad \text{for total thrust per foot of wall} \tag{56}$$

$$M_m = R_m \times (d+2) \quad \text{for overturning moment (note that the moment arm is } d + 2 \text{ since the wall base is 2 feet below ultimate ground level)} \tag{57}$$

where L_o and D may be determined as follows for the 7 and 9-second wave periods with d = 10 feet. Values for d/L_D for the various values of d/L_o may be taken from Table 1, Appendix D.

TABLE 16 - Determination of D and L_D

(1) Period T (seconds)	(2) L_o (feet)	(3) d/L_o	(4) d/L_d	(5) L_d (feet)	(6) D (feet)	(7) D/L_o	(8) D/L_D	(9) L_D
7	251	0.040	0.083	120	16	0.064	0.11	145
9	415	0.024	0.063	160	18	0.043	0.087	207

269. Therefore the shock pressures, thrust, and moments about the base due to the 7 and 9-second waves respectively are given in Table 17.

TABLE 17 - Shock Pressures, Thrusts and Moments

Period T (seconds)	Breaker Height H_D (feet)	Pressure (lbs.per sq.ft.)	Thrust (lbs.per sq.ft.)	Moment (ft.-lbs/ft.)
7	9	6540	19,620	236,000
9	10	4860	16,200	194,500

270. Since the wall is to be founded 2 feet below the ultimate ground line, the static forces on the face of the wall due to wave action may be computed as that force due to a head of water of height (d + 2 + H/2). Therefore, the maximum static pressure will occur at the wall base (the depth d_6) and is given by

$$P_d = w(d+2+H/2) \tag{58}$$

the total thrust per linear foot of wall is given by

$$R_d = P_d \frac{(d + 2 + H/2)}{2} \quad (59)$$

and the overturning moment is given by

$$M_d = R_d \frac{d \pm 2 \pm H/2}{3} \quad (60)$$

271. For the 7 and 9-second waves, these values are

TABLE 18 - Static Pressures, Thrusts and Moments

Period T	Breaker Height H_b	Pressure	Thrust	Moment
(seconds)	(feet)	(lb./ft.²)	lbs./ft.)	(ft.-lbs./ft)
7	9	1,060	8,750	48,000
9	10	1,090	9,360	52,500

The manner in which these breaking wave pressures are applied is illustrated in Figure 94.

272. Broken Waves. - To evaluate the dynamic force due to the 11-second wave, the relationships

$$P_m = (wd/2)(1 - x_1/x_2)^2 \quad \text{for maximum pressures} \quad (61)$$

$$R_{d^-} = P_m\,(h_c) \quad \text{for total thrust per foot of wall} \quad (62)$$

$$M_m = \left(R_{d-}\right)\left(h_c/2 + \left|h_w\right|\right) \quad \text{for maximum moment} \quad (63)$$

may be used, where

d = the breaker depth for the 11-second wave and equals $8 \times 1.5 = 12$ feet . (Table 14)

x_2 = 12' x 20 = 240' = landward limit of uprush

x_1 = 240 - 10 x 20 = 40' distance of wave crest to wall

h_c = elevation of wave crest over still water level = 0.7 x 11 = 8 feet

h_w = elevation of wall base above still water level measured positively upward = -10 feet.

Therefore for the 11-second wave, the dynamic pressure is 223 pounds per square foot, the dynamic thrust would be 1,780 pounds per lineal foot of

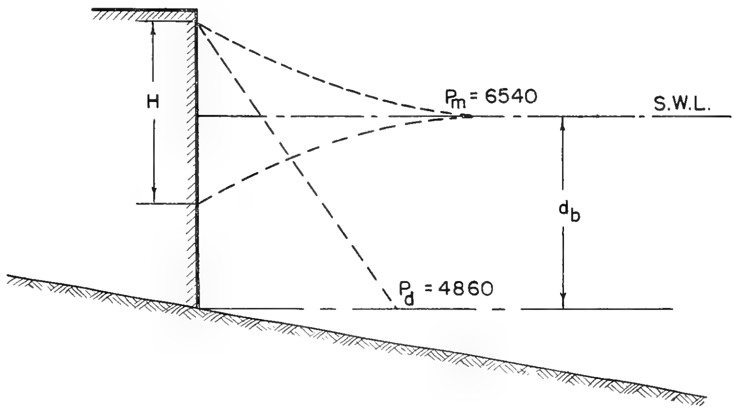


FIGURE 94 - Breaking Wave Pressure Diagram

wall, and the overturning moment would be 25,000 foot-pounds per lineal foot of wall.

273. The equivalent static expressions at the depth d_b are

$$P_s = w(h_c - h_w) \text{ for maximum pressure at the wall base} \quad (64)$$

$$R_s = P_d \frac{(h_c - h_w)}{2} \text{ for total thrust per linear foot of wall} \quad (65)$$

$$M_s = R_d \frac{(h_c - h_w)}{3} \text{ for overturning moment} \quad (66)$$

The static broken wave pressures, forces, and moments are 1,120 pounds per square foot at the base, 10,100 pounds per lineal foot of wall, and 60,500 foot-pounds per lineal foot of wall.

274. Wave Forces. - The preceding computations may be summarized in the following tabulation:

Table 19 - Wave Pressures, Thrusts and Moments About Wall Base

Period T	Breaker Height H_o	Pressure		Thrust	Moments
(seconds)	(feet)	(lbs. per sq. ft.)		(lbs./ft.)	(ft.-lbs/ft)
7	9	Dynamic	6,540	19,620	235,000
		Static	1,060	8,750	48,000
		TOTAL		28,370	284,000
9	10	Dynamic	4,860	16,200	194,500
		Static	1,090	9,360	52,500
		TOTAL		25,560	247,000
11	11	Dynamic	223	1,780	25,000
		Static	1,120	10,100	60,500
		TOTAL		11,880	85,500

Note that in this case the thrusts and moments are highest for the lowest breaking wave heights.

275. Wall Height. - Though the forces due to the 11-second wave would be far less than those due to the 7 and 9-second waves, the wave crest height above sea level would be greater. To prevent significant overtopping, this height would be used for the design wall height. That is, the wall crest would be 8 feet above the still water level.

276. Wall Design. - A wall section conforming to the previously noted general design criteria and to the above is shown in Figure 95.

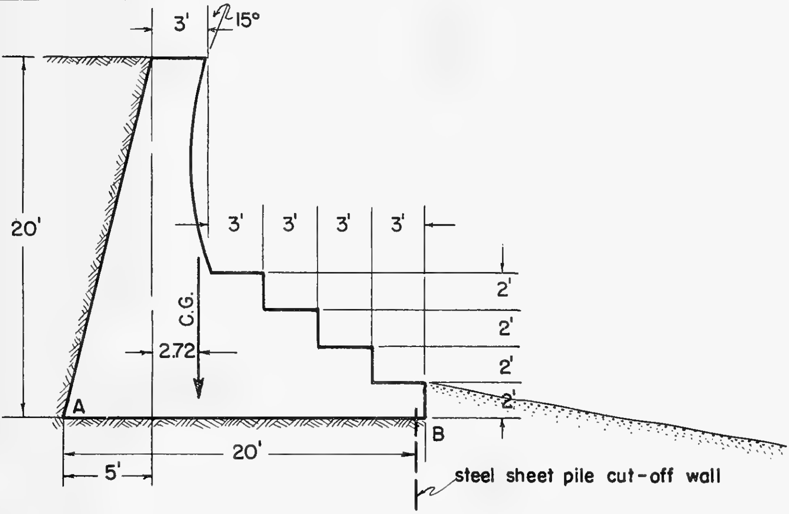


FIGURE 95 - Cross section of concrete wall

The weight of concrete in the wall, considering concrete to have a unit weight of 150 pounds per cubic foot, is 24,500 pounds per lineal foot. The wall center of gravity determined by taking moment areas about A is 7.72 feet seaward of A.

277. Stability Calculations. - Wall Overturning Landward. - Maximum landward overturning moments would occur with maximum wave action on the face of the wall and with the backfill unsaturated. Wave forces and uplift would tend to cause the wall to overturn landward. (From Table 19 the design wave forces are those due to the 7-second period wave.) The overturning moment would be resisted by the moments due to the weight of water over the toe of the seawall, the weight of the wall, active earth pressure on the back of the wall, and the weight of earth on the back of the wall.

278. Wave Forces and Moments. - Total wave forces and moments about A are tabulated below.

TABLE 20 - Wave Forces and Moments

Force	Arm (A)	Moment about A
(lbs. per ft.)	(feet)	(ft.-lbs. per ft.)
Wave (Horizontal) = 28,370		-284,000
Water (Vertical) = 8,850	14.7	+130,000

279. Earth Forces and Moments. - For stability calculations it will be assumed that the backfill is unsaturated. Then the total earth thrust is given by

$$P = \frac{w' h^2}{2} \left(\operatorname{Csc}(\phi_1 + \theta)\right) \left[\frac{\sin(\theta - \phi)}{(1 + n)(\sin\theta)}\right]^2 \qquad (67)$$

where

$$w' = w\left(\frac{h + 2h'}{h}\right)$$

$$n = \sqrt{\frac{\sin(\phi + \phi_1 \sin\phi}{\sin(\phi + \theta)\sin\theta}} \qquad (68)$$

$$h' = \frac{W}{w}$$

and for this wall

$W = 250$ pounds per square foot = the surcharge load

$w = 120$ pounds per cubic foot = the backfill unit weight

$h = 20$ ft. = the backfill height

$\theta = 90 + \tan^{-1}\left(\frac{5}{20}\right) = 104^\circ\ 2'$ = the wall backface angle with the horizontal

$\phi = 25^\circ$ = the backfill internal friction angle

$\phi_1 = 17^\circ$ = the backfill wall friction angle

This thrust will act at a distance $\left(0.375 + \frac{w' - w}{10w}\right)$ H above the base and at an angle $(\theta - 90 + \phi_1)$ with the horizontal. By resolving P into horizontal (P_H) and vertical (P_V) components we may find the thrust and moments about A. These are as follows:

TABLE 21 - Earth Forces and Moments

Force	Arm (from A) (feet)	Moment (ft. - lbs.per ft.)
Earth (Horizontal), P_H = 11,500 (lb/ft)	7.86	+90,500
Earth (Vertical), P_V = 6,950 (lb/ft)	2.36	+16,400

280. Uplift Forces and Moments. - Uplift forces may be computed as a straight line triangular variation, assuming a head of 12.5 feet at the toe and none at the heel.

TABLE 22 - Uplift Forces and Moments

Force (lbs. per foot)	Arm (from A) (feet)	Moments (ft. - lbs.per ft.)
Uplift (Vertical) 8,020	13.33	-107,000

281. Stability Table. - Summarizing the preceding, the following table may be set up:

TABLE 23 - Stability Table, Landward Overturning of Moments

Force	(lbs.per ft.)	Arm (from A) (feet)	Moments about A (ft.-lbs. per ft.)	
Concrete (Vertical)	24,500	7.72	+189,000	
Earth (Horizontal)	11,500	7.86	+ 90,500	
Earth (Vertical)	6,950	2.36	+ 16,400	
Water (Vertical)	8,850	14.7	+130,000	
Wave (Horizontal)	28,370			-284,000
Uplift (Vertical)	8,020	13.33		-107,000
TOTAL			+425,900	-391,000
NET TOTAL			+ 34,900	

The moments which would cause overturning around point A are negative, therefore the wall is stable under wave forces.

282. Stability Calculations. - Wall Toppling Seaward. - Maximum moment causing the wall to overturn seaward would occur with no water in front of the wall, and assuming a saturated backfill and a surcharge of 250 pounds per square foot. The earth forces and moments may be computed

in the same manner as for the unsaturated case. All angles would be the same, but

$$w = 145.7$$
$$h' = 1.72$$

so

$$w' = 171 \text{ pounds per cubic foot}$$

The earth thrust will therefore be in the proportion 171/141 to the force calculated for an unsaturated fill, and will be applied at a height of 7.84 feet above the base. On resolving the thrust into horizontal and vertical components, the forces and moments about B may be tabulated.

TABLE 24 - Earth Forces and Moments

Force (lbs.per ft.)	Arm (from B) (feet)	Moment (ft.-lbs.per ft.)
Earth (Horizontal), $P_H = 13,950$	7.84	+109,000
Earth (Vertical), $P = 8,400$	17.65	+148,000

283. Uplift Forces and Moments. - Uplift may be calculated for conservative conditions by assuming the water level at the toe to be at the base. Then the uplift force diagram is triangular and the thrust and moment about B is given in Table 25.

TABLE 25 - Uplift Forces and Moments

Force	(lbs. per ft.)	Arm (from B) (feet)	Moment (ft.-lbs.per ft.)
Uplift	12,810	13.33	+171,000

284. Stability Table. - Summarizing the preceding, Table 26 may be set up as follows:

TABLE 26 - Stability, Overturning Seaward

Forces	(lbs.per ft.)	Arm (from B) (feet)	Moment about B (ft.-lbs. per ft.)	
Concrete (Vertical)	24,500	12.18		-298,000
Earth (Vertical)	8,400	17.65		-148,000
Earth (Horizontal)	13,950	7.84	+109,000	
Uplift (Vertical)	12,840	13.33	+171,000	
TOTAL			+280,000	-446,000
NET TOTAL				-166,000

The moments which would cause overturning around point B are positive, therefore the wall is stable under saturated earth forces.

285. Internal Stresses. - Overturning Landward. - Although the wall is stable against overturning, calculation of the resultant of the forces indicates that this resultant would fall outside the middle third of the base. The total vertical force of 32,280 pounds per foot downward would be applied 7.16 feet from A. The total horizontal force, which would be 16,870 pounds per foot directed landward, would be applied 11.5 feet above the base. From Figure 96, the resultant of these forces would fall 1.1 feet from A inside the base.

286. It would be possible, by adding to the area of concrete to bring the resultant of the forces within the middle third, or at least to reduce the tension in the base. However, because of the high dynamic pressures of the breaking waves, to eliminate tension completely (or even to materially reduce the tension) would require an excessively large amount of additional concrete. Accordingly the economics of providing the additional concrete as opposed to the provision of tension steel must be weighed locally. No such analysis will be made for the purpose of design illustration. It should also be noted that tension steel for the given design will be required in the front face of the wall.

287. Foundation Pressures. - Overturning Landward. - The bearing pressure of the loads may be calculated from

$$f = \frac{P_V}{A} \pm \frac{M}{Z} \qquad (69)$$

where P_V = sum of vertical loads = 32,280

A = base area (unit length of wall) = 20

M = the total moment about the base centerline
$= 16{,}870 \times 11.5 + 32{,}280(10-7.16) \cong 287{,}000$

Z = the section modulus (unit length of wall) $= \frac{20^2}{6}$

Therefore $f = \frac{32{,}280}{20} \pm \frac{6 \times 287{,}000}{400}$

$\cong 1600 \pm 4300$

The pressure distribution is shown by DC in Figure 97.

288. Bearing Piling. - Since the maximum base pressure exceeds the allowable bearing capacity of the foundation material, piles must be used to support the wall. The design piling load along a section of wall may be calculated by uniformly subtracting the bearing capacity of the foundation material from the total foundation pressure. The remaining foundation pressures must be supported by piles. In this case, subtracting the bearing capacity of the soil as shown by the line EF on Figure 97, piling would be required to support the load DEG, which is

$$F_{DEG} = \frac{1}{2} \times 3900 \times \overline{EG}$$

$\overline{EG}$ being given by $\frac{\overline{EG}}{3900} = \frac{20}{8520}$, $\overline{EG} = 9.1$

Therefore F = 17,700 pounds per foot applied at a distance 1/3 $\overline{EG}$ = 3.03 feet from A.

289. Allowable pile loading varies with the material in which the pile is driven. If an individual pile rests on a hard stratum its bearing capacity is determined by the ultimate strength in compression of the pile material. Essentially the pile becomes a totally laterally supported column. However, when a pile's bearing load is supported primarily by skin friction with the material through which it is driven, its bearing capacity must be determined by empirical means.

290. For most foundation materials on which seawalls will be built, the resistance to a static load is almost the same as that offered to the dynamic load of driving. The allowable bearing load per pile can thus be calculated by relating the depth of penetration of a test pile per blow to the hammer weight. Relationships commonly in use for timber and concrete piling are as follows:

TABLE 27 - Safe Load Per Pile

Driver	Timber	Concrete
Drop Hammer	$P = \frac{2Wh}{S + 1}$	$P = \frac{2Wh}{s\ (1 + W_p/W}$
Steam hammer, single acting	$P = \frac{2Wh}{s + 0.1}$	$P = \frac{2Wh}{s + 0.1\ W_p/W}$
Steam hammer, double acting	$P = \frac{2h\ (W + ap)}{s + 0.1}$	$P = \frac{2\ (W + ap)}{s + 0.1\ W_p/W}$

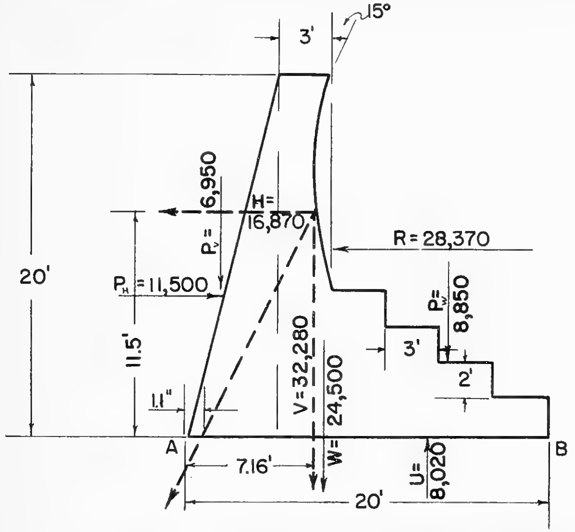


FIGURE 96 - Force resultant overturning landward

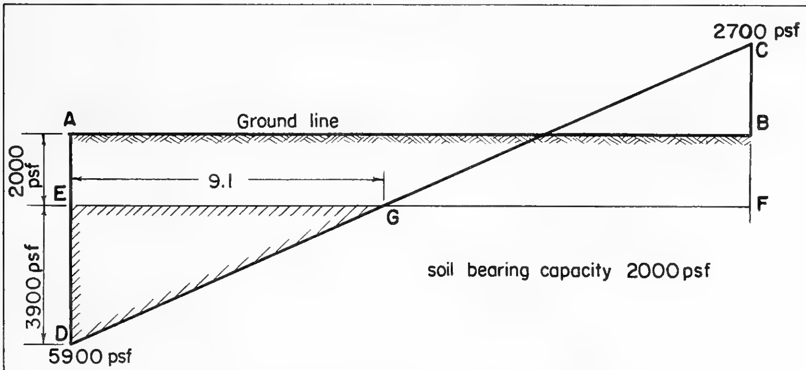


FIGURE 97 - Bearing pressures, overturning landward

P = safe bearing load, pounds

W = hammer weight, pounds

W_p = pile weight, pounds

h = free fall of hammer, feet

a = effective area of piston, inches

p = mean effective stream pressure, pounds per square inch

s = penetration of sinking, inches
(taken as average of last 5 to 10 blows for a drop hammer and as average of last 20 blows for a steam hammer).

291. These formulas will give safe loadings of individual piles provided the individual piles of a group are not so closely spaced that skin friction is materially reduced.*

292. Internal Stresses. - Overturning Seaward. - On obtaining the resultants of vertical and horizontal forces it is found that this resultant would fall within the middle third of the base. The total vertical force would be 20,060 pounds per foot downward and would be applied 3.75 feet from B. The total horizontal thrust which would be 13,950 pounds per foot directed seaward, would be applied 7.89 feet above the base. From Figure 98, the resultant of these forces would fall 8.29 feet from B within the base, or well within the middle third. Therefore, there would be no tension in the base due to the force of the saturated fill.

293. Foundation Pressures. - Overturning Seaward. -

Using $f = \frac{P_v}{A} \pm \frac{M}{Z}$ (see equation 69)

where $P_v = 20{,}060$

$A = 20$

$M = 13{,}950 \times 7.84 - 20{,}060\ (13.75 - 10) = 32{,}000$

$Z = 20^2/6$

$f = \frac{20{,}060}{20} \pm \frac{32{,}000 \times 6}{400}$

$= 1003 \pm 480$

The pressure distribution would be as shown in Figure 99 by the line CD.

* See C. Terzaghi, Trans. ASCE, Vol. 93, p. 290.

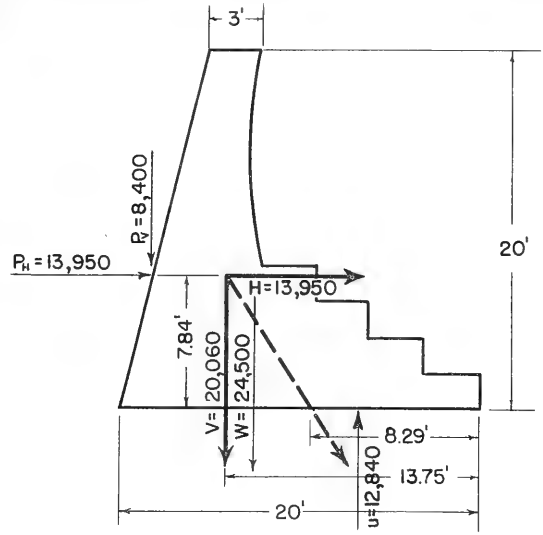


FIGURE 98 - Force resultant overturning seaward

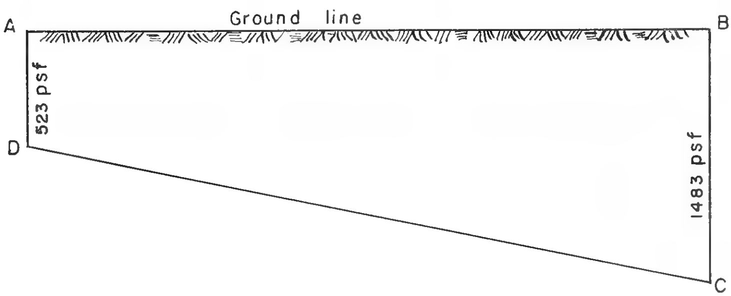


FIGURE 99 — BEARING PRESSURES, OVERTURNING SEAWARD

Since the bearing capacity of the foundation is 2,000 pounds per square foot, no piling would be required under these force conditions.

294. Steel Sheet Pile Cellular Seawall. - A wall is to be placed in a low bluff area, to protect the bluff line from the action of storm waves. The water depth at the toe of the wall is to be 3 feet below low water datum. There is a maximum expected wind set-up of 2 feet. The slope of the bottom seaward of the site of the wall is approximately 1 on 40. The elevation of the top of the bluffs is about 10 feet above low water datum. This would also be the elevation of the top of the backfill. Wave analysis indicates that the maximum deep water wave height expected is 10 feet, and that a wave this high will have a period of 12 seconds. The maximum refraction coefficient is 0.5, at a depth of 50 feet.

295. The bottom is composed of a 2-foot thickness of gravel and shale overlying bedrock. Because of poor penetration possibilities, a sheet pile, cellular type structure was chosen for design.

296. The backfill material has a natural drained unit weight of 130 pounds per cubic foot and an internal friction angle of 25°. The cell fill material (gravel) has a natural drained unit weight of 120 pounds per cubic foot and an internal friction angle of 45°. The coefficient of friction between the cell fill and the bottom is 0.5.

297. Wave Forces. - The deep water wave length of a 12-second wave is $L_o = 5.12\ (12)^2 = 737$ feet. At a depth of 50 feet $d/L_o = 50/737 = 0.068$. From Table 1 of Appendix D the corresponding $d/L = 0.112$ and $H/H'_o = 0.98$. Therefore, the wave height in this depth of water where the refraction coefficient is 0.5 for a wave 10 feet high in deep water is

$$H = H_o \times k \times (H/H'_o) = 10 \times 0.5 \times 0.98 \cong 5 \text{ feet} \tag{70}$$

From this equation, k must be equal to (H'_o/H_o) where H'_o is the deep water wave height which, in the absence of refraction, would give a wave height of H in a depth of 50 feet. This equivalent deep water wave height H'_o is 5 feet.

298. A wave with deep water length of 737 feet and deep water height of 5 feet will have a steepness of $H'_o/L_o = 5/737 = 0.0068$. From Plate 5a, Appendix D, the ratio $d_b/H_o = 2.0$, from which d_b, the breaking depth, is $d_b = 2.0 \times 5 = 10$ feet. The maximum anticipated depth at the wall including tidal rise is 5 feet, which indicates that waves will break before reaching the structure, and that the wave force criteria for broken waves must be used for this deep water design wave.

299. Forces Due to Broken Waves. - Referring to Plate 5b, Appendix D, H_b/H_o for a wave whose deep water steepness $(H_o/L_o) = 0.0068$, moving up a 1:40 slope is $H_b/H_o = 1.5$. Therefore the breaking wave height is 7.5 feet.

300. The dynamic pressure (P_m) and hydrostatic pressure at the base (P_s) are given by

$$P_m = \frac{wd}{2} \left(1 - \frac{x_1}{x_2} \right)^2 \tag{71}$$

and $$P_s = w \, (h_c - h_w) \tag{72}$$

where w = 62.4 pounds per cubic foot = unit weight of water

d = 10 feet = breaking depth, referred to the <u>maximum</u> water level

h_c = 0.7 x H_b = 5.25 feet = the breaking wave crest elevation above the maximum water level

h_w = -5 feet = the elevation, measured positively upward, of the wall base above maximum water level

x_1 = slope x ($d + h_w$) = 40 (10 - 5) = 200 feet = the distance from the line of breakers to the w all

x_2 = slope ($d + h_c$) = 40 (10 + 5.25) = 610 feet = the distance from the line of breakers to the landward limit of uprush

Therefore $P_m = \frac{62.4 \times 10}{2} \left(1 - \frac{200}{610}\right)^2$ = 141 pounds per square foot

and P_s = 62.4 (10.25) = 640 pounds per square foot

The pressure distribution is shown in Figure 100. From this figure, the total thrust R is given by

$$\begin{aligned} R &= (R_d) + R_s \\ &= P_m \, (h_c) + P_s \left(\frac{h_c - h_w}{2}\right) \\ &= 145 \, (5.25) + 658 \left(\frac{10.25}{2}\right) = \\ &= 760 + 3390 = 4150 \text{ pounds per lineal foot of wall} \end{aligned} \tag{73}$$

and the total moment about the base M is given by

$$\begin{aligned} M &= M_m + M_d \\ &= (R_d)\left(\frac{h_c}{2} + |h_w|\right) + R_d \left(\frac{h_c - h_w}{3}\right) \\ &= 760 \left(\frac{5.25}{2} + 5\right) + 3390 \left(\frac{10.25}{3}\right) \end{aligned} \tag{74}$$

= 5800 + 11,600 = 17,400 foot pounds per lineal foot of wall

301. <u>Forces Due to Breaking Waves.</u> - Though the maximum deep water wave breaks before reaching the wall, some lesser height wave will break right at it. This wave height may be approximated by the relationship $d_b/H_b = 1.3$ where d_b is the maximum water depth at the wall. In this case then, $H_b = 5/1.3 = 3.8$ feet. To use the Minikin relationships for forces, it is necessary to determine D and L_D as indicated in paragraph 209. In tabular form, the computations are as follows:

TABLE 28 - Determination of D and L_D for d = 5 feet

L_o (feet)	d/L_o	d/L_d	L_d (feet)	D (feet)	D/L_o	D/L_D	L_D (feet)
737	0.0068	0.033	$\frac{d}{0.033} = 152$	$\frac{152}{40} + 5 = 8.8$	0.0119	0.044	$\frac{D}{0.044} = 200$

302. The wave shock pressure is given by

$$P_m = \frac{101\ H_b w}{L_D} \times d\left(1 + \frac{d}{D}\right) \tag{75}$$

$$= \frac{101 \times 3.8 \times 64.2 \times 5}{200}\left(1 + \frac{5}{8.8}\right) = 967 \text{ pounds per square ft.}$$

and the maximum hydrostatic pressure at the depth d_b (the base) ignoring any water pressure on the wall backface would be

$$P_d = w\left(d_b + \frac{H}{2}\right) = 64.2\ (5 + 1.9) \tag{76}$$

$$= 444 \text{ pounds per square foot.}$$

The manner of application of these pressures is shown in Figure 101.

From this figure, the total thrust is given by

$$R = R_m + R_d$$

$$= P_m\left(\frac{H_b}{3}\right) + \frac{P_d}{2}\left(d + \frac{H}{2}\right) \tag{77}$$

$$= 967 \times \frac{3.8}{3} + 222\,(6.9)$$

$$= 1222 + 1530 = 2752 \text{ pounds per lineal foot of wall.}$$

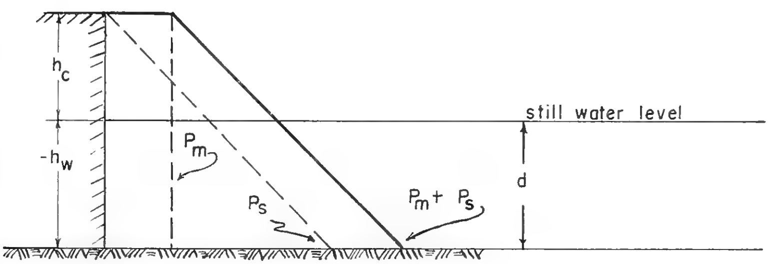


FIGURE 100 - DISTRIBUTION OF PRESSURES; BROKEN WAVES

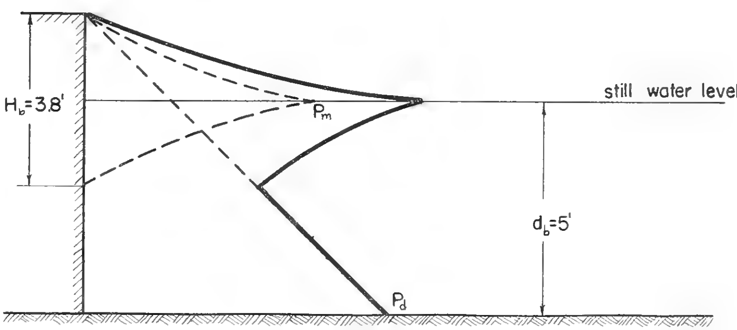


FIGURE 101 - Distribution of pressure; breaking waves

and the total overturning moment would be

$$M = R_m d + \frac{R_d}{3} \quad (d + \frac{H}{2}) \tag{78}$$

$$= 1222 \times 5 + \frac{1530}{3} \quad (6.9)$$

$$= 6110 + 3520$$

$$= 9,630 \text{ foot pounds per lineal foot of wall.}$$

303. Tabulation of Design Wave Forces and Moments. - The forces and moments exerted on the wall by the deep water design wave breaking in 8.8 feet of water are greater than those exerted by some lesser height of wave breaking right on the structure. These larger forces will be used for design.

TABLE 29 - Wave Forces and Moments

Force	Force (pounds per foot)	Moment (foot pounds per foot)
Dynamic	760	5800
Static	3390	11,600
TOTAL	4150	17,400

304. Earth Forces. - The backfill thrust is given by

$$P = \frac{wh^2}{2} \tan^2 \frac{(90 - \phi)}{2} \tag{79}$$

where h = 13 feet = d height of backfill above low water datum

$\phi = 25^{\circ}$

w = 130 pounds per cubic foot = the unit weight of unsaturated fill

= 130 + 0.4 x 64.2 = 155.7 pounds per cubic foot = the unit weight of saturated fill

The point of application of this thrust is at a distance 0.375h above the cell base.

305. Therefore, for an unsaturated backfill

$$P = \frac{[illegible] \times 169}{2} (0.637)^2$$

$$= 4460 \text{ pounds per lineal foot of wall}$$

and for a saturated backfill

$$P = \frac{155.7 \times 169}{2} (0.637)^2$$

$$= 5340 \text{ pounds per lineal foot of wall}$$

each applied at a distance 4.88 feet above the base.

306. The thrust on the sheet pile due to the fill material is similarly calculated with

$$\phi = 45^\circ$$

$w = 120$ pounds per cubic foot for unsaturated fill

$= 120 + 0.4 \times 64.2 = 145.7$ pounds per cubic foot for saturated fill

307. Therefore for an unsaturated cell fill

$$P = \frac{120 \times 169}{2} (0.414)^2$$

$$= 1,730 \text{ pounds per lineal foot of wall}$$

and for a saturated cell fill

$$P = \frac{145.7 \times 169}{2} (0.414)^2$$

$$= 2110 \text{ pounds per lineal foot of wall}$$

308. Note, expecially for the cell fill, that the maximum pressure (not thrust) is given by

$$p = \underline{wh} \tan^2 \left(\frac{90 - \phi}{2}\right)$$

and occurrs at the cell bottom. For the case being considered with a saturated cell fill, this pressure is

$$p = 145.7 \times 13 (0.414)^2$$

$$= 324 \text{ pounds per square foot}$$

and with an unsaturated cell fill

$$p = 120 \times 13 (0.414)^2$$

$$= 267 \text{ pounds per square foot}$$

309. <u>General Design</u>. - The type wall chosen is the so-called diaphragm type illustrated in Figure 102. The dashed lines indicate the dimensions of an equivalent rectangular cell used for stability calculations. The width (b) of the equivalent rectangle is the average width of the actual cell, the length (L) is the length of one cell section; and the distance (r) is the radius of the outside walls. Note that the crosswalls have a slight arc for stability against differential earth pressures due to partial filling of one cell before filling of the adjacent cell is started.

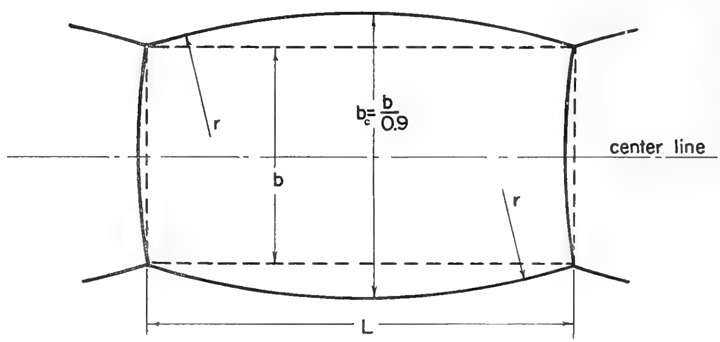


FIGURE 102 - Diaphragm type cell wall

310. Failure criteria for such walls, if the piling has a significant amount of penetration, are difficult to determine. However there will be an adequate factor of safety if they are considered to be open ended boxes resting on the bottom. In the example chosen, the rock stratum is only two feet below the lake bottom and the open ended box calculations will be fairly accurate.

311. <u>Piling Calculations</u>. - Pile interlock tension resistance to cell rupture is directly proportional to the cell fill pressure and the radius of the cell wall. Expressing the fill pressure p in pounds per square foot and the radius in feet, the interlock tension in pounds per linear inch is given by

$$t = \frac{pr}{12} \qquad (80)$$

assuming an M112 section piling would be used, with an allowable interlock tension of 6,000 pounds per inch, the maximum allowable cell wall radius for this problem, with p being the pressure of the saturated cell fill would be

$$r = \frac{12t}{p} = \frac{12 \times 6000}{324}$$

$$= 222 \text{ feet}$$

and for safety will be taken as 50 feet. (Note that any pressure due to hydrostatic water pressure has been ignored, i.e. that the water level for maximum interlock tension is at the wall base.)

312. Stability Calculations - Overturning. - The circular tension per foot of pile developed in the pile interlocks of the outside cell wall is directly proportional to the radius of the wall and to the maximum pressure of the cell fill; that is $t = pr$, where p is the pressure of the fill. The total tension developed in one interlock is $T = t \times h/2 = Pr$, where P is the total thrust due to the cell fill. The tension in the cross wall, if two cells are joined by a standard Y-pile with 120° legs is the same as the tensions in the individual wall arcs, that is $T = Pr$ again.

313. For the cell to overturn, the cross wall piles must slip along the interlocks. Therefore, if f is the coefficient of lock friction, and L is the length of one cell section, the resistance to overturning developed by the cross walls per unit length of structure is

$$S_p = \frac{Prf}{L} \tag{81}$$

314. In addition to this resistance, there is the shear resistance developed by the cell fill itself which is given by $S_f = P \tan \phi$, that is, the lateral fill thrust times the coefficient of internal friction. Therefore the total resistance to overturning per unit length of wall developed by the crosswall and cell fill is

$$S = S_p + S_f \tag{82}$$

$$= P \left(\frac{rf}{L} + \tan \phi\right)$$

315. The exterior overturning moment (M) per unit length of wall about the center of the cell may be replaced by a couple about the neutral axis (also the center of the cell). If it is assumed that pressures along the wall bottom which may cause this couple are linearly distributed along the wall width, the "couple diagram" may be drawn as shown in Figure 103.

316. The couple (F) due to the two pressure diagrams is applied at the centroids of the two triangles, and the moment due to this couple must equal the overturning moment about the neutral axis. Therefore the overturning forces may be represented by

$$F = \frac{3M}{2b} \tag{83}$$

For stability these forces should at least equal the resisting forces due to the crosswalls and fill, so

$$F = \frac{3M}{2b} = S = P\left(\frac{rf}{L} + \tan\phi\right) \tag{84}$$

The minimum width of the equivalent rectangular cell section for stability is

$$b = \frac{3M}{2P\left(\frac{rf}{L} + \tan\phi\right)} \tag{85}$$

where

M = the total overturning moment about the center line

P = the total force due to the cell fill

r = the radius of the outside cell wall

f = the coefficient of interlock friction

L = cell section length

$\tan\phi$ = the coefficient of internal friction of the cell fill.

317. Overturning Seaward. - The maximum overturning moment due to the backfill will be caused by a saturated backfill with no water in front of the wall. From the section on earth forces, this backfill force would be 5,340 pounds per lineal foot of wall applied at a distance 4.48 feet above the base, giving a moment of 23,900 foot-pounds per lineal foot of wall. Assuming an interlock friction coefficient of 0.3, an unsaturated cell fill, and a cell length of 40 feet, the equation for minimum width would become

$$b = \frac{3 \times 23{,}900}{2 \times 1730\left(\frac{50 \times 0.3}{40} + 1\right)} = 15.1 \text{ feet, say 16 feet}$$

the maximum width between outside walls would be $b_c = b/0.9 = 17.8$ feet, say 18 feet.

318. Overturning Landward. - Similarly, the maximum overturning moment due to wave forces will be that due to the maximum wave impinging

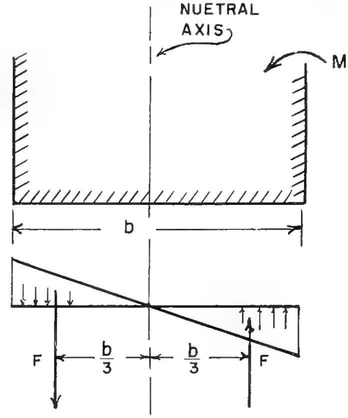


FIGURE 103 – COUPLE REPLACING OVERTURNING MOMENT

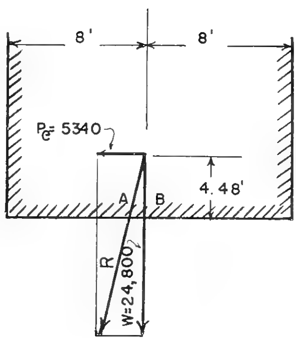


FIGURE 104 – RESULTANT FORCE

on the structure, with no active backfill pressure opposing it. From the section on wave forces, this moment would be 17,400 foot pounds per lineal foot of wall. Since this moment is smaller than that due to earth pressures alone, the wall would be stable against wave attack.

319. Stability: Sliding Seaward. - The weight of the filling material per foot of seawall length is

$$W = Whb \tag{86}$$

where W = the unsaturated cell fill unit weight

h = the fill height

b = the average cell width

so $W = 120 \times 13 \times 16 = 24{,}800$ pounds per foot

The known forces for seaward sliding are as shown in Figure 104.

320. For stability against sliding, the horizontal force divided by the total weight must be less than the coefficient of friction multiplied by the factor of safety. Because of the penetration of the piling, a factor of safety of 1 is satisfactory, or

$$[illegible] < [illegible]$$

or $\frac{5340}{24{,}800} = 0.215$ which is less than the allowable and the wall is stable against sliding seaward.

[illegible]. Stability: Sliding [illegible]ward. - The numerator of the above [illegible] of 4,150 foot pounds per foot is substituted for 5,340. Since the denominator remains the same, the wall [illegible] more stable against wave forces than it is against earth forces.

322. Design Section. - A cell section designed for stability is shown in Figure 105.

[illegible]. [illegible] Sheet pile bulkheads may be con[illegible] where [illegible] is obtainable. For design computa[illegible] may be made.

(a) A surcharge load on a fill may be considered to be re[illegible]ed by an equivalent height of fill which will produce the same pressure as the surcharge at its point of application;

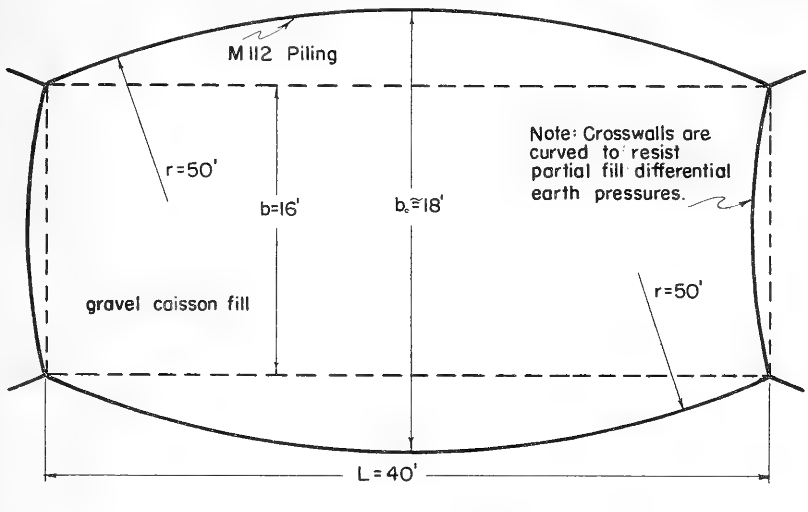


FIGURE 105 - cell section

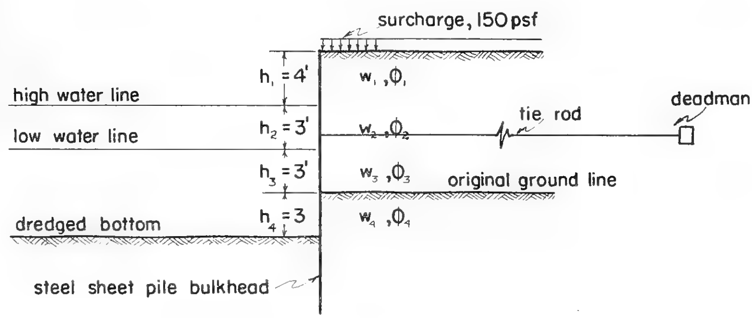


FIGURE 106 - Bulkhead placement

(b) Active pressures may be computed by use of relationship:

$$P_a = w\ h\ \tan^2\left(\frac{90 - \phi}{2}\right) \tag{87}$$

where w = the unit weight of the material

h = the height (including the equivalent height due to surcharge if any) of the fill

ϕ = the internal friction angle of the fill material.

The point of application of the resultant thrust will be through the centroid of the pressure diagram drawn from this relationship;

(c) Maximum passive pressure may be computed by use of the relationship:

$$P_p = w\ h\ \tan^2\left(\frac{90 + \phi}{2}\right) \tag{88}$$

The point of application of the resultant thrust will be through the centroid of the pressure diagram which may be drawn from this relationship. The application of these relationships to the design of sheet pile bulkheads is illustrated in the following example.

324. Design problem. - It is desired to construct a bulkhead in a protected anchorage where the maximum tidal variation is 3 feet. The anchorage is to be dredged to a depth of 6 feet below extreme low water. The original ground line is 3 feet below extreme low water, but the area is to be backfilled to 10 feet above the original ground line or 4 feet above maximum high water. A surcharge load of 150 pounds per square foot is to be designed for.

325. The fill material has a drained unit weight of 120 pounds per cubic foot with an internal friction angle (ϕ) of 30°. Its saturated unit weight is 140 pounds per cubic foot, and its submerged unit weight is 75 pounds per cubic foot, both with an internal friction angle of 25°. The undisturbed ground has a submerged weight of 80 pounds per cubic foot with an internal friction angle of 30°.

326. Figure 106 is a diagram of the desired placement for the proposed sheet pile bulkhead. For design purposes, it will be assumed that the backfill is saturated to the high water line. The heights h_1, h_2, and h_3, refer respectively to the backfill height above the high water line and the original ground line. The height h_4 refers to the height of the original ground above the dredged bottom. The corresponding unit weights and internal friction angles are w_1, and ϕ_1, w_2 and ϕ_2, w_3 and ϕ_3 and w_4 and ϕ_4.

327. Loading Diagram. - A loading diagram may be constructed by drawing horizontal lines at various points of lengths proportional to the pressure intensities (as given by equations 87 and 88) at the points.

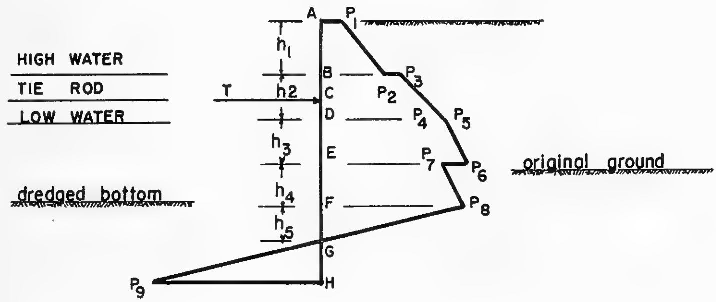


FIGURE 107 - LOADING DIAGRAM FOR MINIMUM PILE LENGTH

Thus in Figure 107

$$p_1 = w_1 \times \frac{W_1}{w_1} \tan^2 \frac{90^\circ - \phi_1}{2} \tag{89}$$

where $\frac{W_1}{w_1}$ = the equivalent surcharge height

$\therefore \quad p_1 = 150 \tan^2 30^\circ$

$= 50$ pounds per square foot

= pressure due to surcharge load alone exerted by the drained fill

$$p_2 = w_1 \left(h_1 + \frac{W_1}{w_1}\right) \tan^2 \left(\frac{90^\circ - \phi_1}{2}\right) \qquad (90)$$

$= (120 \times 4 + 150) \tan^2 30^\circ$

$= 210$ pounds per square foot

= pressure due to surcharge W_1 and drained backfill.

Considering the surcharge plus drained backfill to be a new surcharge applied to the saturated fill with a pressure of $W_2 = w_1 \left(h_1 + \frac{W_1}{w_1}\right) = 630$ pounds per square foot, then

$$p_3 = w_2 \left(\frac{W_2}{w_2}\right) \tan^2 \left(\frac{90^\circ - \phi_2}{2}\right) \qquad (91)$$

$= 630 \tan^2 (32.5^\circ)$

$= 256$ pounds per square foot

= pressure due to surcharge W_2 alone exerted by the saturated fill

$$p_4 = w_2 \left(h_2 + \frac{W_2}{w_2}\right) \tan^2 \left(\frac{90^\circ - \phi_2}{2}\right) \qquad (92)$$

$= (140 \times 3 + 630) \tan^2 (32.5^\circ)$

$= 426$ pounds per square foot

= pressure due to surcharge W_2 and saturated backfill.

328. Below the low water line, hydrostatic pressure on the land side of the wall would be balanced by the water in front of the wall. Accordingly, these pressures may be ignored. Again, considering the surcharge plus the drained backfill plus the saturated fill to be a new surcharge,

$$W_3 = w_2 \left(h_2 + \frac{W_2}{w_2}\right) = 1050 \text{ pounds per square foot.}$$

then $$p_5 = w_3 \times \frac{W_3}{w_3} \tan^2 \left(\frac{90^\circ - \phi_3}{2}\right) \qquad (93)$$

$= 1050 \tan^2 (32.5^\circ)$

$= 426$ pounds per square foot

$=$ pressure due to surcharge W_3 exerted by submerged fill

and $$p_6 = w_3 \left(h_3 + \frac{W_3}{w_3}\right) \tan^2 \left(\frac{90^\circ - \phi_3}{2}\right) \qquad (94)$$

$= (75 \times 3 + 1050) \tan^2 (32.5^\circ)$

$= 518$ pounds per square foot

$=$ pressure due to surcharge W_3 and submerged fill

Using $W_4 = w_3 \left(h_3 + \frac{W_3}{w_3}\right) = 1275$ pounds per square foot

$$p_7 = w_4 \times \frac{W_4}{w_4} \tan^2 \left(\frac{90^\circ - \phi_4}{2}\right) \qquad (95)$$

$= 1275 \tan^2 (30^\circ)$

$= 425$ pounds per square foot

$=$ pressure due to surcharge W_4 exerted by original ground,

and $$p_8 = w_4 \left(h_4 + \frac{W_4}{w_4}\right) \tan^2 \left(\frac{90 - \phi_4}{2}\right) \qquad (96)$$

$= (80 \times 3 + 1275) \tan^2 (30^\circ)$

$= 505$ pounds per square foot

$=$ pressure due to surcharge W_4 and original ground to the dredged bottom at 6 feet below the low water line.

329. At the point of application of p_8, both active pressure on the pile back and passive pressure on the pile face would be applied. The active pressure on the pile back would increase in the same manner as it increased from p_7 to p_8, because there would be no change of material at the level of p_8. From the expressions for p_7 and p_8, this rate of increase of active pressure with increase in depth would be $R_a = w_4 \tan^2 \left(\frac{90 - \phi_4}{2}\right)$ per unit increase in h_4. Similarly, the maximum passive pressure on the pile face would increase with depth at the rate $R_p = w_4 \tan^2 \left(\frac{90 + \phi_4}{2}\right)$ per unit increase in h_4, since at any point h below the level of the dredged bottom, this passive pressure $p_8 = w_4 h \tan^2 \frac{90 + \phi_4}{2}$. The rate of increase

of passive pressure applied on the wall face would be greater than the increase of active pressure applied at the wall back, and the net rate of change or pressure is given by

$$R = R_p - R_a$$

$$= w_4 \left[\tan^2 \left(\frac{90 + \varphi}{2}\right) - \tan^2 \left(\frac{90 - \varphi}{2}\right) \right] \tag{97}$$

$$= 80 \left[3.00 - 0.33 \right]$$

= 214 pounds per square foot per foot of depth

= rate of <u>decrease</u> of outward pressures

330. The point on the loading diagram at which the earth pressures would be zero is labeled G and its distance h_5 below the dredged bottom may be found by dividing p_8 by R.

$$h_5 = p_8/R = \frac{505}{214} \tag{98}$$

= 2.36 feet

= depth of zero loading point below the dredged bottom.

331. The magnitude of the concentrated force (T) on the wall due to the deadman is dependent on the depth of penetration of the pile.

332. <u>Pile Length.</u> - A pile which is just long enough to support the backfill will have a larger cross section than one, somewhat longer, which is designed for minimum bending moment. The choice between the two is a matter of economics.

333. <u>Pile Design -- Minimum Pile Length.</u> - For optimum pile design, the point of application of the deadman thrust should be such that the bending moment at this point equals the maximum bending moment in the pile below it. To attain this optimum design, the maximum moment below an assumed point of application must be found and compared with that at the assumed point. If these moments are not equal, the process must be repeated. For the purpose of this illustrative problem, only the methods of calculating the bending moments will be carried out.

334. Assuming the point of application (C) of the deadman tension (T) to be 1 foot above the low water line, the moment M_c of all the forces about this point between the surface and point C is given by

$$M_c = \left[p_8h_5\ (1 + h_3+h_4+\frac{h_5}{3})\right] + \left[p_7h_4\ (1+h_3+\frac{h_4}{2}) + \frac{p_8-p_7}{2}\ h_4\ (1+h_3+\frac{2}{3}\ h_4)\right]$$

$$+ \left[p_5h_3\ (1+\frac{h_3}{2}) + (\frac{p_6-p_5}{2})h_3\ (1+\frac{2h_3}{3})\right] + \left[p_c x 1 x \frac{1}{2} + \frac{p_4-p_c}{2}\ x\ 1\ x\frac{2}{3}\right] \qquad (99)$$

$$- \left[p_3 x 2 x \frac{1}{2} + \frac{p_c-p_3}{2}\ x\ 2\ x\frac{2}{3}\right] - \left[(p_1h_1)\ (2+\frac{h_1}{2}) + (\frac{p_2-p_1}{2})\ h_1\ (2+\frac{h_1}{3})\right]$$

where $p_c = p_4 - (\frac{p_4-p_3}{h_2}) = 369$ pounds per square foot

$$M_C = \left[4620 + 7720 + 3600 + 209\right] - \left[331 + 1865\right]$$

$= 13,953$ foot pounds per foot of wall

335. For stability, the sum of all moments about C must be zero. Then if H is the point of deepest pile penetration,

$$M_C = (\overline{CG} + \frac{2h_6}{3})\ (h_6\ x\ \frac{h_6\ x\ R}{2}) \qquad (100)$$

where h_6 x R = p_9

Therefore $(h_6)^2\ (\overline{CG} + \frac{2}{3}\ h_6) = 2\frac{M_C}{R}$ (101)

or in this case $(h_6)^2\ (9.36 + \frac{2}{3}h_6) = \frac{13,953\ x\ 2}{214} \cong 131$

from which $h_6 \cong 3.4$ feet.

336. Since point G is itself 2.36 feet below the dredged bottom, the pile must penetrate a distance 3.4 + 2.4 = 5.8 feet below the dredged bottom, at which point the pressure p_9 would be

p_9 = R x h_6 = 214 x 3.4 = 728 pounds per square foot. (102)

The minimum length of pile for stability with the deadman tie-rod one foot above the low water line would be 18.8 feet.

337. Maximum Bending Moment for Minimum Length Pile. - The maximum bending moment in the wall would occur at that point above the dredged bottom where the shear passes through zero. The shear S_F at the dredged bottom F is given by the sum of the forces below that point, which sum is the algebraic sum of the

areas of the pressure diagrams below that point. Thus this shear

$$S_F = \frac{p_9 h_6}{2} - \frac{p_8 h_5}{2} \tag{103}$$

$$= \frac{728 \times 3.4}{2} - \frac{505 \times 2.36}{2}$$

$$= 1235 - 595 = 640 \text{ pounds per foot of wall.}$$

338. Above this point, the shear would decrease as pressure diagram area is added. The quantity $\frac{p_8 - p_7}{h_4} = 26.7$ pounds per square foot per foot of wall would be rate of change of pressure over h_4, and calling this rate R_4 the equation for finding the point of zero shear when this point is within h_4 is

$$S_F = (p_8 - R_4 Z_1) Z_1 + \frac{R_4 Z_1^2}{2} \tag{104}$$

or $R_4 Z_1^2 - 2p_8 Z_1 + 2S_F = 0$

where Z_1, is the distance of this zero shear point above the dredged bottom F. Solving for Z_1

$$Z_1 = \frac{p_8 - \sqrt{p_8^2 - 2 R_4 S_F}}{R4} \tag{105}$$

$$= 1.31 \text{ feet}$$

The pressure at Z would be $p_Z = p_8 - R_4 Z = 470$ pounds per square foot.

339. The bending moment M_F at the dredged bottom is the sum of the moments of the individual areas p_9 HG and p_8 FG

$$M_F = \frac{Rh_6^2}{2} (h_5 + \frac{2}{3} h_6) - \frac{h_5^2 p_8}{6} = \frac{Rh_6^2}{6} (2h_6 + 3h_5) - \frac{h_5^2 p_8}{6} \tag{106}$$

(Note: R is the rate of change of pressure between p_8 and p_9)

$$M_F = \frac{214 \times (3.4)^2}{2} (2.36 + 2.26) - \frac{(236)^2 (505)}{6}$$

M_F = 5230 foot pounds per foot of wall

340. Figure 108 shows the loading, shear, and moment diagrams between F and Z_1. At any point x above F, the shear S is given by S_F less the area of the loading diagrams between F and x or

$$S = S_F - (p_8 X - \frac{R_4 X^2}{2}) \tag{107}$$

Integrating the area under this curve between the limits of 0 and Z_1, the shear diagram area A_s is found to be

$$A_s = \frac{Z_1}{2}\left(S_F - \frac{p_8 Z_1^2}{2} + \frac{R_4 Z_1^2}{6}\right)$$

or, since $S_F = p_8 Z_1 - \frac{R_4 Z_1^2}{2}$, and $p_{Z_1} = p_8 - R_4 Z_1$

$$A_s = \frac{Z_1^2}{6}\left(p_8 + 2p_{Z_1}\right) \tag{108}$$

Since the area of the shear diagram between points F and Z_1 is the total increase of moment between these points, the bending moment at Z is

$$M_Z = M_F + \frac{Z_1^2}{6}(p_8 + 2p_{Z_1}) \tag{109}$$

$$= 5230 + (\frac{1.31^2}{6}) \quad (505 + 940)$$

$$= 5230 + 415 = 5645 \text{ foot-pounds per foot of wall}$$

= the maximum bending moment.

341. <u>Bending Moment At the Deadman Tie for Minimum Length Pile.</u> - Above the point of application of the deadman tension (T) the shear and moment diagrams are similarly analyzed. Thus in Figure 109 the shear at any point X_1 below A down to B is given by

$$S = p_1 X_1 + \left(\frac{p_2 - p_1}{h_1}\right)\frac{X_1^2}{2} \tag{110}$$

from which the shear at B = $S_B = \frac{h_1}{2} p_1 + p_2$ = 520 pounds per foot of wall.

The area of the shear diagram between A and B is

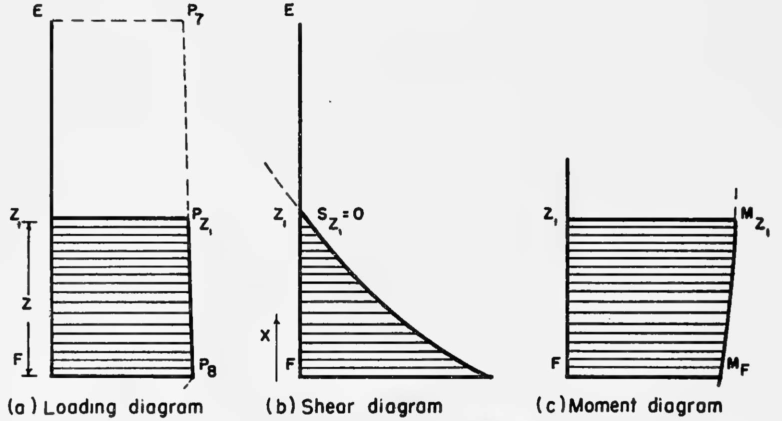


FIGURE 108 LOADING, SHEAR, AND BENDING MOMENT DIAGRAMS BETWEEN POINTS F AND Z_1

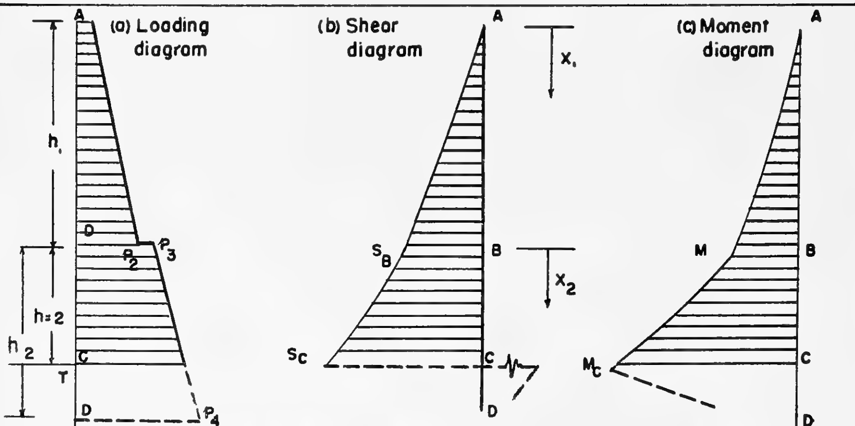


FIG. 109 LOADING, SHEAR, & BENDING MOMENT DIAGRAMS ABOVE DEADMAN TIE

$$A_{S_1} = \frac{h_1^{\,2}}{6} \left(2p_1 + p_2 \right) = 825 \text{ foot-pounds per foot of wall} \tag{111}$$

which is also the moment M_B, at B_7 since the bending moment at A is zero.

342. Similarly the shear at any point X_2 below B down to C is given by

$$S = S_B + p_3 X_2 + \left(\frac{p_c - p_3}{h_2}\right) \frac{X_2^{\,2}}{2} \tag{112}$$

The area of the diagram between B and C is

$$A_{S_2} = S_B \times h + \frac{h^2}{6} \left(2p_3 + p_c\right) = 1627 \text{ foot-pounds per foot of wall} \tag{113}$$

(Note: $p_c = \dfrac{p_4 - p_3}{h_2} = 369$ pounds per square foot)

The bending moment at C is

$$M_C = M_B + A_{S_2} \tag{114}$$

$$= 825 + 1627 = 2452 \text{ foot pounds foot of wall.}$$

343. For this trial, it can be seen that the maximum positive bending moment in the pile is greater than that at the deadman tie. Lowering the point of application of the tie tension would bring these two values closer, but would put the tie under water where increase in construction costs would more than offset the saving in material costs.

344. The tension T in the tie rod is found by summing algebraically the forces due to the various earth pressure polygons.

$$T = \left[\left(\frac{p_1 + p_2}{2}\right) h_1 + \left(\frac{p_3 + p_4}{2}\right) h_2 + \left(\frac{p_5 + p_6}{2}\right) h_3 + \left(\frac{p_7 + p_8}{2}\right) h_4 + \frac{p_8}{2} \times h_5 \right] - \left[\frac{p_9}{2} \times h_6 \right] \tag{115}$$

$$= (520 + 1022 + 1418 + 390 + 595) = 1240$$

$$= 4945 - 1240 = 3705 \text{ pounds per foot of wall}$$

$$= F_a - F_p$$

F_a is the seaward (active) earth force, and F_p is the landward (passive) force. Note that the point of application of the force F_a is a distance ℓ below the tie rod given by

$$\ell_3' = \frac{M_c}{F_a} = \frac{13,953}{4,945} = 2.82 \text{ feet} \tag{116}$$

where M_c is given by equation (157).

345. Pile Design - Minimum Bending Moment. - By driving the pile somewhat deeper, a cantilever would be formed near the pile bottom, which would develop passive pressures in the backfill. The loading diagram would be changed as shown in Figure 110.

346. For optimum pile design, that is for the least length of pile for which the maximum bending moment would be smallest, pressures p_9 and p_{10} must be as large as can be developed by the ground at these points. This may be proven for p_9 by finding the expression for the maximum negative bending moment. This maximum moment would occur below G, the point of zero pile load. Calling the moment at G, M_G, and the shear at the same point, S_G, the maximum moment would occur at the point of zero shear (say Z_2), and would equal M_G plus the area of the shear diagram between G and Z_2. Referring to Figure 111, the loading, shear, and bending moment diagrams for this section of pile, the shear at any point Z below G would be $S_z = S_G - RZ^2/2$ where R is the rate of change of pressure between p_8 and p_9. At $Z = Z_2$, this shear would be zero, therefore

$$Z_2 = \frac{2S_G}{R} \quad . \quad \text{The moment at } Z_2 \text{ is}$$

$$M_{Z_2} = M_G + A_s \tag{117}$$

$$= M_C + \frac{2}{3} S_c \frac{2S_G}{R}$$

$$= \text{the maximum negative moment.}$$

347. Now M_G and S_G are functions of the pressure area p_8 G F; as this area increases, both M_G and S_G will increase. These would be smallest when the rate of pressure change R is greatest (some lesser rate would appear as the dashed line in Figure 111 and would cause point G to be lowered). But the greatest possible rate of pressure change between p_8 and p_9 would make p_9 as large as the ground would permit as that point, which was to be shown.

348. Referring again to Figure 110, one of the earth forces which cause the pressure p_9 to be developed is that due to the pressure triangle p_{10} I J.

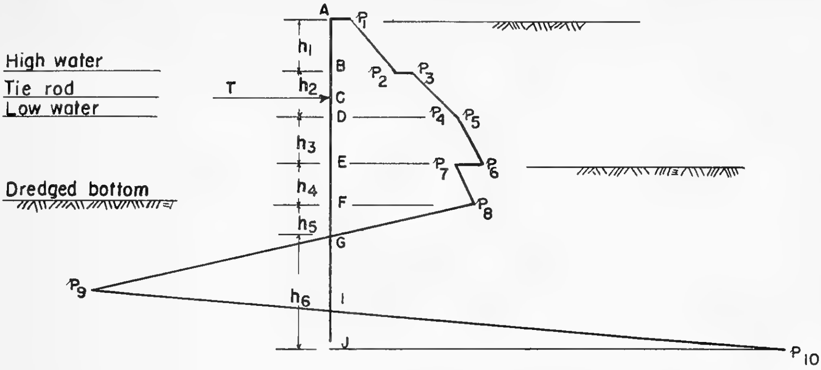


FIGURE 110 LOADING DIAGRAM FOR MINIMUM BENDING MOMENT

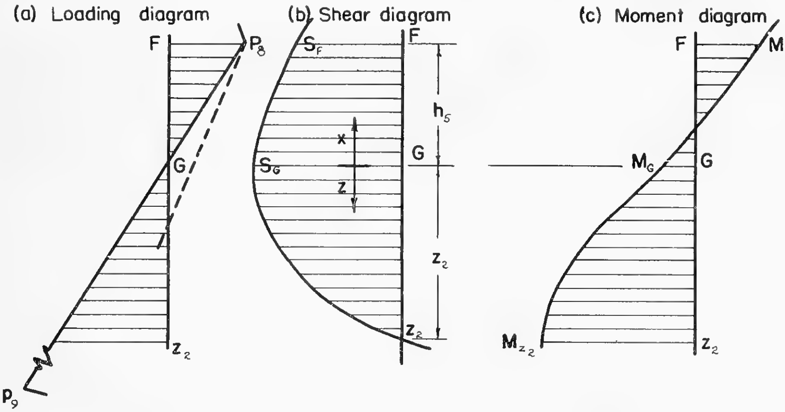


FIGURE 111 - Loading, shear, and moment diagrams below point F

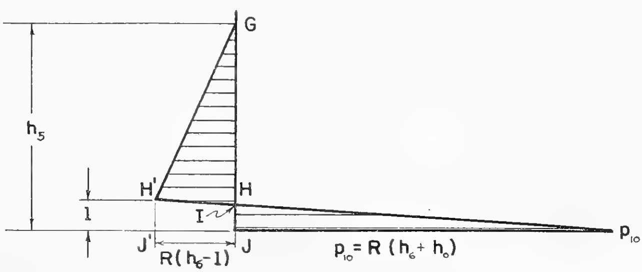


FIGURE 112 - Loading diagram below point G

Whatever this force is, the least length of pile I J over which it is applied would be that length corresponding to the highest possible point of application of the pressure p_{10}. But the magnitude of p_{10} at this highest point can be no greater than the pressure which may be generated by the earth. Therefore for minimum pile length, p_{10} must be as large as this maximum permissible earth pressure.

349. The maximum rate of pressure change between p_8 and p_9 has already been determined as

$$R = w_4\left[\tan^2\left(\frac{90 + \phi_4}{2}\right) - \tan^2\left(\frac{90 - \phi_4}{2}\right)\right] \tag{118}$$

= 214 pounds per square foot of depth (equation 97)

and the distance between F and G with this rate was found to be $h_5 = 2.36$. (Equation 98) The maximum positive moment in terms of the moment at F, the dredged bottom, has been shown to be

$$M_{Z_1} = M_F + \left[\frac{Z_1^2}{2} x (p_R + 2p_{Z_1})\right]$$

(equation 109). It may be seen from Figure 111 that in going from F to G, the moments increase negatively. M_G is related to M_F by $M_G = M_F - |A_s|$ where A_s is the area of the shear diagram between F and G.

Therefore $$M_{Z_1} = M_G + A_s + \frac{Z_1^2}{2}(p_8 + 2p_{Z_1}) \tag{119}$$

The magnitude of the shear at any point x above G up to F at which point $x = h_5$, is given by

$$S = S_G - \frac{R_x^2}{2} \tag{120}$$

therefore since $Rh_5 = p_8$ the area A_s is

$$A_s = S_G h_5 - \frac{p_8 h_5^2}{6} \tag{121}$$

The maximum positive moment in terms of M_G and S_G

$$M_{Z_1} = M_G + S_G h_5 - \frac{p_8 h_5^2}{6} + \frac{Z_1^2}{2}(p_8 + 2p_{Z_1}) \tag{122}$$

and the maximum negative moment (equation 117) is

$$M_{Z_2} = M_G + \frac{2}{3} S_G \sqrt{2S_G/R}$$

350. In the above equations, S_G is given by the sum of all forces above G or

$$S_G = F_a - T \tag{123}$$

$$= 4945 - T \text{ (see equation 115)}$$

and the moment M_G is

$$M_G = \frac{M_C}{I} (\overline{CG} - 1) - \overline{CG} \times T \tag{124}$$

$$= \frac{13,953}{2.82} (9.36 - 2.82) - 9.36\ T$$

$$= 32,300 - 9.36\ T \quad \text{(See equation 99 and 116)}$$

In addition, h_5 = 2.36 feet (equation 98)

p_8 = 505 pounds per square foot (equation 96)

R = 214 pounds per square foot per foot of depth (equation 97)

$p_{Z_1} = p_8\ R_4\ Z_1$; R_4 = 26.7 pounds per square foot per foot of depth (equation 104)

and $Z_1 = \dfrac{p_8 - \sqrt{p_8^2 - 2R_4S_F}}{R_4}$ (equation 105)

where $S_F = S_G - \dfrac{Rh_5^2}{2} = (4945 - T) - 595 = 4350 - T$ (equation 123)

therefore $Z_1 = \dfrac{505 - \sqrt{(505)^2 - 53.4\ (4350 - T)}}{26.7}$

$$= 18.9 - \sqrt{32.3 + 0.075\ T}$$

351. For optimum design the maximum bending moments should be equal. The exact solution of the equation $M_{Z_1} = M_{Z_2}$ is difficult, and it is best solved by trial and error, assuming for the first trial $M_G = 0$. Then

$$T = \frac{32,300}{9.36} = 3450 \text{ pounds per foot of wall}$$

$$S_G = 4945 - T = 1495 \text{ pounds per foot of wall}$$

$$Z_1 = 1.8 \text{ feet}$$

352. Substituting these values in the expression for the maximum moments, the maximum positive moment is found to be

$$M_{Z_1} = 5355 \text{ foot-pounds per foot of wall}$$

and the maximum negative moment is found to be

$$M_{Z_2} = 3680 \text{ foot-pounds per foot of wall}$$

353. By decreasing the value of T (that is, making M_G negative), these moments may be equated. Through trial and error the correct values of T, M_G, S_G, and Z_1 are found to be approximately

$$T = 3350 \text{ pounds per foot of wall}$$
$$M_G \cong 1000 \text{ foot-pounds per foot of wall}$$

$$S_G = 1595 \text{ pounds per foot of wall}$$

$$Z_1 \cong 2.05 \text{ feet.}$$

The bending moments $M_{Z_1} = M_{Z_2} \cong 5100$ foot pounds per foot run of wall.

354. For stability, the forces below G must equal S_G, and the moments below G must equal M_G. It has been shown that the pressure p_{10} for minimum pile length must be as great as can be generated by the earth at that point. Now the maximum passive pressure at the depth of p_{10} is that due to a "surcharge" consisting of all the earth above point G, applied to an undisturbed ground mass of height h_6. This surcharge load is given by

$$W_p = w_4 \; (h_4 + h_5) + \frac{W_4}{w_4} \tag{125}$$

where W_4 = 1275 pounds per square foot = the surcharge load of the earth above h_4

$w_4 = w_5 = w_6 = 80$ pounds per cubic foot = the unit weight of the submerged ground

$h_4 = 3$ feet

$h_5 = 2.36$ feet

therefore W_p = 1705 pounds per square foot.

The maximum passive pressure p_p at the depth of p_{10} is applied at the wall back and is given by

$$p_p = w_6 \left(h_6 + \frac{W_p}{w_6}\right) \tan^2 \left(\frac{90 + \phi_6}{2}\right) \tag{126}$$

where $\phi_6 = 30^o$

355. This active pressure at the depth of p_{10} at the wall face is due to a surcharge composed of the ground above G applied to the earth mass of height h_6. The surcharge in this case is

$$W_a = w_5 h_5 \tag{127}$$

$$= 80 \times 2.36 = 186.4 \text{ pounds per square foot}$$

The maximum active pressure p_a at the depth of p_{10} is applied at the wall face and is given by

$$P_a = w_6 \left(h_6 + \frac{W_a}{w_6}\right) \tan^2 \left(\frac{90 - \phi}{2}\right) \tag{128}$$

356. The actual pressure p_{10} is given by

$$p_{10} = p_p - p_a \tag{129}$$

$$= w_6 h_6 \tan^2 \left(\frac{90 + \phi_6}{2}\right) - \tan^2 \left(\frac{90 - \phi_6}{2}\right) + w_p \tan^2 \left(\frac{90 + \phi_6}{2}\right) - W_a \tan^2 \left(\frac{90 - \phi_6}{2}\right)$$

$$= Rh_6 + 1705\ (3) - 186.4\ (0.33)$$

$$= 214\ h_6 + 5053 = 214\ (h_6 + 23.6)$$

which may be written symbolically

$$p_{10} = R\ (h_6 + h_o)$$

The loading diagram for this section of piling is shown in Figure 112

357. The sum of all forces below G must equal the shear S_G at G. Summing the forces, represented by the areas GH' H, H'H JJ', and H'J' p_{10}

and equating them to S_G we have

$$R\left(\frac{h_6 - \ell}{2}\right)^2 + R\ (h_6 - \ell)\ x\ \ell - R(h_6 - \ell) + R\ (h_6 + h_o)\ \frac{\ell}{2} = S_G$$

which, when solved for ℓ gives

$$\ell = \left(\frac{Rh_6^{\ 2} - 2S_G}{R\ (h_o + 2h_6)}\right) \tag{130}$$

358. Similarly the sum of all moments below G must equal the moment M_G at G. Using the same areas

$$R\frac{(h_6 - \ell)^2}{2} \times \frac{2}{3}\ (h_6 - \ell) + R(h_6\ell - \ell^2)\ (h_6 - \frac{\ell}{2}) - R(2h_6 + h_o - \ell)$$

$$\times\ \frac{\ell}{2} \times (h_6 - \frac{1}{3}) = M_G$$

which may be reduced to

$$2h_6^{\ 3} - \ell\ (6h_6^{\ 2} + 3h_oh_6) + \ell^2\ (h_o + 2h_6) = \frac{6M_G}{R}$$

Substituting the solution for ℓ from equation (188) the final expression for h_6 is

$$R^2\ h_6^{\ 3}\ (h_6 + h_o) - 2R\ 3M_G(2h_6 + h_o) + S_Gh_6(4h_6 + 3h_o) - 4S_G^{\ 2} = 0 \tag{131}$$

Substituting the previously determined values for R, h_o, M_G, and S_G in this equation we have approximately

$$45.8\ h_6^{\ 4} + 1080h_6^{\ 3} - 2{,}730h_6^{\ 2} - 50{,}970h_6 = 40{,}480$$

which when solved for h_6 gives

$$h_6 \cong 7.4 \text{ feet}$$

The length of piling to G is 15.36 feet, and the total length of pile of minimum moment is about 22.8 feet.

359. Final Pile Design. - When designed for minimum length, the maximum bending moment in the pile was M_z = 5645 foot pounds per foot of wall, and the length needed was 18.8 feet. When designed for minimum bending moment, M_z was reduced to 5,100 foot-pounds per foot of wall, but the length needed for this reduced moment was 22.8 feet, an increase of 4 feet. The lightest pile section which can withstand a moment of 5,100 foot-pounds per foot of wall is an MP 115 which with a section modulus of 5.4 is good for a bending moment of 6,300 foot-pounds per foot. Since this is larger than the maximum moment which would occur in the pile when designed for minimum length, no saving is introduced by designing for minimum moment.

360. Accordingly, the design pile would be an MP 115 (or an MZ 22 which has the same weight per square foot of wall but greatly increased strength) and the total pile length would be 18.8 (19) feet.

361. Deadman Design. - If the backfill is such that dependable passive resistance would be developed, either an intermittent or a continuous deadman could be used instead of one (say) of sheet pile which would probably be more expensive. The deadman may be of reinforced concrete, steel, timber, or any other material which would develop the required tie rod tensions over the life of the structure. The size of deadman required may be found by reference to Figure 113.

362. The deadman would experience an active pressure on its back (the side farthest from the bulkhead wall) which would increase at the rate

$$R_a = w_1 \tan^2 \left(\frac{90 - \phi_1}{2}\right)$$

and a passive pressure on its front which would increase at the rate of

$$R_p = w_1 \tan^2 \left(\frac{90 - \phi_1}{2}\right);$$

assuming the entire deadman to be above the line of permanent saturation. The total rate of pressure increase then is

$$R_d = R_p - R_a \tag{132}$$

$$= w_1 \left[\tan^2 \left(\frac{90 + \phi_1}{2}\right) - \tan^2 \left(\frac{90 - \phi_1}{2}\right)\right]$$

$$= 120 \left[3.00 - 0.33\right]$$

$$= 321 \text{ pounds per square foot per foot of depth.}$$

363. The total earth pressure on the deadman, given by the area of the trapezoid p_K K L p_L must equal the tie-rod tension per unit width of

wall (T) times the length of wall supported by each deadman, divided by the deadman width. In the case of a continuous deadman this earth force must equal T. Assuming a continuous deadman,

$$T = \left(\frac{p_K + p_L}{2}\right) \times d_d \tag{133}$$

where $p_K = R_d \times d_K$

and $p_L = p_K + R_d \times d_d = R_d \; (d_K + d_d)$

Therefore

$$T = \frac{R_d \; (2d_K + d_d)}{2} \times d_d$$

if we call $\frac{T}{R_d} = \frac{T}{321} = T'$ and solve for d_K

$$d_K = \frac{2T' - d_d^{\,2}}{2d_d} \tag{134}$$

364. In addition, the sum of the moments of these forces about (say) K must equal zero.

$$T \; (d_T - d_K) = \frac{p_K d_d^{\,2}}{2} + R_d d_d \times \frac{d_d}{2} \times \frac{2d_d}{3} \tag{135}$$

$$= \frac{R_d d_K d_d^{\,2}}{2} + \frac{R_d d_d^{\,3}}{3}$$

which, when the value of d_K and T' above are substituted, gives

$$d_d^{\,4} - 12 \; d_t \; T' \; d_d + 12 \; (T')^2 = 0 \tag{136}$$

an equation in d_d which must be solved by trial and error.

[illegible]. This quartic equation may have as many as two positive roots but no more. One of these roots will be slightly small than $d_d = \sqrt[3]{12 d_T T'}$

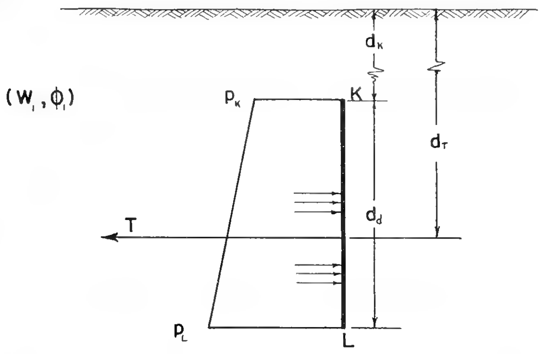


FIGURE 113.- Deadman loading diagram

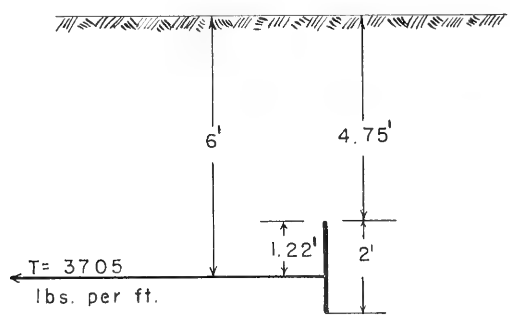


FIGURE 114 DEADMAN DIMENSIONS

but there may be another root smaller than this. In the case of the pile designed for minimum length where T = 3705 pounds per foot of wall, the quantity $\sqrt[3]{12 d_T T'} \cong 9.4$ feet and a positive root does exist between $d_d = 8$ and 9. If this value for d_d is substituted in equation 48, d_K is found to be negative, which is impossible.

366. However investigation shows that there is another root very close to $d_d = 2$. This root is the applicable solution.

Substituting in the equation for d_K

$$d_K = \frac{2T' - d_d^{\,2}}{2d_d} = \frac{23.05 - 4}{4}$$

$$= 4.76 \text{ feet.}$$

The deadman dimensions and placement are shown on Figure 114.

367. <u>Sheet Pile Deadman.</u> - If the backfill is not dependable for passive resistance, a sheet pile deadman, or one which depends on active as well as passive pressures must be used. The analysis for a deadman of this type is similar to the analysis of the lower part of the pile wall designed for minimum moment. (See Figure 115)

368. By summing forces and moments about K as done previously, d_d is given by

$$\left[d_d\ (p_L + p_K) - 2T'\right]^2 = 2p_L\left[d_d^{\,2}\ (p_L + 2p_K) - 6T'\ (d_d - d_T + d_K)\right] \quad (137)$$

d_m and d_o are given by

$$d_m = \frac{d_d\ (p_K + p_L) - 2T'}{2p_L} \quad (138)$$

$$d_o = \frac{d_m \times p_L}{2p_L - Rdd_m} \quad (139)$$

The maximum negative and maximum positive moments in the pile are respectively

$$M_n = \frac{\left(d_T - d_K\right)^2}{6} \times \left[3p_K + R_d\ (d_T - d_K)\right] \quad (140)$$

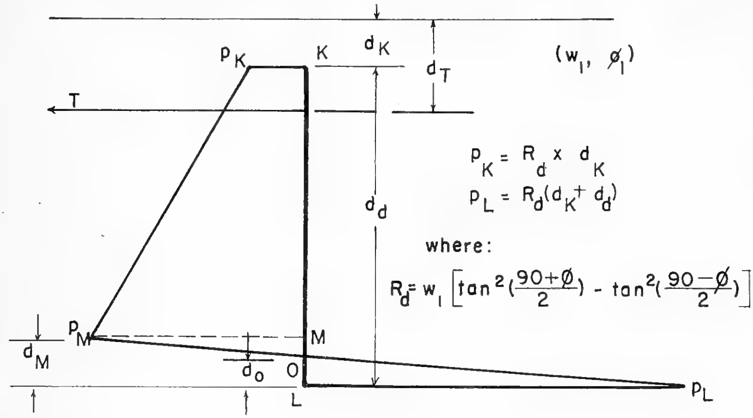


FIGURE 115 SHEET PILE DEADMAN

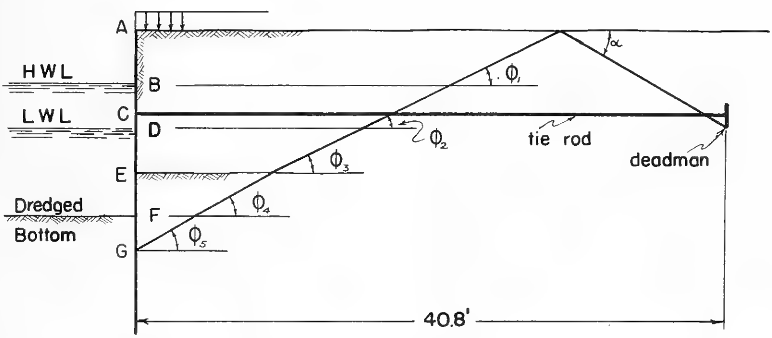


FIGURE 116 – Deadman distance from wall

$$M_p = \frac{4p_L\, d_m^{\,3}}{6(p_L - R_d d_m)} \qquad (141)$$

369. Distance of Deadman From Wall. - The deadman should be located outside the possible failure planes of the various layers of earth fill and also outside the intersection of its own failure plane to the surface with the line of possible failure of the earth fill. Since in failing, the wall would rotate seaward about the point of zero loading, the mimimum distance is determined - as in Figure 116 - by dividing the heights of the various fill components by the tangents of the various internal friction angles (failure angles) and summing the result; i.e. for the wall to fail the component fill failure planes must coincide.

370. For the case being considered, the minimum distance of the deadman from the wall is given by

$$L = \frac{h_5}{\tan \phi_5} + \frac{h_4}{\tan \phi_4} + \frac{h_3}{\tan \phi_3} + \frac{h_2}{\tan \phi_2} + \frac{h_1}{\tan \phi_1} + \frac{(d_K + d_d)}{\tan \alpha}$$

where $\tan \alpha = \sqrt{1 + \tan \phi_1^{\,2}} - \tan \phi_1 = 0.58$

$$\text{therefore } L = \frac{2.36}{\tan 30^\circ} + \frac{3}{\tan 30^\circ} + \frac{3}{\tan 25^\circ} + \frac{3}{\tan 25^\circ} + \frac{4}{\tan 30^\circ} + \frac{6.78}{0.58}$$

$$= 4.08 + 5.2 + 6.44 + 6.44 + 6.94 + 11.70$$

$= 40.8$ feet from the wall.

OFFSHORE BREAKWATERS

GENERAL CONSIDERATIONS

371. Because of their high cost, offshore breakwaters have been little used solely for shore protection. However, because they have use as a protective structure, because they may be required to provide a sand trap for by-passing material, and because they may be used to retain the toe of a beach fill, design data are included for typical examples of the most frequently constructed types. Other types may be designed by using the methods illustrated for the design of seawalls.

372. Types and Description. - Several conventional types of permanent offshore breakwaters are now in general use, varying as to material of which constructed and as to cross section. The principal construction materials are stone, concrete, steel and timber. In addition fascine mattresses, asphalt, and bitumen have occasionally been used (Kuiper 1951). The selection of a material or combination of materials for breakwater construction is based on cost and adaptability as pertains to conditions

imposed by a given site.

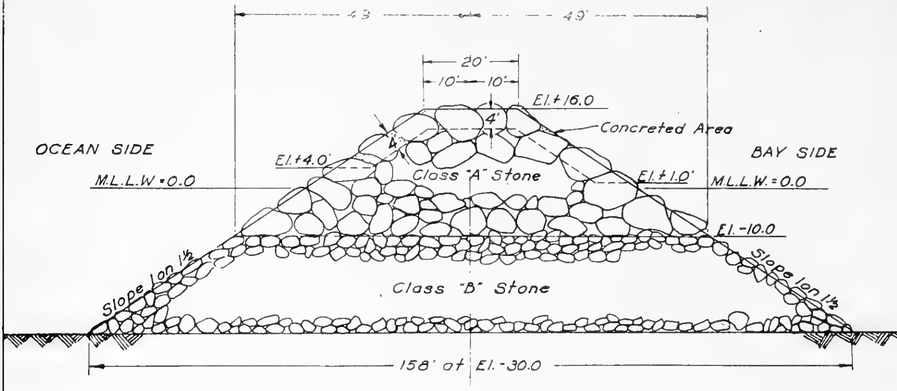
373. Rubble Mound. - The rubble-mound structure is adaptable to any depth of water and may be placed on practically any kind of foundation. This type of structure is used extensively throughout the United States and almost exclusively on the Pacific Coast (Kaplan and Pape, 1950). The chief advantages of the random stone structure are: damage is easily repaired, settlement of the structure results only in readjustment of component stones rather than the incipient failure of the entire structure, and the reflected wave action is less severe than that from a solid wall structure. The chief disadvantage of rubble-mound construction is the large quantity of material required, which results in a high first cost if satisfactory material is mot available within economic hauling distance.

374. The rubble-mound breakwater is a more or less heterogeneous assemblage of natural stones of different sizes and shapes either dumped at random or laid in courses. A typical cross section is shown in Figure 117. Side slopes and stone sizes are so designed that the structure will resist the expected wave action.

375. Composite Earth and Stone. - The composite type of random stone structure is designed for relative economy and rapidity of construction. The principal difference from the rubble-mound breakwater lies in the substitution of sand and clay or incipient shale for the base and core material. Care must be used in selecting the clay or earth material with which the sand sub-base is covered. This type of construction was used for the 32,000-foot Los Angeles-Long Beach detached breakwater. As shown in Figure 118 this structure consisted of a sand inner core covered by a hard clay mound, all being armored by a graded rubble-stone covering. If satisfactory material can be secured from an approved or useful dredging area, savings may be effected.

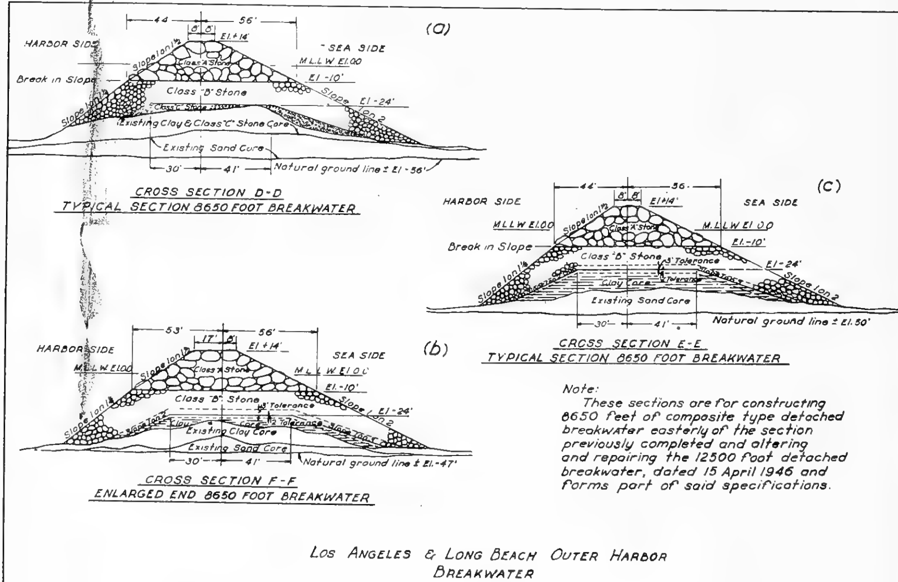
376. Stone and Concrete. - The stone and concrete type structure is a combination of rubble-mound and concrete-wall types, ranging from a rubble mound, in which the voids in the upper part of the structure are filled with concrete, to massive concrete superstructures on rubble-mound substructures. The rubble mound usually is used either as a foundation for a vertical or nearly vertical concrete wall or as the main structure surmounted by a low concrete superstructure with a vertical, curved, stepped, or inclined face. The use of a composite concrete and stone structure reduces the quantity of material required and may be economical in great depths.

377. Where the bottom is subject to scour, care must be used to prevent the superstructure from being undermined, as storm waves tend, in their recoil down the face, to displace materials of the mound at the toe of the superstructure. Rubble-mound foundations require some time to become stable, and should be placed one or more years before construction of the superstructure. Stone and concrete breakwaters, when properly designed and constructed, give satisfactory service in withstanding heavy



RUBBLE-MOUND BREAKWATER
TYPICAL SECTION
MORRO BAY HARBOR, CALIFORNIA

FIGURE 117



LOS ANGELES & LONG BEACH OUTER HARBOR
BREAKWATER
COMPOSITE EARTH & STONE

FIGURE 118

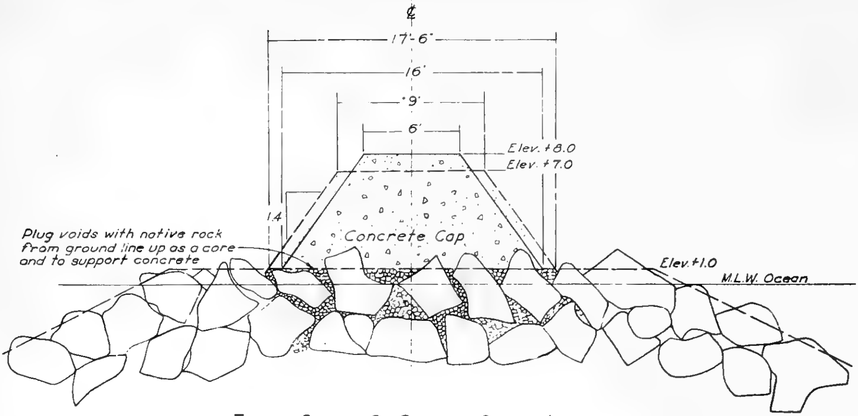
sea action. A cross section of a typical structure is shown in Figure 119.

378. Concrete Caissons. - Breakwaters of this type are built of reinforced concrete shells, which are floated into position, settled upon a prepared foundation, filled with stone or sand to give stability, and then capped with concrete slabs or cap stones. These breakwaters may be constructed with or without parapet walls. In general, concrete caissons are of two types; one type having a bottom of reinforced concrete which is an integral part of the caisson, the other type not having a permanent bottom. The bottom opening of the latter type is closed with a temporary wooden bottom which is removed after the caisson is placed on the foundation. Stone, which is used to fill the compartments, combine with the foundation material to provide additional resistance against horizontal movement. A typical section is shown in Figure 120 of concrete caisson breakwater.

379. Caissons are suitable for depths ranging from 10 to 35 feet. The foundations must be prepared to support the structure and to withstand scour at the base. This foundation usually consists of a mat or mound of rubble-stone. Piles may be used to support the structure. Heavy riprap is usually placed along the caissons to protect against scour, horizontal displacement, or weaving when the caisson is supported on piling. Considerable labor and adequate floating plant are required to prepare a rubble-mound foundation. The top of the mound, where the caisson is to be placed, must be leveled by diver. For bottomless caissons, strips of crushed stone are placed on the longitudinal footings of the caisson and only these strips need be leveled.

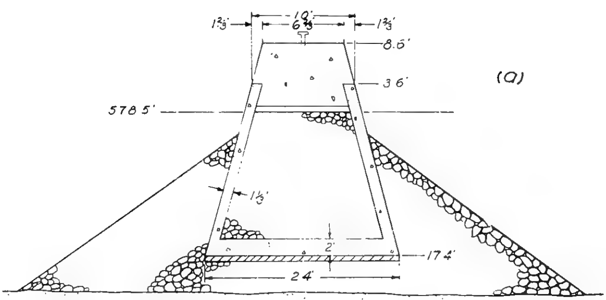
380. Sheet Piling. - Timber or concrete sheet piling has been used for breakwater construction at locations where storm waves are not severe. Steel sheet piling is used for breakwaters in several types of structures, which include a single row of piling with or without pile buttressess; a single row of sheet piling arranged so that the row of piling acts as a buttressed wall, but requires no more piling than a straight wall; double walls of sheet piling held together with tie rods with the space between the walls filled with stone or sand (usually separated into compartments by cross walls if sand is used); and cellular steel sheet pile structures which are modifications of the double-wall type. Examples of pile breakwaters are shown in Figures 121 to 124 inclusive.

381. Cellular steel sheet pile breakwaters require little maintenance and are suitable for construction in depths up to 40 feet and on all kinds of foundations. Steel sheet pile structures possess, under some conditions, advantages of economy and speed of construction, but during construction are subject to storm damage. Corrosive action is the principal disadvantage in sea water. If abrasive action of sand and water continually wears away the corroded metal, leaving fresh steel exposed, the life of the piling may not exceed 10 years. However, if the

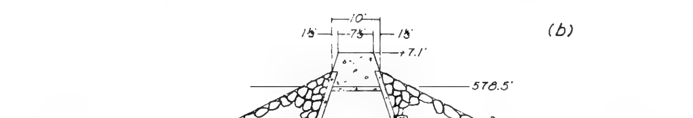


TYPICAL SECTION OF CONCRETE CAPPED JETTY
LAKE WORTH INLET, FLORIDA

FIGURE 119

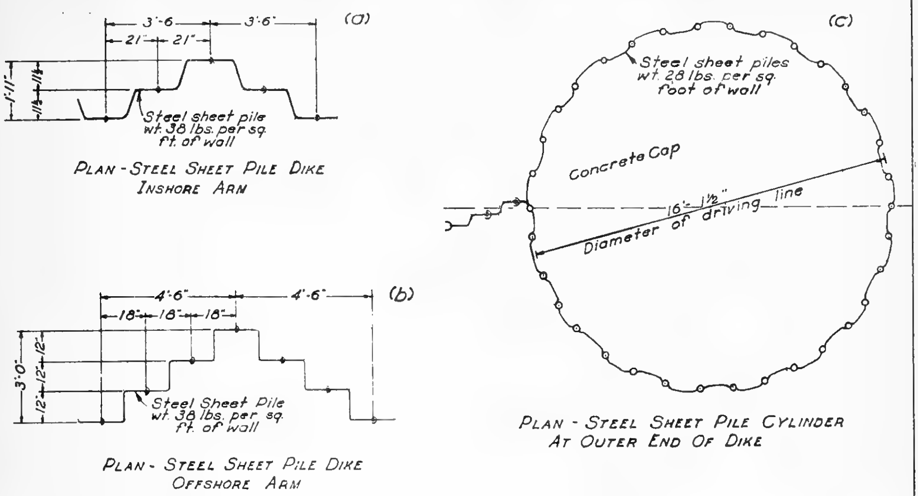


TYPICAL SECTION - NORTH AND SOUTH BREAKWATER
MILWAUKIE HARBOR, WISCONSIN



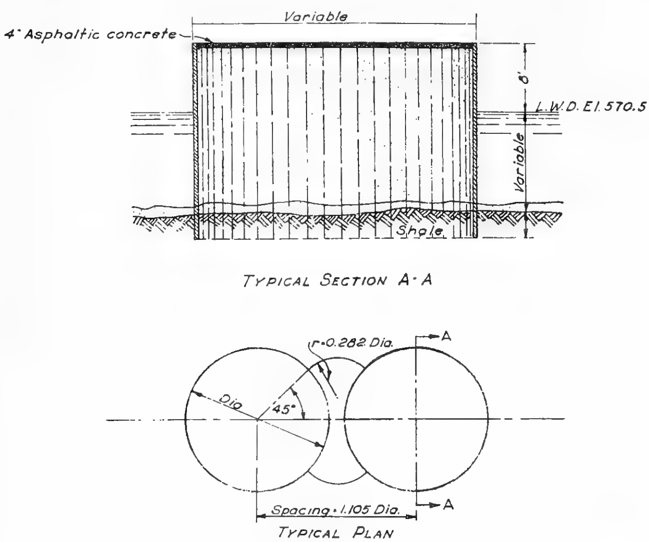
TYPICAL SECTION - SOUTH BREAKWATER
MUSKEGON HARBOR, WISCONSIN

FIGURE 120



PORT O'CONNOR DIKES - PORT O'CONNOR, TEXAS

FIGURE 121



CIRCULAR TYPE OF CELLULAR BREAKWATER

FIGURE 122

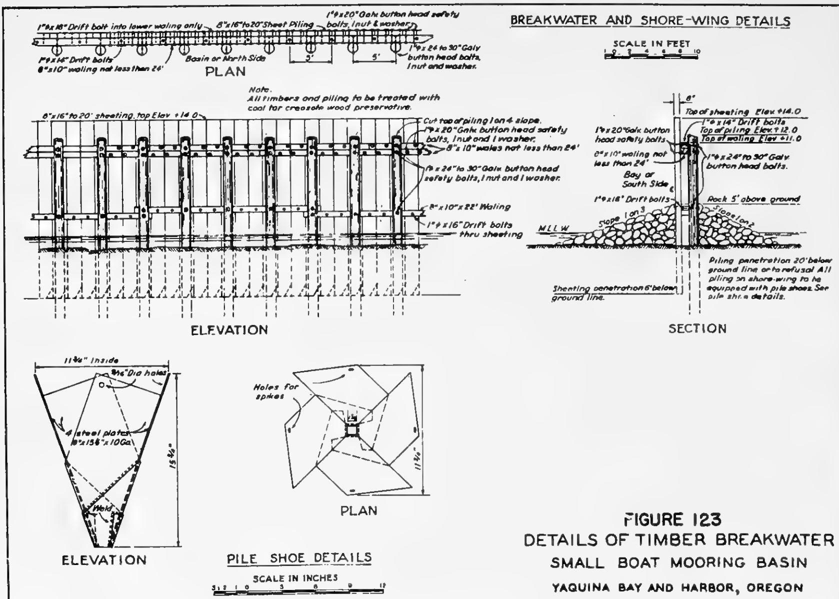
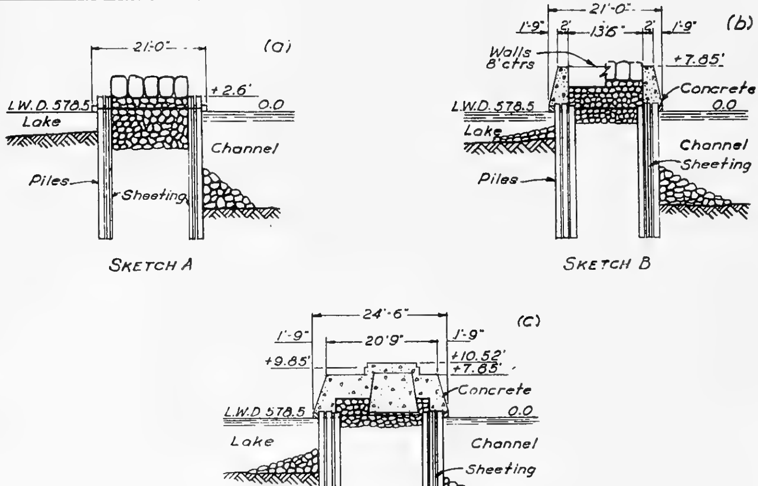


FIGURE 123
DETAILS OF TIMBER BREAKWATER
SMALL BOAT MOORING BASIN
YAQUINA BAY AND HARBOR, OREGON



TYPICAL SECTIONS - WEST PIER
MICHIGAN CITY HARBOR, INDIANA
FIGURE 124

corrosion is left undisturbed the piling may last 35 years or longer. If stone is used to fill the structure, its life will be greater than with sand filling, as the holes which first corrode through the web will have to be of considerable size before the stone will leach out and reduce the stability of the structure.

382. Crib Types. - Crib structures are constructed of timber in compartments, some of which are floored. The cribs are floated into position and settled upon a prepared foundation by filling the floored compartments with stone. The unfloored compartments are then filled with stone for stability. The structure then is capped with timber, concrete, or capstones. The superstructure and decking of cribs set on a rubble-mound foundation are often of timber to allow for settlement of the crib. When decay of the timber requires replacement, concrete may be used, as the crib will probably have settled into permanent position by that time.

383. Timber crib structures are suitable for depths of 50 feet or more. The foundation is prepared in a manner similar to that for concrete caissons. Timber structures are not suitable in salt water where marine borers are present. However, in fresh water timber continually submerged or saturated does not decay and the structure will last many years. Examples of timber crib breakwaters are shown in Figure 125 and 126.

384. Solid Fill. - A solid fill structure is sometimes constructed where it is desired to prevent wave action passing through the structure. One common type of solid fill breakwater consists of hydraulically deposited sand fill between stone mounds, with an armor of heavy rubblestone on the seaward side to protect the smaller material in the stone mounds against wave action. An example of solid fill breakwater is shown in Figure 127.

385. Asphaltic Materials. - In some instances the core and capstone have been consolidated to various depths by forcing hot asphaltic concrete into the interstices between the stones. Typical sections of structures where asphaltic concrete has been used are shown on Figure 128. Although the asphaltic concrete appears to serve its purpose in the cases shown, adjacent sections indicate that the use of portland cement concrete would probably have been superior. Asphaltic concrete used at other locations such as the Columbia River and Los Angeles Harbor has proven ineffective.

386. Selection of Type. - The selection of the type of offshore breakwater is dependent on many factors. Principal factors which may affect the selection of type include natural forces to be resisted, foundations, availability of material, desired life, and cost. Probably all are reflected in the cost. The desired life is also a function of the use of the structure.

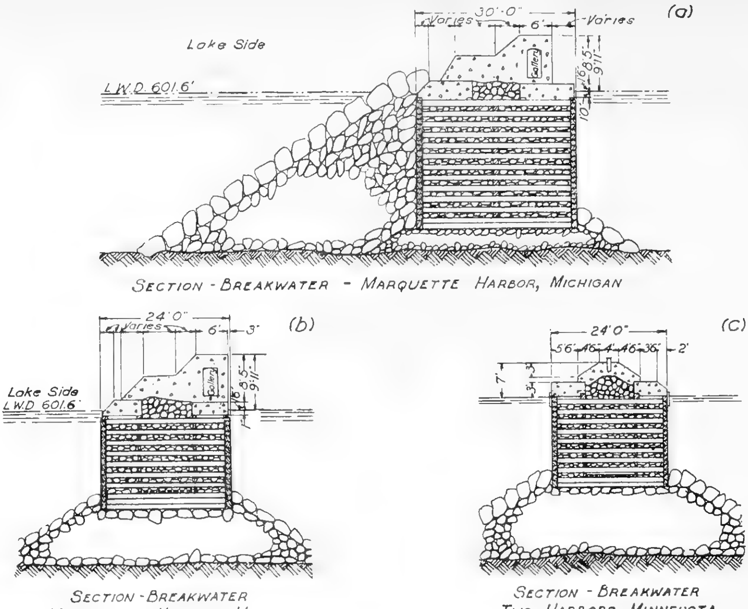
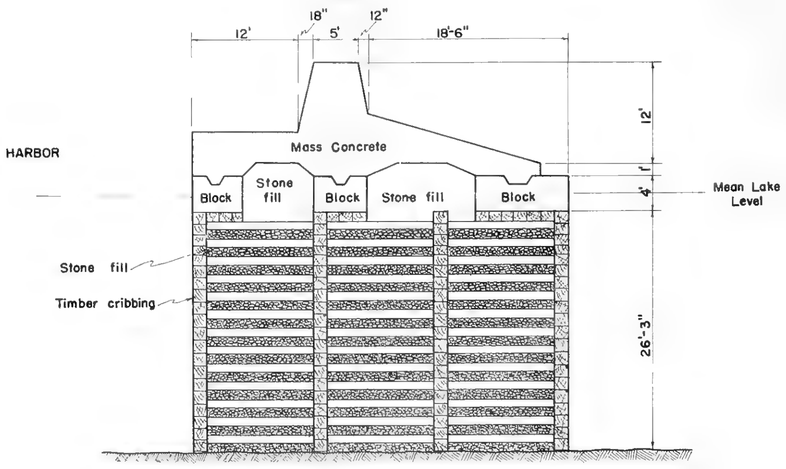


FIGURE 125



BREAKWATER AT HARBOR BEACH, MICH.
TIMBER CRIB TYPE

FIGURE 126

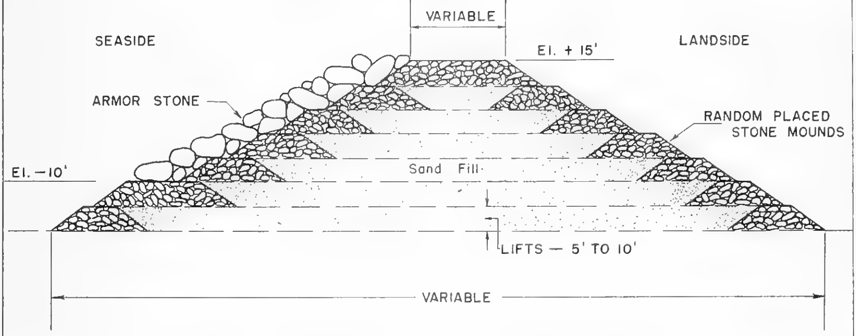
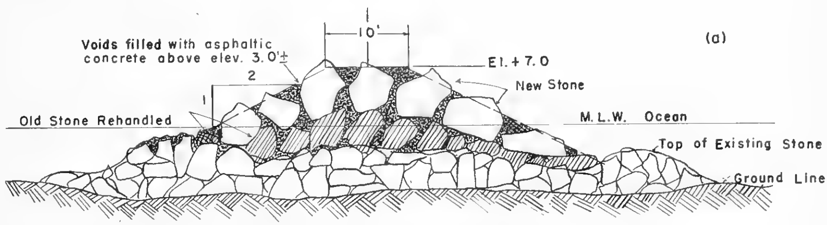
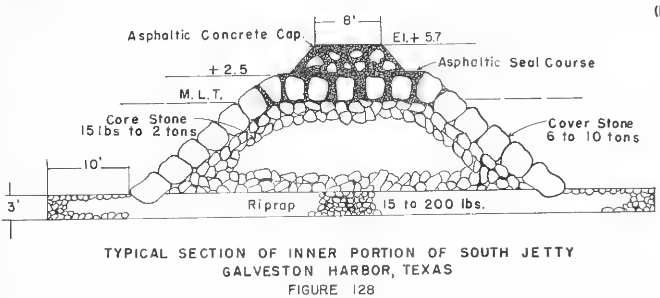


FIGURE 127.- TYPICAL SECTION — SOLID FILL BREAKWATER



TYPICAL SECTION - STONE AND ASPHALT CONCRETE JETTY

LAKE WORTH INLET, FLORIDA
100 FEET BUILT AT OUTER END OF NORTH JETTY



TYPICAL SECTION OF INNER PORTION OF SOUTH JETTY
GALVESTON HARBOR, TEXAS
FIGURE 128

387. The principal natural force to be resisted by a detached breakwater is wave attack. This is of primary importance in determining the type of structure. Heavy wave action would require one of the more massive types of structure, whereas sheet piling might suffice where wave attack is light. In some few instances involving submerged breakwaters to retain a sand beach, the earth pressures also must be taken into consideration.

388. The type of foundation may be the governing factor in the selection of type. For instance a rock bottom would not permit use of one of the sheet pile types. Also, with a readily erodible bottom, some flexible type of structure such as a random stone mound would probably prove desirable. Timber crib structures are moderately flexible to uneven settlement. Obviously, undermining of the structure by scour of an erodible foundation would be most detrimental to a caisson or masonry wall type breakwater.

389. Availability of materials may also dictate the type of structure by its effect on cost. Lack of stone within economic hauling distance may make mandatory use of some other material. In many instances steel sheet pile cannot be obtained due to shortages, hence concrete or timber must be used.

390. The desired life of the structure also dictates its type. Obviously, an untreated timber structure could not be installed on a sea coast where a structure is desired to last 50 years. Conversely, a permanent stone structure would not be constructed if the need for protection was of a temporary nature.

391. Finally, the choice of structure would depend on either its first cost or its annual cost. All of the factors discussed affect the cost in one way or another. Generally the structure selected would be that one which would accomplish the desired purpose at the lowest annual cost. Under some circumstances a structure with a somewhat higher annual cost might be selected to secure a substantial reduction in the first cost.

TYPE OF PROBLEMS

392. Caisson Type.* - By application of methods for forecasting and determining the characteristics of the design wave, described in the section on wave action, the following data have been found:

H = 13 feet = design wave height at the structure.
L = 150 feet = wave length at the structure.

*From Corps of Engineers, Engineering Manual for Civil Works Construction, "Design of Miscellaneous Structures." "Breakwaters and Jetties", Part CXXIX, Chapter 4, June 1952.

T = 5.4 seconds = wave period.

d = 26 feet = depth of water below datum at the structure.

The wave direction is normal to the breakwater. The depth of water being twice the wave height, the waves will not break on or before the structure and the Sainflou method for computing wave pressures from non-breaking waves can be used. The pressure diagram developed by this method is shown in Figure 129.

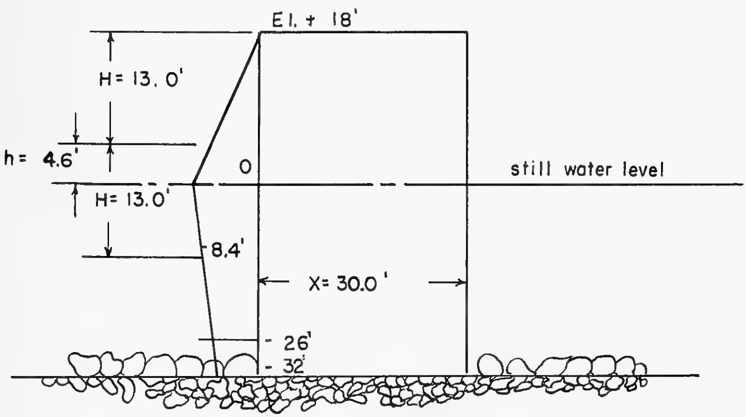


FIGURE 129-PRESSURE DIAGRAM FOR CAISSON TYPE BREAKWATER

393. The height (h_o) of the mean level of clapotis (orbit center) above the still water level, is taken from the graph, Figure 75 using the value of $2d/L = \frac{2(26)}{150} = 0.346$. In like manner the value of P_1 is taken from Figure 76. (See pages 103 and 104)

h_o = 4.6 feet

P_1 = 490 pounds - the pressure the clapotis adds or substracts from the still water pressure.

The upper and lower limits reached by the clapotis are

$h_o + H = 4.6 + 13 = 17.6$ feet above still water level.

$$h_o - H = 4.6 - 13 = 8.4 \text{ below still water level.}$$

Accordingly, to obstruct the oscillating wave completely, the breakwater should rise to not less than 17.6 feet above still water level.

394. With d = 32 feet, R_e and M_e may be found by

$$R_e = \frac{(d + H + h_o) \quad (wd + P_1)}{2} - \frac{wd^2}{2} \tag{143}$$

$$M_e = \frac{(d + h_o + H)^2 \quad (wd + P_1)}{6} - \frac{wd^3}{6} \tag{144}$$

so

$$R_e = \frac{(32 + 17.6) \left[62.5 \ (32) + 490\right]}{2} - \frac{62.5 \ (32)^2}{2}$$

$$R_e = 28{,}750 \text{ pounds.}$$

$$M_e = \frac{(49.6)^2 \left(62.5 \ (32) + 490\right)}{6} - \frac{62.5 \ (32)^3}{6}$$

$$M_e = 679{,}630 \text{ foot-pounds}$$

395. The width of the concrete structure is found by setting up the weight of the breakwater in terms of the unknown width, x, using submerged weights below still water level. If 150 pounds per cubic foot is the weight of the concrete in air and (150 - 62.5) = 87.5 pounds per cubic foot the weight of concrete in water, the equation is

$$\text{weight} = 18X \ (150) + 32X \ (150 - 62.5) = 5500X \tag{145}$$

If, for stability, it is required that the resultant of wave pressure and weight must fall within the middle third of the base, then

$$\frac{5500X^2}{6} = 679{,}630 + \frac{490X^2}{3}$$

$$906.5X^2 - 163.3X^2 = 685{,}700$$

$$X = 30 \text{ feet.}$$

and the weight = 5500X = 165,000 pounds

As the resultant cuts the base at the edge of the middle third, the factor of safety against overturning is 3 and the entire structure will be in compression.

396. To determine the factor of safety against sliding, multiply the effective downward force acting on the structure by a coefficient of static friction and divide by the horizontal thrust of the wave force. If the foundation is not dressed smooth, a friction coefficient of 0.5 or 0.6 is generally considered adequate. The factor of safety should not be less than 2.0.

$$\frac{\text{wt} \times 0.5}{R_e} = \frac{165{,}000 \times 0.5}{28{,}750} = 2.87 \tag{146}$$

which exceeds two and is satisfactory.

397. As the structure is symmetrical, the resultant of the vertical loads cuts the center of the base of the structure at a distance of 15 feet from the inner edge. The distance Z from the inner edge to the point where the resultant cuts the base, assuming uplift pressures in a triangular distribution, is given by

$$\begin{aligned} 165{,}000 - 490 \times 20 &= \frac{679{,}630}{15 - Z} \\ Z &= 10.3 \text{ feet.} \end{aligned} \tag{147}$$

398. The maximum pressure against the foundation would be at the inside edge with maximum wave conditions. If Z is the distance between the inside edge of the base and the point where all the resultant forces intersect the base, V is the effective downward force per unit length of wall, G is the pressure on one square unit of foundation, and A is the width of the breakwater, then the ground pressure at the shoreward edge of the breakwater is:

$$G = \frac{2V}{A} \left(2 - \frac{3Z}{A}\right) \tag{148}$$

$$G = \frac{2\ (165{,}000)}{30} \left(\frac{2 - 3\ (10.3)}{30}\right) = 10{,}700 \text{ pounds.}$$

This pressure would be large for most foundation conditions. Should this ground pressure be too great for the bearing power of the bed at the site, the base width of the structure must be increased until the pressure is within the allowable load.

399. Rubble-Stone Breakwaters. - The design of a rubble-stone breakwater is best illustrated by numerical example. It is assumed that a breakwater is to be constructed in sea water in a depth of 30 feet. The stone available locally has a specific gravity of 2.65. The quarry is capable of producing adequate quantities of stone weighing as much as 15 tons.

400. Referring to the methods of paragraphs 222 , the wave height at the structure's location, will be given by

$$H = H_o \left(\frac{H}{H'_o}\right) \sqrt{b_o / b} \tag{149}$$

Assume $H_o = 17$, $L_o = 184$, $(T = 6)$, $\sqrt{b_o/b} = 0.84$ and $d = 30$, $d/L_o = 0.163$, and from Table 1, Appendix D, $d/L = 0.194$ and $H/H'_o = 0.913$. Therefore, from equation (149), $H = 13$ feet. From Plates 7a and 12, Appendix D, stable slopes for various stone weights may be fixed in the following manner.

TABLE 30 - Computations for Stable Above Surface Slopes

Slope	W/K' (Fig. 7a)	K' (Fig. 12)	W (pounds)	W (tons)
1 on 3	3.3×10^5	0.033	11,000	5.5
1 on 2	8.2×10^5	0.019	15,600	7.8
1 on 1.5	2.7×10^6	0.015	40,400	20.2

Since 15-ton stone would be stable on a slope somewhere between 1 on 1.5 and 1 on 2, the slope of 1 on 1.75 will be chosen for this design. (Further refinements may be calculated by use of the equation

$$W = \frac{234\ K'\ H^3}{(1.09 \cos \alpha - \sin \alpha)^3\ (1.62)^3} \tag{150}$$

and reference to Table 15)

401. Sub-surface slopes may be determined by substituting, for H in equation (150) for any depth, an hypothetical wave height H' given by

$$H' = \frac{\pi H_s^2}{L_o \sinh \frac{2\pi d}{L}} \tag{151}$$

in which H_s is determined by extending equation (149) to points over the breakwater slope. Starting at depth 13 (one wave height below the surface) the computation for H' is illustrated in the following table.

TABLE 31 - Computation of H_s and H' for various values of d

$H_o = 17$, $L_o = 184$ $b_o/b = 0.84$					
d	d/L_o	$\sinh \frac{2\pi d}{L}$	(H/H'_o)	H_s*	H'*
13	0.0705	0.7823	0.9704	13.9	5.4
20	0.109	1.076	0.9263	13.2	2.6
30	0.163	1.546	0.9130	13.1	1.2

* H_s and H' are plotted on Figure 130

402. Assuming that 6-ton stone is available and will be placed in the section between the elevations of -13 and -20 feet, and that quarry run stone averaging 1/2 ton would be used below elevation -20 feet, stable slopes may be found from Plate 9a, Appendix D with K' = 0.015. With the 6 ton stone, $W/K' = 8 \times 10^5$ and with H' = 5.4 feet (d = 13 feet) the stable slope is found to be about 1 on 1-1/4. With the 1/4 ton stone $W/K' = 3.3 \times 10^4$ and with H' = 2.6 feet (d = 20 feet) the stable slope is found to be about 1 on 1.4.

403. In order to obstruct the waves effectively the breakwater crest should be at an elevation of about 19 1/2 feet (H x 1-1/2) above still water level. If only partial obstruction is mandatory the crest elevation may be set somewhat lower. The choice between the two must be made on a use and economic basis. Crest widths are determined mainly by the intended use of the breakwater or construction methods employed; in this case (say) 20 feet. Landward side slopes ordinarily should not be less than 1 on 1-1/2 to the depth of one wave height (13 feet), thence 1 on 1-1/4 to the bottom. With these criteria and the computed sea side slopes of 1 on 1.75 to elevation -13, 1 on 1.25 to elevation -20, and 1 on 1.4 to the bottom, a stable profile may be drawn as in Figure 131. Note that end slopes should be designed in the same manner as sea side slopes.

404. Composite Type Breakwaters. A composite type breakwater is one comprised of two or more types and materials or of two or more materials. There have been many types of composite breakwaters designed and constructed where it has appeared that a saving in time, material, or cost would be effected thereby. One type of composite breakwater consisted of a shale or earth core incased in a rubble-mound covering. Another type is comprised of a concrete caisson wall set on a rubble-mound base.

405. As wave action is affected by the profile of the substructure, the characteristics of the wave which act on the superstructure must be determined accordingly. Should it be desired to place the base of the

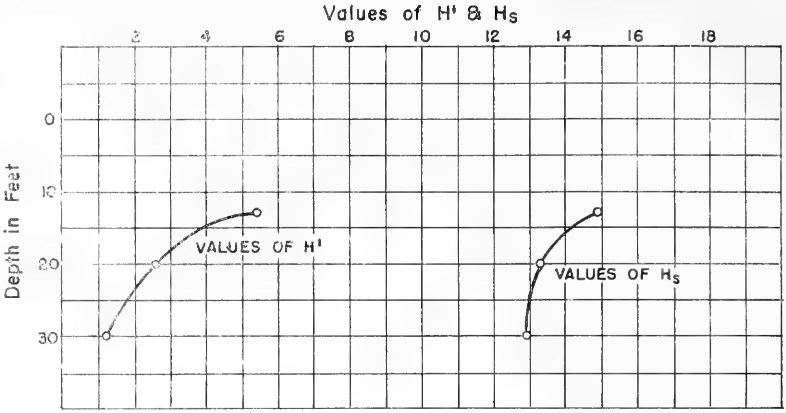


FIGURE 130 - VALUES OF H' AND H_s IN TERMS OF DEPTH (d)

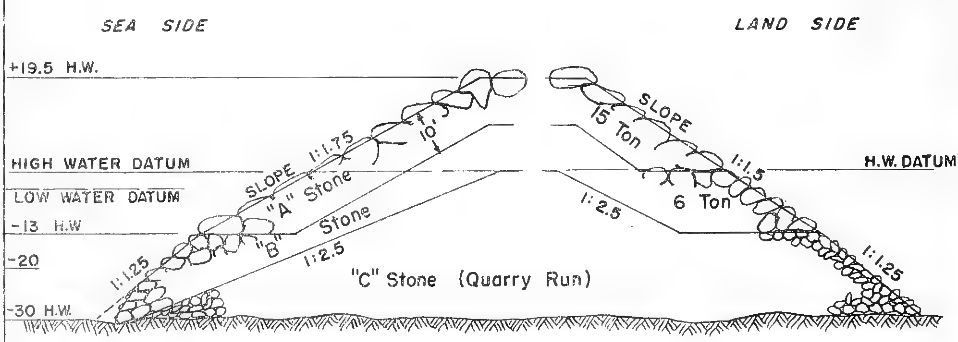


FIGURE 131 - PROFILE OF BREAKWATER SIDE SLOPES

concrete caisson breakwater described in the previous example (see paragraph 392) on a rubble-mound substructure at elevation -10 feet, values for H and H' should be found for this depth by Iribarren's method. H' is used in computing the weight of the individual stone at elevation -10 feet, and H is the shallow water wave height at the superstructure. The wave height H should be used in the stability analysis of the concrete superstructure. Assuming the same conditions as in the previous examples, the wave characteristics are computed to be:

$H' = 8.1 \quad H = 14.3 \quad d = 10 \quad d/L_o = 0.0543$

$d/L = 0.09859 \quad L = 101.4$

Using these wave data, the concrete caisson would be designed according to the procedure described in paragraphs 392 to 398.

SAND BY-PASSING

406. It is often desired to nourish beaches downdrift from a natural or artificial barrier within the littoral zone, the trapped material acting as a source of supply. Such a littoral barrier may be a jettied entrance to a harbor, a natural inlet, an offshore breakwater, or a shore connected breakwater. A sand by-passing plant is a plant designed to mechanically transport littoral drift past a barrier to a point on the downdrift shore from which it will again be moved by natural forces. Several methods for by-passing sand have been considered with a view to reducing the cost of the operation. Methods considered at various times include the following:

a. Fixed land-based dredging plants;
b. Portable land-based dredging plants;
c. Eductor method with a fixed plant;
d. Floating plant.

407. In this section those methods, already in operation, will be described.

408. Fixed Plant. - A small fixed plant has been used for some time at South Lake Worth Inlet, Florida. (see Figure 132). After construction of jetties at South Lake Worth Inlet, erosion occurred on the downcoast beach. A seawall and groin field failed to protect the shore line and a small by-passing plant was placed in operation in 1937. The basis of design of the pump and pumping plant as initially installed was not related to the rate of littoral drift along the shore. It was designed to transport the quantity of sand over a period of 2 years estimated as required to fill the groins and give adequate protection to the seawall. Pumping was continued until 1942.

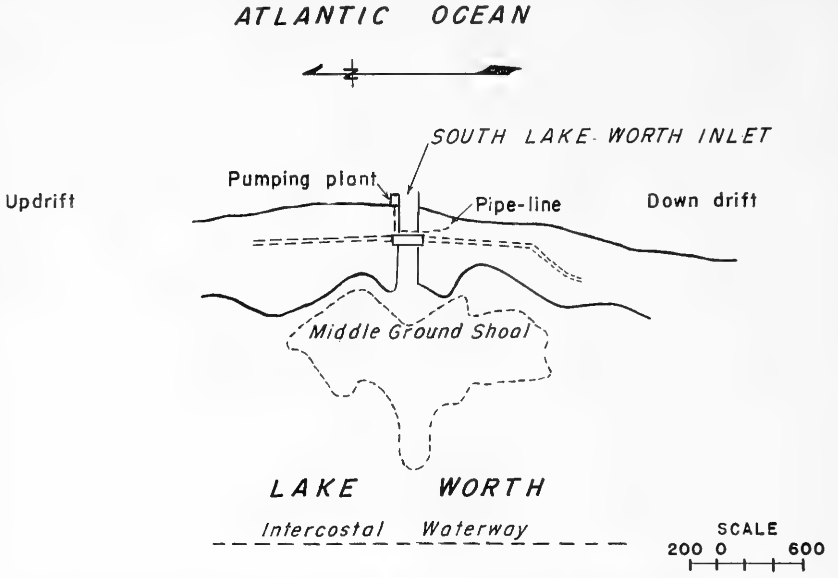


FIGURE 132 – SOUTH LAKE WORTH INLET BYPASSING PLANT

409. The pumping plant consisted essentially of an 8- inch suction line, a 6-inch 65 h.p. Diesel-driven centrifugal pump, and about 1,200 feet of 6-inch discharge line. The installation had a capacity of about 55 cubic yards an hour and averaged 48,000 cubic yards a year during 4 years of operation. The normal rate of littoral drift during that same period was on the order of 225,000 cubic yards a year. At the end of 5 years the beach was restored for a distance of over a mile downcoast.

410. During the 3-year period, 1942-1945, pumping was discontinued with the result that severe erosion of the beaches south of the inlet again occurred. Accordingly, in 1945, the plant was again placed in operation. In an attempt to reduce shoaling in the entrance channel, the size of the installation was increased to a 10-inch suction line, 8-inch pump with 275 h.p. Diesel motor, and 8-inch discharge line. Observations indicate that a larger pump is required and the installation of a 12-inch pump and discharge line is under consideration.

411. A by-passing plant on a large scale has been constructed at Salina Cruz, Mexico. Salina Cruz is an artificial harbor in the Gulf of Tehuantepec on the Pacific coast of Mexico. (See Figure 133) This plant essentially consists of six 18-inch suction pipes operating through two dredge pumps. Each dredge pump is motivated by a 450 h.p. Diesel engine. An 18-inch discharge line crosses the entrance to the inner harbor on a drawspan and discharges on the beach downcoast from the eastern breakwater.

412. The Salina Cruz installation immediately encountered operational difficulties. The suction pipes of the plant were not long enough to open a channel in the beach that would permit the free entrance of the sand to the dredge. An attempt has been made to open this channel by means of a dragline operating across the beach by a cable supported from two posts. As yet it is not known whether operational difficulties have been overcome.

413. In theory, the plant is designed with a capacity somewhat greater than the average rate of littoral drift. Its purpose is to pump initially a sufficient quantity to pull the shore line back to the future alinement shown on Figure 133. Thereafter the plant would pump all material coming to it along the coast. This is expected to prevent shoaling of the harbor as well as to prevent damage of downdrift areas. The plant has cost approximately $2 million to date. It remains to be seen if it can be made to function as designed.

414. Portable Land-Based Plant. - Because of the wide range in the daily rate of littoral drift, any by-passing plant designed to pump substantially all of the material moving along the coast, must necessarily have a capacity of several times the average daily rate computed from an annual basis. Thus, after the operating sequence has been established the plant would be required to operate only a fraction of the time - probably not over 4 to 5 hours a day. In an effort to reduce plant costs, consideration has been given to a portable land-based dredging plant. This plant would probably be operated from a fixed pier or other foundation. It would have to be highly portable so that it could be rapidly and easily assembled, disassembled, and transported between sites. By working full time, such a plant would be able to by-pass sand from several sites, provided adequate sand traps were constructed to impound littoral material between dredging. However, no known working plants of this nature are presently in operation.

415. Eductors. The successful use of eductors in moving 14 million cubic yards of sand from dunes to the beach during the preparation of the Hyperion sewage treatment plant site on Santa Monica Bay, California, led to a consideration of the possibilities of its use for by-passing sand. In principal the eductor is merely a venturi activated by a high pressure jet. It can be effectively used to move sand for distances up to 2,000 feet, depending on the size of the eductor and high pressure jet.

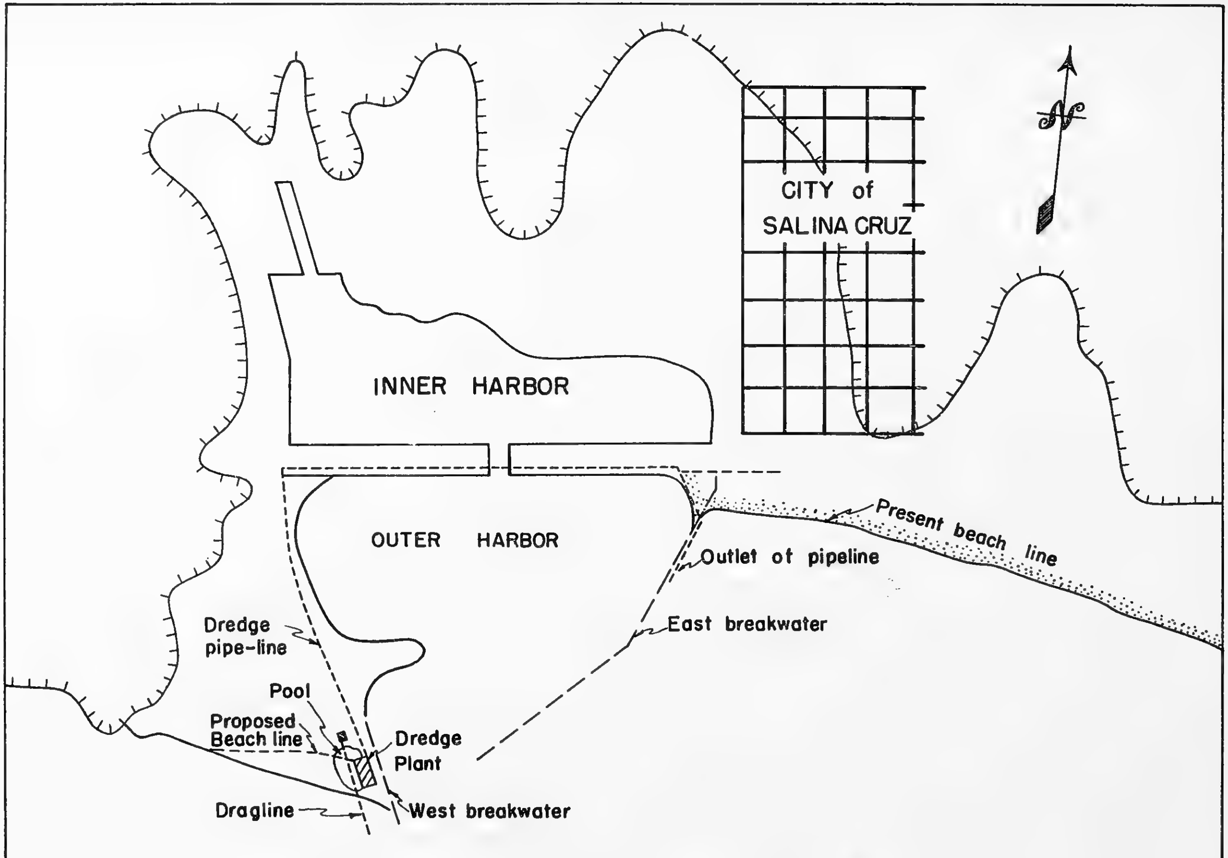


FIGURE 133-FIXED DREDGE, SALINA CRUZ, MEXICO

The efficiency of the eductor is somewhat lower than that of an ordinary dredge pump with a suction pipe. Studies to date have failed to indicate sufficient savings in construction or operating labor costs as to offset the lower efficiency and show an overall saving in cost per cubic yard of material moved.

416. Floating Plant. - To date no satisfactory method has been found which offers any improvement on the proven methods of periodic movement of beach material by hydraulic pipe-line dredge. Sand has been successfully by-passed by this type of plant at Santa Barbara, California, for the last 15 years. (See Figure 134.)

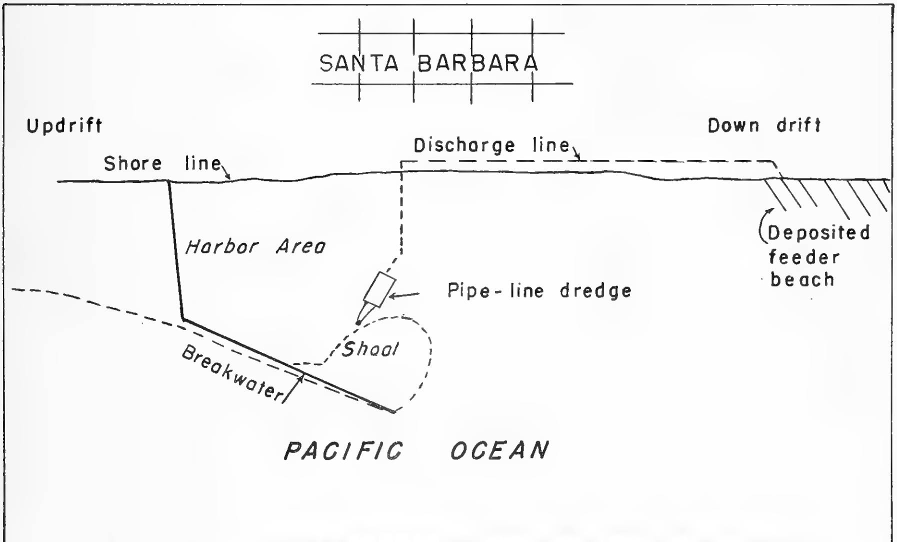


FIGURE 134 — SANTA BARBARA HARBOR-SAND BYPASSING

The rate of littoral drift at Santa Barbara has been determined to be about 270,000 cubic yards a year. It moves downcoast. This drift was completely intercepted by a shore-connected breakwater, constructed in 1927. After its construction, the upcoast angle of the breakwater filled in and material passed along and around the seaward end of the structure, settling in the protected harbor area. The resultant shoal extended across the harbor entrance towards shore. Coincident with the accretion of material in and upcoast of the harbor, erosion began on the downcoast side of the harbor. Within a period of 10 years, the erosion zone extended a distance of 10 miles downcoast and the damage amounted to millions of dollars.

417. A project was inaugurated in 1935 for the maintenance of navigable depths in the harbor entrance and the restoration of the downcoast beaches by cooperation of the United States and the city and county of Santa Barbara. The first dredging in 1935 was done by hopper dredge. The material was deposited in 18 feet of water in a mound parallel to shore. It was hoped that the material would be moved ashore by wave action, a hope that was never realized. Subsequent dredging has been accomplished at 2 to 3 year intervals by hydraulic pipe-line dredge. In these operations, amounts ranging from 500,000 to 800,000 cubic yards of material were dredged and deposited directly on the beach downcoast of the zone of influence of the breakwater. Within a second 10-year interval the beaches had been re-established and the shore line stabilized along its new alinement. The action at Santa Barbara has adequately demonstrated the efficacy of periodic by-passing of material to maintain normal littoral drift to the downdrift beaches.

418. Two things are required for the effective use of floating plant for periodic by-passing of sand: (1) a sand trap of sufficient size as to retain littoral material between dredging operations; and (2) a protected area within which the suction dredge can operate in safety. Both conditions were satisfied by the breakwater at Santa Barbara. The Santa Monica breakwater (see Figure 68) has provided similar protection, as well as a sand trap, from which similar by-passing has been accomplished.

419. In some few instances, dredging with floating plant has been attempted on the open coast by confining operations to calm periods. Even so, much lost time has ensued and damage has been severe with the results that contractors either submit high prices or no bids at all on this kind of work. If conditions are favorable, consideration may well be given to the construction of a combination sand trap and protective structure.

SAND DUNES

420. The problem of dune control resolves itself into two fundamental objectives: (1) the stabilization and maintenance of sand dunes at locations where they exist naturally; and (2) the inducement of dune formation where they do not exist naturally, by the direct and permanent stoppage or impounding of sand before the location to be protected, and the stabilization of the dune formation after its creation.

421. Wind blown sand accumulates in several distinctive ways and characteristic forms. Any appreciable accrual is loosely termed a dune. Strictly, according to the most generally accepted definition, a true dune is one capable of moving freely as a unit and which can exist independently of any fixed surface structure. The loose definition is used herein. Sand accumulations caused directly by fixed obstructions in the path of the wind, unlike true dunes, are dependent for their continued existence on the presence of the obstacle which causes and fixes them so they cannot move away.

422. When an object is placed so as to interrupt the wind flow, the air path in front and behind the obstacle is divided into two parts by a somewhat ill-defined surface of discontinuity. Outside this surface the air stream flows smoothly by; but the volume within the wind shadow of the obstacle is filled with swirls and vortices of air whose average forward velocity is less than that of the air stream outside. Downwind from the obstacle, the forward velocity of the air inside the shadow gradually increases and the shadow fades away to merge eventually with the general flow of the wind. The sand grains which strike the obstacle rebound from it and come to rest in the relatively stagnant air in front. When the resulting heap has grown up so that its slopes stand at the limiting angle of repose (about 34°) all additional material slides down the slope to join the sand stream passing along the side of the obstacle.

423. Dune Building. - Dunes can be caused to form by the use of sand fences or pentration oil.

424. Sand fences can be constructed in movable sections or made of individual pickets driven into the sand. The width and length of the pickets may vary but the spacing of the pickets is important, with no more than 50 per cent of the surface covered. For best results the space between the pickets should equal the width of the pickets. In order to widen the crest of the dune, and facilitate establishment of vegetation, two lines of fence about 30 feet apart should be used. The use of a single fence tends to make dunes with a sharp crest unfavorable to establishment of cover. As the dune builds up on the fence, the fence can be raised until the desired height is attained. The belt can be broadened by shifting the second fence windward as the dune grows, or by the addition of a third fence.

425. The sand fences should be constructed normal to the prevailing wind direction unless it is desired to cause the sand to move longitudinally along the fence to fill in low gaps. In this case the fences may be built slightly quartering to the general shifting direction of the sand movement or panelling may be resorted to.

426. Panelling, to divert or stop sand, consists of curved or flat barriers in a single slant or in V arrangement. As the line of the slant or V approaches the normal to the wind direction, there is more and more

stoppage, and less and less diversion of sand. These barriers require frequent changing or cleaning. The dune development behind diversion fences is shown in Figure 135.

427. Oil penetration coats of hot crude oil may be used to stabilize the face of the dune and cause the dune to widen. Penetration is deeper when the upper few inches of sand are dry. Oiling is expensive and special equipment is necessary for highest efficiency. Oiling is effective for only a few years and must be repeated frequently. Oiling increases the saltation coefficient of material flowing over the surface. The sand blows up and over the top, depositing in front of the slip face in successive advancing fill increments. This widening of the dune continues until a streamlined body is built downwind from the paved face. The entire mass is permanently stored for as long as the oiled surface remains intact. Figure 136 shows the successive steps of dune development behind an oiled surface.

428. Dune Stabilization. - After protective dunes have been formed they should be stabilized with vegetation. This is expensive in the beginning but minimizes future difficulty. The most satisfactory plants are long lived perennials, with extensive root systems, that spread rapidly either vegetatively or by seed or both, and maintain surface growth even though sand is accumulating around them to increasing depths. Such plants are not numerous but in practically every section a few satisfactory ones are obtainable at reasonable costs.

429. Usually grasses are available and can be transplanted from naturally established plantings. Transplanting is most satisfactory by using small clumps $\frac{1}{2}$ to 1 inch in diameter, setting about 8 to 10 inches deep in staggering rows with plants in rows about 18 inches apart. Staggering the rows prevents direct wind action over long areas and offers more opportunity to hold sand than straight line planting. Dunes are low in fertility and contain practically no organic matter. Nitrogeneous fertilizers are stimulating and while not effective very long in the sand, they are of benefit in inducing vigor and enabling newly set plants to get firmly established the first year. The use of coarse fertilizers as manure is generally not practicable because its effect is so slow that little of its value would be realized.

430. Brush barriers may be used effectively to control the sand on newly filled areas while grasses, shrubs, or trees are gaining a foothold. These barriers are usually built in rows 4 feet apart, crosswise with the prevailing wind, about 2 feet high, 1 to 2 feet wide, and anchored firmly with stakes. Any kind of brush will suffice, although evergreen material is most satisfactory.

431. Covers for Various Regions. - In the Great Lakes region, the best grasses for use in sand binding are European beach grass, long-leafed reed grass, European dunegrass, Redfield grass, and Northern Wheatgrass. All are native to the region except European beach grass

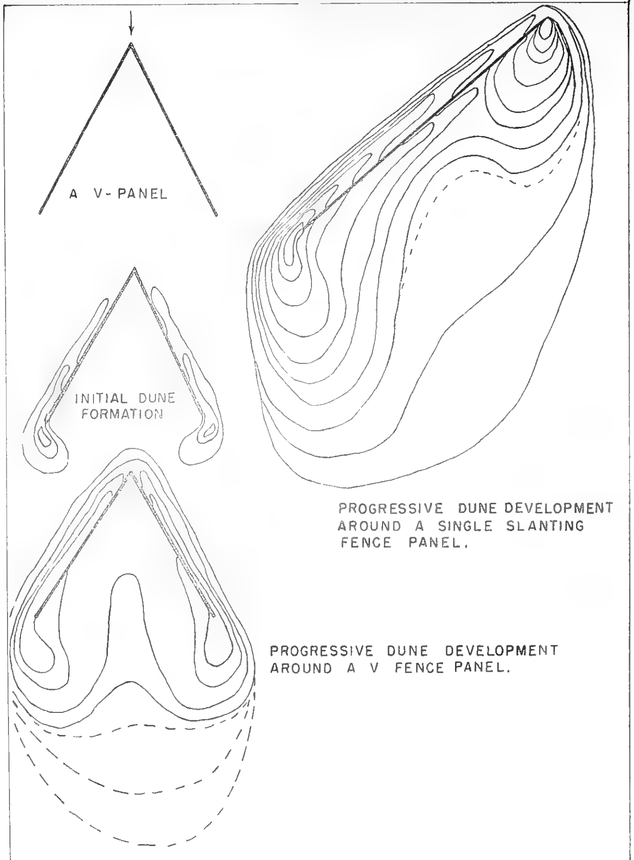


FIGURE 135-DUNE DEVELOPMENT AROUND FENCE PANELS.

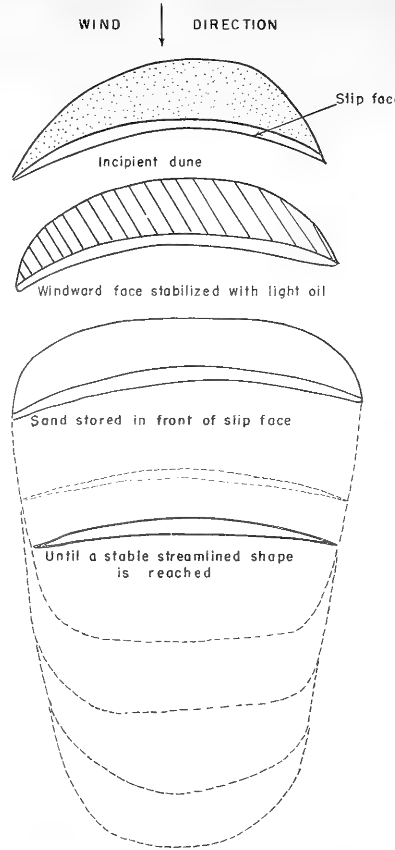


FIGURE 136-DEVELOPMENT OF AN OILED DUNE

which has been introduced at many points and is not uncommon.

432. On the North Atlantic coast from Massachusetts to Maryland the most serviceable grass for sand binding is beach grass. European dunegrass and salt marsh grass are not uncommon and when available locally may be used to advantage. The most serviceable shrubs are Scotch broom and northern bayberry.

433. From Virginia to Florida on the Atlantic coast, bitter panic, Bermuda grass, seaside oat, and salt marsh grass are among the best herbaceous plants. On the Florida coast, Saint Augustine grass has been found well adapted to sandy areas. Bermuda grass will probably be found the most generally satisfactory sand-binding grass, although this can often be supplemented to advantage with centipede grass, long-leafed reedgrass, seaside oat, Japanese lawngrass, and others.

434. In the northern Pacific coastal area, beachgrass, seaside grass, and sea lyme grass have been found most serviceable. Seashore lupin and beach pea are common in this area and very efficient sand binders. In the southern Pacific coastal area from San Francisco to Santa Barbara, the beach grasses are the most serviceable. Sea lyme grass and several species of Mesembryanthemum are effectively used.

435. Scientific names of the more commonly used grasses and shrubs are as follows:

Common Name	Scientific Name
Long-leafed reedgrass	Calamovilfa longifolia
European dunegrass	Elymus arenarius
Redfield grass	Redfieldia flexuosa
Northern wheatgrass	Agropyron dasystachyum
Beach grasses	Amophila arenaria, Ammophila breviligulta
Salt marsh grass	Spartina alterniflora
Scotch broom	Cytisus scoparius
Northern Bayberry	Myrica pennsylvanica
Bitter panic	Panicum amarum
Bermuda grass	Cynodon dactylon
Seaside oat	Uniola paniculata
St. Augustine grass	Stenotaphrum secundatum
Centipede grass	Eremochloa ophiuroides
Japanese lawngrass	Zoysia japonica
Seaside grass	Poa macrantha
Sea lyme grass	Elymus arenarius
Seashore lupin	Lupinus littoralis
Beach pea	Lathyrus maritimus

GROINS

GENERAL CONSIDERATIONS

436. Scope. - In the past, groins generally have been built, not designed. As the result, many have been too light to withstand the forces to be met and have failed. Others have been overbuilt to such an extent as to be uneconomic. Because of the many unknowns involved, this section on design is an attempt at standardization on a safe basis rather than any presentation of an exact design analysis. The types of structures analyzed include a steel sheet pile groin, a concrete block groin and a rubble-stone groin. Modifications of these would be similarly designed.

437. Type and Description of Groins. - Nearly as many different types of groins have been constructed as there are people to design and build them. The following list includes a few of the more representative types which have been found satisfactory through actual experience. The selection of the type to use is dependent on many interrelated factors. The various types of groins listed below are not necessarily in the order in which they would be recommended.

a. Concrete permeable;
b. Concrete impermeable;
c. Steel sheet pile;
d. Rubble-stone;
e. Stone block;
f. Timber

438. Concrete Permeable Groins. - Of the various types of permeable concrete groins, that patented by Sydney M. Wood probably has been the most widely used. Accordingly, it has been taken as typical of this type of groin. This groin consists of precast reinforced concrete units shaped like flat dumbbells. See Figure 137. These units are threaded on piles to form cribs. The piles contribute materially to the stability of the groin and may be used to support erecting equipment. As part of the theory of permeable groins is that the permeability may be varied, the longitudinal members may vary with respect to the number of vertical lugs. The lengths of the units range from 6 to 14 feet and the corresponding weights from 3/4 to 1½ tons. The width of the groin is equal to the length of the units. The 14-foot lengths are used below water; varying the lengths from the surface of the water to the top of the groin makes a sloping side and a top width of 6 feet. The piles serve as guides to position the units and lock them together. Each group of four piles and connecting units comprise an independent crib unit, permitting a measure of unequal settlement without failure of the structure.

439. Concrete Impermeable Groins. - Impermeable concrete groins may be either articulated or solid throughout their entire length. In general, the articulated type appears to be the most practicable as it will permit of unequal settlement. Also, being handled in smaller units, it is

simpler to construct as costly forming is avoided. Figures 138, 139, and 140 show three types of impermeable concrete groins. Figure 138 shows a combination concrete and stone semi-articulated type. Figure 139 shows an all concrete articulated type. Figure 140 shows an all concrete type originally articulated and later solidified with a concrete cap.

440. In Figure 138, the precast reinforced concrete groin is constructed of vertical transverse blocks shaped like a capitol "I", 6 feet wide and 4 feet long. The height of the unit is equal to the width of the unit but may vary with location. Horizontal panels 2 feet high, 8 inches thick, and 10 feet long, with notches on either end, are placed in between the transverse blocks. Rock fill is then placed in between the horizontal panels and covered with a 6-inch concrete slab cast in place. The concrete slab can be used to tie the entire structure into a single unit.

441. In Figure 139, the groin is composed of concrete precast trapezoidal blocks, each 3 feet 3 inches high and weighing 5 tons. Two types of blocks are used. Type A, shown in Figure 139 has a length of 5 feet, a top width of 2 feet and bottom width of 6 feet. Type B, not shown but similar to type A, has a length of 3 feet 10 inches, a top width of 3 feet 2 inches, and a bottom width of 7 feet 2 inches. The block at the offshore end is the same as type A but with a sloping offshore face. The steel joints and reinforcing, shown in Figure 139, are the same for both type A and type B. A patent on the joint is held by Harrison Weber. The reinforcing bars are welded to the I-beam and the channels on opposite ends of the block, and are placed as one unit in the form for casting the concrete block.

442. The blocks are placed on the beach surface, the flexible nature of the joints enabling the blocks to conform with the ground slope and to undergo a measure of settlement. The height of groin can be increased by maintaining the relationship between height and base width. As some settlement occurred where this type of groins was originally placed on a sandy beach, a mat of crushed stone was used as a foundation in later installations.

443. Figure 140 shows another type of concrete block groin which is simple to cast and place, and which has served satisfactorily in some locations. The height of groin can be changed by changing the length and base width of the concrete blocks. Concrete channel-ways are cast in the block as a guide in placing and to tie the blocks together. No special type of end blocks is required. After the blocks have settled for a period of time, the concrete cap can be poured. This cap will increase the height of the groin somewhat, and tie all the component parts together into a monolithic groin.

44 . <u>Steel Sheet Pile Groins</u>. - Satisfactory steel sheet pile groins have been constructed with straight web, arch web, or M or Z sections. All of these are made with interlocking joints that do not pull apart

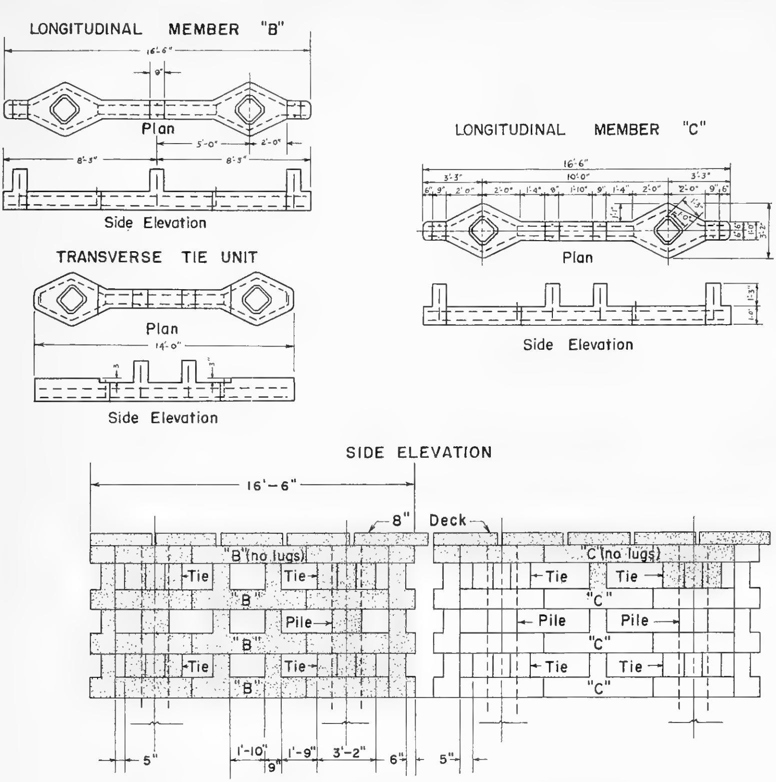


FIGURE 137 CONCRETE PERMEABLE GROIN (SYDNEY M. WOOD)

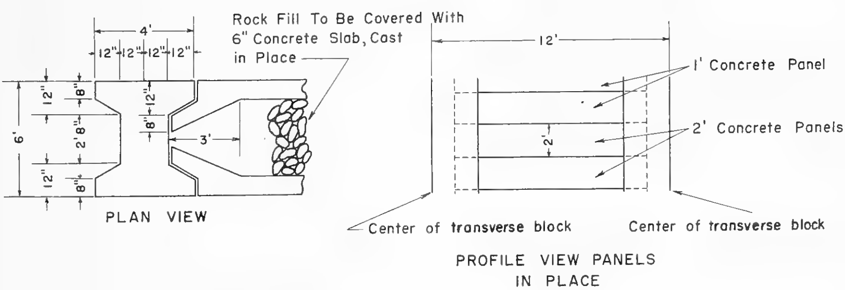


FIGURE 138-COMBINED PRECAST CONCRETE AND ROCK FILL GROIN

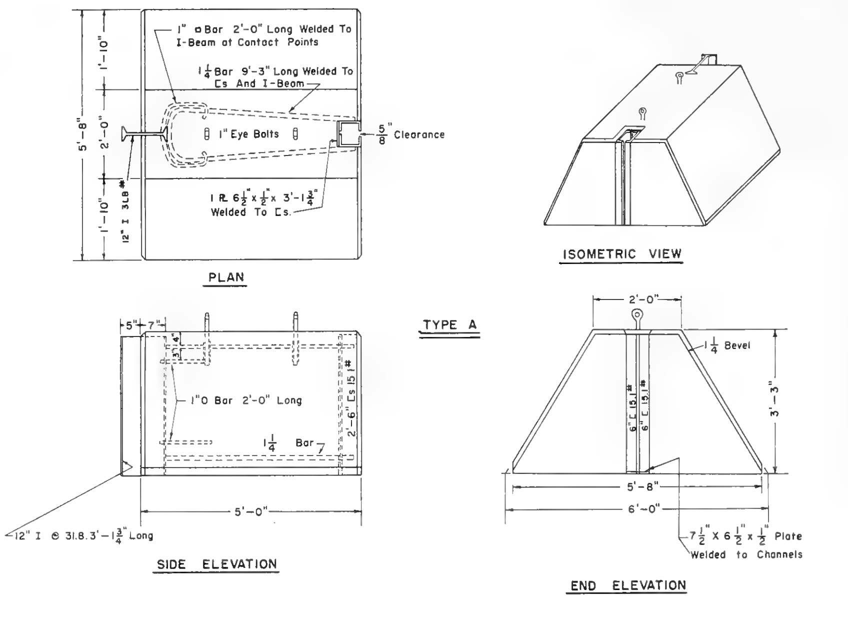


FIGURE 139 CONCRETE BLOCK GROIN (HARRISON WEBER)

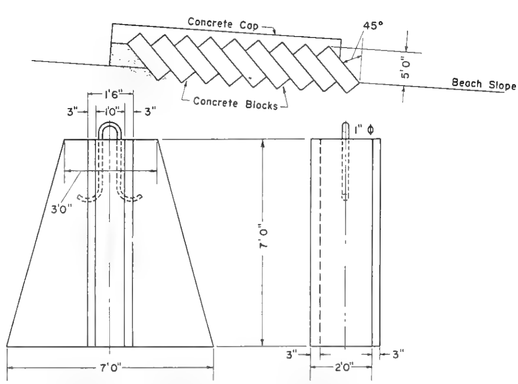


FIGURE 140 CONCRETE BLOCK GROIN (C. L. LEEDS)

when subjected to blows from the waves and provide positive sand-tight connections between adjacent piles. The type of pile selected is governed by the wave action to be encountered. The section modulus increases as the depth of the arch increases in the arch web, M, and Z sections. The steel sheet pile groin is usually constructed with steel or horizontal timber wales along the top of the steel pile and in some cases, vertical round timber piles or brace piles are bolted to the outside of the wales for added support. The round piles are not always required with the M and Z sections but would ordinarily be used with the flat or arch web sections. A typical design for a steel sheet pile groin is shown in Figure 141. The round pile and timbers should be creosoted to maximum treatment for use in waters infested with marine borers. This would not be as necessary in the Great Lakes. In some instances the life of the steel sheet pile has been indefinitely prolonged by pouring concrete slabs on either side of the sheet pile after holes have been scoured through the piling by moving sand.

445. The cellular type of steel sheet-pile groin is being used more extensively, especially on the Great Lakes where foundation conditions are a problem and adequate pile penetration cannot be obtained. A typical cellular type groin is shown in Figure 142. This groin is comprised of cells of varying sizes, each consisting of semicircular walls connected by cross diaphragms. Each cell is filled with sand or stone to increase stability. A concrete slab may or may not be poured over the top to provide a walkway or platform and to retain the fill material.

446. Stone Groins. - Stone groins have been successfully constructed of either rubble or of cut stone blocks. Either type can be made permeable or impermeable. The impermeable rubble-stone groin is constructed with a core of quarry run stone including sufficient fine material to make it sand tight, and with caps of stone sufficiently heavy to protect the structure from anticipated wave damage. The random or rubble-stone mound is usually constructed with 1 on $1\frac{1}{2}$ side and end slopes and a top width of 5 feet or more. A typical stone groin is shown in Figure 143. The size of stone used in the core may vary depending upon the source. The layer of cover stone should be a minimum of 3 feet thick with individual stones weighing from 1 to 4 tons or more depending on the wave action to be resisted; averaging approximately 3 tons.

447. Several variations to the all-stone groin have been used. In some instances, to increase the impermeability of the structure, a diaphragm of timber Wakefield piling or steel sheet piling has been included. Also in many instances the shore end of the structure has been constructed either of timber or steel sheet piling, thus reducing the overall cost of the structure without reducing its economic life or its ability to resist severe wave attack or erosion. Generally the timber or steel section does not extend seaward of the crest of berm. Where it is not exposed to the action of marine borers, untreated timber may give a considerable length of service.

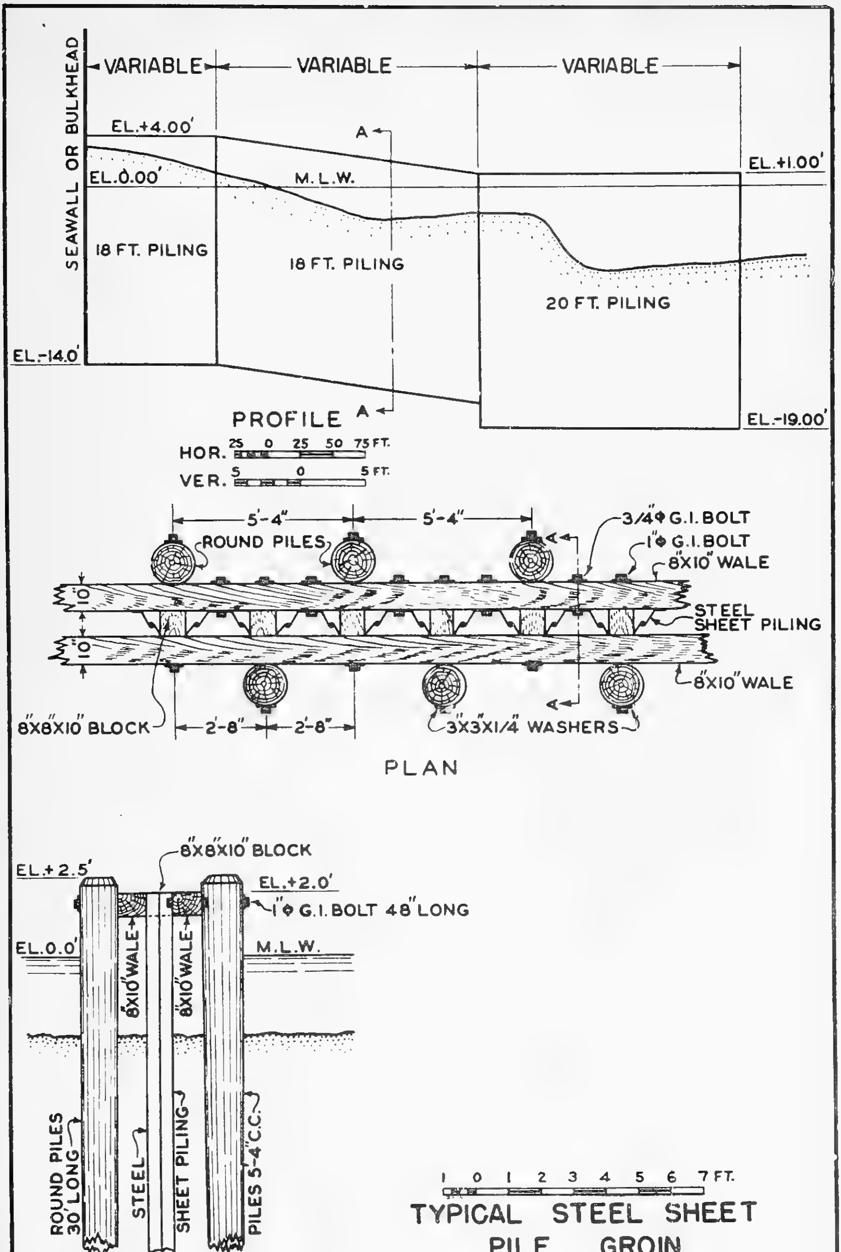
448. One satisfactory method of sealing an all-stone groin to make it impermeable is to fill the voids between the stones with concrete grout. This also increases the stability of the structure to resist wave action. The grout should be placed over the entire exposed surface of the groin and forced into the voids to the sand line. A rich mix should be used (7 sacks of cement to the cubic yard). If exposed to wave action while setting, an admixture of calcium chloride may be added to expedite the set. Where building sand and fresh water are not available, beach sand and sea water have been used satisfactorily.

449. Permeable stone groins differ from impermeable stone groins in several ways. The permeable groin is constructed with core stone large enough to keep it from being sand and water tight. The side slopes are usually 1 on $1\frac{1}{2}$, but the top width is about 3 feet, or the minimum required for stability. The section would be similar to that shown in Figure 143.

450. Stone block groins are usually more or less permeable although they have been made impermeable by placing a steel sheet pile diaphragm down the center of the stone blocks. The individual blocks range in weight from 2 to 6 tons but must be of sufficient size to withstand wave action. The height of groin along the profile is easily varied by adding another row of the stone blocks. Typical sections are shown in Figure 144.

451. <u>Timber Groins.</u> - The most common type of timber groin is an impermeable structure composed of sheet piling supported by wales and round piles. All timbers and piles should be given the maximum recommended pressure treatment of coal tar cresote. All holes should preferably be drilled before treatment. A typical timber groin is shown in Figure 145. The round timber piles forming the primary support of the groin should be a minimum of 12 inches in diameter at the butt. Stringers or wales, which are bolted to the piling horizontally, should be at least 8 inches by 10 inches, preferably cut and drilled before creosoting. The sheet piling is usually either of the Wakefield or the splined type, supported between the wales in a vertical position and secured to the wales with bolts. The plane of the sheeting is vertical. Although usually vertical, the piling may be driven at an angle in this plane.

452. <u>Asphaltic Groins.</u> - A number of attempts have been made to use asphaltic materials for groins or jetty construction. Although there have been some notable instances where the asphalt has served the desired purpose reasonably well, the experience at other locations would indicate very indifferent success. Accordingly, it is believed the use of asphaltic materials should be given consideration only if large economic advantages appear to justify the risk involved. One type of asphaltic groin that has appeared to afford some evidence of success, has an apron built around it to prevent backwash from undermining the



TYPICAL STEEL SHEET PILE GROIN

FIGURE 141

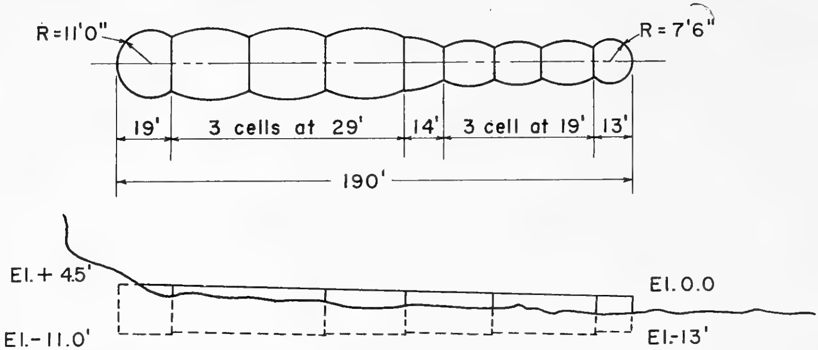


FIGURE 142 -TYPICAL CELLULAR STEEL GROIN

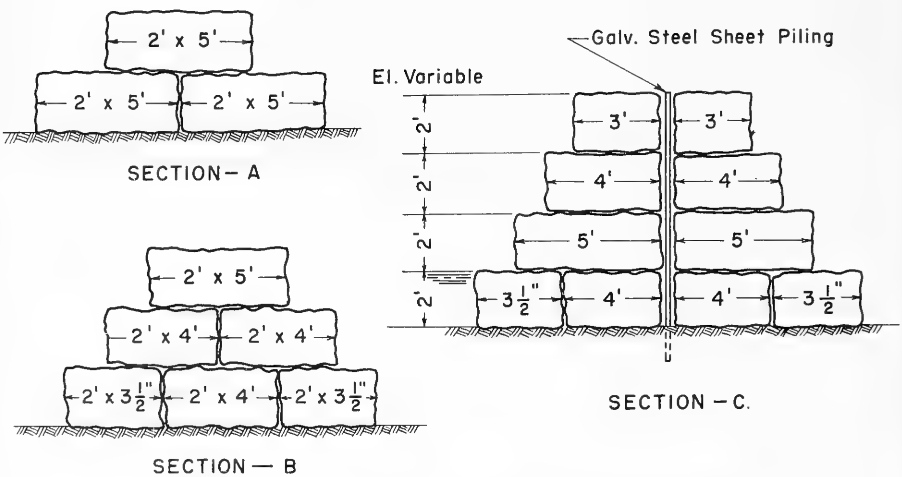
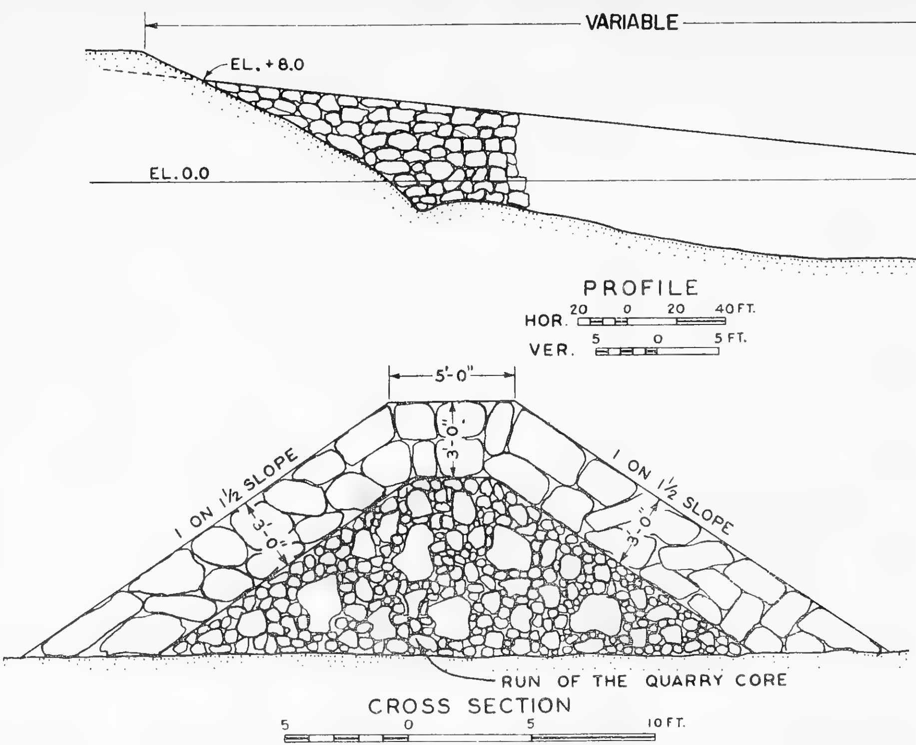


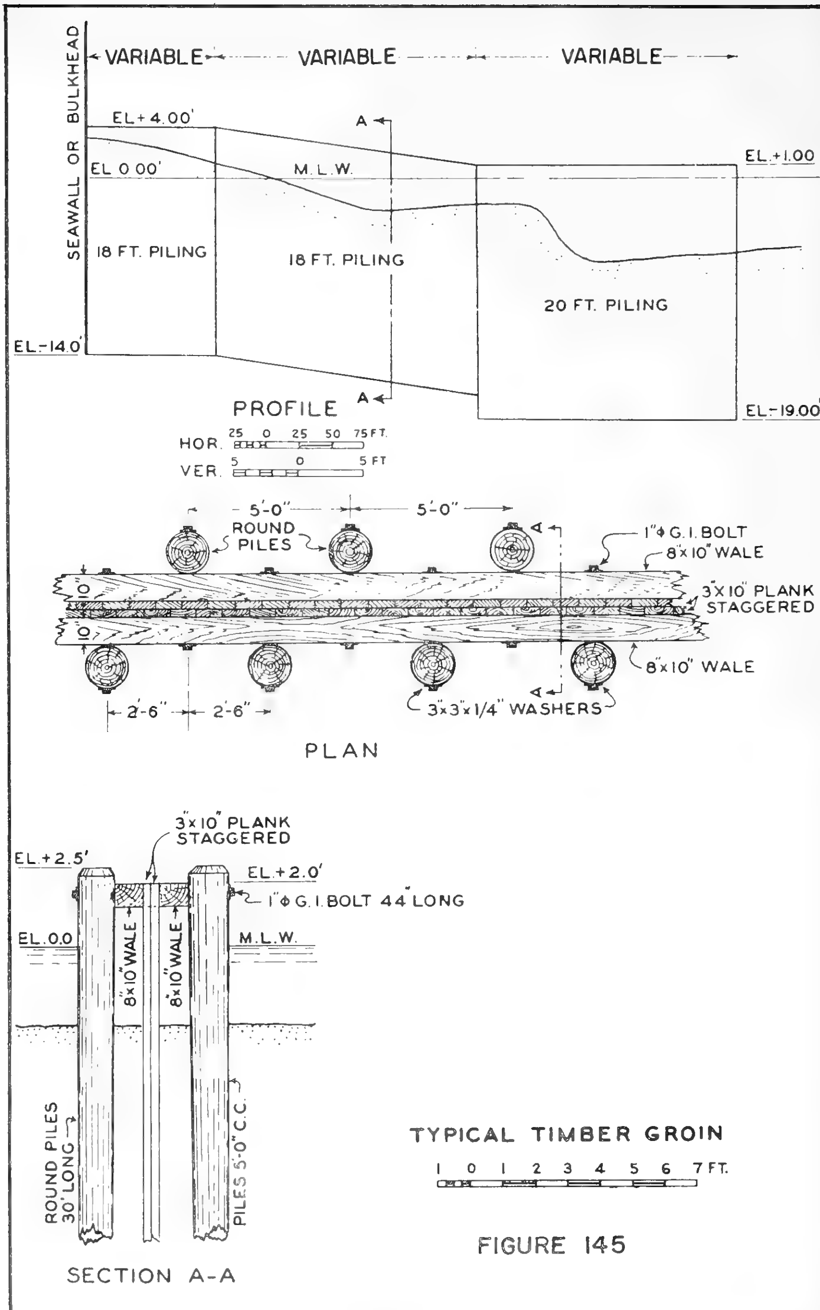
FIGURE 144 - TYPICAL STONE BLOCK GROINS



TYPICAL STONE GROIN

SCALES AS SHOWN

FIGURE 143



TYPICAL TIMBER GROIN

FIGURE 145

structure until sand has built up around the groin itself. Creosoted timber piles eight to ten feet long are driven on -8 feet centers along the centerline of the groin starting at high water line and extending to low water line with about four feet extending above the surface of the sand. Hot sand-asphalt mix is placed around the piles in a straight line to a height of about 4 feet, the base of the groin being about six feet wide. The apron, 15 feet wide and 2 inches thick, is constructed along both sides of the groin and around the seaward end. The asphalt is placed at a temperature between 250° and 300° F and tamped while hot. The aprons on the sides are rolled with a small hand roller. A typical section of this type of groin is shown in Figure 146.

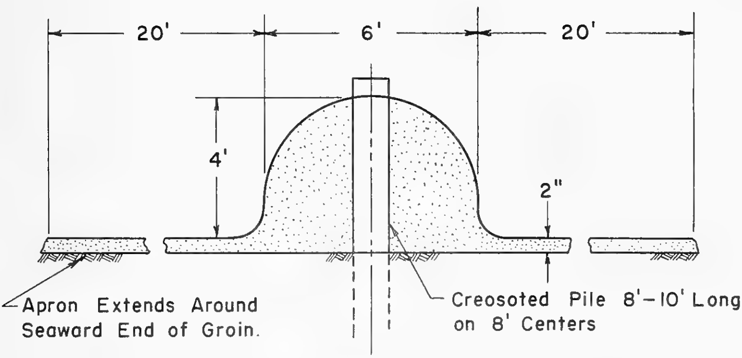


FIGURE 146 - ASPHALT GROIN USING LOCAL BEACH SAND

453. Selection of Type. - Every beach has its inherent physical characteristics. These include the range of tide, littoral drift, beach composition, wave characteristics, tidal currents, marine borers, and beach dimensions and slope. Other factors to be considered include beach and upland topography, the character of the foundation, the availability and cost of construction materials, maintenance costs, economic life, and the use made of the upland areas. After planning considerations have indicated that use of groins is practicable, the selection of groin type is affected in overlapping and varying degrees by the foregoing inter-related factors.

454. With consideration to beach and upland topography, a steep beach is an indication of heavy wave action and generally requires substantial structures, usually of concrete, stone, or steel. Under these conditions light timber groins will not be adequate seaward of the crest of berm. However, timber can be used for the inshore section above the crest of berm if the beach is backed by sandy areas such as sand dunes. On gently sloping beaches, lighter construction of timber, steel, or concrete generally can be used except in the vicinity of inlets, the cheaper construction being justified on the basis of its economic life. A thorough consideration of foundation materials is essential to the selection of groin type. Borings and probings should be taken to determine the subsurface conditions for penetration of piling. Where the foundations are poor or where little penetration is possible, some type of cellular or stone groin may be indicated despite its greater cost. Good penetration may indicate the economy of sheet piling, provided materials are available.

455. Availability of materials affects selection of type because of the economic aspects. It may happen that the material, which would normally be the most economic with full consideration given to life of the material and maintenance costs, is not available except at a cost that would make some other material or type of construction more economic. This involves the question of the economic life of the material together with the annual cost of maintenance to attain that economic life. The first costs of timber groins and of steel sheet pile groins, in that order, are nearly always less than for other types of construction. The concrete groin is considerably more expensive, but usually costs less than does the massive stone groin. However, concrete and stone groins require less maintenance and have much longer economic life than do the timber or steel sheet pile groins. With this in mind, and the amount of funds available for initial construction, the period during which protection will be required must be studied before deciding on a particular type.

456. No universal plan of protection can be prescribed because of the wide variation in conditions at each location. However, the information required in a problem and the general factors to be considered to determine the groin type are generally the same and depend on the location and characteristics of the area for detail. The groins constructed at Coney Island, New York, are used as a typical example to demonstrate the selection of type. These groins were designed to retain the artificially placed protective beach. Profiles taken before fill or groin construction was started showed a general slope of about 1 on 24, which would be considered steep and ordinarily require heavy stone structures. However, good penetration could be obtained throughout the area, and because the fill would be added as the groin construction progressed, providing protection from heavy waves to the landward ends of the groins, timber impermeable groins 160 feet long, heavily creosoted to resist attack by marine borers, were chosen for the inshore section. Seaward of this timber section, the groins were constructed of stone for a

distance of 200 feet to withstand heavy waves and prevent scour or damage to the timber sections. The stone was transported to the site by barges and lighters from quarries at Rockport, Massachusetts. When the work was finished the beach between high and low water had a moderate slope and extended about 320 feet seaward of the original shore line. The experience with this beach and the groins was satisfactory over a period of 30 years, during which several hurricanes occurred.

457. Forces Acting on Groins. - The forces acting on a groin include earth pressure and wave action. Earth pressures are determined from the differential ground elevations as developed in Part I. Wave pressures are determined from the application of the design wave to the structure. Determination of the characteristics of the design wave is discussed in the section on wave heights.

STRUCTURAL DESIGN

458. Vertical Sheet Pile Groins. - This type of groin may be constructed of timber, concrete, or steel, depending on life expectancy, cost, availability of materials, etc. The sample design included herein is for a steel structure. Similar methods would be used for either timber groins or concrete sheet pile groins. The conditions assumed represent about the maximum that a groin would be required to withstand.

459. Loading Conditions. - The most severe loading conditions would occur when the water level was at extreme low, and under the assumption that the fill on the updrift side of the groin was saturated with sea water (see Figure 147). With a 9-foot differential fill at the groin, the loading diagram against the piling will be as shown in Figure 147 B. On the updrift side the forces shown are active pressures. From the top of the piling to extreme low water (ground line), this active pressure increases uniformly at a rate of

$$P = wh^2 \tan^2\left[(1/2)(90 - \phi)\right] \qquad (152)$$

where for $h = 1$ foot

$w = 110$

$\phi = 25^\circ$ (Beach sand)

$\tan^2 1/2\ (90 - \phi) = 0.41$

$p = 110 \times 0.41 = 45$ pounds.

Assuming the voids in the sand saturated with sea water or $0.4 \times 64 = 26$ pounds, the weight of the beach fill per cubic foot would be $45 + 26$ or 71 pounds.

460. From extreme low water (ground line) to the bottom of the piling, the further increase in pressure per foot of depth is

$p = 64 + (110 - 0.6 \times 64)\ (0.41)$

$p = 93$ pounds

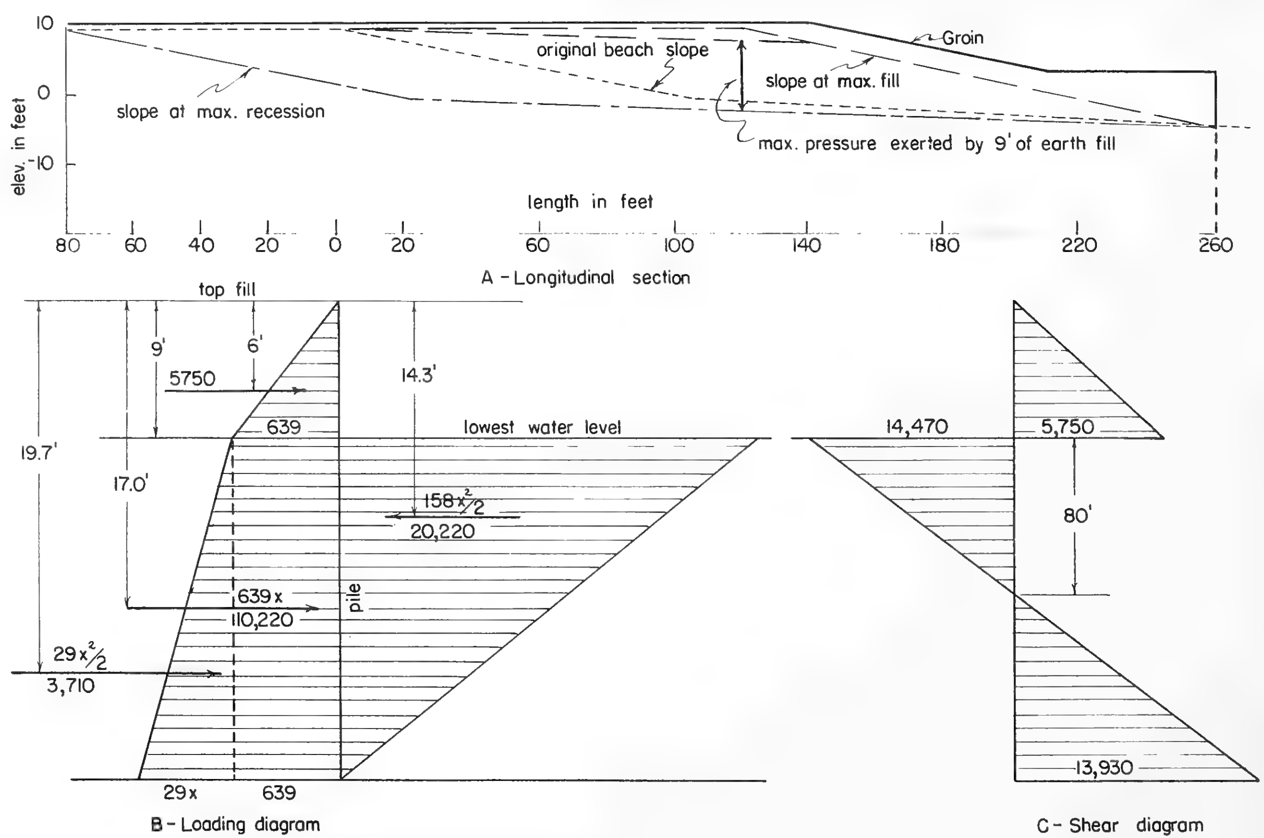


FIGURE 147- Earth Loading On Groins

As the water pressure is equal on both sides of the sheet piling, the net active increase in pressure per foot of depth would equal $93-64 = 29$ pounds. The total active pressure at the foot of the sheet piling would equal

$$9 \times 71 + 29x \text{ or } 639 + 29x$$

where x is the distance from the ground line to the bottom of the pile.

461. Passive Resistance. - The resistance to the active pressure is provided by the passive resistance of the earth on the downdrift side of the groin. Assuming the passive resistance of the piling to overturning at the bottom of the pile would be zero, it would reach a maximum near ground surface. Assuming a uniform variation, the passive resistance per foot of depth would be

$$R_p = wh^2 \tan^2 \left[(1/2)(90 + \emptyset)\right] \tag{153}$$

where $\emptyset = 25^\circ$

$$R_p = \left[110 - (0.6 \times 64)\right](1)^2 (2.46) = 158 \text{ pounds.}$$

At the ground surface the passive resistance would be 158x.

462. Length of Sheet Piling to Resist Earth Pressure. - As there are no vertical forces which act against the sheet piling, only two conditions of statics must be fulfilled:

$$\sum H = 0 \text{ and } \sum M = 0$$

From the loading diagram Figure 147B, the minimum length of pile penetration that would satisfy the condition of $\sum H = 0$ can be found by

$$5750 + 639x + \frac{29x^2}{2} = \frac{158x^2}{2}$$

Collecting terms

$$64x^2 - 639x - 5750 = 0$$

or

$$x^2 - 10x - 90 = 0$$

$$x = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a} \tag{154}$$

$$x = +\frac{10 \pm \sqrt{100 + 360}}{2}$$

$$= \frac{10 \pm 21.4}{2} = \frac{31.4}{2} = 16 \text{ feet of penetration}$$

For $\Sigma H = 0$

$$\begin{aligned} & \ 5{,}750 \\ 16 \times 639 &= 10{,}220 \\ \frac{29 \times \overline{16}^2}{2} &= 3{,}710 \\ & \overline{19{,}680} \text{ pounds} \end{aligned}$$

$$\frac{158 \times \overline{16}^2}{2} = 20{,}220 \text{ pounds}$$

which is close on the safe side.

For $\Sigma M = 0$ Taking moments around the top of the pile:

$$6 \times 5750 = 34{,}500$$

$$17 \times 10{,}220 = 173{,}800 \qquad 20{,}220 \times 14.3 = 289{,}000 \text{ foot-pounds}$$

$$19.7 \times 3{,}710 = \frac{73{,}200}{281{,}500} \text{ foot-pounds}$$

which also is close enough and on the safe side. This computed pile penetration approximately equals the rule of thumb that the penetration should be twice the unsupported loaded height.

463. Determination of Pile Section. - The points of zero shear will occur at the ground line and at a point "x" below the ground line such that

$$\frac{x}{14{,}470} = \frac{16}{28{,}400}$$

$$x = 8.0 \text{ feet}$$

The maximum moment will occur at points of zero shear. At the ground line

$$M = 3 \times 5{,}750 = 17{,}250 \text{ foot-pounds}$$

At 8 feet below the ground line

$$M = (871 \times 8 \times 4) + \left(\frac{29 \times \overline{8}^2}{2} \times 5.3\right) - \left(\frac{158 \times \overline{8}^2}{2} \times 2.7\right)$$

$$= 27{,}773 + 4918 - 13{,}651 = 19{,}040 \text{ foot-pounds.}$$

This is the maximum moment.

The section modulus is

$$S_m = \frac{M}{f_s} = \frac{19,040 \times 12}{14,000} = 16.7 \text{ inches}^3 \qquad (153)$$

From a steel pile handbook, an MZ22 or MP110 will be found to suffice. A somewhat lighter section may be used when the customary practice is followed of using wales at the top of the sheet pile with secondary support furnished by round timber pile driven outside the wales. Timber sections or concrete pile sections can similarly be selected from respective handbooks. In all probability a timber section would not be designed for a differential loading as high as 9 feet, but could be used for lighter loadings.

464. Wave Pressures. - A groin must also resist wave action during periods of storm waves. The maximum wave force would occur immediately following construction at the point where the unsupported height of the pile would be a maximum. However, at this point there would be no earth loading to resist. As the earth pressure increased, the length of pile which could be acted upon by wave action would decrease. Further, during brief periods when the wave action might strike from the downdrift side, the pile would be supported by the fill on the updrift side of the groin. Accordingly, maximum wave action and maximum earth pressure would not apply simultaneously, but the pile should be designed either for one or for the other.

465. A wave will generally break in water ranging from 1.1 to 1.5 times the height of the wave. Assuming a 6-foot tide, the maximum wave that could break against the structure would be at about station 1 + 60 where the groin would extend from 2 feet below to 9 feet above mean low water. Using a breaking depth of 1.3h with a 6-foot tide, the water depth of 8 feet would permit a maximum wave of 6 feet to impinge on the groin.

466. The period of the design wave would be determined from the wave study. Assuming this period has been found as 10 seconds, the length in deep water is

$$L_o = 5.12\ T^2 = 512 \text{ feet} \qquad (156)$$

and the wave length at the site may be found as a function of d/L_o. From Table 1, Appendix D, for d/L_o of $8.0/512 = 0.0156$, $d/L = 0.050$. Therefore, $L = d/0.050 = 8.0/0.050 = 160$ feet.

467. From the profile, the beach slope seaward of the groin was found to be 1 on 20. With D = to the depth of water one wave length from the seaward end of the groin

$$D = 8.0 + \frac{160}{20} = 16.0 \text{ feet}$$

for $$D/L_o = \frac{16.0}{512} = 0.031$$

$$D/L_D = 0.073 \text{ and } L_D = 219 \text{ feet}$$

468. The total wave pressure of a wave normal to a vertical wall is comprised of dynamic pressure and static pressure, in which the dynamic pressure of the breaking wave is

$$P_m = \frac{\pi g H_W}{L_D} \left[\frac{d(D + d)}{D} \right] \qquad (157)$$

$$P_m = \frac{\pi g \times 6 \times 64.2}{219} \left[\frac{8}{16} \times 24 \right] = 2140 \text{ pounds per square foot}$$

The static pressure is

$$P_s = \frac{wH}{2} = \frac{64.2 \times 6}{2} = 193 \text{ pounds per square foot.}$$

469. Figure 148 shows the wave pressure diagram for the wave breaking on the groin.

470. The maximum wave pressure P is

$$P = \left(P_m \times \frac{H}{3}\right) + P_s \left(d + \frac{H}{4}\right) \qquad (158)$$

$$= \left(2140 \times \frac{6}{3}\right) + 193 \left(8 + \frac{6}{4}\right) = 4280 + 1830$$

$$= 6110 \text{ pounds per foot of wall}$$

This force acts at an angle (α) to the structure. The angle (α) that the waves strike is determined from the refraction diagram. The maximum angle of possible wave attack should be used. See Figure 149. Assuming this angle as 30 degrees, the component of force normal to the structure per unit length of structure would be

$$P_n = \left(P_m \times \frac{H}{3}\right) \sin^2\alpha + P_s \left(d + \frac{H}{4}\right)$$

$$= 4280 \times 0.25 + 1830 = 2{,}900 \text{ pounds per linear foot of wall.}$$

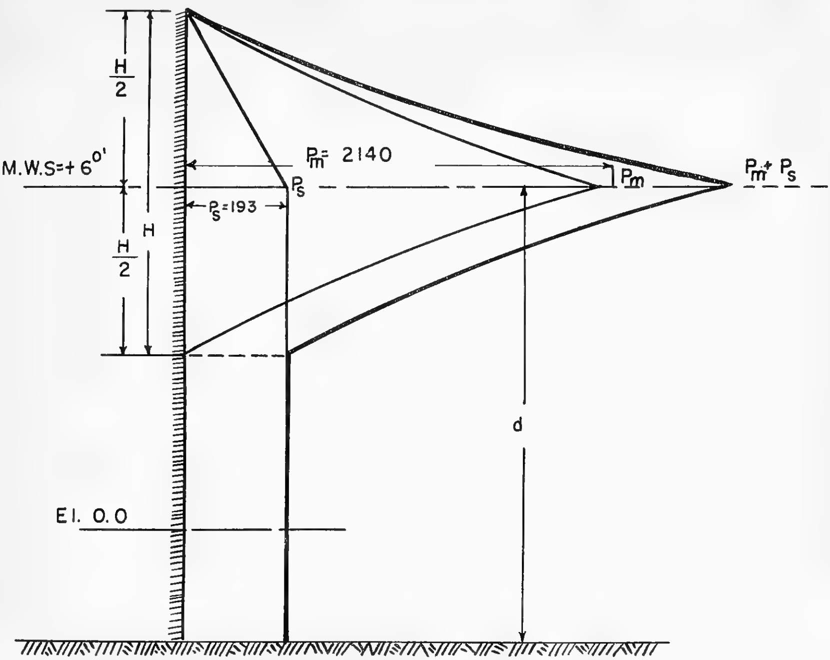


FIGURE 148 – WAVE PRESSURE DIAGRAM

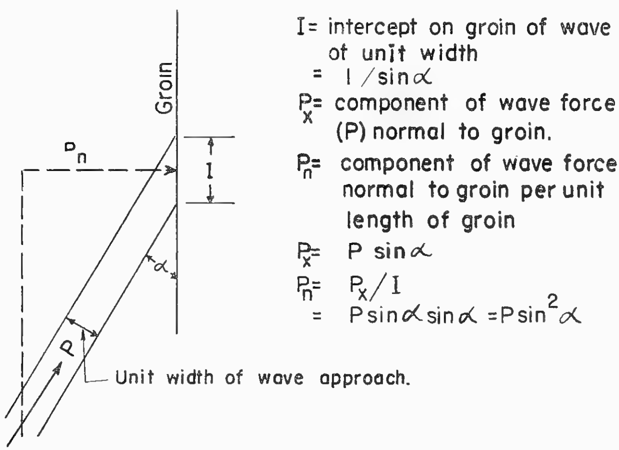


FIGURE 149.- ANGLE OF WAVE APPROACH

471. The bending moment would be

$$M = \frac{P_n \, Hd \sin^2\alpha}{3} + \frac{P_s \, d^2}{2} + \frac{P_s \, H}{4} \left(d \quad \frac{H}{6}\right) \qquad (159)$$

$$= 1070 \, d + \frac{193 \times 64}{4} + \frac{193 \times 6 \times 9}{4}$$

$$= 8{,}600 + 6{,}200 + 2{,}000 = 17{,}400 \text{ foot-pounds per foot of wall.}$$

472. As the bending moments resulting from wave action are less than those resulting from earth pressure, the sections computed in paragraphs 462 and 463 would be used.

473. Concrete and Stone Block Groins. - This type of groin is not as adaptable to building beaches as sheet pile structures or rubble-stone mounds. It is used principally to stabilize a beach in its existing position with only slight accretion expected on the updrift side. With articulated block groins or single stone block groins, the differential height of the updrift and downdrift beach lines is limited to about 5 feet. Accordingly the groin would be so placed with reference to the beach surface that sand would spill over the top of the structure before erosion had undermined it on the downdrift side.

474. Maximum Forces. - Again the forces acting on the structure would be either earth pressures or wave forces or combination thereof. No combination of these forces would exceed the maximum that would occur by application of either singly. The maximum earth pressure would occur with the groin full on one side and unsupported on the other. The maximum wave pressure would occur at the seaward end of the groin where there would be no supporting fill on either side, and where the groin would be subject to attack by the largest wave. The end block would be subject to the full impact of the waves. Landward blocks would be subject to reduced wave forces by reason of the angle of wave approach.

475. Earth Pressures. - Maximum earth pressures would occur with one side full and the other side of the groin empty. See Figure 147. For $\emptyset = 25^\circ$, $\theta = 111^\circ$, $\emptyset_1 = 22^\circ$, and $p = 0^\circ$.

$$P = 1/2 \left[\frac{\sin \theta - \emptyset}{(1 + n) \sin \theta}\right]^2 \frac{wh^2}{\sin (\emptyset_1 + \theta)} \qquad (160)$$

$$n = \sqrt{\frac{\sin (\emptyset + \theta_1) \ \sin (\emptyset - p)}{\sin (\emptyset_1 + \theta) \ \sin (\theta - p)}}$$

$$= \sqrt{\frac{.731 \times .423}{.731 \times .934}} = 0.67$$

$$P = 1/2 \left[\frac{.998}{1.67\ (.936)}\right]^2 \quad \frac{71(5)^2}{.731} \quad = 0.41\ (2430)$$

$$= 1{,}000 \text{ pounds}$$

476. The maximum moment of horizontal pressure would be:

$$M_h = 1000 \times 5/3 = 1{,}670 \text{ foot-pounds}$$

If the vertical component of earth pressure is neglected, which may be done unless it has a large effect on stability, the vertical load of the wall passes through the centerline of the base, and the above moment becomes the moment around the centerline of the base.

477. Assuming a block with a 7-foot base, a 3-foot top width, and 5 feet high, the weight per linear foot in air would be

$$5\,\frac{(7 + 3)}{2} \times 150 = 3{,}750 \text{ pounds}$$

With this block, the stress in the outer fibre would be

$$f = \frac{P}{A} \pm \frac{6M}{bd^2} = \frac{3750}{1 \times 7} \pm \frac{6(1670)}{(7)^2} = 536 \pm 205 \qquad (161)$$

$$= 536 + 205 = 741 \text{ pounds per square foot.}$$

$$= 536 - 205 = 331 \text{ pounds per square foot.}$$

These pressures are satisfactory both for concrete and for the foundations.

478. To check the possibility of sliding:

$$\frac{P}{W} = \frac{1000}{3750} = 0.27$$

This is less than the coefficient of friction for masonry on sand and is considered satisfactory.

479. Wave Pressures. - With a 5-foot high block, the maximum wave conditions would occur when the water is about 3.7 feet deep at the groin, at which time a wave 3.7/1.3 = 2.8 feet high would be possible. Using a 10-second wave and a beach slope of 1 on 20

$$\frac{d}{L_o} = \frac{3.7}{512} = .0073 \text{ and } \frac{d}{L} = .034$$

$$\therefore\ L = d/.034 = 3.7/0.034 = 109 \text{ feet}$$

$$D = 3.7 + \frac{109}{20} = 9.2 \text{ feet}$$

$$\frac{D}{L} = \frac{9.2}{512} = .018 \text{ and } \frac{D}{L_D} = .055$$

Therefore

$$L_D = 9.2/.055 = 167 \text{ feet.}$$

480. The total dynamic pressure if the groin were normal to the direction of wave approach would be determined from equation 157 or

$$P_m = \frac{\pi g\ 2.8 \times 64.2}{167} \times \frac{3.7\ (9.2 + 3.7)}{9.2}$$

$$P_m = 109 \times 5.2 = 567 \text{ pounds per square foot,}$$

and the static pressure is

$$P_s = \frac{wH}{2} = \frac{64.2 \times 2.8}{2} = 90 \text{ pounds per square foot.}$$

These pressures would be applied in a manner similar to that shown in Figure 148.

481. The total maximum wave pressure if the groin were normal to wave approach by equation (6) would be

$$P_m = (567 \times \frac{2.8}{3}) + 90\ (3.7 + \frac{2.8}{4})$$

$$= 530 + 396 = 926 \text{ pounds}$$

As the dynamic pressure acts at an angle (α) with the structure, the total force normal to the structure per unit length of structure would be, with α = 30 degrees.

$$P_n = 530 \sin^2 \alpha + 396$$

$$= 133 + 396 = 529 \text{ pounds per foot of groin.}$$

482. The moments around the base of the block by equation (159) would be

$$M = 133 \times 3.7 + \frac{90 \times \overline{3.7}^2}{2} + \frac{90 \times 2.8}{4}\ (3.7 + 4.7)$$

$$M = 492 + 616 + 528 = 1636 \text{ foot-pounds.}$$

then $\quad f = \frac{3750}{7} \pm \frac{6 \times 1630}{49}$

$$= 536 \pm 200$$

$$= 536 + 200 = 736 \text{ pounds per square foot.}$$

$$536 - 200 = 336 \text{ pounds per square foot.}$$

and $\quad \frac{P}{W} = \frac{529}{3750} = 0.14$

The assumed design is also safe for wave forces.

483. Rubble-Stone Groins. - Assume a rubble-stone groin to end in 4 feet of water with a 5-foot tide. From the wave studies, the wave with the greatest energy is a 10-second wave with a wave height of 10 feet in deep water. The wave orthogonals approach the structure at an angle of $\alpha = 11^\circ$.

484. Maximum Design Wave Height. - The highest wave that can strike the seaward end of the groin is

$$H = \frac{4 \text{ (depth of water)} + 5 \text{ (tide)}}{1.3} = 7 \text{ feet}$$

Any larger waves will break before reaching the structure and only a reformed wave or wave uprush will be propagated forward. Inspection of the orthogonal pattern indicates a uniform bottom with little divergence so that 7-foot waves can occur where the deep water design wave is 10 feet.

485. Along the sides of the groin, however, the wave energy between two adjacent orthogonals is spread out along a considerable length of groin such that with orthogonals 1 foot apart, the length along the groin would be

$$x = \frac{1}{\sin \alpha} = \frac{1}{.191} = 5.24$$

Accordingly, the wave energy impinging on the side of the groin is only 1/5 of the wave energy impinging on the end. Also, H is proportional to the $\sqrt{e}$ or the equivalent height of wave striking the side of the structure would be

$$H_e = \frac{H\sqrt{.20}}{\sqrt{1}} = 7 \times .45 = 3.2 \text{ feet.}$$

The stability of the structure is dependent on the stability of each individual stone. Because of the irregularities of the surface, the equivalent wave is able to impinge on any single stone with the same force as if the wave were normal to the structure. Accordingly, no further reduction is made.

486. Structure Design. - The rubble-stone groin is designed without regard to differential ground lines except to be certain that the shore end is carried far enough inshore to insure that the structure will not be flanked. Should one side scour, the stones in the groin will settle and readjust themselves. Should settlement prove excessive, some maintenance may be required. In some instances the width of the groin may be determined by construction methods, and in other instances may be determined by the size of capstone required.

487. Stone Sizes On Side and End Slopes. - Assume, in this case, that the specific gravity of the stone is 2.70. A 10-second wave has a deep water wave length of 512 feet. At a depth d of 9 feet, $d/L_o = 9/512 = 0.0176$ and from Table 1, Appendix D, $d/L = 0.054$. Referring to plate 12 of Appendix D, with $d/L = 0.054$ and a slope of 1 on 1.5, an extrapolated 1 on 1.5 slope curve gives for K' approximately K' = 0.03. Plate 9b Appendix D gives for W/K' under a 7 foot wave attack, $W/K' = 4 \times 10^5$, and under a 3.2 foot wave attack $W/K' = 3.7 \times 10^4$. With K' = 0.03 the stable stone weights for the 7 and 3.2 foot wave heights are approximately 6 tons and 0.55 tons respectively.

488. The stone dimensions determined for a slope of 1 on 1.5 would seem reasonable to obtain from most quarries. However, should stone as large as 6 tons not be available, a flatter end slope could be tried. To allow for ravelling and settlement at the end of the groin, the seaward 25 feet (minimum) should be constructed with stone of 6 tons minimum size on side slopes and top. Smaller stone, well graded to form sand tight core should be used inside the armor or capstone. Landward of this end section, the stone cap and slope stone should be as large as the quarry will economically produce with a minimum size of 0.55 ton. A typical groin of this type is shown in Figure 143.

APPENDIX A

GLOSSARY OF TERMS

APPENDIX A

GLOSSARY OF TERMS

ACCRETION - May be either NATURAL or ARTIFICIAL. Natural accretion is the gradual build-up of land over a long period of time solely by the action of the forces of nature, on a BEACH by deposition of water- or air-borne material. Artificial accretion is a similar build-up of land by reason of an act of man, such as the accretion formed by a groin, breakwater, or beach fill deposited by mechanical means. Also AGGRADATION.

ADVANCE (OF A BEACH) - (1) a continuing seaward movement of the shore line; (2) A net seaward movement of the shore line over a specified time. Also PROGRESSION.

AGE, WAVE - The ratio of wave velocity to wind velocity (in wave forecasting theory).

ALLUVIUM - Soil (sand, mud, or similar detrital material) deposited by flowing water, or the deposits formed thereby.

ALONGSHORE - Same as LONGSHORE.

AMPLITUDE, WAVE - (1) in hydrodynamics, one-half the wave height; (2) in engineering usage, loosely, the wave height from crest to trough.

ARTIFICIAL NOURISHMENT - The process of replenishing a beach by artificial means, e.g. by the deposition of dredged materials.

ATOLL - A ring-like "coral" island or islands encircling or nearly encircling a lagoon. It should be noted that the term "coral" island for most of these tropical islands is incorrect as calcareous algae (Lithothamnion) often forms much more than 50% of them.

ATOLL REEF - A ring-shaped, coral reef, often carrying low sand islands, enclosing a body of water.

AWASH - (1) (Nautical) Condition of an object which is nearly flush with the water level; (2) (Common usage) Condition of being tossed about or washed by waves or tide.

BACKBEACH - See BACKSHORE.

BACKRUSH - The seaward return of the water following the uprush of the waves. For any given tide stage the point of farthest return seaward of the backrush is known as the LIMIT of BACKRUSH or LIMIT of BACKWASH. (See Figure A-2)

BACKSHORE - That zone of the shore or beach lying between the foreshore and the coast line and acted upon by waves only during severe storms, especially when combined with exceptionally high water. Also BACKBEACH. (See Figure A-1)

BACKWASH - (1) See BACKRUSH; (2) Water or waves thrown back by an obstruction such as a ship, breakwater, cliff, etc.

BANK - (1) The rising ground bordering a lake, river, or sea, on a river designated as right or left as it would appear facing downstream; (2) An elevation of the sea floor of large area, surrounded by deeper water, but safe for surface navigation; a submerged plateau or shelf, a shoal, or shallow.

BAR - An offshore ridge or mound of sand, gravel, or other unconsolidated material submerged at least at high tide, especially at the mouth of a river or estuary, or lying a short distance from and usually parallel to, the beach. (See Figure A-2 and A-9)

BAR, BAYMOUTH - A bar extending partially or entirely across the mouth of a bay. (See Figure A-9)

BAR, CUSPATE - A crescent shaped bar uniting with shore at each end. It may be formed by a single spit growing from shore turning back to again meet the shore, or by two spits growing from shore uniting to form a bar of sharply cuspate form. (See Figure A-9)

BARRIER BEACH - A bar essentially parallel to the shore, the crest of which is above high water. Also OFFSHORE BARRIER. (See Figure A-9)

BARRIER REEF - A reef which roughly parallels land but is some distance offshore, with deeper water intervening.

BASIN, BOAT - A naturally or artificially enclosed or nearly enclosed body of water where small craft may lie.

BAY - A recess in the shore or an inlet of a sea or lake between two capes or headlands, not as large as a gulf but larger than a cove. See also BIGHT, EMBAYMENT. (See Figure A-9)

BAYMOUTH BAR - A bar extending partially or entirely across the mouth of a bay. (See Figure A-9)

BAYOU - A minor sluggish waterway or estuarial creek, tributary to, or connecting, other streams or bodies of water. Its course is usually through lowlands or swamps.

BEACH (n.) (1) The zone of unconsolidated material that extends landward from the low water line to the place where there is marked change in material or physiographic form....or to the line of permanent vegetation (usually the effective limit of storm waves). The seaward limit of the beach - unless otherwise specified - is the mean low water line. A beach includes FORESHORE and BACKSHORE; (2) Sometimes, the material which is in more or less active transport, alongshore or on-and-off shore, rather than the zone. (See Figure A-1)

BEACH ACCRETION - See ACCRETION.

BEACH, BARRIER - A bar essentially parallel to the shore, the crest of which is above high water level. Also OFFSHORE BARRIER.

BEACH BERM - A nearly horizontal portion of the beach or backshore formed by the deposit of material by wave action. Some beaches have no berms, others have one or several.

BEACH CUSP - One of a series of low mounds of beach material separated by crescent shaped troughs spaced at more or less regular intervals along the beach face. Also CUSP.

BEACH EROSION - The carrying away of beach materials by wave action, tidal currents, or littoral currents, or by wind.

BEACH FACE - The section of the beach normally exposed to the action of the wave uprush. The FORESHORE zone of a BEACH. (Not synonymous with SHOREFACE). (See Figure A-2)

BEACH, FEEDER - An artificially widened beach serving to nourish downdrift beaches by natural littoral currents or forces.

BEACH RIDGE - An essentially continuous mound of beach material behind the beach that has been heaped up by wave or other action. Ridges may occur singly or as a series of approximately parallel deposits. In England they are called FULLS.

BEACH SCARP - An almost vertical slope along the beach caused by erosion by wave action. It may vary in height from a few inches to several feet, depending on wave action and the nature and composition of the beach. (See Figure A-1)

BEACH WIDTH - The horizontal dimension of the beach as measured normal to the shoreline.

BENCH - (1) A level or gently sloping erosion plane inclined seaward; (2) A nearly horizontal area at about the level of maximum high water on the sea side of a dike.

BENCH MARK (B.M.) - A fixed point used as a reference for elevations.

BERM, BEACH - A nearly horizontal portion of a beach formed by the deposit of material by wave action. Some beaches have no berms, others have one or several. (See Figure A-1)

BERM CREST - The seaward limit of a berm. Also BERM EDGE. (See Figure A-1)

BIGHT - A slight indentation in the shore line of an open coast or of a bay, usually crescent shaped. (See Figure A-8)

BLIND ROLLERS - Long, high swells which have increased in height, almost to the breaking point, as they pass over shoals or run in shoaling water.

BLUFF - A high steep bank or cliff.

BOLD COAST - A prominent land mass that rises steeply from the sea.

BORE - A tidal flood with a high, abrupt front. (e.g. such as occurs in the Amazon in South America, the Hugli in India, and in the Bay of Fundy.) Also EAGER.

BOTTOM - The ground or bed under any body of water; the bottom of the sea. (See Figure A-1)

BOTTOM (NATURE OF) - The composition or character of the bed of an ocean or other body of water; (e.g. clay, coral, gravel, mud, ooze, pebbles, rock, shells, shingle, hard, or soft.)

BOULDER - A rounded rock more than 12 inches in diameter; larger than a cobble stone.

BREAKER - A wave breaking on the shore, over a reef, etc. Breakers may be (roughly) classified into three kinds although there is much overlapping:

Spilling breakers break gradually over quite a distance;
Plunging breakers tend to curl over and break with a crash; and
Surging breakers peak up, but then instead of spilling or plunging they surge up the beach face. (See Figure A-4)

BREAKER DEPTH - The still water depth at the point where the wave breaks. Also BREAKING DEPTH. (See Figure A-2)

BREAKWATER - A structure protecting a harbor, anchorage, or basin from waves.

BULKHEAD - A structure separating land and water areas, primarily designed to resist earth pressures. See also SEAWALL.

BUOY - A float; especially a floating object moored to the bottom, to mark a channel, anchor, shoal rock, etc. Some common types:

A nun or nut buoy is conical in shape;
A can buoy is squat, and cylindrical or nearly cylindrical above water and conical below water;
A spar buoy is a vertical, slender spar anchored at one end;
A bell buoy is one having a bell operated mechanically or by the action of waves, usually marking shoals or rocks;
A whistling buoy is similarly operated, marking shoals or channel entrances;
A dan buoy carries a pole with a flag or light on it.

BUOYANCY - The resultant of upward forces, exerted by the water on a submerged or floating body, equal to the weight of the water displaced by this body.

CANAL - An artificial watercourse cut through a land area for use in navigation, irrigation, etc.

CANYON - (1) (Oceanographical) A deep submarine depression of valley form with relatively steep sides; (2) (Geographical) A deep gorge or ravine with steep sides, often with a river flowing at the bottom of it.

CAPE - A relatively extensive land area jutting seaward from a continent or large island which prominently marks a change in, or interrupts notably, the coastal trend; a prominent feature. (See Figure A-8)

CAPILLARY WAVE - A wave whose velocity of propagation is controlled primarily by the surface tension of the liquid in which the wave is travelling. Water waves of length less than one inch are considered to be capillary waves.

CAUSEWAY - A raised road, across wet or marshy ground or across water.

CAUSTIC - In refraction of waves, the name given to the curve to which adjacent orthogonals of waves, refracted by a bottom whose contour lines are curved, are tangents. The occurrence of a caustic always marks a region of crossed orthogonals and high wave convergence.

CAY - See KEY.

CHANNEL - (1) A natural or artificial waterway of perceptible extent which either periodically or continuously contains moving water, or which forms a connecting link between two bodies of water; (2)

The part of a body of water deep enough to be used for navigation through an area otherwise too shallow for navigation; (3) A large strait, as the English Channel; (4) The deepest portion of a stream, bay, or strait through which the main volume or current of water flows.

CHARACTERISTIC WAVE HEIGHT - See SIGNIFICANT WAVE HEIGHT.

CHART DATUM - The plane or level to which soundings on a chart are referred, usually taken to correspond to a low water stage of the tide. See also DATUM PLANE.

CHOP - The short-crested waves that may spring up quickly in a fairly moderate breeze, and break easily at the crest. Also WIND CHOP.

CLAPOTIS - (1) The French equivalent for a type of STANDING WAVE; (2) In American usage it is usually associated with the standing wave phenomenon caused by the reflection of a wave train from a breakwater, bulkhead, or steep beach.

CLAY - See SOIL CLASSIFICATION.

CLIFF - A high, steep face of rock; a precipice. See also SEA CLIFF.

COAST - A strip of land of indefinite width (may be several miles) that extends from the seashore inland to the first major change in terrain features. (See Figure 1)

COASTAL AREA - The land and sea area bordering the shore line. (See Figure 1)

COASTAL PLAIN - The plain composed of horizontal or gently sloping strata of elastic materials fronting the coast and generally representing a strip of recently emerged sea bottom.

COAST LINE - (1) Technically, the line that forms the boundary between the COAST and the SHORE; (2) Commonly, the line that forms the boundary between the land and the water.

COBBLE (COBBLESTONE) - See SOIL CLASSIFICATION.

COMBER - (1) A deep water wave whose crest is pushed forward by a strong wind, much larger than a whitecap; (2) A long-period spilling breaker.

CONTINENTAL SHELF - The zone bordering a continent extending from the line of permanent immersion to the depth (usually about 100 fathoms) where there is a marked or rather steep descent toward the great depths.

CONTOUR - (1) A line connecting the points, on a land or submarine surface, that have the same elevation; (2) In topographic or hydrographic work, a line connecting all points of equal elevation above or below a datum plane.

CONTROLLING DEPTH - The least depth of water in the navigable parts of a waterway, which limits the allowable draft of vessels.

CONVERGENCE - (1) In refraction phenomena, the decreasing of the distance between orthogonals in the direction of wave travel. This denotes an area of increasing wave height and energy concentration; (2) In wind set-up phenomena, the increase in set-up observed over that which would occur in an equivalent rectangular basin of uniform depth, caused by changes in planform or depth; also the decrease in basin width or depth causing such increase in set-up.

CORAL - The calcareous skeletons of various anthozoans and a few hydrozoans; also these skeletons when solidified into a stony mass. Many tropical islands, reefs, and atolls are formed of coral.

COVE - A small sheltered recess in a shore or coast, often inside a larger embayment. (See Figure A-8)

CREST LENGTH, WAVE - The length of a wave along its crest. Sometimes called CREST WIDTH.

CREST OF BERM - The seaward limit of a berm. Also BERM EDGE. (See Figure A-1)

CREST OF WAVE - (1) The highest part of a wave; (2) That part of the wave above still water level. (See Figure A-3)

CREST WIDTH, WAVE - See CREST LENGTH, WAVE.

CURRENT - A flow of water.

CURRENT, COASTAL - One of the offshore currents flowing generally parallel to the shore line with a relatively uniform velocity (as compared to the littoral currents). They are not related genetically to waves and resulting surf but may be composed of currents related to distribution of mass in ocean waters (or local eddies), wind-driven currents and/or tidal currents.

CURRENT, DRIFT - A broad, shallow, slow-moving ocean or lake current.

CURRENT, EBB - The movement of the tidal current away from shore or down a tidal stream.

CURRENT, EDDY - A circular movement of water of comparatively limited area formed on the side of a main current. Eddies may be created at points where the main stream passes projecting obstructions.

CURRENT, FEEDER - The current which flows parallel to shore before converging and forming the neck of a rip current. See also RIP.

CURRENT, FLOOD - The movement of the tidal current toward the shore or up a tidal stream.

CURRENT, INSHORE - Any current inside the breaker zone.

CURRENT, LITTORAL - The nearshore currents primarily due to wave action, e.g. Longshore currents and Rip currents. See also CURRENT, NEARSHORE.

CURRENT, LONGSHORE - The inshore current moving essentially parallel to the shore, usually generated by waves breaking at an angle to the shore line.

CURRENT SYSTEM, NEARSHORE - The current system caused primarily by wave action in and near the breaker zone and which consists of four parts: the shoreward mass transport of water; longshore currents; seaward return flow, including rip currents; and the longshore movement of the expanding heads of rip currents.

CURRENT, OFFSHORE - (1) Any current in the offshore zone; (2) Any current flowing away from shore.

CURRENT, PERIODIC - A current, caused by the tide-producing forces of the moon and the sun, which is a part of the same general movement of the sea manifested in the vertical rise and fall of the tides. Also CURRENT, TIDAL.

CURRENT, PERMANENT - A current that runs continuously independent of the tides and temporary causes. Permanent currents include the fresh water discharge of a river and the currents that form the general circulatory systems of the oceans.

CURRENT, RIP - A narrow current of water flowing seaward through the breaker zone. A rip current consists of three parts: (1) The "feeder currents" flowing parallel to the shore inside the breakers; (2) The "neck" - where the feeder currents converge and flow through the breakers in a narrow band or "rip"; and (3) The "head" - where the current widens and slackens outside the breaker line. Also RIP SURF. (See Figure A-7)

CURRENT, STREAM - A narrow, deep, and fast-moving ocean current.

CURRENT, TIDAL - A current, caused by the tide-producing forces of the moon and the sun, which is a part of the same general movement of the sea manifested in the vertical rise and fall of the tides. Also CURRENT, PERIODIC. See also CURRENT, FLOOD AND CURRENT, EBB.

CUSP - One of a series of naturally formed low mounds of beach material separated by crescent-shaped troughs spaced at more or less regular intervals along the beach face. Also BEACH CUSP. (See Figure A-7)

CUSPATE BAR - A crescent-shaped bar uniting with shore at each end. It may be formed by a single spit growing from shore turning back to again meet the shore, or by two spits growing from shore uniting to form a bar of sharply cuspate form.

CYCLOIDAL WAVE - A very steep, symmetrical wave whose crest forms an angle of 120°. The wave form is that of a cycloid. A trochoidal wave of maximum steepness. See also WAVE, TROCHOIDAL.

DAILY RETARDATION (OF TIDES) - The amount of time by which corresponding tidal phases grow later day by day (averages approximately 50 minutes).

DATUM, CHART - See CHART DATUM.

DATUM PLANE - The horizontal plane to which soundings, ground elevations, or water surface elevations are referred. Also REFERENCE PLANE. The plane is called a TIDAL DATUM when defined by a certain phase of the tide. The following datums are ordinarily used on hydrographic charts:

MEAN LOW WATER - Atlantic Coast (U.S.), Argentina, Sweden, and Norway;
MEAN LOWER LOW WATER - Pacific Coast (U.S.);
MEAN LOW WATER SPRINGS - Great Britain, Germany, Italy, Brazil, and Chile.

LOW WATER DATUM - Great Lakes (U.S. and Canada);
LOWEST LOW WATER SPRINGS - Portugal;
LOW WATER INDIAN SPRINGS - India and Japan (See INDIAN TIDE PLANE)

LOWEST LOW WATER - France, Spain, and Greece.

A common datum used on topographic maps is based upon MEAN SEA LEVEL. See also BENCH MARK.

DEBRIS LINE - A line near the limit of storm wave uprush marking the seaward limit of debris deposits.

DECAY DISTANCE - The distance through which waves travel after leaving the generating area.

DECAY OF WAVES - The change that waves undergo after they leave a generating area (fetch) and pass through a calm, or region of lighter winds. In the process of decay, the significant wave height decreases and the significant wave length increases.

DEEP WATER - Water of depth such that surface waves are little affected by conditions on the ocean bottom. It is customary to consider water deeper than one-half the surface wave length as deep water.

DEFLATION - The removal of material from a beach or other land surface by wind action.

DELTA - An alluvial deposit, usually triangular, at the mouth of a river.

DEPTH - The vertical distance from the still water level (or datum as specified) to the bottom.

DEPTH OF BREAKING - The still water depth at the point where the wave breaks. Also BREAKER DEPTH.

DEPTH CONTOUR - See CONTOUR.

DEPTH, CONTROLLING - The least depth of water in the navigable parts of a waterway, which limits the allowable draft of vessels.

DEPTH FACTOR -See SHOALING COEFFICIENT.

DERRICK STONE - Stone of a sufficient size as to require handling in individual pieces by mechanical means, generally 1 ton up.

DIFFRACTION OF WATER WAVES - The phenomenon by which energy is transmitted laterally along a wave crest. When a portion of a train of waves is interrupted by a barrier such as a breakwater, the effect of diffraction is manifested by propagation of waves into the sheltered region within the barrier's geometric shadow.

DIKE (DYKE) - A wall or mound built around a low-lying area to prevent flooding.

DIURNAL - Daily, recurring once each day. (e.g. lunar day or solar day).

DIURNAL TIDE - A tide with one high water and one low water in a tidal day. (See Figure A-10)

DIVERGENCE - (1) In refraction phenomena, the spreading of orthogonals in the direction of wave travel. This denotes an area of decreasing wave height and energy concentration; (2) In wind set-up phenomena, the decrease in set-up observed under that which would occur in an equivalent rectangular basin of uniform depth, caused by changes in planform or depth. Also the increase in basin width or depth causing such decrease in set-up.

DOWNCOAST - In United States usage, the coastal direction generally trending towards the south.

DOWNDRIFT - The direction of predominant movement of littoral materials.

DRIFT (noun) - (1) The speed at which a current runs; (2) Also, floating material deposited on a beach (driftwood); (3) A deposit of a continental ice sheet, as a DRUMLIN; (4) Sometimes used as an abbreviation of LITTORAL DRIFT.

DRIFT CURRENT - A broad, shallow, slow moving ocean or lake current.

DUKW - (pronounced duck) Amphibian Truck, $2\frac{1}{2}$ ton, 6 x 6.

DUNES - Ridges or mounds of loose, wind-blown material, usually sand. (See Figure A-7)

DURATION - In wave forecasting, the length of time the wind blows in essentially the same direction over the FETCH (generating area).

DURATION, MINIMUM - The time necessary for steady state wave conditions to develop for a given wind velocity over a given fetch length.

EAGER - See BORE.

EBB CURRENT - The movement of the tidal current away from shore or down a tidal stream.

EBB TIDE - A non-technical term referring to that period of tide between a high water and the succeeding low water; falling tide. (See Figure A-10)

ECHO SOUNDER - A survey instrument that determines the depth of water by measuring the time required for a sound signal to travel to the bottom and return. It may be either "sonic" or "supersonic" depending on the frequency of sound wave, the "sonic" being generally within the audible ranges (under 15,000 cycles per sec.).

EDDY - A circular movement of water formed on the side of a main current. Eddies may be created at points where the main stream passes projecting obstructions.

EDDY CURRENT - See EDDY.

EELGRASS - A submerged marine plant with very long, narrow leaves, abundant along the North Atlantic Coast. See also KELP and SEAWEED.

EMBANKMENT - An artificial bank, mound, dike or the like, built to hold back water, carry a roadway, etc.

EMBAYED - Formed into a bay or bays, as an embayed shore.

EMBAYMENT - An indentation in a shore line forming an open bay.

ENERGY COEFFICIENT - The ratio of the energy in a wave per unit crest length transmitted forward with the wave at a point in shallow water to the energy in a wave per unit crest length transmitted forward with the wave in deep water. On refraction diagrams this is equal to the ratio of the distance between a pair of orthogonals at a selected point to the distance between the same pair of orthogonals in deep water. Also the square of the REFRACTION COEFFICIENT.

ENTRANCE - The avenue of access or opening to a navigable channel.

EROSION - The wearing away of land by the action of natural forces. (See also SCOUR). On a BEACH, by carrying away of beach material by wave action, tidal currents, or littoral currents or by the action of the wind (see DEFLATION).

ESCARPMENT - A more or less continuous line of cliffs or steep slopes facing in one general direction which are caused by erosion or faulting. Also SCARP. (See Figure A-1)

ESTUARY - (1) That portion of a stream influenced by the tide of the body of water into which it flows; (2) A bay, as the mouth of a river, where the tide meets the river current.

FAIRWAY - The parts of a waterway kept open and unobstructed for navigation.

FATHOM - A unit of measurement used for soundings. It is equal to 6 feet (1.83 meters).

FATHOMETER - The copyrighted trade name for a type of echo sounder.

FEEDER BEACH - An artificially widened beach serving to nourish downdrift beaches by natural littoral currents or forces.

FEEDER CURRENT - The current which flows parallel to shore before converging and forming the neck of a rip current. See also RIP.

FETCH - (1) In wave forecasting, the continuous area of water over which the wind blows in essentially a constant direction. Sometimes used synonymously with FETCH LENGTH. Also GENERATING AREA; (2) In wind set-up phenomena, for inclosed bodies of water, the distance between the points of maximum and minimum water surface elevations. This would usually coincide with the longest axis in the general wind direction.

FETCH LENGTH - In wave forecasting, the horizontal distance (in the direction of the wind) over which the wind blows.

FIRTH - A narrow arm of the sea; also the opening of a river into the sea.

FJORD (FIORD) - A long narrow arm of the sea between highlands.

FLOOD CURRENT - The movement of the tidal current toward the shore or up a tidal stream.

FLOOD TIDE - A non-technical term referring to that period of tide between low water and the succeeding high water; a rising tide. (See Figure A-10)

FOAM LINE - The front of a wave as it advances shoreward, after it has broken. (See Figure A-4)

FOLLOWING WIND - In wave forecasting, wind blowing in the same direction that waves are travelling.

FORESHORE - The part of the shore, lying between the crest of the seaward berm (or the upper limit of wave wash at high tide) and the ordinary low water mark, that is ordinarily traversed by the uprush and backrush of the waves as the tides rise and fall. (See Figure 1)

FREEBOARD - The additional height of a structure above design high water level to prevent overflow. Also, at a given time the vertical distance between the water level and the top of the structure. On a ship, the distance from the water line to main deck or gunwale.

FRESHET - A rapidly rising flood in a stream resulting from snow melt or rainfall.

FRINGING REEF - A reef attached to an insular or continental shore.

GENERATING AREA - In wave forecasting, the continuous area of water surface over which the wind blows in essentially a constant direction. Sometime used synonymously with FETCH LENGTH. Also FETCH.

GENERATION OF WAVES - (1) the creation of waves by natural or mechanical means; (2) In wave forecasting, the creation and growth of waves caused by a wind blowing over a water surface for a certain period of time. The area involved is called the GENERATING AREA or FETCH.

GEOMETRIC MEAN DIAMETER - The diameter equivalent of the arithmetic mean of the logarithmic frequency distribution. In the analysis of beach sands it is taken as that grain diameter determined graphically by the intersection of a straight line through selected boundary sizes (generally points on the distribution curve where 16 and 84 percent of the sample by weight is coarser) and a vertical line through the median diameter of the sample.

GEOMETRIC SHADOW - In wave diffraction theory, the area outlined by drawing straight lines paralleling the direction of wave approach through the extremities of the protective structure. It differs from the actual protected area to the extent that the diffraction and refraction effects modify the wave pattern.

GEOMORPHOLOGY - That branch of both physiography and geology which deals with the form of the earth, the general configuration of its surface, and the changes that take place in the evolution of land forms.

GRADIENT (GRADE) - See SLOPE. With reference to winds or current, the rate of increase or decrease in speed, usually in the vertical, or the curve which represents this rate.

GRAVEL - See SOIL CLASSIFICATION.

GRAVITY WAVE - A wave whose velocity of propagation is controlled primarily by gravity. Water waves of a length greater than 2 inches are considered gravity waves.

GROIN (BRIT. GROYNE) - A shore protective structure (built usually perpendicular to the shore line) to trap littoral drift or retard erosion of the shore. It is narrow in width (measured parallel to the shore line), and its length may vary from less than one hundred to several hundred feet (extending from a point landward of the shore line out into the water). Groins may be classified as permeable or impermeable; impermeable groins having a solid or nearly solid structure, permeable groins having openings through them of sufficient size to permit passage of appreciable quantities of littoral drift.

GROUND SWELL - A long high ocean swell. Also this swell as it rises to prominent height in shallow water. Not usually so high or dangerous as BLIND ROLLERS.

GROUND WATER - Subsurface water occupying the zone of saturation. In a strict sense the term is applied only to water below the WATER TABLE.

GROUP VELOCITY - The velocity at which a wave group travels. In deep water, it is equal to one-half the velocity of the individual waves within the group.

GULF - A relatively large portion of sea, partly enclosed by land.

GUT - (1) A narrow passage such as a strait or inlet.
(2) A channel in otherwise shallower water, generally formed by water in motion.

HARBOR (BRIT. HARBOUR) - A protected part of a sea, lake, or other body of water used by vessels as a place of safety and/or for the transfer of passengers and cargo between water and land carriers. See also PORT.

HEAD (HEADLAND) - A point or portion of land jutting out into the sea, a lake, or other body of water; a cape or promontory; now, usually specifically, a promontory especially bold and cliff-like.

HEAD OF RIP - The section of a rip current that has widened out seaward of the breakers. See also CURRENT, RIP: CURRENT, FEEDER: and NECK (RIP).

HEIGHT OF WAVE - The vertical distance between a crest and the preceding trough. (See Figure A-3) See also SIGNIFICANT WAVE HEIGHT.

HIGH TIDE: HIGH WATER (HW) - The maximum height reached by each rising tide. See TIDE. (See Figure A-10)

HIGH WATER OF ORDINARY SPRING TIDES (HWOST) - A tidal datum appearing in some British publications, based on high water of ordinary spring tides.

HIGHER HIGH WATER (HHW) - The higher of the two high waters of any tidal day. The single high water occurring daily during periods when the tide is diurnal is considered to be a higher high water. (See Figure A-10)

HIGHER LOW WATER (HLW) - The higher of two low waters of any tidal day. (See Figure A-10)

HIGH WATER - See HIGH TIDE.

HIGH WATER LINE - In strictness, the intersection of the plane of mean high water with the shore. The shore line delineated on the nautical charts of the Coast and Geodetic Survey is an approximation of the mean high water line.

HINDCASTING, WAVE - The calculation from historic synoptic wind charts of the wave characteristics that probably occurred at some past time.

HINTERLAND - The region inland from the coast.

HOOK - A spit or narrow cape, turned landward at the outer end, resembling a hook in form.

HYDRAULIC JUMP - In fluid flow, a change in flow conditions accompanied by a stationary, abrupt turbulent rise in water level in the direction of flow. A type of STATIONARY WAVE.

HYDROGRAPHY - (1) A configuration of an underwater surface including its relief, bottom materials, coastal structures, etc. and
(2) The description and study of sea, lakes, rivers, and other waters.

IMPERMEABLE GROIN - See under GROIN.

INDIAN SPRING LOW WATER - The approximate level of the mean of lower low waters at spring tides, used principally in the Indian Ocean and along the east coast of Asia. Also INDIAN TIDE PLANE.

INDIAN TIDE PLANE - The datum of INDIAN SPRING LOW WATER.

INLET - A short, narrow waterway connecting a bay, lagoon, or similar body of water with a large parent body of water. An arm of the sea (or other body of water), that is long compared to its width, and that may extend a considerable distance inland. See also TIDAL INLET.

INSHORE (ZONE) - In beach terminology, the zone of variable width extending from the shore face through the breaker zone. (See Figure 1)

INSHORE CURRENT - Any current in or landward of the breaker zone.

INSULAR SHELF - The zone surrounding an island extending from the line of permanent immersion to the depth (usually about 100 fathoms) where there is a marked or rather steep descent toward the great depths.

INTERNAL WAVES - Waves that occur within a fluid whose density changes with depth, either abruptly at a sharp surface of discontinuity (an interface) or gradually. Their amplitude is greatest at the density discontinuity or, in the case of a gradual density change, somewherein the interior of the fluid and not at the free upper surface where the surface waves have their maximum amplitude.

ISTHMUS - A narrow strip of land, bordered on both sides by water, that connects two larger bodies of land.

JETTY - (1) (U. S. usage) On open seacoasts, a structure extending into a body of water, and designed to prevent shoaling of a channel by littoral materials, and to direct and confine the stream or tidal flow. Jetties are built at the mouth of a river or tidal inlet to help deepen and stabilize a channel. (2) (British usage) Jetty is synonymous with "wharf" or "pier".

KELP - The general name for several species of large seaweeds. A mass or growth of large seaweed or any of various large brown seaweeds.

KEY - A low insular bank of sand, coral, etc., as one of the islets off the southern coast of Florida, Also CAY.

KINETIC ENERGY (OF WAVES) - In a progressive oscillatory wave, a summation of the energy of motion of the particles within the wave. This energy does not advance with the wave form.

KNOLL - (1) A submerged elevation of rounded shape rising from the ocean floor, but less prominent than a seamount. (2) A small rounded hill.

KNOT - (Abbreviation kt. or kts.) The unit of speed used in navigation. It is equal to 1 nautical mile (6,080.20 feet) per hour.

LAGGING - See DAILY RETARDATION (OF TIDES)

LAGOON - A shallow body of water, as a pond or lake, which usually has a shallow, restricted outlet to the sea. (See Figures A-8 and A-9).

LAND BREEZE - A light wind blowing from the land caused by unequal cooling of land and water masses.

LAND-SEA BREEZE - The combination of a land breeze and a sea breeze as a diurnal phenomenon.

LANDLOCKED - An area of water enclosed, or nearly enclosed, by land, as a bay, a harbor, etc. (thus, protected from the sea).

LANDMARK - A conspicuous object - natural or artificial - located near or on land which aids in fixing the position of an observer.

LEADLINE - A line, wire, or cord used in sounding. It is weighted at one end with a plummet (sounding lead). Also SOUNDING LINE.

LEE - (1) Shelter, or the part or side sheltered or turned away from the wind or waves. (2) (Chiefly nautical) The quarter or region toward which the wind blows.

LEEWARD - The direction toward which the wind is blowing; the direction toward which waves are travelling.

LENGTH OF WAVE - The horizontal distance between similar points on two successive waves measured perpendicularly to the crest. (See Figure A-3).

LEVEE - A dike or embankment for the protection of land from inundation.

LIMIT OF BACKRUSH) See BACKWASH.
LIMIT OF BACKWASH

LITTORAL - Of or pertaining to a shore, especially of the sea. A coastal region.

LITTORAL CURRENT - The current near the shore of an ocean or other body of water.

LITTORAL DEPOSITS - Deposits of littoral drift.

LITTORAL DRIFT - The material moved in the littoral zone under the influence of waves and currents.

LITTORAL TRANSPORT - The movement of material along the shore in the littoral zone by waves and currents.

LONGSHORE CURRENT - A current in the surf zone moving essentially parallel to the shore, usually generated by waves breaking at an angle to the shore line.

LOWER HIGH WATER (LHW) - The lower of the two high waters of any tidal day. (See Figure A-10).

LOWER LOW WATER (LLW) - The lower of the two low waters of any tidal day. The single low water occurring daily during periods when the tide is diurnal is considered to be a lower low water. (See Figure A-10).

LOW TIDE (LOW WATER, LW) - The minimum height reached by each falling tide. See TIDE. (See Figure A-10)

LOW WATER DATUM - An approximation to the plane of mean low water that has been adopted as a standard reference plane. See also DATUM PLANE.

LOW WATER LINE - The intersection of any standard low tide datum plane with the shore.

LOW WATER OF ORDINARY SPRING TIDES (LWOST) - A tidal datum appearing in some British publications, based on low water of ordinary spring tides.

MANGROVE - A particular kind of tropical tree or shrub with thickly matted roots, confined to low-lying brackish areas.

MARIGRAM - A graphic record of the rise and fall of the tide.

MARSH - A tract of soft, wet or periodically inundated land, generally treeless and usually characterized by grasses and other low growth.

MARSH, SALT - A marsh periodically flooded by salt water.

MASS TRANSPORT - The net transfer of water by wave action in the direction of wave travel. See under ORBIT.

MEAN DIAMETER, GEOMETRIC - The diameter equivalent of the arthmetic mean of the logarithmic frequency distribution. In the analysis of beach sands it is taken as that grain diameter determined graphically by the intersection of a straight line through selected boundary sizes (generally points on the distribution curve where 16 and 84 percent of the sample by weight is coarser) and a vertical line through the median diameter of the sample.

MEAN HIGHER HIGH WATER (MHHW) - The average height of the higher high waters over a 19-year period. For shorter periods of observation, corrections are applied to eliminate known variations and reduce the result to the equivalent of a mean 19 year value.

MEAN HIGH WATER (MHW) - The average height of the high waters over a 19-year period. For shorter periods of observations, corrections are applied to eliminate known variations and reduce the result to the equivalent of a mean 19-year value.
All high water heights are included in the average where the type of tide is either semidiurnal or mixed. Only the higher high water heights are included in the average where the type of tide is diurnal. So determined, mean high water in the latter case is the same as mean higher high water.

MEAN HIGH WATER SPRINGS - The average height of the high waters occurring at the time of spring tide. Frequently abbreviated to High Water Springs.

MEAN LOWER LOW WATER (MLLW) - Frequently abbreviated lower low water. The average height of the lower low waters over a 19-year period. For shorter periods of observations, corrections are applied to eliminate known variations and reduce the result to the equivalent of a mean 19-year value.

MEAN LOW WATER (MLW) - The average height of the low waters over a 19-year period. For shorter periods of observations, corrections are applied to eliminate known variations and reduce the result to the equivalent of a mean 19-year value.
All low water heights are included in the average where the type of tide is either semidiurnal or mixed. Only the lower low water heights are included in the average where the type of tide is diurnal. So determined, mean low water in the latter case is the same as mean lower low water.

MEAN LOW WATER SPRINGS - Frequently abbreviated low water springs. The average height of low waters occurring at the time of the spring tides. It is usually derived by taking a plane depressed below the half-tide level by an amount equal to one-half the spring range of tide, necessary corrections being applied to reduce the result to a mean value. This plane is used to a considerable extent for hydrographic work outside of the United States and is the plane of reference for the Pacific approaches to the Panama Canal.

MEAN SEA LEVEL - The average height of the surface of the sea for all stages of the tide over a 19-year period, usually determined from hourly height readings. See also SEA LEVEL DATUM.

MEAN TIDE LEVEL - Also called half-tide level. A plane midway between mean high water and mean low water.

MEDIAN DIAMETER - The diameter which marks the division of a given sample into two equal parts by weight, one part containing all grains larger than that diameter and the other part containing all grains smaller.

MINIMUM DURATION - The time necessary for steady state wave conditions to develop for a given wind velocity over a given fetch length.

MIXED TIDE - A type of tide in which the presence of a diurnal wave is conspicuous by a large inequality in either the high or low water heights with two high waters and two low waters usually occurring each tidal day. In strictness all tides are mixed but the name is usually applied without definite limits to the tides intermediate to those predominantly semidiurnal and those predominantly diurnal. (See Figure A-10).

MOLE - In coastal terminology, a massive solid-fill structure of earth, (generally revetted), masonry, or large stone. It may serve as a breakwater or pier.

MONOLITHIC - Like a single stone or block. Therefore in (say) breakwaters, the type of construction in which the structure's component parts are bound together to act as one.

MUD - A fluid-to-plastic mixture of finely divided particles of solid material and water.

NAUTICAL MILE - The length of a minute of arc, 1/21,600 of an average great circle of the earth. Generally one minute of latitude is considered equal to one nautical mile. The accepted United States value is 6,080.20 feet., approximately 1.15 times as long as the statute mile of 5,280 feet. Also GEOGRAPHICAL MILE.

NEAP TIDE - A tide occurring near the time of quadrature of the moon. The neap tidal range is usually 10 to 30 percent less than the mean tidal range.

NEARSHORE (ZONE) - In beach terminology an indefinite zone extending seaward from the shore line somewhat beyond the breaker zone. It defines the area of NEARSHORE CURRENTS. The SHOREFACE. (See Figure 1)

NEARSHORE CIRCULATION - The ocean circulation pattern composed of the NEARSHORE CURRENTS and COASTAL CURRENTS. See under CURRENT.

NEARSHORE CURRENT SYSTEM - The current system caused primarily by wave action in and near the breaker zone, and which consists of four parts: the shoreward mass transport of water; long-shore currents; seaward return flow, including rip currents; and the longshore movement of the expanding heads of rip currents.

NECK - The narrow band of water flowing seaward through the surf. Also RIP.

NIP - The cut made by waves in a shore line of emergence.

NODAL ZONE - An area at which the predominant direction of the littoral drift changes.

NOURISHMENT - The process of replenishing a beach. It may be brought about by natural means, e.g. littoral drift, or by artificial means, e.g. by the deposition of dredged materials.

OCEANOGRAPHY - That science treating of the oceans, their forms, physical features, and phenomena.

OFFSHORE (n. or adj.) - (1) In beach terminology, the comparatively flat zone of variable width, extending from the breaker zone to the seaward edge of the continental shelf. (2) A direction seaward from the shore (See Figure 1).

OFFSHORE CURRENT - (1) Any current in the offshore zone. (2) Any current flowing away from shore.

OFFSHORE WIND - A wind blowing seaward from the land in the coastal area.

ONSHORE - A direction landward from the sea.

ONSHORE WIND - A wind blowing landward from the sea in the coastal area.

OPPOSING WIND - In wave forecasting, a wind blowing in the opposite direction to that in which the waves are travelling.

ORBIT - In water waves, the path of a water particle affected by the wave motion. In deep water waves the orbit is nearly circular and in shallow water waves the orbit is nearly elliptical. In general, the orbits are slightly open in the direction of wave motion giving rise to MASS TRANSPORT. (See Figure A-3)

ORBITAL CURRENT - The flow of water accompanying the orbital movement of the water particles in a wave. Not to be confused with wave-generated LITTORAL CURRENTS. (SeeFigure A-3)

ORTHOGONAL - On a refraction diagram, a line drawn perpendicular to the wave crests. (See Figure A-6).

OSCILLATION - A periodic motion to and fro, or up and down.

OSCILLATORY WAVE - A wave in which each individual particle oscillates about a point with little or no permanent change in position. The term is commonly applied to progressive oscillatory waves in which only the form advances, the individual particles moving in closed or nearly closed orbits. Distinguished from a WAVE of TRANSLATION. See Also ORBIT.

OUTFALL - (1) The vent of a river, drain, etc. (2) A structure extending into a body of water for the purpose of discharging sewage, storm runoff, or cooling water.

OVERWASH - That portion of the uprush that carries over the crest of a berm or of a structure.

PARAPET - A low wall built along the edge of a structure as on a seawall or quay.

PASS - In hydrographic usage a navigable channel, through a bar, reef, or shoal, or between closely adjacent islands.

PARTICLE VELOCITY - For waves, the velocity induced by wave motion with which a specific water particle moves.

PEBBLES - See SOIL CLASSIFICATION.

PENINSULA - An elongated portion of land nearly surrounded by water, and connected to a larger body of land.

PERIODIC CURRENT - A current caused by the tide-producing forces of the moon and the sun, a part of the same general movement of the sea that is manifested in the vertical rise and fall of the tides. See also CURRENT, FLOOD AND CURRENT, EBB.

PERMAFROST - Permanently frozen subsoil.

PERMANENT CURRENTS - A current that runs continuously independent of the tides and temporary causes. Permenent currents include the fresh water discharge of a river and the currents that form the general circulatory systems of the oceans.

PERMEABLE GROIN - See under GROIN.

PETROGRAPHY - The description and systematic classification of rocks.

PIER - A structure, extending out into the water from the shore, to serve as a landing place, a recreational facility, etc., rather than to afford coastal protection.

PILE - A long, slender piece of wood, concrete, or metal to be driven or jetted into the earth or sea bed to serve as a support or protection.

PILING - A group of piles.

PILE, SHEET - A pile with a generally flat cross-section to be driven into the ground or sea bed and meshed or interlocked with like members to form a diaphragm, wall, or bulkhead.

PINNACLE - A tall, slender, pointed, rocky mass. See also REEF PINNACLE.

PLAIN - An extent of level or nearly level land.

PLAIN, COASTAL - A plain fronting the coast and generally representing a strip of recently emerged sea bottom.

PLANFORM - The outline or shape of a body of water as determined by the still water line.

PLATEAU - An elevated plain, table land, or flat-topped region of considerable extent.

PLUNGE POINT - (See Figure A-1) (1) For a plunging wave, the point at which the wave curls over and falls; (2) The final beraking point of the waves just before they rush up on the beach.

PLUNGING BREAKER - See under BREAKER.

POINT - The extreme end of a cape; or the outer end of any land area protruding into the water, usually less prominent than a cape.

PORT - A place where vessels may discharge or receive cargo; may be the entire harbor including its approaches and anchorages or may be the commercial part of a harbor, where the quays, wharves, facilities for transfer of cargo, docks, repair shops, etc. are situated.

POTENTIAL ENERGY OF WAVES - In a progressive oscillatory wave, the energy resulting from the elevation or depression of the water surface from the undisturbed level. This energy advances with the wave form.

PROFILE, BEACH - The intersection of the ground surface with a vertical plane; may extend from the top of the dune line to the seaward limit of sand movement. (See Figure A-1)

PROGRESSION - See ADVANCE.

PROGRESSIVE WAVE - A wave which is manifested by the progressive movement of the wave form.

PROMONTORY - A high point of land projecting into a body of water; a headland.

PROPAGATION OF WAVES - The transmission of waves through water.

PROTOTYPE - In laboratory usage, the original structure, concept, or phenomenon used as a basis for constructing a scale model or copy.

QUAY (pronounced KEY) - A stretch of paved bank, or a solid artificial landing place parallel to the navigable waterway, for use in loading and unloading vessels.

QUICKSAND - Loose, yielding, wet sand which offers no support to heavy objects. The upward flow of the water has a velocity that eliminates contact pressures between the sand grains, and causes the sand-water mass to behave like a fluid.

RECESSION (OF A BEACH) - (1) A continuing landward movement of the shore line. (2) A net landward movement of the shore line over a specified time. Also RETROGRESSION.

REEF - A chain or range of rock or coral, elevated above the surrounding bottom of the sea, generally submerged and dangerous to surface navigation.

REEF, ATOLL - A ring-shaped, coral reef, often carrying low sand islands, enclosing a body of water.

REEF, BARRIER - A reef which roughly parallels land but is some distance offshore, with deeper water intervening.

REEF, FRINGING - A reef attached to an insular or continental shore.

REEF, SAND - Synonymous with BAR.

REFERENCE PLANE - See DATUM PLANE.

REFERENCE POINT - A specified location (in plan and/or elevation) to which measurements are referred.

REFERENCE STATION - A station for which tidal constants have previously been determined and which is used as a standard for the comparison of simultaneous observations at a second station; also a station for which independent daily predictions are given in the tide or current tables from which corresponding predictions are obtained for other stations by means of differences or factors.

REFLECTED WAVE - The wave that is returned seaward when a wave impinges upon a very steep beach, barrier, or other reflecting surface.

REFRACTION OF WATER WAVES - (1) The process by which the direction of a wave moving in shallow water at an angle to the contours is changed. The part of the wave advancing in shallower water moves more slowly than that part still advancing in deeper water, causing the wave crest to bend toward alinement with the underwater contours. (2) The bending of wave crests by currents. (See Figure A-5)

REFRACTION COEFFICIENT - The square root of the ratio of the spacing between adjacent orthogonals in deep water and in shallow water at a selected point. When multiplied by the SHOALING FACTOR, this becomes the WAVE HEIGHT COEFFICIENT or the ratio of the refracted wave height at any point to the deep water wave height. Also the square root of the ENERGY COEFFICIENT.

REFRACTION DIAGRAM - A drawing showing positions of wave crests and/or orthogonals in a given area for a specific deep water wave period and direction. (See Figure A-6).

RETARDATION - The amount of time by which corresponding tidal phases grow later day by day (averages approximately 50 minutes).

RETROGRESSION OF A BEACH - (1) A continuing landward movement of the shore line; (2) A net landward movement of the shore line over a specified time. Also RECESSION.

REVETMENT - A facing of stone, concrete, etc., built to protect a scarp, embankment or shore structure against erosion by wave action or currents.

RIA - A long narrow inlet, with depth gradually diminishing inward.

RIDE-UP - See RUN-UP.

RIDGE, BEACH - An essentially continuous mound of beach material that has been heaped up by wave or other action. Ridges may occur singly or as a series of approximately parallel deposits. (See Figure A-7) In England they are called FULLS.

RILL MARKS - Tiny drainage channels in a beach caused by the flow seaward of water left in the sands of the upper part of the beach after the retreat of the tide or after the dying down of storm waves.

RIP - A body of water made rough by waves meeting an opposing current, particularly a tidal current; often found where tidal currents are converging and sinking. A TIDE RIP.

RIPARIAN - Pertaining to the banks of a body of water.

RIPARIAN RIGHTS - The rights of a person owning land containing or bordering on a watercourse or other body of water in or to its banks, bed, or waters.

RIP CURRENT - A strong surface current of short duration flowing seaward from the shore. It usually appears as a visible band of agitated water and is the return movement of water piled up on the shore by incoming waves and wind. With the seaward movement concentrated in a limited band its velocity is somewhat accentuated. A rip consists of three parts: the FEEDER CURRENT flowing parallel to the shore inside the breakers; the NECK, where the feeder currents converge and flow through the breakers in a narrow band or "rip"; and the HEAD, where the current widens and slackens outside the breaker line. A rip current is often miscalled a RIP TIDE. Also RIP SURF. (See Figure A-7)

RIP SURF - See under RIP CURRENT

RIPPLE - (1) The ruffling of the surface of water, hence a little curling wave or undulation. (2) A wave controlled to a significant degree by both surface tension and gravity. See WAVE, CAPILLARY and WAVE, GRAVITY.

RIPPLE MARKS - Small, fairly regular ridges in the bed of a waterway or on a land surface caused by water currents or wind. As their form is approximately normal to the direction of current or wind, they indicate both the presence and the direction of currents or winds.

RIPRAP - A layer, facing, or protective mound of stones randomly placed to prevent erosion, scour, or sloughing of a structure or embankment; also the stone so used.

RISE, TIDAL - The height of tide as referred to the datum of a chart.

ROADSTEAD - (Nautical) A sheltered area of water near shore where vessels may anchor in relative safety. Also ROAD.

ROCK - (1) (Engineering) A natural aggregate of mineral particles connected by strong and permanent cohesive forces. In igneous and metamorphic rocks, it consists of interlocking crystals; in sedimentary rocks, of closely packed mineral grains, often bound together by a natural cement. Since the terms "strong" and "permanent" are subject to different interpretations, the boundary between rock and soil is necessarily an arbitrary one. (2) (Geological) The material that forms the essential part of the earth's solid crust, and includes loose incoherent masses, such as a bed of sand, gravel, clay or volcanic ash, as well as the very firm, hard, and solid masses of granite, sandstone, limestone, etc. Most rocks are aggregates of one or more minerals, but some are composed entirely of glassy matter, or of a mixture of glass and minerals.

ROLLER - An indefinite term, sometimes considered to be one of a series of long-crested, large waves which roll in upon a coast, as after a storm.

RUBBLE - (1) Loose angular water-worn stones along a beach. (2) Rough, irregular fragments of broken rock.

RUNNEL - A corrugation (trough) of the foreshore (or the bottom just offshore), formed by wave and/or tidal action. Larger than the trough between ripple marks.

RUN-UP - The rush of water up a structure on the breaking of a wave. Also UPRUSH. The amount of run-up is the vertical height above still water level that the rush of water reaches.

SALT MARSH - A marsh periodically flooded by salt water.

SAND - See SOIL CLASSIFICATION.

SAND BAR - (1) See BAR. (2) In a river, a ridge of sand built up to or near the surface by river currents.

SAND REEF - Synonymous with BAR.

SCARP - A more or less continuous line of cliffs or steep slopes facing in one general direction which are caused by erosion or faulting. Also ESCARPMENT.

SCARP, BEACH - An almost vertical slope along the beach caused by erosion by wave action. It may vary in height from a few inches to several feet, depending on wave action and the nature and composition of the beach.

SCOUR - Erosion, especially by moving water. See also EROSION.

SEA - (1) An ocean, or alternatively a large body of (usually) salt water less than an ocean; (2) Waves caused by wind at the place and time of observation; (3) State of the ocean or lake surface in regard to waves.

SEA (STATE OF) - Description of the sea surface with regard to wave action.

SEA BREEZE - (1) A breeze blowing from the sea toward the land; (2) A light wind blowing toward the land caused by unequal heating of land and water masses.

SEA CLIFF - A cliff situated at the seaward edge of the coast.

SEA LEVEL - See MEAN SEA LEVEL.

SEA MOUNT - A submarine mountain rising more than 500 fathoms above the ocean floor.

SEA PUSS - A dangerous longshore current, a rip current, caused by return flow, loosely the submerged channel or inlet through a bar caused by those currents.

SEASHORE - The SHORE of a sea or ocean.

SEAWALL - A structure separating land and water areas primarily designed to prevent erosion and other damage due to wave action. See also BULKHEAD.

SEICHE - A periodic oscillation of a body of water whose period is determined by the resonant characteristics of the containing basin as controlled by its physical dimensions. These periods generally range from a few minutes to an hour or more. (Originally the term was applied only to lakes but now also to harbors, bays, oceans, etc).

SEISMIC SEA WAVE - (TSUNAMI) - A generally long period wave caused by an underwater seismic disturbance or volcanic eruption. Commonly misnamed "tidal wave".

SEMIDIURNAL TIDES - A tide with two high and two low waters in a tidal day, with comparatively little diurnal inequality. (See Figure A-10)

SET OF CURRENT - The direction toward which a current flows.

SET-UP, WIND - (1) The vertical rise in the still water level on the leeward side of a body of water caused by wind stresses on the surface of the water; (2) The difference in still water level between the windward and the leeward sides of a body of water caused by wind stresses on the surface of the water; (3) Synonymous with WIND TIDE. WIND TIDE is usually reserved for use on the ocean and large bodies of water. WIND SET-UP is usually reserved for use on reservoirs and samll bodies of water. (See Figure A-11).

SHALLOW WATER - (1) Commonly; water of such a depth that surface waves are noticeably affected by bottom topography. It is customary to consider water of depths less than half the surface wave length as shallow water. See TRANSITIONAL WATER; (2) More strictly; in hydrodynamics, with regard to progressive gravity waves, water in which the depth is less than 1/25th the wave length. Also called VERY SHALLOW WATER.

SHEET PILE - See under PILE.

SHELF, CONTINENTAL - The zone bordering a continent extending from the line of permanent immersion to the depth (usually about 100 fathoms) where there is a marked or rather steep descent toward the great depths.

SHELF, INSULAR - The zone surrounding an island extending from the line of permanent immersion to the depth (usually about 100 fathoms) where there is a marked or rather steep descent toward the great depths.

SHINGLE - (1) Loosely and commonly; any beach material coarser than ordinary gravel, especially any having flat or flattish pebbles; (2) Strictly and accurately; beach material of smooth, well-rounded pebbles that are roughly the same size. The spaces between pebbles are not filled with finer materials. Shingle gives out a musical note when stepped on.

SHOAL (noun) - A detached elevation of the sea bottom comprised of any material except rock or coral, and which may endanger surface navigation.

SHOAL (verb) - (1) to <u>become</u> shallow gradually; (2) to <u>cause</u> to become shallow; (3) to <u>proceed</u> from a greater to a lesser depth of water.

SHOALING COEFFICIENT - The ratio of the height of a wave in water of any depth to its height in deep water with the effect of refraction eliminated. Sometimes SHOALING FACTOR or DEPTH FACTOR. See also ENERGY COEFFICIENT and REFRACTION COEFFICIENT.

SHORE - The strip of ground bordering any body of water which is ordinarily exposed, or covered by tides and/or waves. A shore of unconsolidated material is usually called a BEACH. (See Figure A-1)

SHORE FACE - The narrow zone seaward from the low tide SHORELINE permanently covered by water, over which the beach sands and gravels actively oscillate with changing wave conditions.

SHORE LINE - The intersection of a specified plane of water with the shore or beach. (e.g. the high water shore line would be the intersection of the plane of mean high water with the shore or beach). The line delineating the shore line on U. S. Coast and Geodetic Survey nautical charts and surveys approximates the mean high water line.

SIGNIFICANT WAVE - A statistical term denoting waves with the average height and period of the one-third highest waves of a given wave group. The composition of the higher waves depends upon the extent to which the lower waves are considered. Experience so far indicates that a careful observer who attempts to establish the character of the higher waves will record values which approximately fit the definition. A wave of significant wave period and significant wave height.

SIGNIFICANT WAVE HEIGHT - The average height of the one-third highest waves of a given wave group. Note that the composition of the highest waves depends upon the extent to which the lower waves are considered. In wave record analysis, the average height of the highest 1/3 of a selected number of waves, this number being determined by dividing the time of record by the significant period. Also CHARACTERISTIC WAVE HEIGHT.

SIGNIFICANT WAVE PERIOD - An arbitrary period generally taken as the period of the 1/3 highest waves within a given group. Note that the composition of the highest waves depends upon the extent to which the lower waves are considered. In wave record analysis, this is determined as the average period of the most frequently recurring of the larger well-defined waves in the record under study.

SILT - See SOIL CLASSIFICATION.

SLACK TIDE (SLACK WATER) - The state of a tidal current when its velocity is near zero, especially the moment when a reversing current changes direction and its velocity is zero. Sometimes considered the intermediate period between ebb and flood currents during which the velocity of the currents is less than 0.1 knot. See TIDAL STAND.

SLIP - A space between two piers, wharves, etc. for the berthing of vessels.

SLOPE - The degree of inclination to the horizontal. Usually expressed as a ratio, such as 1:25 or 1 on 25, indicating 1 unit rise in 25 units of horizontal distance; or in a decimal fraction (0.04); degrees (2° 18'); or percent (4%). It is sometimes described by such adjectives as; steep, moderate, gentle, mild, or flat.

SLOUGH - (pronounced - sloo): (1) A small muddy marshland or tidal waterway which usually connects other tidal areas; (2) A tide-land or bottom-land creek.

SOIL CLASSIFICATION (Size) - An arbitary division of a continuous scale of sizes such that each scale unit or grade may serve as a convenient class interval for conducting the analysis or for expressing the results of an analysis. There are many classifications used; some of those most often used are presented below:

1. Wentworth's Size Classification

Grade Limits (Diameters)	Name
Above 256 mm.	Boulder
256 - 64 mm.	Cobble
64 - 4 mm.	Pebble
4 - 2 mm.	Granule
2 - 1 mm.	Very coarse sand
1 - 1/2 mm.	Coarse sand
1/2 - 1/4 mm.	Medium sand
1/4 - 1/8 mm.	Fine sand
1/8 - 1/16 mm.	Very fine sand
1/16-1/256 mm.	Silt
Below 1/256 mm.	Clay

2. U. S. Army Corps of Engineers' Classification

Grade Limits (Diameters)	Name
Above 305 mm.	Boulders
76 mm - 305 mm.	Cobbles
19 mm - 76 mm.	Coarse gravel
4.7 mm - 19 mm.	Fine gravel
1.9 mm - 4.7 mm	Coarse sand
0.42 - 1.9 mm.	Medium sand
0.074 - 0.42 mm.	Fine sand
0.001 - 0.074 mm.	Silt or clay

3. U. S. Bureau of Soils Classification

Grade Limits (Diameters)	Name
2 - 1 mm.	Fine gravel
1 - 1/2 mm.	Coarse sand
1/2 - 1/4 mm.	Medium sand
1/4 - 1/10 mm.	Fine sand
1/10 - 1/20 mm.	Very fine sand
1/20 - 1/200 mm.	Silt
Below 1/200 mm.	Clay

4. Atterberg's Size Classification

Grade Limits (Diameters)	Name
2,000 - 200 mm.	Blocks
200 - 20 mm.	Cobbles
20 - 2 mm.	Pebbles
2 - 0.2 mm.	Coarse sand
0.2 - 0.02 mm.	Fine sand
0.02 - 0.002 mm.	Silt
Below 0.002 mm.	Clay

SOLITARY WAVE - A wave consisting of a single elevation (above the water surface) of height not necessarily small compared to the depth, and neither followed nor preceded by another elevation or depression of the water surfaces.

SORTING COEFFICIENT - A coefficient used in describing the distribution of grain sizes in a sample of unconsolidated material. It is defined as $S_o = \sqrt{Q_1/Q_3}$, where Q_1 is that diameter which has 75% of the cumulative size-frequency (by wt.) distribution smaller than itself and 25% larger than itself, and Q_3 is that diameter having 25% smaller and 75% larger than itself.

SOUND (noun) - (1) A wide waterway between the mainland and an island, or a wide waterway connecting two sea areas. See also STRAIT. (2) A relatively long arm of the sea or ocean forming a channel between an island and a mainland or connecting two larger bodies, as a sea and the ocean, or two parts of the same body; usually wider and more extensive than a STRAIT.

SOUND (verb) - To measure or ascertain the depth of water as with sounding lines.

SOUNDING - A measured depth of water. On hydrographic charts the soundings are adjusted to a specific plane of reference (SOUNDING DATUM).

SOUNDING DATUM - The plane to which soundings are referred. See also DATUM, CHART.

SOUNDING LINE - A line, wire, or cord used in sounding. It is weighted at one end with a plummet (sounding lead). Also LEADLINE.

SPILLING BREAKER - See under BREAKER.

SPIT - A small point of land or submerged ridge running into a body of water from the shore. (See Figure A-9).

SPRING TIDE - A tide that occurs at or near the time of new and full moon and which rises highest and falls lowest from the mean level.

STAND OF TIDE - An interval at high or low water when there is no sensible change in the height of the tide. The water level is stationary at high and low water for only an instant, but the change in level near these times is so slow that it is not usually perceptible. See TIDE, SLACK.

STANDING WAVE - A type of wave in which the surface of the water oscillates vertically between fixed points, called nodes, without progression. The points of maximum vertical rise and fall are called antinodes or loops. At the nodes, the underlying water particles exhibit no vertical motion but maximum horizontal motion. At the antinodes the underlying water particles have no horizontal motion and maximum vertical motion. They may be the result of two equal progressive wave trains travelling through each other in opposite directions. Sometimes called STATIONARY WAVE.

STATIONARY WAVE - A wave of essentially stable form which does not move with respect to a selected reference point; a fixed swelling. Sometimes called STANDING WAVE.

STILL WATER LEVEL - The elevation of the surface of the water if all wave action were to cease. (See Figure A-3)

STONE - (1) Rock or rocklike matter used as a material for building; (2) A small piece of rock or a specific piece of rock.

STONE, DERRICK - Stone of a sufficient size as to require handling in individual pieces by mechanical means, generally 1 ton up.

STORM TIDE - The rise of water accompanying a storm caused by wind stresses on the water surface. See also SET-UP, WIND.

STRAIT - A relatively narrow waterway between two larger bodies of water. See also SOUND (noun).

STREAM - (1) A course of water flowing along a bed in the earth; (2) A current in the sea formed by wind action, water density differences, etc. (Gulf Stream) See also CURRENT, STREAM.

SURF - The wave activity in the area between the shore line and the outermost limit of breakers.

SURF BEAT - Irregular oscillations of the nearshore water level, with periods of the order of several minutes.

SURF ZONE - The area between the outermost breaker and the limit of wave uprush. (See Figures A-2 and A-5).

SURGE - (1) The name applied to wave motion with a period intermediate between that of the ordinary wind wave and that of the tide, say from $\frac{1}{2}$ to 60 minutes. It is of low height; usually less than 0.3 foot. See also SEICHE. (2) In fluid flow, long interval variations in velocity and pressure, not necessarily periodic, perhaps even transient in nature.

SURGING BREAKER - See under Breaker.

SWAMP (noun) - A tract of wet spongy land, frequently inundated by fresh or salt water, and characteristically dominated by trees and shrubs.

SWAMP (verb) - To overset, sink, or fill up a craft with water.

SWASH - The rush of water up onto the beach following the breaking of a wave. Also UPRUSH, RUN-UP.

SWASH CHANNEL - (1) On the open shore, a channel cut by flowing water in its return to the parent body (e.g. a rip channel); (2) A secondary channel passing through or shoreward of an inlet or river bar. (See Figure A-9).

SWASH MARK - The thin wavy line of fine sand, mica scales, bits of seaweed, etc. left by the uprush when it recedes from its upward limit of movement on the beach face.

SWELL - Wind-generated waves that have advanced into regions of weaker winds or calm.

TERRACE - A horizontal or nearly horizontal natural or artificial topographic feature interrupting a steeper slope, sometimes occurring in a series.

TIDAL CURRENT - A current caused by the tide-producing forces of the moon and the sun, a part of the same general movement of the sea that is manifested in the vertical rise and fall of the tides. Also CURRENT, PERIODIC. See also CURRENT, FLOOD and CURRENT, EBB.

TIDAL DATUM - See DATUM, CHART, and DATUM PLANE.

TIDAL DAY - The time of the rotation of the earth with respect to the moon, or the interval between two successive upper transits of the moon over the meridian of a place, about 24.84 solar hours (24 hours and 50 minutes) in length or 1.035 times as great as the mean solar day. (See Figure A-10)

TIDAL FLATS - Marshy or muddy land areas which are covered and uncovered by the rise and fall of the tide.

TIDAL INLET - (1) A natural inlet maintained by tidal flow; (2) Loosely any inlet in which the tide ebbs and flows. Also TIDAL OUTLET.

TIDAL PERIOD - The interval of time between two consecutive like phases of the tide. (See Figure A-10)

TIDAL POOL - A pool of water remaining on a beach or reef after recession of the tide.

TIDAL PRISM - The total amount of water that flows into the harbor or out again with movement of the tide, excluding any fresh water flow.

TIDAL RANGE - The difference in height between consecutive high and low waters. (See Figure A-10)

TIDAL RISE - The height of tide as referred to the datum of a chart. (See Figure A-10)

TIDAL WAVE - See TSUNAMI.

TIDE - The periodic rising and falling of the water that results from gravitational attraction of the moon and sun acting upon the rotating earth. Although the accompanying horizontal movement of the water resulting from the same cause is also sometimes called the tide, it is preferable to designate the latter as TIDAL CURRENT, reserving the name tide for the vertical movement.

TIDE, DAILY RETARDATION OF - The amount of time by which corresponding tides grow later day by day.

TIDE, DIURNAL - A tide with one high water and one low water in a tidal day. (See Figure A-10)

TIDE, EBB - That period of tide between a high water and the succeeding low water; falling tide. (See Figure A-10)

TIDE, FLOOD - That period of tide between low water and the succeeding high water; a rising tide. (See Figure A-10)

TIDE, MIXED - A type of tide in which the presence of a diurnal wave is conspicuous by a large inequality in either the high or low water heights with two high waters and two low waters usually occurring each tidal day. In strictness all tides are mixed but the name is usually applied without definite limits to the tides intermediate to those predominantly semidiurnal and those predominantly diurnal. (See Figure A-10).

TIDE, NEAP - A tide occurring near the time of quadrature of the moon. The neap tidal range is usually 10 to 30 percent less than the mean tidal range.

TIDE, SEMIDIURNAL - A tide with two high and two low waters in a tidal day, with comparatively little diurnal inequality. (See Figure A-10)

TIDE, SLACK - The state of a tidal current when its velocity is near zero, especially the moment when a reversing current changes direction and its velocity is zero. Sometimes considered the intermediate period between ebb and flood currents during which the velocity of the currents is less than 0.1 knot. See TIDAL STAND. Also SLACK WATER.

TIDE, SPRING - A tide that occurs at or near the time of new and full moon and which rises highest and falls lowest from the mean level.

TIDE, STORM - The rise of water accompanying a storm caused by wind stresses on the water surface. See also SET-UP, WIND.

TOMBOLO - An area of unconsolidated material, deposited by wave action or currents, that connects a rock, or island, etc. to the mainland or to another island. (See Figure A-9)

TOPOGRAPHY - The configuration of a surface including its relief, the position of its streams, roads, buildings, etc.

TRAINING WALL - A wall or jetty to direct current flow.

TRANSITIONAL ZONE
TRANSITIONAL WATER - In regard to progressive gravity waves, water whose depth is less than 1/2 but not more than 1/25 the wave length. Often called SHALLOW WATER.

TROCHOIDAL WAVE - A progressive oscillatory wave whose form is that of a prolate cycloid or trochoid. It is approximated by waves of small amplitude. See also WAVE, CYCLOIDAL.

TROUGH OF WAVE - The lowest part of a wave form between successive crests. Also that part of a wave below still water level. (See Figure A-3).

TSUNAMI - A generally long period wave caused by underwater seismic disturbance or volcanic eruption. Commonly misnamed "tidal wave".

UNDERTOW - A current, below water surface, flowing seaward; also the receding water below the surface from waves breaking on a shelving beach. Actually "undertow" is largely mythical. As the backwash of each wave flows down the beach, a current is formed which flows seaward, however, it is a periodic phenomenon. The most common phenomena expressed as "undertow" are actually the rip currents in the surf.

UNDERWATER GRADIENT - The slope of the sea bottom. See also SLOPE.

UNDULATION - A continuously propagated motion to and fro, in any fluid or elastic medium, with no permanent translation of the particles themselves.

UPCOAST - In United States usage; the coastal direction generally trending towards the north.

UPDRIFT - The direction opposite that of the predominant movement of littoral materials.

UPLIFT - The upward water pressure on the base of a structure or pavement.

UPRUSH - The rush of water up onto the beach following the breaking of a wave. Also SWASH, RUN-UP. (See Figure A-2).

VALLEY, SEA - A submarine depression of broad valley form without the steep side slopes which characterize a canyon.

VALLEY, SUBMARINE - A prolongation of a land valley into or across the continental or insular shelf, which generally gives evidence of having been formed by stream erosion.

VARIABILITY OF WAVES - (1) The variation of heights and periods between individual waves within a wave train. (Wave trains are not composed of waves of equal height and period , but rather of heights and periods which vary in a statistical manner). (2) The variation in direction of propagation of waves leaving the generating area. (3) The variation in height along the crest. This is usually called "variation along the wave".

VELOCITY OF WAVES - The speed with which an individual wave advances.

VERY SHALLOW WATER - See SHALLOW WATER.

VISCOSITY - Internal friction due to molecular cohesion in fluids. The internal properties of a fluid which offer resistance to flow.

WATER LINE - A juncture of land and sea. This line migrates, changing with the tide or other fluctuation in the water level. Where waves are present on the beach, this line is also known as the limit of backrush. (Approximately the intersection of the land with the still water level).

WAVE, STATIONARY - A wave of essentially stable form which does not move with respect to a selected reference point.

WAVE STEEPNESS - The ratio of a wave's height to its length.

WAVE TRAIN - A series of waves from the same direction.

WAVE OF TRANSLATION - A wave in which the water particles are permanently displaced to a significant degree in the direction of wave travel. Distinguished from an OSCILLATORY WAVE.

WAVE, TROCHOIDAL - A progressive oscillatory wave whose form is that of a prolate cycloid or trochoid. It is approximated by waves of small amplitude. See also WAVE, CYCLOIDAL.

WAVE TROUGH - The lowest part of a wave form between successive crests. Also that part of a wave below still water level.

WAVE VARIABILITY - (1) The variation of heights and periods between individual waves within a wave train. (Wave trains are not composed of waves of equal height and period, but rather of heights and periods which vary in a statistical manner). (2) The variation in direction of propagation of waves leaving the generating area. (3) The variation in height along the crest. This is usually called "variation along the wave".

WAVE VELOCITY - The speed with which an individual wave advances.

WAVE, WIND - A wave that has been formed and built up by the wind.

WAVES, INTERNAL - Waves that occur within a fluid whose density changes with depth, either abruptly at a sharp surface of discontinuity (an interface) or gradually. Their amplitude is greatest at the density discontinuity or, in the case of a gradual density change, somewhere in the interior of the fluid and not at the free upper surface where the surface waves have their maximum amplitude.

WHARF - A structure built on the shore of a harbor, river, canal, etc., so that vessels may lie alongside to receive and discharge cargo, passengers, etc.

WHITECAP - On the crest of a wave, the white froth caused by wind.

WIND - The horizontal natural movement of air; air naturally in motion with any degree of velocity.

WIND CHOP - The short-crested waves that may spring up quickly in a fairly moderate breeze, and break easily at the crest.

WAVE HINDCASTING - The calculation from historic synoptic wind charts of the wave characteristics that probably occurred at some past time.

WAVE LENGTH - The horizontal distance between similar points on two successive waves measured perpendicularly to the crest.

WAVE, OSCILLATORY - A wave in which each individual particle oscillates about a point with little or no permanent change in position. The term is commonly applied to progressive oscillatory waves in which only the form advances, the individual particles moving in closed or nearly closed orbits. Distinguished from a WAVE of TRANSLATION. See also ORBIT.

WAVE PERIOD - The time for a wave crest to traverse a distance equal to one wave length. The time for two successive wave crests to pass a fixed point. See also SIGNIFICANT WAVE PERIOD.

WAVE, PROGRESSIVE - A wave which is manifested by the progressive movement of the wave form.

WAVE PROPAGATION - The transmission of waves through water.

WAVE RAY - See ORTHOGONAL.

WAVE, REFLECTED - The wave that is returned seaward when a wave impinges upon a very steep beach or barrier.

WAVE REFRACTION - (1) The process by which the direction of a train of waves moving in shallow water at an angle to the contours is changed. The part of the wave train advancing in shallower water moves more slowly than that part still advancing in deeper water, causing the wave crests to bend toward alinement with the underwater contours. (See Figures A-5 and A-6). (2) The bending of wave crests by currents.

WAVE, SEISMIC - A TSUNAMI. A generally long period wave caused by an underwater seismic disturbance or volcanic eruption. Commonly misnamed "tidal wave".

WAVE, SOLITARY - A wave consisting of a single elevation (above the water surface) of height not necessarily small compared to the depth and neither followed nor preceded by another elevation or depression of the water surfaces.

WAVE, STANDING - A type of wave in which there are nodes, or points of no vertical motion and maximum horizontal motion, between which the water oscillates vertically. The points of maximum vertical motion and least horizontal motion are called antinodes or loops. It is caused by the meeting of two similar wave groups travelling in opposing directions.

WAVE, STATIONARY - A wave of essentially stable form which does not move with respect to a selected reference point.

WAVE STEEPNESS - The ratio of a wave's height to its length.

WAVE TRAIN - A series of waves from the same direction.

WAVE OF TRANSLATION - A wave in which the water particles are permanently displaced to a significant degree in the direction of wave travel. Distinguished from an OSCILLATORY WAVE.

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WAVE VELOCITY - The speed with which an individual wave advances.

WAVE, WIND - A wave that has been formed and built up by the wind.

WAVES, INTERNAL - Waves that occur within a fluid whose density changes with depth, either abruptly at a sharp surface of discontinuity (an interface) or gradually. Their amplitude is greatest at the density discontinuity or, in the case of a gradual density change, somewhere in the interior of the fluid and not at the free upper surface where the surface waves have their maximum amplitude.

WHARF - A structure built on the shore of a harbor, river, canal, etc., so that vessels may lie alongside to receive and discharge cargo, passengers, etc.

WHITECAP - On the crest of a wave, the white froth caused by wind.

WIND - The horizontal natural movement of air; air naturally in motion with any degree of velocity.

WIND CHOP - The short-crested waves that may spring up quickly in a fairly moderate breeze, and break easily at the crest.

WIND, FOLLOWING - In wave forecasting, wind blowing in the same direction that waves are travelling.

WIND, OFFSHORE - A wind blowing seaward over the coastal area.

WIND, ONSHORE - A wind blowing landward over the coastal area.

WIND, OPPOSING - In wave forecasting, wind blowing in the opposite direction to that in which the waves are travelling.

WIND SET-UP - (1) The vertical rise in the still water level on the leeward side of a body of water caused by wind stresses on the surface of the water; (2) The difference in still water level between the windward and the leeward sides of a body of water caused by wind stresses on the surface of the water; (3) Synonymous with WIND TIDE. WIND TIDE is usually reserved for use on the ocean and large bodies of water. WIND SET-UP is usually reserved for use on reservoirs and smaller bodies of water (See Figure A-11).

WIND TIDE - See WIND SET-UP.

WINDWARD - The direction from which the wind is blowing.

WIND WAVES (1) Waves being formed and built up by the wind, (2) loosely, any wave generated by wind.

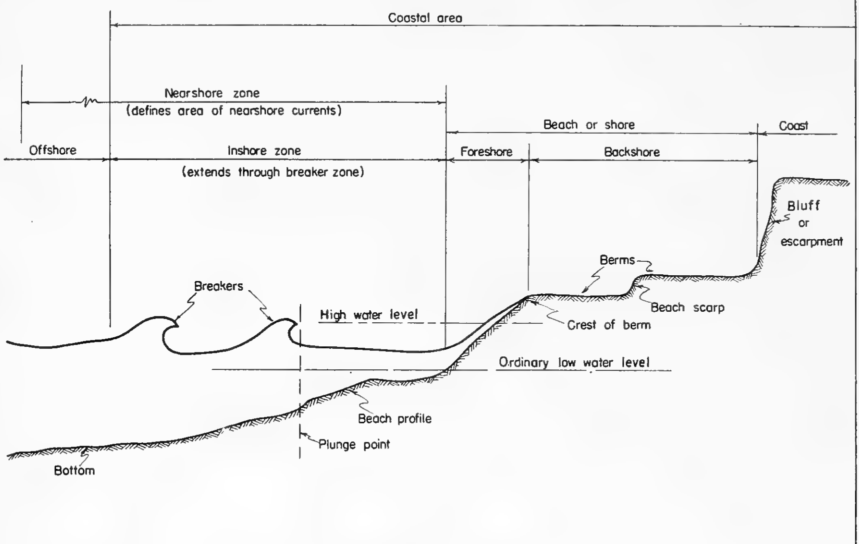


FIGURE A-1 — Beach profile — related terms

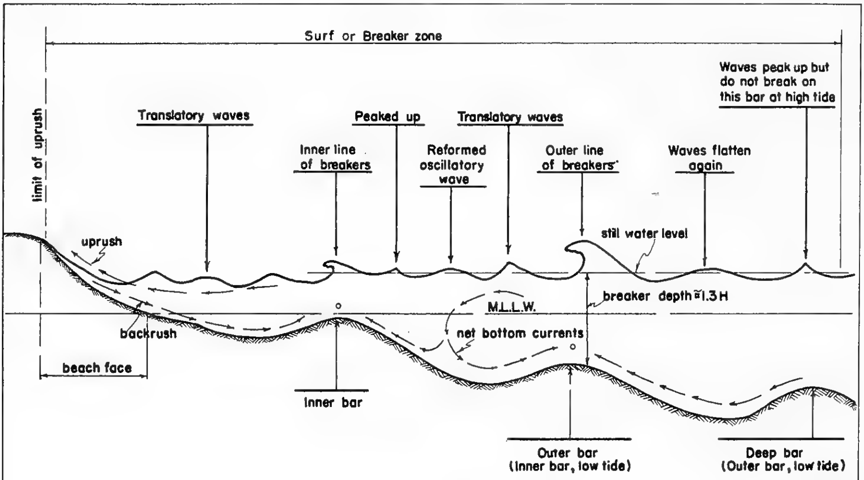


FIGURE A-2 Waves and currents in the breaker zone

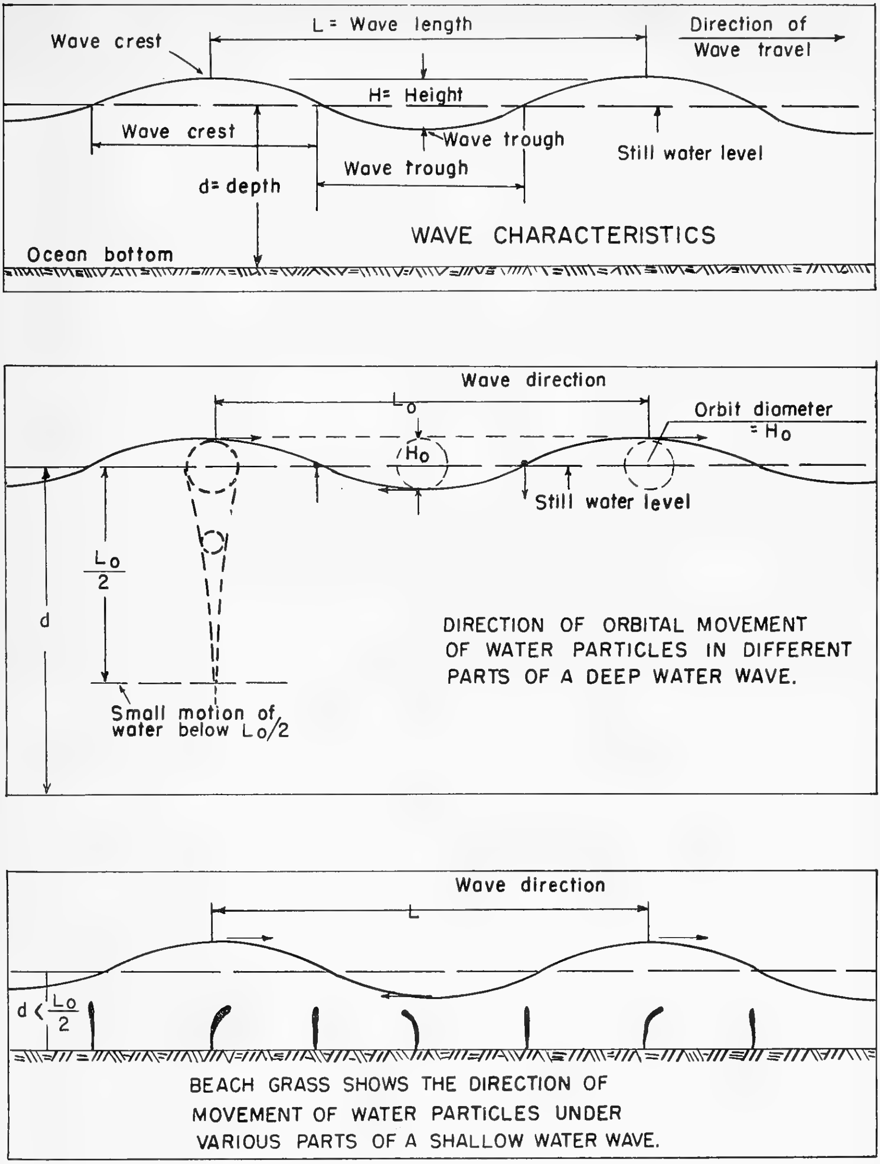
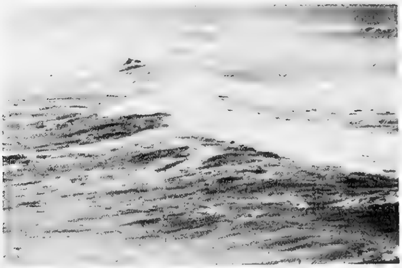
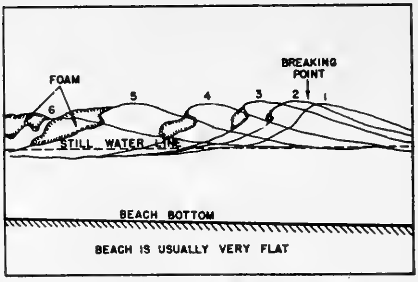


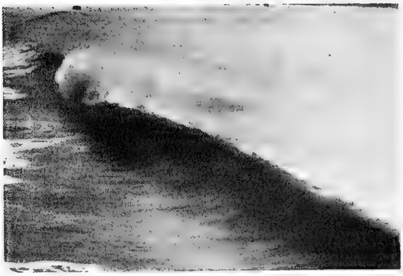
FIGURE A-3 WAVE CHARACTERISTICS AND DIRECTION OF WATER PARTICLE MOVEMENT



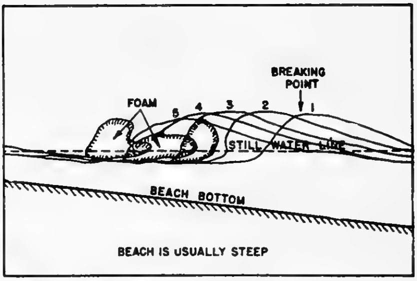
SPILLING BREAKER



SKETCH SHOWING THE GENERAL CHARACTER OF SPILLING BREAKERS

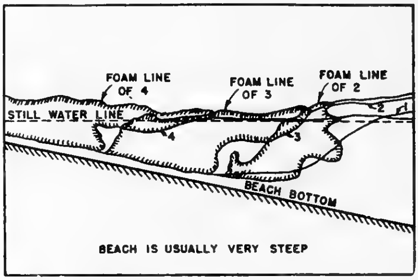


PLUNGING BREAKER



SKETCH SHOWING THE GENERAL CHARACTER OF PLUNGING BREAKERS

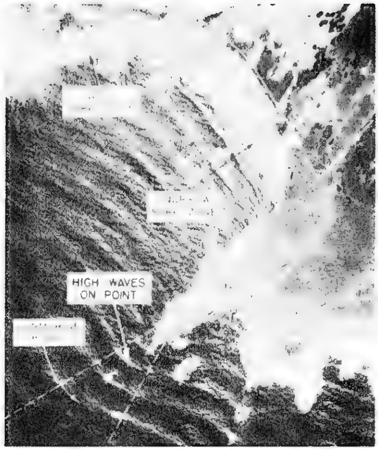
SURGING BREAKER



SKETCH SHOWING THE GENERAL CHARACTER OF SURGING BREAKERS

Both Photographs And Diagrams Of The Three Types Of Breakers Are Presented Above. The Sketches Consist Of A Series Of Profiles Of The Wave Form As It Appears Before Breaking, During The Breaking, And After Breaking. The Numbers Opposite The Profile Lines Indicate The Relative Times Of The Occurrences.

FIGURE A-4 BREAKER TYPES



Pt. Pinos, California

Waves moving over a submarine ridge concentrate to give large wave heights on a point.

Halfmoon Bay, California

Note the increasing width of the surf zone with increasing degree of exposure to the south.

Purisima Pt., California

Refraction of waves around a headland produces low waves and a narrow surf zone where bending is greatest.

FIGURE A-5 REFRACTION OF WAVES

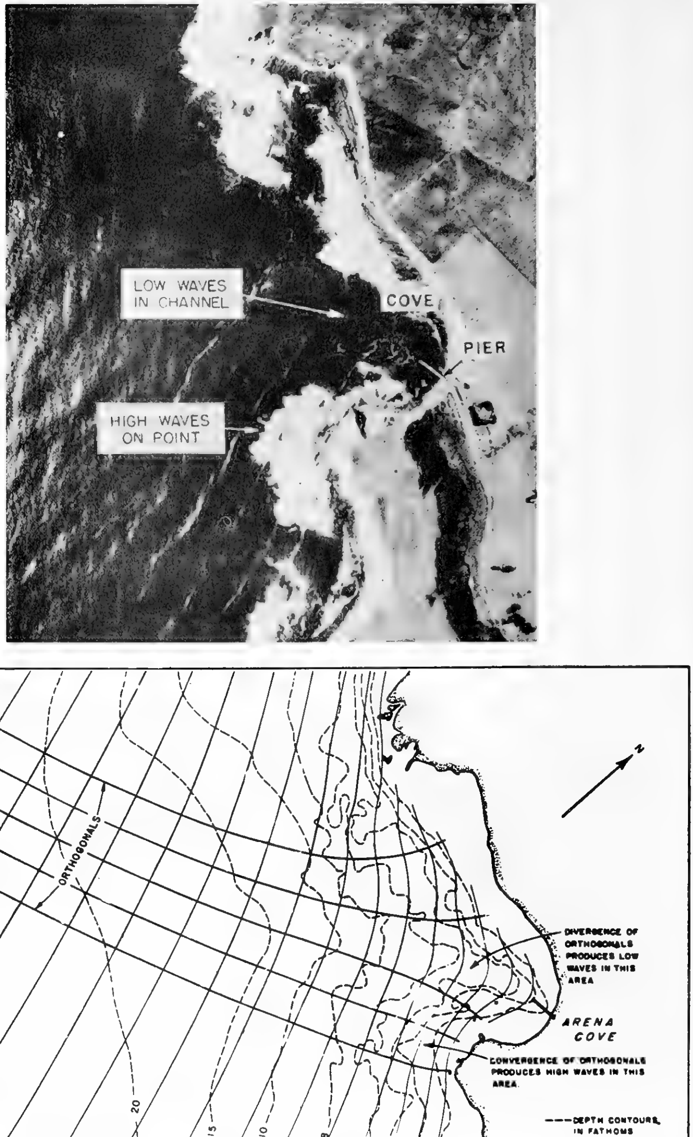


FIGURE A-6 REFRACTION DIAGRAM

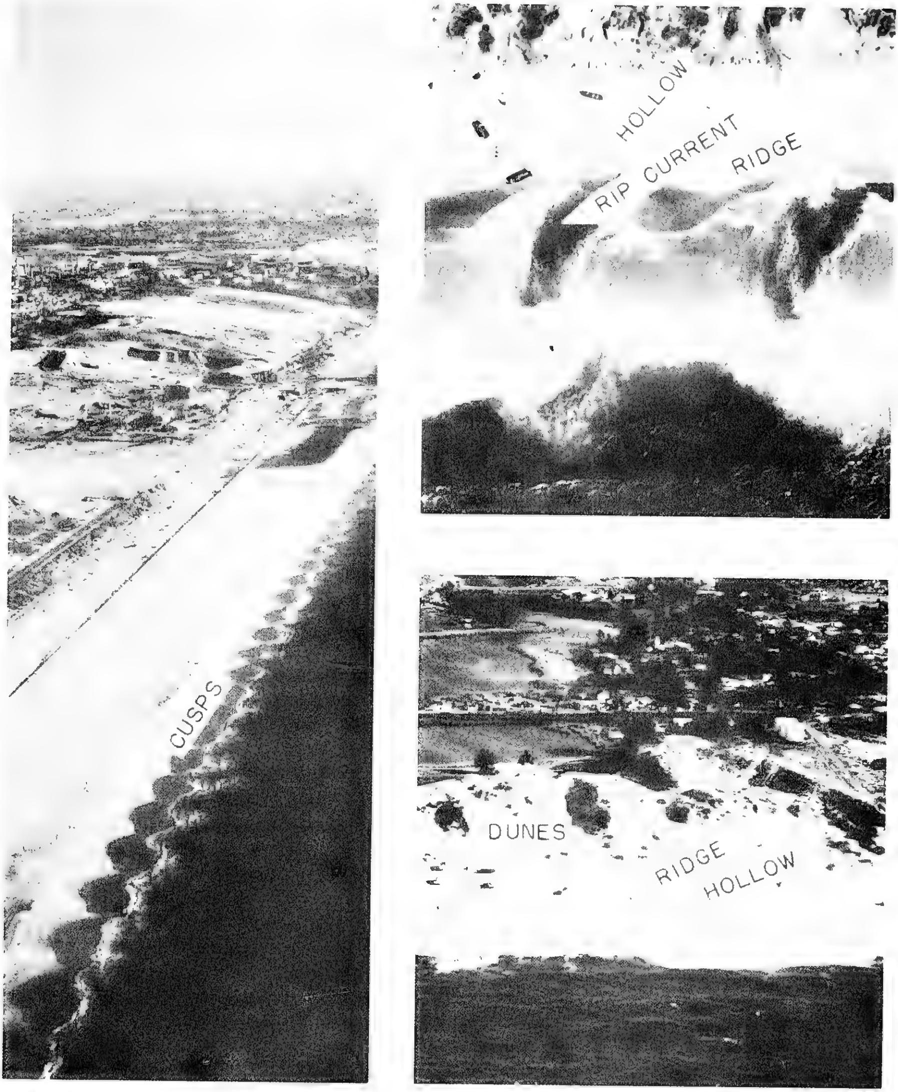


FIGURE A-7 BEACH FEATURES

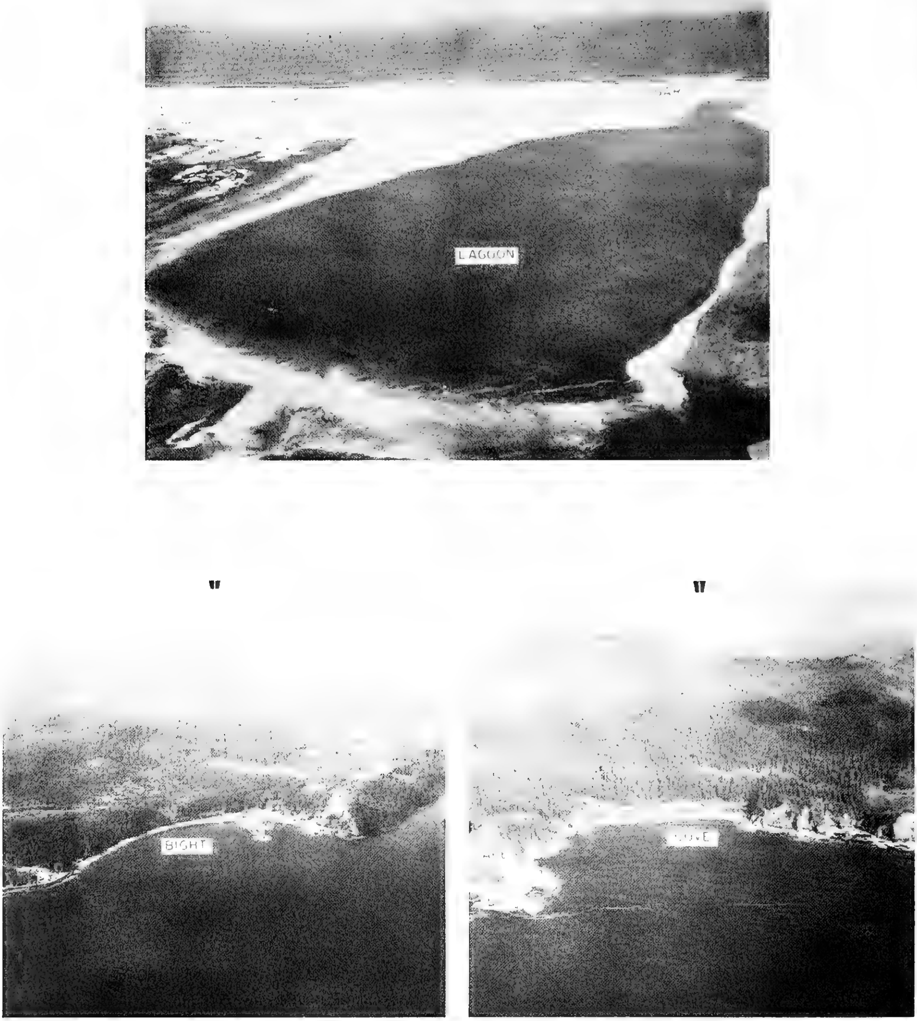


FIGURE A-8 SHORELINE FEATURES

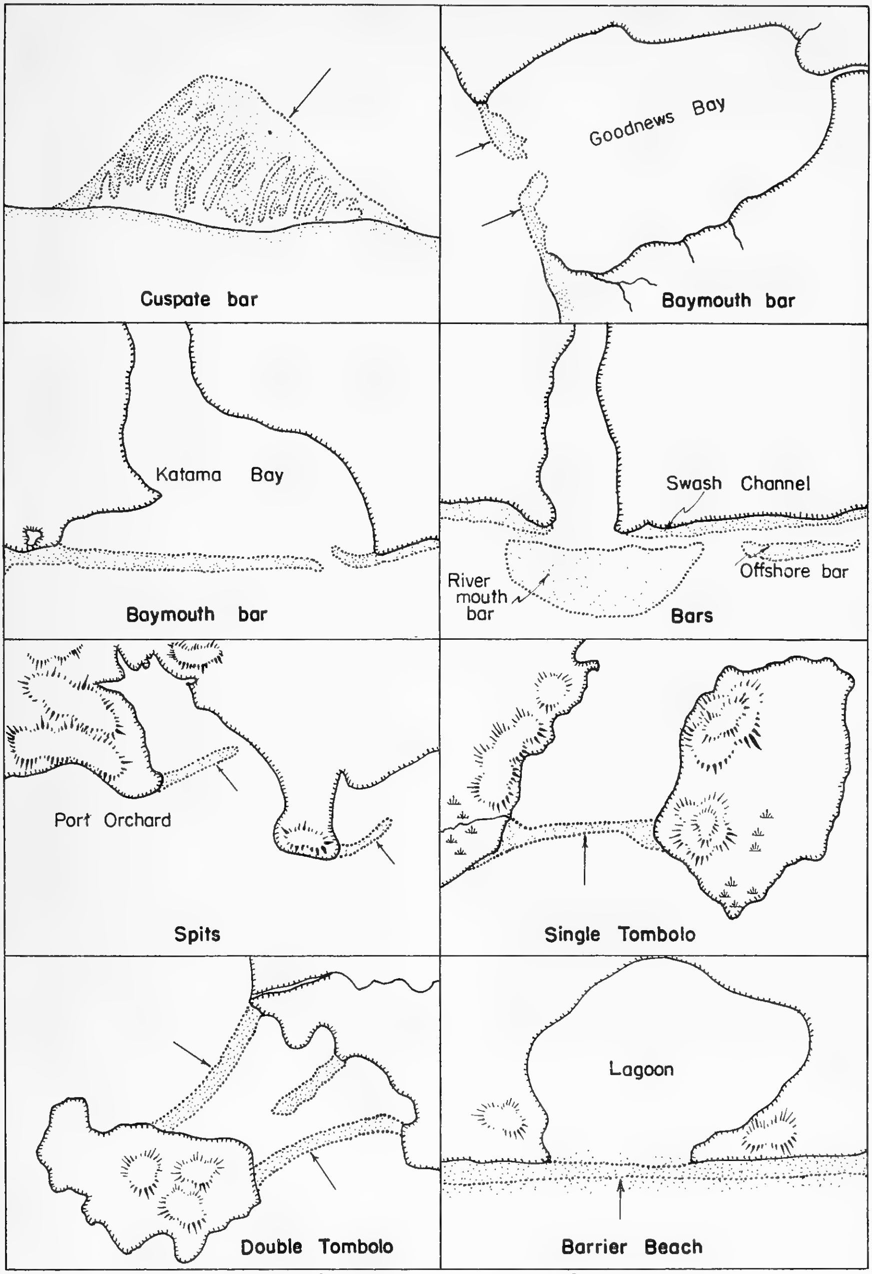


FIGURE A-9 Bar and beach forms

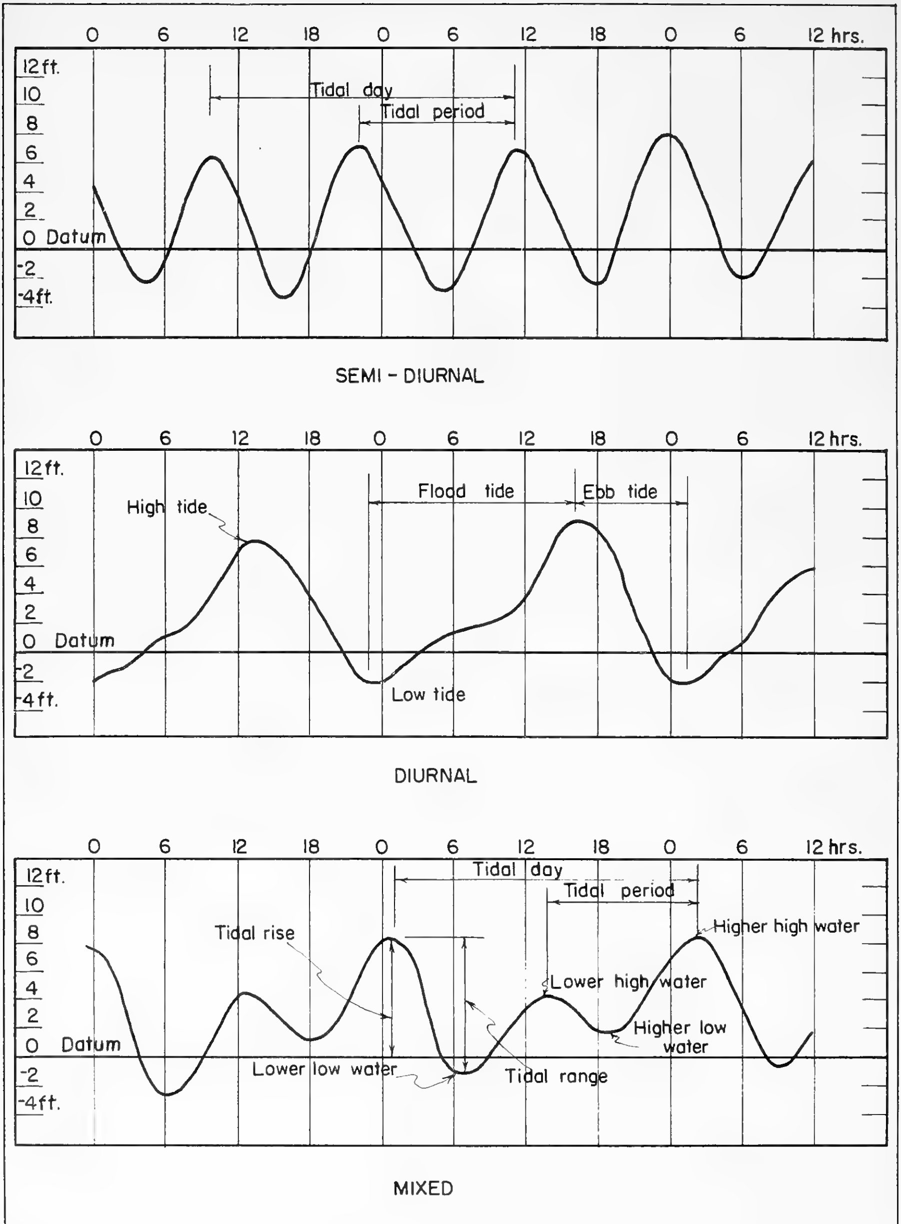


FIGURE A-10 Types of tides

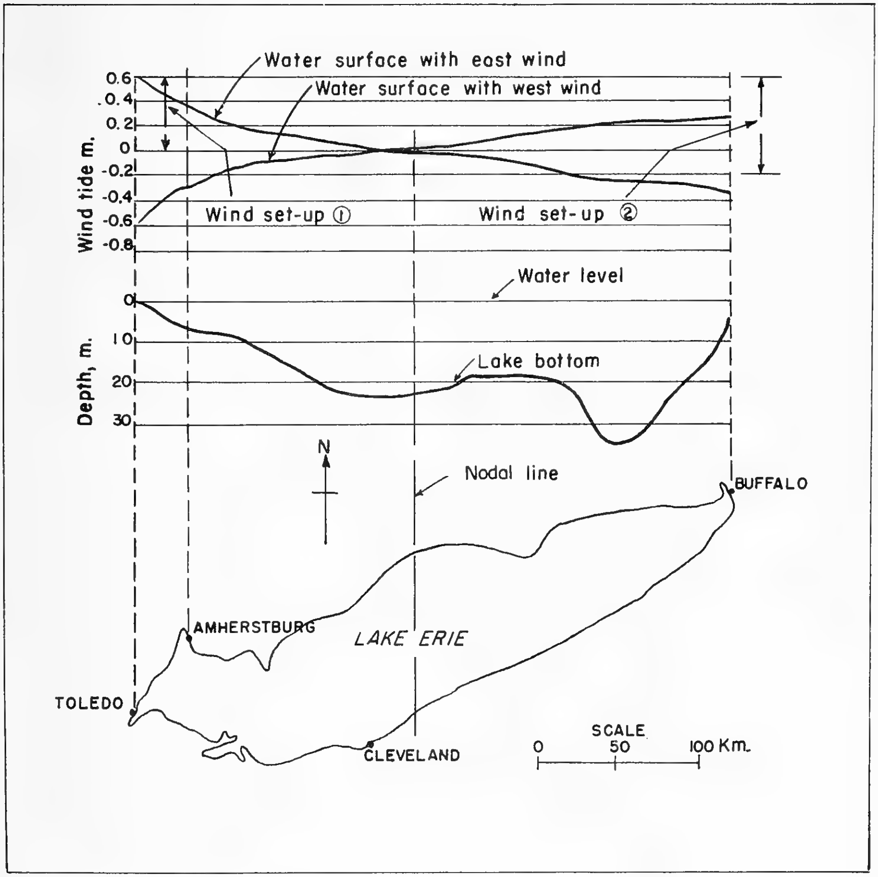


FIGURE A-11 SET-UP OR WIND TIDE

APPENDIX B

LIST OF COMMON SYMBOLS

APPENDIX B

LIST OF COMMON SYMBOLS

Symbol	Definition	Basic Units (in F, L, T System)	Example in English Units
A	Area	L^2	$feet^2$
a	Acceleration, Also:	L/T^2	ft/sec^2
	Amplitude	L	feet
a_z	Horizontal displacement of a water particle	L	feet
a'	Length of semi-major axis of orbit of water particle	L	feet
-a	Subscript "a" may refer to active earth pressures	--	--
$\_{av}$	Subscript "av" refers to average	--	--
B	Breakwater gap width	L	feet
B.F.	Beaufort wind force	--	--
b	Length of wave crest between orthogonals (measured perpendicular to the local direction to travel)	L	feet
b_z	Vertical displacement of a water particle	L	feet
b'	Length of semi-minor axis of orbit of water particle	L	feet
$\_b$	Subscript "b" refers to breaking wave conditions	--	--
C,c	Wave velocity	L/T	ft/sec
C_d	Coefficient of drag	--	--
C_g	Group velocity	L/T	ft/sec
C_H	Velocity of waves of finite height	L/T	ft/sec

Symbol	Definition	Basic Units (in F, L, T System)	Example in English Units
C_M	Coefficient of mass	--	--
C_o	Deep water wave velocity	L/T	ft/sec
D	Decay distance Also: Diameter Also: A depth	L L L	Nautical miles feet feet
D_d	Coefficient of shoaling	--	--
D_e	Effective decay distance	L	nautical miles
d	Depth of water, measured from the still water level to the bottom	L	feet or fathoms
d_b	Depth of water at a breaker's position	L	feet
E	Mean total energy of one wave per unit length of crest	LF/L	ft-lbs/ft of crest
E_k	Mean kinetic energy of one wave per unit length f crest	LF/L	ft-lbs/ft of crest
E_p	Mean potential energy of one wave per unit length of crest	LF/L	ft-lbs/ft of crest
F	Force, one of the basic units Also: a function of one or more variables, as F(x,y). Also: Fetch length	F -- L	pounds -- nautical miles
F(H)	Percent of wave heights below the height H.	--	--
F_H	Horizontal component of force	F	pounds
F_m	Minimum fetch length	L	nautical miles
F_V	Vertical component of force	F	pounds
$F_X(x)$	Prob. $(X \leq x)$ = cumulative distribution function of the random variable X.	--	--

Symbol	Definition	Basic Units (in F,L, T System)	Example in English Units
f	Function of one or more variables, as $f(x,y)$ Also: Coriolis parameter ($f = 2 \Omega \sin \phi$)	-- $1/T$	-- radian/sec
f'_c	Compressive strength of concrete	F/L^2	psi
g	Acceleration of gravity Also: Function of one or more variables, as $g(x,y)$	L/T^2 --	ft/sec^2 --
H	Wave height Also: Horizontal component of force	L F	feet pounds
$H_{1/10}$	Average height of the highest one-tenth of the waves for a specified period of time	L	feet
$H_{1/3}$ also H_s	Average height of the highest one-third of the waves for a specified period of time	L	feet
H_b	Wave height on breaking	L	feet
H_D	Significant wave height at end of decay distance	L	feet
H_F	Significant wave height at end of fetch	L	feet
H_{av}	Average of the wave heights for a specified period of time	L	feet
H_m	Highest wave for a specified period of time	L	feet
h	A height	L	feet
h_o	Elevation of mean level of clapotis above SWL	L	feet
J	Distance between two underwater contours as used in the orthogonal methods of wave refraction coefficient determination	L	feet

Symbol	Description	Basic Units (in F,L, T System)	Example in English Units
K	Sub-surface pressure response coefficient	--	--
	Also: Any constant	--	--
	Also: Refraction coefficient	--	--
K_d	Refraction coefficient	--	--
K_b	Refraction coefficient for breakers	--	--
K'	Diffraction coefficient	--	--
i	Beach slope, as specified	L/L	$\frac{\text{Ver. rise (ft)}}{\text{Hor. dist. (ft)}}$
	Also: $\sqrt{-1}$	--	--
k	$\sqrt{m^2 + n^2}$, where $m = 2\pi/L$ and $n = 2\pi/L'$ (short crested waves)	1/L	1/feet
	Also: $2\pi/L$ (long crested waves)	1/L	1/feet
L	Wave length (distance between two successive crests in the direction of propagation)	L	feet
	Also: Length, one of the basic units	L	feet
L_b	Wave length on breaking	L	feet
L_D	Wave length at end of decay	L	feet
L_F	Wave length at end of fetch	L	feet
L'	Wave length in the crest direction (short-crested theory)	L	feet
ℓ	a length	L	feet
ℓ'	Crest length (the linear distance measured along the wave crest between consecutive intersections of the crest and the still water level)	L	feet
M	Mass, a basic unit in the (M,L,T) system	$\frac{FT^2}{L}$	slugs
	Also: Energy coefficient		
	Also: Moment	LF	foot-pounds

Symbol	Definition	Basic Units (in F,L, T System)	Example in English Units
m	$2\pi/L$	1/L	1/feet
N	A number	--	--
n	Ratio of group velocity to wave velocity	--	--
	Also: Crest interval in refraction drawings	--	--
	Also: $2 \pi/L'$	1/L	1/feet
$\_o$	Subscript "o" refers to deep water conditions	--	--
P	Power transmitted by one wave per unit length of crest	$\frac{LF/T}{L}$	ft-lbs/sec per foot
	Also: Pressure	F/L^2	pounds/square foot
	Also: Force	F	pounds
p	Probability	--	--
	Also: Sub-surface pressure associated with wave motion	F/L^2	pounds/square foot
	Also: Atmospheric pressure	F/L^2	milibars
	Also: Any pressure	F/L^2	pounds/square foot
	Also: Angle of earth fill surface to the horizontal	--	degrees
Prob (A) or p (A)	The probability of a statement A	--	--
$\_p$	Subscript "p" may refer to passive earth pressures	--	--
q	Water particle velocity in direction of interest	L/T	ft/sec
R	A resultant force	F	pounds
	Also: Distance between contours measured along an orthogonal	L	feet
R_e	Reynolds number	--	--

Symbol	Definition	Basic Units (in F,L, T System)	Example in English Units
r	Radius	L	feet
	Also: Coefficient of energy partition	--	--
	Also: Distance from end of breakwater to point (x,y) in diffraction theory	L	feet
S	Distance from the ocean bottom to a water particle		
	Also: Sum of the real parts of diffraction function $\Sigma s + i\Sigma w$	--	--
	Also: Specific gravity	--	--
s	Sheltering coefficient	$1/L^3$	$1/ft^3$
	Also: Specific gravity	--	--
	Also: Real part of diffraction Function $f(-u) = s + iw$	--	--
	Also: Slope as specified	L/L	$\frac{\text{Ver. rise (ft)}}{\text{Hor. dist. (ft)}}$
$\_\_{s}$	Subscript "s" refers to surface terms	--	--
T	Wave period	T	seconds
	Also: Temperature	θ	degrees F.
	Also: Length, a basic unit	L	feet
T_C	Significant wave period--the average period for the well defined series of highest waves on a record	T	seconds
T_D	Significant wave period at end of decay	T	seconds
T_F	Significant wave period at end of fetch	T	seconds
t	A time	T	seconds
t_D	Travel time of waves (from end of fetch to end of decay distance)	T	hours

Symbols	Definition	[illegible] ([illegible] F,L, [illegible])	Example in English [illegible]
t_d	Wind duration, interval of time wind blows at constant velocity in generating waves	T	hours
t_v	Modulus of [illegible] due to viscosity	T	hours
t_{min}	Minimum duration of wind in fetch area	T	hours
U	Velocity of surface wind	L/T	knots
$\overline{U}$	Velocity of mass transport	L/T	ft/sec
U'	Horizontal velocity of motion left after wave motion has been destroyed (Gerstner Theory).	L/T	ft/sec
	Also: Approximate velocity of surface wind	L/T	knots
u	Water particle (horizontal component positive in the direction of wave advance) orbital velocity	L/T	ft/sec
	Also: Upper limit of integral term in solutions of diffraction problem	--	--
V	A velocity	L/T	ft/sec
	Also: A volume	L^3	$feet^3$
V_f	Velocity of storm or fetch front	L/T	knots
V_g	Geostrophic wind velocity	L/T	knots
v	Water particle (vertical component) orbital velocity	L/T	ft/sec
W	Weight	F	pounds
	Also: Work performed by one wave per unit length of crest	FL/L	ft-lbs/ft of crest
w	Unit weight (specific weight) of water	F/L^3	lbs/ft^3
	Also: Imaginary part of [illegible] function f(-u) = S + iw	--	--

Symbol	Definition	Basic Unit (in F,L, T System)	Example in English Units
X	Extraneous force in x direction	F	pounds
x	Coordinate, usually horizontal	L	feet
x_o	Equilibrium position of particle, horizontal coordinate	L	feet
$\overline{x}$	Horizontal displacement of particles from equilibrium position	L	feet
Y	Extraneous force in y direction	F	pounds
y	A coordinate, usually vertical	L	feet
y_o	Equilibrium position of particle, vertical coordinate	L	feet
$\overline{y}$	Vertical displacement of particle from equilibrium position	L	feet
Z	Extraneous force in z direction	F	pounds
	Also: Section modulus per length of wall	L^3/L	ft^2
Z_t	Time between successive weather maps	T	hours
	Also: Greenwich mean time	T	hours
z	A coordinate, usually horizontal, perpendicular to x and y	L	feet
α (alpha)	Angle of wave crest to bottom contours	--	degrees
	Also: Angle of wave approach, measured between the shore line and the line of wave advance	--	degrees
	Also: Angle between gradient and surface winds	--	degrees
	Also: Phase difference between axis of "f" and "g" terms in diffraction theory	--	radians
α_3	Skewness coefficient	--	--

Symbol	Definition	Basic Units (in F,L, T System)	Example in English Units
β (beta)	Angular position of the water particle when the maximum horizontal particle velocity occurs.	--	degrees
	Also: Wave age, the ratio of wave velocity to the velocity of the generating wind	--	--
	Also: An angle	--	degrees
γ^2 (gamma)	Resistance coefficient applicable to wind	--	--
Δ (delta)	Change	--	as stated
δ (delta)	Wave steepness, H/L	--	--
η (eta)	Wave surface elevation	L	feet
θ (Theta)	Angle of wall face with horizontal (for earth pressures)	--	degrees
	Also: Angular displacement	--	Radians
	Also: Temperature, a basic unit	θ	degrees F.
	Also: Angle of variability in the direction of wave travel	--	degrees
	Also: Angle of the face of a structure with the horizontal	L	degrees
λ (lambda)	Wave length	L	feet
μ (mu)	Arithmetic mean	--	appropriate units
	Also: Absolute viscosity	FT/L^2	$lb\text{-}sec/ft^2$
	Also: Coefficient of friction	--	--
ν (nu)	Kinematic viscosity	L^2/T	ft^2/sec
π (pi)	3.1416 - Ratio of circumference of a circle to the diameter	--	--
ρ (rho)	Correlation coefficient	--	--
	Also: Mass density, any substance	FT^2/L^4 (M/L^3)	$slugs/ft^3$

Symbol	Definition	Basic Units (in F,L, T System)	Example in English Units
ρ_a	Mass density of air	FT^2/L^4	
		(M/L^3)	slugs/ft^3
ρ_w	Mass Density of water	FT^2/L^4 (M/L^3)	slugs ft^3
σ (sigma)	Standard deviation	--	appropriate units
	Also: $2\pi/T$	$1/T$	1/sec
σ_w	Surface tension of water	LF/L^2	ft-lbs/ft^2
τ (tau)	Drag force per unit area	F/L^2	lbs/ft^2
ϕ (phi)	Velocity potential	L^2/T	ft^2/sec
	Also: Angle of internal friction of earth	--	degrees
	Also: Phase of diffracted wave	--	radians
ψ (psi)	Aximuth of direction of wave travel, in the direction of travel	--	degrees
	Also: Stream function	L^2/T	ft^2/sec
	Also: Direction from which the wave comes	--	Compass direction (true)
Ω (omega)	Angular velocity of the earth	$1/T$	radians/sec
ω (omega)	Angular velocity	$1/T$	radian/sec

APPENDIX C

BIBLIOGRAPHY

APPENDIX C

BIBLIOGRAPHY

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APPENDIX D

MISCELLANEOUS TABLES
AND GRAPHS

APPENDIX D

MISCELLANEOUS TABLES AND GRAPHS

PLATES AND TABLES

Table D-1 - Functions of d/L for Even Increments of d/L_o

Table D-2 - Functions of d/L_o for Even Increments of d/L

d/L_o = ratio of the depth of water at any specific location to the wave length in deep water.

d/L = ratio of the depth of water at any specific location to the wave length at that same location.

K = a pressure response factor used in connection with underwater pressure instruments, where

$$K = H'/H = P/P_o = \frac{\cosh 2\pi d/L\ (1 - \frac{Z}{d})}{\cosh 2\pi d/L}$$

where P is the pressure flucutation at a depth Z below still water, P_o is the surface pressure fluctuation, d is the depth of water from still water level to the ocean bottom, L is the wave length in any particular depth of water, and H' is the corresponding variation of head at a depth Z. The values of K shown in the tables are for the instrument placed on the bottom where

$$K = \frac{1}{\cosh 2\pi d/L}$$

n = the fraction of wave energy that travels forward with the wave form: i.e., with the wave velocity C rather than the group velocity C_G.

$$n = \frac{1}{2} \quad \frac{1 + 4\pi d/L}{\sinh 4\pi d/L}$$

C_G/C_o = ratio of group velocity to deep-water wave velocity where

$$\frac{C_G}{C_o} = \frac{C_G}{C} \cdot \frac{C}{C_o} = \tanh 2\pi d/L$$

H/H'_o = ratio of the wave height in shallow water to what its wave height would have been in deep water if unaffected by refraction.

$$\frac{H}{H'_o} = \sqrt{\frac{1}{2} \cdot \frac{1}{n} \cdot \frac{1}{C/C_o}}$$

M = an energy coefficiant defined as

$$M = \frac{\pi^2}{2 \tanh^2 \frac{2\pi d}{L}}$$

TABLE D-1

FUNCTIONS OF d/L FOR EVEN INCREMENTS OF d/L_o

from 0.0001 to 1.000

d/L_o	d/L	$2\pi d/L$	TANH* $2\pi d/L$	SINH $2\pi d/L$	COSH $2\pi d/L$	K	$4\pi d/L$	SINH $4\pi d/L$	COSH $4\pi d/L$	n	C_G/C_o	H/H_o'	M
0	0	0	0	0	1	1	0	0	1	1	0	∞	∞
.0001000	.003990	.02507	.02506	.02507	1.0003	.9997	.05014	.05016	1.001	.9998	.02506	4.467	7,855
.0002000	.005643	.03546	.03544	.03547	1.0006	.9994	.07091	.07097	1.003	.9996	.03543	3.757	3,928
.0003000	.006912	.04343	.04340	.04344	1.0009	.9991	.08686	.08697	1.004	.9994	.04336	3.395	2,620
.0004000	.007982	.05015	.05011	.05018	1.0013	.9987	.1003	.1005	1.005	.9992	.05007	3.160	1,965
.0005000	.008925	.05608	.05602	.05611	1.0016	.9984	.1122	.1124	1.006	.9990	.05596	2.989	1,572
.0006000	.009778	.06144	.06136	.06148	1.0019	.9981	.1229	.1232	1.008	.9988	.06128	2.856	1,311
.0007000	.01056	.06637	.06627	.06642	1.0022	.9978	.1327	.1331	1.009	.9985	.06617	2.749	1,124
.0008000	.01129	.07096	.07084	.07102	1.0025	.9975	.1419	.1424	1.010	.9983	.07072	2.659	983.5
.0009000	.01198	.07527	.07513	.07534	1.0028	.9972	.1505	.1511	1.011	.9981	.07499	2.582	874.3
.001000	.01263	.07935	.07918	.07943	1.0032	.9969	.1587	.1594	1.013	.9979	.07902	2.515	787.0
.001100	.01325	.08323	.08304	.08333	1.0035	.9966	.1665	.1672	1.014	.9977	.08285	2.456	715.6
.001200	.01384	.08694	.08672	.08705	1.0038	.9962	.1739	.1748	1.015	.9975	.08651	2.404	656.1
.001300	.01440	.09050	.09026	.09063	1.0041	.9959	.1810	.1820	1.016	.9973	.09001	2.357	605.8
.001400	.01495	.09393	.09365	.09407	1.0044	.9956	.1879	.1890	1.018	.9971	.09338	2.314	562.6
.001500	.01548	.09723	.09693	.09739	1.0047	.9953	.1945	.1957	1.019	.9969	.09663	2.275	525
.001600	.01598	.1004	.1001	.1006	1.0051	.9949	.2009	.2022	1.020	.9967	.09977	2.239	493
.001700	.01648	.1035	.1032	.1037	1.0054	.9946	.2071	.2086	1.022	.9965	.1028	2.205	463
.001800	.01696	.1066	.1062	.1068	1.0057	.9943	.2131	.2147	1.023	.9962	.1058	2.174	438
.001900	.01743	.1095	.1091	.1097	1.0060	.9940	.2190	.2207	1.024	.9960	.1087	2.145	415
.002000	.01788	.1123	.1119	.1125	1.0063	.9937	.2247	.2266	1.025	.9958	.1114	2.119	394
.002100	.01832	.1151	.1146	.1154	1.0066	.9934	.2303	.2323	1.027	.9956	.1141	2.094	376
.002200	.01876	.1178	.1173	.1181	1.0069	.9931	.2357	.2379	1.028	.9954	.1161	2.070	359
.002300	.01918	.1205	.1199	.1208	1.0073	.9928	.2410	.2433	1.029	.9952	.1193	2.047	343
.002400	.01959	.1231	.1225	.1234	1.0076	.9925	.2462	.2487	1.031	.9950	.1219	2.025	329
.002500	.02000	.1257	.1250	.1260	1.0079	.9922	.2513	.2540	1.032	.9948	.1243	2.005	316
.002600	.02040	.1282	.1275	.1285	1.0082	.9919	.2563	.2592	1.033	.9946	.1268	1.986	304
.002700	.02079	.1306	.1299	.1310	1.0085	.9916	.2612	.2642	1.034	.9944	.1292	1.967	292
.002800	.02117	.1330	.1323	.1334	1.0089	.9912	.2661	.2692	1.036	.9942	.1315	1.950	282
.002900	.02155	.1354	.1346	.1358	1.0092	.9909	.2708	.2741	1.037	.9939	.1338	1.933	272
.003000	.02192	.1377	.1369	.1382	1.0095	.9906	.2755	.2790	1.038	.9937	.1360	1.917	263
.003100	.02228	.1400	.1391	.1405	1.0098	.9903	.2800	.2837	1.040	.9935	.1382	1.902	255
.003200	.02264	.1423	.1413	.1427	1.0101	.9900	.2845	.2884	1.041	.9933	.1404	1.887	247
.003300	.02300	.1445	.1435	.1449	1.0104	.9897	.2890	.2930	1.042	.9931	.1425	1.873	240
.003400	.02335	.1467	.1456	.1472	1.0108	.9893	.2934	.2976	1.043	.9929	.1446	1.860	233
.003500	.02369	.1488	.1477	.1494	1.0111	.9890	.2977	.3021	1.045	.9927	.1466	1.847	226
.003600	.02403	.1510	.1498	.1515	1.0114	.9887	.3020	.3065	1.046	.9925	.1487	1.834	220
.003700	.02436	.1531	.1519	.1537	1.0117	.9884	.3061	.3109	1.047	.9923	.1507	1.822	214
.003800	.02469	.1551	.1539	.1558	1.0121	.9881	.3103	.3153	1.049	.9921	.1527	1.810	208
.003900	.02502	.1572	.1559	.1579	1.0124	.9878	.3144	.3196	1.050	.9919	.1546	1.799	203
.004000	.02534	.1592	.1579	.1599	1.0127	.9875	.3184	.3238	1.051	.9917	.1565	1.788	198
.004100	.02566	.1612	.1598	.1619	1.0130	.9872	.3224	.3280	1.052	.9915	.1584	1.777	193
.004200	.02597	.1632	.1617	.1639	1.0133	.9869	.3263	.3322	1.054	.9912	.1602	1.767	189
.004300	.02628	.1651	.1636	.1659	1.0137	.9865	.3302	.3362	1.055	.9910	.1621	1.756	184
.004400	.02659	.1671	.1655	.1678	1.0140	.9862	.3341	.3403	1.056	.9908	.1640	1.746	180
.004500	.02689	.1690	.1674	.1698	1.0143	.9859	.3380	.3444	1.058	.9906	.1658	1.737	176
.004600	.02719	.1708	.1692	.1717	1.0146	.9856	.3417	.3483	1.059	.9904	.1676	1.727	172
.004700	.02749	.1727	.1710	.1736	1.0149	.9853	.3454	.3523	1.060	.9902	.1693	1.718	169
.004800	.02778	.1745	.1728	.1754	1.0153	.9849	.3491	.3562	1.062	.9900	.1711	1.709	165
.004900	.02807	.1764	.1746	.1773	1.0156	.9846	.3527	.3601	1.063	.9898	.1728	1.701	162
.005000	.02836	.1782	.1764	.1791	1.0159	.9843	.3564	.3640	1.064	.9896	.1746	1.692	159
.005100	.02864	.1800	.1781	.1809	1.0162	.9840	.3599	.3678	1.066	.9894	.1762	1.684	156
.005200	.02893	.1818	.1798	.1827	1.0166	.9837	.3635	.3715	1.067	.9892	.1779	1.676	153
.005300	.02921	.1835	.1815	.1845	1.0169	.9834	.3670	.3753	1.068	.9889	.1795	1.669	150
.005400	.02948	.1852	.1832	.1863	1.0172	.9831	.3705	.3790	1.069	.9887	.1811	1.662	147
.005500	.02976	.1870	.1848	.1880	1.0175	.9828	.3739	.3827	1.071	.9885	.1827	1.654	145
.005600	.03003	.1887	.1865	.1898	1.0178	.9825	.3774	.3864	1.072	.9883	.1843	1.647	142
.005700	.03030	.1904	.1881	.1915	1.0182	.9822	.3808	.3900	1.073	.9881	.1859	1.640	140
.005800	.03057	.1921	.1897	.1932	1.0185	.9818	.3841	.3937	1.075	.9879	.1874	1.633	137
.005900	.03083	.1937	.1913	.1949	1.0188	.9815	.3875	.3972	1.076	.9877	.1890	1.626	135

*Also: b_s/a_s, C/C_o, L/L_o

d/L_o	d/L	$2\pi d/L$	TANH $2\pi d/L$	SINH $2\pi d/L$	COSH $2\pi d/L$	K	$4\pi d/L$	SINH $4\pi d/L$	COSH $4\pi d/L$	n	C_G/C_o	H/H_o'	M
.006000	.03110	.1954	.1929	.1967	1.0192	.9812	.3908	.4008	1.077	.9875	.1905	1.620	133
.006100	.03136	.1970	.1945	.1983	1.0195	.9809	.3941	.4044	1.079	.9873	.1920	1.614	130
.006200	.03162	.1987	.1961	.2000	1.0198	.9806	.3973	.4079	1.080	.9871	.1935	1.607	128
.006300	.03188	.2003	.1976	.2016	1.0201	.9803	.4006	.4114	1.081	.9869	.1950	1.601	126
.006400	.03213	.2019	.1992	.2033	1.0205	.9799	.4038	.4148	1.083	.9867	.1965	1.595	124
.006500	.03238	.2035	.2007	.2049	1.0208	.9796	.4070	.4183	1.084	.9865	.1980	1.589	123
.006600	.03264	.2051	.2022	.2065	1.0211	.9793	.4101	.4217	1.085	.9863	.1994	1.583	121
.006700	.03289	.2066	.2037	.2081	1.0214	.9790	.4133	.4251	1.087	.9860	.2009	1.578	119
.006800	.03313	.2082	.2052	.2097	1.0217	.9787	.4164	.4285	1.088	.9858	.2023	1.572	117
.006900	.03338	.2097	.2067	.2113	1.0221	.9784	.4195	.4319	1.089	.9856	.2037	1.567	116
.007000	.03362	.2113	.2082	.2128	1.0224	.9781	.4225	.4352	1.091	.9854	.2051	1.561	114
.007100	.03387	.2128	.2096	.2144	1.0227	.9778	.4256	.4386	1.092	.9852	.2065	1.556	112
.007200	.03411	.2143	.2111	.2160	1.0231	.9774	.4286	.4419	1.093	.9850	.2079	1.551	111
.007300	.03435	.2158	.2125	.2175	1.0234	.9771	.4316	.4452	1.095	.9848	.2093	1.546	109
.007400	.03459	.2173	.2139	.2190	1.0237	.9768	.4346	.4484	1.096	.9846	.2106	1.541	108
.007500	.03482	.2188	.2154	.2205	1.0240	.9765	.4376	.4517	1.097	.9844	.2120	1.536	106
.007600	.03506	.2203	.2168	.2221	1.0244	.9762	.4406	.4549	1.099	.9842	.2134	1.531	105
.007700	.03529	.2218	.2182	.2236	1.0247	.9759	.4435	.4582	1.100	.9840	.2147	1.526	104
.007800	.03552	.2232	.2196	.2251	1.0250	.9756	.4464	.4614	1.101	.9838	.2160	1.521	102
.007900	.03576	.2247	.2209	.2265	1.0253	.9753	.4493	.4646	1.103	.9836	.2173	1.517	101
.008000	.03598	.2261	.2223	.2280	1.0257	.9750	.4522	.4678	1.104	.9834	.2186	1.512	100
.008100	.03621	.2275	.2237	.2295	1.0260	.9747	.4551	.4709	1.105	.9832	.2199	1.508	98.6
.008200	.03644	.2290	.2250	.2310	1.0263	.9744	.4579	.4741	1.107	.9830	.2212	1.503	97.5
.008300	.03666	.2304	.2264	.2324	1.0266	.9741	.4607	.4772	1.108	.9827	.2225	1.499	96.3
.008400	.03689	.2318	.2277	.2338	1.0270	.9737	.4636	.4803	1.109	.9825	.2237	1.495	95.2
.008500	.03711	.2332	.2290	.2353	1.0273	.9734	.4664	.4834	1.111	.9823	.2250	1.491	94.1
.008600	.03733	.2346	.2303	.2367	1.0276	.9731	.4691	.4865	1.112	.9821	.2262	1.487	93.0
.008700	.03755	.2360	.2317	.2381	1.0280	.9728	.4719	.4896	1.113	.9819	.2275	1.482	91.9
.008800	.03777	.2373	.2330	.2396	1.0283	.9725	.4747	.4927	1.115	.9817	.2287	1.478	90.9
.008900	.03799	.2387	.2343	.2410	1.0286	.9722	.4774	.4957	1.116	.9815	.2300	1.474	89.9
.009000	.03821	.2401	.2356	.2424	1.0290	.9718	.4801	.4988	1.118	.9813	.2312	1.471	88.9
.009100	.03842	.2414	.2368	.2438	1.0293	.9815	.4828	.5018	1.119	.9811	.2324	1.467	88.0
.009200	.03864	.2428	.2381	.2452	1.0296	.9712	.4855	.5049	1.120	.9809	.2336	1.463	87.1
.009300	.03885	.2441	.2394	.2465	1.0299	.9709	.4882	.5079	1.122	.9807	.2348	1.459	86.1
.009400	.03906	.2455	.2407	.2479	1.0303	.9706	.4909	.5109	1.123	.9805	.2360	1.456	85.2
.009500	.03928	.2468	.2419	.2493	1.0306	.9703	.4936	.5138	1.124	.9803	.2371	1.452	84.3
.009600	.03949	.2481	.2431	.2507	1.0309	.9700	.4962	.5168	1.126	.9801	.2383	1.448	83.5
.009700	.03970	.2494	.2443	.2520	1.0313	.9697	.4988	.5198	1.127	.9799	.2394	1.445	82.7
.009800	.03990	.2507	.2456	.2534	1.0316	.9694	.5014	.5227	1.128	.9797	.2406	1.442	81.8
.009900	.04011	.2520	.2468	.2547	1.0319	.9691	.5040	.5257	1.130	.9794	.2417	1.438	81.0
.01000	.04032	.2533	.2480	.2560	1.0322	.9688	.5066	.5286	1.131	.9792	.2429	1.435	80.2
.01100	.04233	.2660	.2598	.2691	1.0356	.9656	.5319	.5574	1.145	.9772	.2539	1.403	73.1
.01200	.04426	.2781	.2711	.2817	1.0389	.9625	.5562	.5853	1.159	.9751	.2643	1.375	67.1
.01300	.04612	.2898	.2820	.2938	1.0423	.9594	.5795	.6125	1.173	.9731	.2743	1.350	62.1
.01400	.04791	.3010	.2924	.3056	1.0456	.9564	.6020	.6391	1.187	.9710	.2838	1.327	57.8
.01500	.04964	.3119	.3022	.3170	1.0490	.9533	.6238	.6651	1.201	.9690	.2928	1.307	54.0
.01600	.05132	.3225	.3117	.3281	1.0524	.9502	.6450	.6906	1.215	.9670	.3014	1.288	50.8
.01700	.05296	.3328	.3209	.3389	1.0559	.9471	.6655	.7158	1.230	.9649	.3096	1.271	47.9
.01800	.05455	.3428	.3298	.3495	1.0593	.9440	.6856	.7405	1.244	.9629	.3176	1.255	45.3
.01900	.05611	.3525	.3386	.3599	1.0628	.9409	.7051	.7650	1.259	.9609	.3253	1.240	43.0
.02000	.05763	.3621	.3470	.3701	1.0663	.9378	.7242	.7891	1.274	.9588	.3327	1.226	41.0
.02100	.05912	.3714	.3552	.3800	1.0698	.9348	.7429	.8131	1.289	.9568	.3399	1.213	39.1
.02200	.06057	.3806	.3632	.3898	1.0733	.9317	.7612	.8368	1.304	.9548	.3468	1.201	37.4
.02300	.06200	.3896	.3710	.3995	1.0768	.9287	.7791	.8603	1.319	.9528	.3535	1.189	35.9
.02400	.06340	.3984	.3786	.4090	1.0804	.9256	.7967	.8837	1.335	.9508	.3600	1.178	34.4
.02500	.06478	.4070	.3860	.4184	1.0840	.9225	.8140	.9069	1.350	.9488	.3662	1.168	33.1
.02600	.06613	.4155	.3932	.4276	1.0876	.9195	.8310	.9310	1.366	.9468	.3722	1.159	31.9
.02700	.06747	.4239	.4002	.4367	1.0912	.9164	.8478	.9530	1.381	.9448	.3781	1.150	30.8
.02800	.06878	.4322	.4071	.4457	1.0949	.9133	.8643	.9760	1.397	.9428	.3838	1.141	29.8
.02900	.07007	.4403	.4138	.4546	1.0985	.9103	.8805	.9988	1.413	.9408	.3893	1.133	28.8

d/L_o	d/L	$2\pi d/L$	TANH $2\pi d/L$	SINH $2\pi d/L$	COSH $2\pi d/L$	K	$4\pi d/L$	SINH $4\pi d/L$	COSH $4\pi d/L$	n	C_G/C_o	H/H'_o	M
.03000	.07135	.4483	.4205	.4634	1.1021	.9073	.8966	1.022	1.430	.9388	.3947	1.125	27.9
.03100	.07260	.4562	.4269	.4721	1.1059	.9042	.9124	1.044	1.446	.9369	.4000	1.118	27.1
.03200	.07385	.4640	.4333	.4808	1.1096	.9012	.9280	1.067	1.462	.9349	.4051	1.111	26.3
.03300	.07507	.4717	.4395	.4894	1.1133	.8982	.9434	1.090	1.479	.9329	.4100	1.104	25.6
.03400	.07630	.4794	.4457	.4980	1.1171	.8952	.9588	1.113	1.496	.9309	.4149	1.098	24.8
.03500	.07748	.4868	.4517	.5064	1.1209	.8921	.9737	1.135	1.513	.9289	.4196	1.092	24.19
.03600	.07867	.4943	.4577	.5147	1.1247	.8891	.9886	1.158	1.530	.9270	.4242	1.086	23.56
.03700	.07984	.5017	.4635	.5230	1.1285	.8861	1.0033	1.180	1.547	.9250	.4287	1.080	22.97
.03800	.08100	.5090	.4691	.5312	1.1324	.8831	1.018	1.203	1.564	.9230	.4330	1.075	22.42
.03900	.08215	.5162	.4747	.5394	1.1362	.8801	1.032	1.226	1.582	.9211	.4372	1.069	21.90
.04000	.08329	.5233	.4802	.5475	1.1401	.8771	1.047	1.248	1.600	.9192	.4414	1.064	21.40
.04100	.08442	.5304	.4857	.5556	1.1440	.8741	1.061	1.271	1.617	.9172	.4455	1.059	20.92
.04200	.08553	.5374	.4911	.5637	1.1479	.8711	1.075	1.294	1.636	.9153	.4495	1.055	20.46
.04300	.08664	.5444	.4964	.5717	1.1518	.8688	1.089	1.317	1.654	.9133	.4534	1.050	20.03
.04400	.08774	.5513	.5015	.5796	1.1558	.8652	1.103	1.340	1.672	.9114	.4571	1.046	19.62
.04500	.08883	.5581	.5066	.5876	1.1599	.8621	1.116	1.363	1.691	.9095	.4607	1.042	19.23
.04600	.08991	.5649	.5116	.5954	1.1639	.8592	1.130	1.386	1.709	.9076	.4643	1.038	18.85
.04700	.09098	.5717	.5166	.6033	1.1679	.8562	1.143	1.409	1.728	.9057	.4679	1.034	18.49
.04800	.09205	.5784	.5215	.6111	1.1720	.8532	1.157	1.433	1.747	.9037	.4713	1.030	18.15
.04900	.09311	.5850	.5263	.6189	1.1760	.8503	1.170	1.456	1.766	.9018	.4746	1.026	17.82
.05000	.09416	.5916	.5310	.6267	1.1802	.8473	1.183	1.479	1.786	.8999	.4779	1.023	17.50
.05100	.09520	.5981	.5357	.6344	1.1843	.8444	1.196	1.503	1.805	.8980	.4811	1.019	17.19
.05200	.09623	.6046	.5403	.6421	1.1884	.8415	1.209	1.526	1.825	.8961	.4842	1.016	16.90
.05300	.09726	.6111	.5449	.6499	1.1926	.8385	1.222	1.550	1.845	.8943	.4873	1.013	16.62
.05400	.09829	.6176	.5494	.6575	1.1968	.8356	1.235	1.574	1.865	.8924	.4903	1.010	16.35
.05500	.09930	.6239	.5538	.6652	1.2011	.8326	1.248	1.598	1.885	.8905	.4932	1.007	16.09
.05600	.1003	.6303	.5582	.6729	1.2053	.8297	1.261	1.622	1.906	.8886	.4960	1.004	15.84
.05700	.1013	.6366	.5626	.6805	1.2096	.8267	1.273	1.646	1.926	.8867	.4988	1.001	15.60
.05800	.1023	.6428	.5668	.6880	1.2138	.8239	1.286	1.670	1.947	.8849	.5015	.9985	15.36
.05900	.1033	.6491	.5711	.6956	1.2181	.8209	1.298	1.695	1.968	.8830	.5042	.9958	15.13
.06000	.1043	.6553	.5753	.7033	1.2225	.8180	1.311	1.719	1.989	.8811	.5068	.9932	14.91
.06100	.1053	.6616	.5794	.7110	1.2270	.8150	1.3231	1.744	2.011	.8792	.5094	.9907	14.70
.06200	.1063	.6678	.5834	.7187	1.2315	.8121	1.336	1.770	2.033	.8773	.5119	.9883	14.50
.06300	.1073	.6739	.5874	.7256	1.2355	.8093	1.348	1.795	2.055	.8755	.5143	.9860	14.30
.06400	.1082	.6799	.5914	.7335	1.2402	.8063	1.360	1.819	2.076	.8737	.5167	.9837	14.11
.06500	.1092	.6860	.5954	.7411	1.2447	.8035	1.372	1.845	2.098	.8719	.5191	.9815	13.92
.06600	.1101	.6920	.5993	.7486	1.2492	.8005	1.384	1.870	2.121	.8700	.5214	.9793	13.74
.06700	.1111	.6981	.6031	.7561	1.2537	.7977	1.396	1.896	2.144	.8682	.5236	.9772	13.57
.06800	.1120	.7037	.6069	.7633	1.2580	.7948	1.408	1.921	2.166	.8664	.5258	.9752	13.40
.06900	.1130	.7099	.6106	.7711	1.2628	.7919	1.420	1.948	2.189	.8646	.5279	.9732	13.24
.07000	.1139	.7157	.6144	.7783	1.2672	.7890	1.432	1.974	2.213	.8627	.5300	.9713	13.08
.07100	.1149	.7219	.6181	.7863	1.2721	.7861	1.444	2.000	2.236	.8609	.5321	.9694	12.92
.07200	.1158	.7277	.6217	.7937	1.2767	.7833	1.455	2.026	2.260	.8591	.5341	.9676	12.77
.07300	.1168	.7336	.6252	.8011	1.2813	.7804	1.467	2.053	2.284	.8572	.5360	.9658	12.62
.07400	.1177	.7395	.6289	.8088	1.2861	.7775	1.479	2.080	2.308	.8554	.5380	.9641	12.48
.07500	.1186	.7453	.6324	.8162	1.2908	.7747	1.490	2.107	2.332	.8537	.5399	.9624	12.34
.07600	.1195	.7511	.6359	.8237	1.2956	.7719	1.502	2.135	2.357	.8519	.5417	.9607	12.21
.07700	.1205	.7569	.6392	.8312	1.3004	.7690	1.514	2.162	2.382	.8501	.5435	.9591	12.08
.07800	.1214	.7625	.6427	.8386	1.3051	.7662	1.525	2.189	2.407	.8483	.5452	.9576	11.95
.07900	.1223	.7683	.6460	.8462	1.3100	.7634	1.537	2.217	2.432	.8465	.5469	.9562	11.83
.08000	.1232	.7741	.6493	.8538	1.3149	.7605	1.548	2.245	2.458	.8448	.5485	.9548	11.71
.08100	.1241	.7799	.6526	.8614	1.3198	.7577	1.560	2.274	2.484	.8430	.5501	.9534	11.59
.08200	.1251	.7854	.6558	.8687	1.3246	.7549	1.571	2.303	2.511	.8413	.5517	.9520	11.47
.08300	.1259	.7911	.6590	.8762	1.3295	.7522	1.583	2.331	2.537	.8395	.5533	.9506	11.36
.08400	.1268	.7967	.6622	.8837	1.3345	.7494	1.594	2.360	2.563	.8378	.5548	.9493	11.25
.08500	.1277	.8026	.6655	.8915	1.3397	.7464	1.605	2.389	2.590	.8360	.5563	.9481	11.14
.08600	.1286	.8080	.6685	.8989	1.3446	.7437	1.616	2.418	2.617	.8342	.5577	.9469	11.04
.08700	.1295	.8137	.6716	.9064	1.3497	.7409	1.628	2.448	2.644	.8325	.5591	.9457	10.94
.08800	.1304	.8193	.6747	.9141	1.3548	.7381	1.639	2.478	2.672	.8308	.5605	.9445	10.84
.08900	.1313	.8250	.6778	.9218	1.3600	.7353	1.650	2.508	2.700	.8290	.5619	.9433	10.74

Table D-1 Cont'd

d/L_o	d/L	$2\pi d/L$	TANH $2\pi d/L$	SINH $2\pi d/L$	COSH $2\pi d/L$	K	$4\pi d/L$	SINH $4\pi d/L$	COSH $4\pi d/L$	n	C_G/C_o	H/H_o'	M
.09000	.1322	.8306	.6808	.9295	1.3653	.7324	1.661	2.538	2.728	.8273	.5632	.9422	10.65
.09100	.1331	.8363	.6838	.9372	1.3706	.7296	1.672	2.568	2.756	.8255	.5645	.9411	10.55
.09200	.1340	.8420	.6868	.9450	1.3759	.7268	1.684	2.599	2.785	.8238	.5658	.9401	10.46
.09300	.1349	.8474	.6897	.9525	1.3810	.7241	1.695	2.630	2.814	.8221	.5670	.9391	10.37
.09400	.1357	.8528	.6925	.9600	1.3862	.7214	1.706	2.662	2.843	.8204	.5682	.9381	10.29
.09500	.1366	.8583	.6953	.9677	1.3917	.7186	1.717	2.693	2.873	.8187	.5693	.9371	10.21
.09600	.1375	.8639	.6982	.9755	1.3970	.7158	1.728	2.726	2.903	.8170	.5704	.9362	10.12
.09700	.1384	.8694	.7011	.9832	1.4023	.7131	1.739	2.757	2.933	.8153	.5716	.9353	10.04
.09800	.1392	.8749	.7039	.9908	1.4077	.7104	1.750	2.790	2.963	.8136	.5727	.9344	9.962
.09900	.1401	.8803	.7066	.9985	1.4131	.7076	1.761	2.822	2.994	.8120	.5737	.9335	9.884
.1000	.1410	.8858	.7093	1.006	1.4187	.7049	1.772	2.855	3.025	.8103	.5747	.9327	9.808
.1010	.1419	.8913	.7120	1.014	1.4242	.7022	1.783	2.888	3.057	.8086	.5757	.9319	9.734
.1020	.1427	.8967	.7147	1.022	1.4297	.6994	1.793	2.922	3.088	.8069	.5766	.9311	9.661
.1030	.1436	.9023	.7173	1.030	1.4354	.6967	1.805	2.956	3.121	.8052	.5776	.9304	9.590
.1040	.1445	.9076	.7200	1.037	1.4410	.6940	1.815	2.990	3.153	.8036	.5785	.9297	9.519
.1050	.1453	.9130	.7226	1.045	1.4465	.6913	1.826	3.024	3.185	.8019	.5794	.9290	9.451
.1060	.1462	.9184	.7252	1.053	1.4523	.6886	1.837	3.059	3.218	.8003	.5803	.9282	9.384
.1070	.1470	.9239	.7277	1.061	1.4580	.6859	1.848	3.094	3.251	.7986	.5812	.9276	9.318
.1080	.1479	.9293	.7303	1.069	1.4638	.6833	1.858	3.128	3.284	.7970	.5820	.9269	9.254
.1090	.1488	.9343	.7327	1.076	1.4692	.6806	1.869	3.164	3.319	.7954	.5828	.9263	9.191
.1100	.1496	.9400	.7352	1.085	1.4752	.6779	1.880	3.201	3.353	.7937	.5836	.9257	9.129
.1110	.1505	.9456	.7377	1.093	1.4814	.6752	1.891	3.237	3.388	.7920	.5843	.9251	9.068
.1120	.1513	.9508	.7402	1.101	1.4871	.6725	1.902	3.274	3.423	.7904	.5850	.9245	9.009
.1130	.1522	.9563	.7426	1.109	1.4932	.6697	1.913	3.312	3.459	.7888	.5857	.9239	8.950
.1140	.1530	.9616	.7450	1.117	1.4990	.6671	1.923	3.348	3.494	.7872	.5864	.9234	8.891
.1150	.1539	.9670	.7474	1.125	1.5051	.6645	1.934	3.385	3.530	.7856	.5871	.9228	8.835
.1160	.1547	.9720	.7497	1.133	1.5108	.6619	1.944	3.423	3.566	.7840	.5878	.9223	8.780
.1170	.1556	.9775	.7520	1.141	1.5171	.6592	1.955	3.462	3.603	.7824	.5884	.9218	8.726
.1180	.1564	.9827	.7543	1.149	1.5230	.6566	1.966	3.501	3.641	.7808	.5890	.9214	8.673
.1190	.1573	.9882	.7566	1.157	1.5293	.6539	1.977	3.540	3.678	.7792	.5896	.9209	8.621
.1200	.1581	.9936	.7589	1.165	1.5356	.6512	1.987	3.579	3.716	.7776	.5902	.9204	8.569
.1210	.1590	.9989	.7612	1.174	1.5418	.6486	1.998	3.620	3.755	.7760	.5907	.9200	8.518
.1220	.1598	1.004	.7634	1.182	1.5479	.6460	2.008	3.659	3.793	.7745	.5913	.9196	8.468
.1230	.1607	1.010	.7656	1.190	1.5546	.6433	2.019	3.699	3.832	.7729	.5918	.9192	8.419
.1240	.1615	1.015	.7678	1.198	1.5605	.6407	2.030	3.740	3.871	.7713	.5922	.9189	8.371
.1250	.1624	1.020	.7700	1.207	1.5674	.6381	2.041	3.782	3.912	.7698	.5926	.9186	8.324
.1260	.1632	1.025	.7721	1.215	1.5734	.6356	2.051	3.824	3.952	.7682	.5931	.9182	8.278
.1270	.1640	1.030	.7742	1.223	1.5795	.6331	2.061	3.865	3.992	.7667	.5936	.9178	8.233
.1280	.1649	1.036	.7763	1.231	1.5862	.6305	2.072	3.907	4.033	.7652	.5940	.9175	8.189
.1290	.1657	1.041	.7783	1.240	1.5927	.6279	2.082	3.950	4.074	.7637	.5944	.9172	8.146
.1300	.1665	1.046	.7804	1.248	1.5990	.6254	2.093	3.992	4.115	.7621	.5948	.9169	8.103
.1310	.1674	1.052	.7824	1.257	1.6060	.6228	2.104	4.036	4.158	.7606	.5951	.9166	8.061
.1320	.1682	1.057	.7844	1.265	1.6124	.6202	2.114	4.080	4.201	.7591	.5954	.9164	8.020
.1330	.1691	1.062	.7865	1.273	1.6191	.6176	2.125	4.125	4.245	.7575	.5958	.9161	7.978
.1340	.1699	1.068	.7885	1.282	1.6260	.6150	2.135	4.169	4.288	.7560	.5961	.9158	7.937
.1350	.1708	1.073	.7905	1.291	1.633	.6123	2.146	4.217	4.334	.7545	.5964	.9156	7.897
.1360	.1716	1.078	.7925	1.300	1.640	.6098	2.156	4.262	4.378	.7530	.5967	.9154	7.857
.1370	.1724	1.084	.7945	1.308	1.647	.6073	2.167	4.309	4.423	.7515	.5969	.9152	7.819
.1380	.1733	1.089	.7964	1.317	1.654	.6047	2.177	4.355	4.468	.7500	.5972	.9150	7.781
.1390	.1741	1.094	.7983	1.326	1.660	.6022	2.188	4.402	4.514	.7485	.5975	.9148	7.744
.1400	.1749	1.099	.8002	1.334	1.667	.5998	2.198	4.450	4.561	.7471	.5978	.9146	7.707
.1410	.1758	1.105	.8021	1.343	1.675	.5972	2.209	4.498	4.607	.7456	.5980	.9144	7.671
.1420	.1766	1.110	.8039	1.352	1.681	.5947	2.219	4.546	4.654	.7441	.5982	.9142	7.636
.1430	.1774	1.115	.8057	1.360	1.688	.5923	2.230	4.595	4.663	.7426	.5984	.9141	7.602
.1440	.1783	1.120	.8076	1.369	1.696	.5898	2.240	4.644	4.751	.7412	.5986	.9140	7.567
.1450	.1791	1.125	.8094	1.378	1.703	.5873	2.251	4.695	4.800	.7397	.5987	.9139	7.533
.1460	.1800	1.131	.8112	1.388	1.710	.5847	2.261	4.746	4.850	.7382	.5989	.9137	7.499
.1470	.1808	1.136	.8131	1.397	1.718	.5822	2.272	4.798	4.901	.7368	.5990	.9136	7.465
.1480	.1816	1.141	.8149	1.405	1.725	.5798	2.282	4.847	4.951	.7354	.5992	.9135	7.432
.1490	.1825	1.146	.8166	1.415	1.732	.5773	2.293	4.901	5.001	.7339	.5993	.9134	7.400

Table D-1 Cont'd

d/L_o	d/L	$2\pi d/L$	TANH $2\pi d/L$	SINH $2\pi d/L$	COSH $2\pi d/L$	K	$4\pi d/L$	SINH $4\pi d/L$	COSH $4\pi d/L$	n	C_G/C_o	H/H'_o	M
.1500	.1833	1.152	.8183	1.424	1.740	.5748	2.303	4.954	5.054	.7325	.5994	.9133	7.369
.1510	.1841	1.157	.8200	1.433	1.747	.5723	2.314	5.007	5.106	.7311	.5994	.9133	7.339
.1520	.1850	1.162	.8217	1.442	1.755	.5699	2.324	5.061	5.159	.7296	.5995	.9132	7.309
.1530	.1858	1.167	.8234	1.451	1.762	.5675	2.335	5.115	5.212	.7282	.5996	.9132	7.279
.1540	.1866	1.173	.8250	1.460	1.770	.5651	2.345	5.169	5.265	.7268	.5996	.9132	7.250
.1550	.1875	1.178	.8267	1.469	1.777	.5627	2.356	5.225	5.320	.7254	.5997	.9131	7.221
.1560	.1883	1.183	.8284	1.479	1.785	.5602	2.366	5.283	5.376	.7240	.5998	.9130	7.191
.1570	.1891	1.188	.8301	1.488	1.793	.5577	2.377	5.339	5.432	.7226	.5999	.9129	7.162
.1580	.1900	1.194	.8317	1.498	1.801	.5552	2.387	5.398	5.490	.7212	.5998	.9130	7.134
.1590	.1908	1.199	.8333	1.507	1.809	.5528	2.398	5.454	5.544	.7198	.5998	.9130	7.107
.1600	.1917	1.204	.8349	1.517	1.817	.5504	2.408	5.513	5.603	.7184	.5998	.9130	7.079
.1610	.1925	1.209	.8365	1.527	1.825	.5480	2.419	5.571	5.660	.7171	.5998	.9130	7.052
.1620	.1933	1.215	.8381	1.536	1.833	.5456	2.429	5.630	5.718	.7157	.5998	.9130	7.026
.1630	.1941	1.220	.8396	1.546	1.841	.5432	2.440	5.690	5.777	.7144	.5998	.9130	7.000
.1640	.1950	1.225	.8411	1.555	1.849	.5409	2.450	5.751	5.837	.7130	.5998	.9130	6.975
.1650	.1958	1.230	.8427	1.565	1.857	.5385	2.461	5.813	5.898	.7117	.5997	.9131	6.949
.1660	.1966	1.235	.8442	1.574	1.865	.5362	2.471	5.874	5.959	.7103	.5996	.9132	6.924
.1670	.1975	1.240	.8457	1.584	1.873	.5339	2.482	5.938	6.021	.7090	.5996	.9132	6.900
.1680	.1983	1.246	.8472	1.594	1.882	.5315	2.492	6.003	6.085	.7076	.5995	.9133	6.876
.1690	.1992	1.251	.8486	1.604	1.890	.5291	2.503	6.066	6.148	.7063	.5994	.9133	6.853
.1700	.2000	1.257	.8501	1.614	1.899	.5267	2.513	6.130	6.212	.7050	.5993	.9134	6.830
.1710	.2008	1.262	.8515	1.624	1.907	.5243	2.523	6.197	6.275	.7036	.5992	.9135	6.807
.1720	.2017	1.267	.8529	1.634	1.915	.5220	2.534	6.262	6.342	.7023	.5991	.9136	6.784
.1730	.2025	1.272	.8544	1.644	1.924	.5197	2.544	6.329	6.407	.7010	.5989	.9137	6.761
.1740	.2033	1.277	.8558	1.654	1.933	.5174	2.555	6.395	6.473	.6997	.5988	.9138	6.738
.1750	.2042	1.282	.8572	1.664	1.941	.5151	2.565	6.465	6.541	.6984	.5987	.9139	6.716
.1760	.2050	1.288	.8586	1.675	1.951	.5127	2.576	6.534	6.610	.6971	.5985	.9140	6.694
.1770	.2058	1.293	.8600	1.685	1.959	.5104	2.586	6.603	6.679	.6958	.5984	.9141	6.672
.1780	.2066	1.298	.8614	1.695	1.968	.5081	2.597	6.672	6.747	.6946	.5982	.9142	6.651
.1790	.2075	1.304	.8627	1.706	1.977	.5058	2.607	6.744	6.818	.6933	.5980	.9144	6.631
.1800	.2083	1.309	.8640	1.716	1.986	.5036	2.618	6.818	6.891	.6920	.5979	.9145	6.611
.1810	.2092	1.314	.8653	1.727	1.995	.5013	2.629	6.890	6.963	.6907	.5977	.9146	6.591
.1820	.2100	1.320	.8666	1.737	2.004	.4990	2.639	6.963	7.035	.6895	.5975	.9148	6.571
.1830	.2108	1.325	.8680	1.748	2.013	.4967	2.650	7.038	7.109	.6882	.5974	.9149	6.550
.1840	.2117	1.330	.8693	1.758	2.022	.4945	2.660	7.113	7.183	.6870	.5972	.9150	6.530
.1850	.2125	1.335	.8706	1.769	2.032	.4922	2.671	7.191	7.260	.6857	.5969	.9152	6.511
.1860	.2134	1.341	.8718	1.780	2.041	.4899	2.681	7.267	7.336	.6845	.5967	.9154	6.492
.1870	.2142	1.346	.8731	1.791	2.051	.4876	2.692	7.345	7.412	.6832	.5965	.9155	6.474
.1880	.2150	1.351	.8743	1.801	2.060	.4854	2.702	7.421	7.488	.6820	.5963	.9157	6.456
.1890	.2159	1.356	.8755	1.812	2.070	.4832	2.712	7.500	7.566	.6808	.5961	.9159	6.438
.1900	.2167	1.362	.8767	1.823	2.079	.4809	2.723	7.581	7.647	.6796	.5958	.9161	6.421
.1910	.2176	1.367	.8779	1.834	2.089	.4787	2.734	7.663	7.728	.6784	.5955	.9163	6.403
.1920	.2184	1.372	.8791	1.845	2.099	.4765	2.744	7.746	7.810	.6772	.5952	.9165	6.385
.1930	.2192	1.377	.8803	1.856	2.108	.4743	2.755	7.827	7.891	.6760	.5950	.9167	6.368
.1940	.2201	1.383	.8815	1.867	2.118	.4721	2.765	7.911	7.974	.6748	.5948	.9169	6.351
.1950	.2209	1.388	.8827	1.879	2.128	.4699	2.776	7.996	8.059	.6736	.5946	.9170	6.334
.1960	.2218	1.393	.8839	1.890	2.138	.4677	2.787	8.083	8.145	.6724	.5944	.9172	6.317
.1970	.2226	1.399	.8850	1.901	2.148	.4655	2.797	8.167	8.228	.6712	.5941	.9174	6.300
.1980	.2234	1.404	.8862	1.913	2.158	.4633	2.808	8.256	8.316	.6700	.5938	.9176	6.284
.1990	.2243	1.409	.8873	1.924	2.169	.4611	2.819	8.346	8.406	.6689	.5935	.9179	6.268
.2000	.2251	1.414	.8884	1.935	2.178	.4590	2.829	8.436	8.495	.6677	.5932	.9181	6.253
.2010	.2260	1.420	.8895	1.947	2.189	.4569	2.840	8.524	8.583	.6666	.5929	.9183	6.237
.2020	.2268	1.425	.8906	1.959	2.199	.4547	2.850	8.616	8.674	.6654	.5926	.9186	6.222
.2030	.2277	1.430	.8917	1.970	2.210	.4526	2.861	8.708	8.766	.6642	.5923	.9188	6.206
.2040	.2285	1.436	.8928	1.982	2.220	.4504	2.872	8.803	8.860	.6631	.5920	.9190	6.191
.2050	.2293	1.441	.8939	1.994	2.231	.4483	2.882	8.897	8.953	.6620	.5917	.9193	6.176
.2060	.2302	1.446	.8950	2.006	2.242	.4462	2.893	8.994	9.050	.6608	.5914	.9195	6.161
.2070	.2310	1.451	.8960	2.017	2.252	.4441	2.903	9.090	9.144	.6597	.5911	.9197	6.147
.2080	.2319	1.457	.8971	2.030	2.263	.4419	2.914	9.187	9.240	.6586	.5908	.9200	6.133
.2090	.2328	1.462	.8981	2.042	2.274	.4398	2.925	9.288	9.342	.6574	.5905	.9202	6.119

Table D-1 Cont'd

d/L_o	d/L	$2\pi d/L$	TANH $2\pi d/L$	SINH $2\pi d/L$	COSH $2\pi d/L$	K	$4\pi d/L$	SINH $4\pi d/L$	COSH $4\pi d/L$	n	C_G/C_o	H/H'_o	M
.2100	.2336	1.468	.8991	2.055	2.285	.4377	2.936	9.389	9.442	.6563	.5901	.9205	6.105
.2110	.2344	1.473	.9001	2.066	2.295	.4357	2.946	9.490	9.542	.6552	.5898	.9207	6.091
.2120	.2353	1.479	.9011	2.079	2.307	.4336	2.957	9.590	9.642	.6541	.5894	.9210	6.077
.2130	.2361	1.484	.9021	2.091	2.318	.4315	2.967	9.693	9.744	.6531	.5891	.9213	6.064
.2140	.2370	1.489	.9031	2.103	2.329	.4294	2.978	9.796	9.847	.6520	.5888	.9215	6.051
.2150	.2378	1.494	.9041	2.115	2.340	.4274	2.989	9.902	9.952	.6509	.5884	.9218	6.037
.2160	.2387	1.500	.9051	2.128	2.351	.4253	2.999	10.01	10.06	.6498	.5881	.9221	6.024
.2170	.2395	1.506	.9061	2.142	2.364	.4232	3.010	10.12	10.17	.6488	.5878	.9223	6.011
.2180	.2404	1.511	.9070	2.154	2.375	.4211	3.021	10.23	10.28	.6477	.5874	.9226	5.999
.2190	.2412	1.516	.9079	2.166	2.386	.4191	3.031	10.34	10.38	.6467	.5871	.9228	5.987
.2200	.2421	1.521	.9088	2.178	2.397	.4171	3.042	10.45	10.50	.6456	.5868	.9231	5.975
.2210	.2429	1.526	.9097	2.192	2.409	.4151	3.052	10.56	10.61	.6446	.5864	.9234	5.963
.2220	.2438	1.532	.9107	2.204	2.421	.4131	3.063	10.68	10.72	.6436	.5861	.9236	5.951
.2230	.2446	1.537	.9116	2.218	2.433	.4111	3.074	10.79	10.84	.6425	.5857	.9239	5.939
.2240	.2455	1.542	.9125	2.230	2.444	.4091	3.085	10.91	10.95	.6414	.5854	.9242	5.927
.2250	.2463	1.548	.9134	2.244	2.457	.4071	3.095	11.02	11.07	.6404	.5850	.9245	5.915
.2260	.2472	1.553	.9143	2.257	2.469	.4051	3.106	11.15	11.19	.6394	.5846	.9248	5.903
.2270	.2481	1.559	.9152	2.271	2.481	.4031	3.117	11.27	11.31	.6383	.5842	.9251	5.891
.2280	.2489	1.564	.9161	2.284	2.493	.4011	3.128	11.39	11.44	.6373	.5838	.9254	5.880
.2290	.2498	1.569	.9170	2.297	2.506	.3991	3.138	11.51	11.56	.6363	.5834	.9258	5.869
.2300	.2506	1.575	.9178	2.311	2.518	.3971	3.149	11.64	11.68	.6353	.5830	.9261	5.858
.2310	.2515	1.580	.9186	2.325	2.531	.3952	3.160	11.77	11.81	.6343	.5826	.9264	5.848
.2320	.2523	1.585	.9194	2.338	2.543	.3932	3.171	11.90	11.93	.6333	.5823	.9267	5.838
.2330	.2532	1.591	.9203	2.352	2.556	.3912	3.182	12.03	12.07	.6323	.5819	.9270	5.827
.2340	.2540	1.596	.9211	2.366	2.569	.3893	3.192	12.15	12.19	.6313	.5815	.9273	5.816
.2350	.2549	1.602	.9219	2.380	2.581	.3874	3.203	12.29	12.33	.6304	.5811	.9276	5.806
.2360	.2558	1.607	.9227	2.393	2.594	.3855	3.214	12.43	12.47	.6294	.5807	.9279	5.796
.2370	.2566	1.612	.9235	2.408	2.607	.3836	3.225	12.55	12.59	.6284	.5804	.9282	5.786
.2380	.2575	1.618	.9243	2.422	2.620	.3816	3.236	12.69	12.73	.6275	.5800	.9285	5.776
.2390	.2584	1.623	.9251	2.436	2.634	.3797	3.247	12.83	12.87	.6265	.5796	.9288	5.766
.2400	.2592	1.629	.9259	2.450	2.647	.3779	3.257	12.97	13.01	.6256	.5792	.9291	5.756
.2410	.2601	1.634	.9267	2.464	2.660	.3760	3.268	13.11	13.15	.6246	.5788	.9294	5.746
.2420	.2610	1.640	.9275	2.480	2.674	.3741	3.279	13.26	13.30	.6237	.5784	.9298	5.736
.2430	.2618	1.645	.9282	2.494	2.687	.3722	3.290	13.40	13.44	.6228	.5780	.9301	5.727
.2440	.2627	1.650	.9289	2.508	2.700	.3704	3.301	13.55	13.59	.6218	.5776	.9304	5.718
.2450	.2635	1.656	.9296	2.523	2.714	.3685	3.312	13.70	13.73	.6209	.5272	.9307	5.710
.2460	.2644	1.661	.9304	2.538	2.728	.3666	3.323	13.85	13.88	.6200	.5768	.9310	5.701
.2470	.2653	1.667	.9311	2.553	2.742	.3648	3.334	14.00	14.04	.6191	.5764	.9314	5.692
.2480	.2661	1.672	.9318	2.568	2.755	.3629	3.344	14.15	14.19	.6182	.5760	.9317	5.684
.2490	.2670	1.678	.9325	2.583	2.770	.3610	3.355	14.31	14.35	.6173	.5756	.9320	5.675
.2500	.2679	1.683	.9332	2.599	2.784	.3592	3.367	14.47	14.51	.6164	.5752	.9323	5.667
.2510	.2687	1.689	.9339	2.614	2.798	.3574	3.377	14.62	14.66	.6155	.5748	.9327	5.658
.2520	.2696	1.694	.9346	2.629	2.813	.3556	3.388	14.79	14.82	.6146	.5744	.9330	5.650
.2530	.2705	1.700	.9353	2.645	2.828	.3537	3.399	14.95	14.99	.6137	.5740	.9333	5.641
.2540	.2714	1.705	.9360	2.660	2.842	.3519	3.410	15.12	15.15	.6128	.5736	.9336	5.633
.2550	.2722	1.711	.9367	2.676	2.856	.3501	3.421	15.29	15.32	.6120	.5732	.9340	5.624
.2560	.2731	1.716	.9374	2.691	2.871	.3483	3.432	15.45	15.49	.6111	.5728	.9343	5.616
.2570	.2740	1.722	.9381	2.707	2.886	.3465	3.443	15.63	15.66	.6102	.5724	.9346	5.608
.2580	.2749	1.727	.9388	2.723	2.901	.3447	3.454	15.80	15.83	.6093	.5720	.9349	5.600
.2590	.2757	1.732	.9394	2.739	2.916	.3430	3.465	15.97	16.00	.6085	.5716	.9353	5.592
.2600	.2766	1.738	.9400	2.755	2.931	.3412	3.476	16.15	16.18	.6076	.5712	.9356	5.585
.2610	.2775	1.744	.9406	2.772	2.946	.3394	3.487	16.33	16.36	.6068	.5707	.9360	5.578
.2620	.2784	1.749	.9412	2.788	2.962	.3376	3.498	16.51	16.54	.6060	.5703	.9363	5.571
.2630	.2792	1.755	.9418	2.804	2.977	.3359	3.509	16.69	16.73	.6052	.5699	.9367	5.563
.2640	.2801	1.760	.9425	2.820	2.992	.3342	3.520	16.88	16.91	.6043	.5695	.9370	5.556
.2650	.2810	1.766	.9431	2.837	3.008	.3325	3.531	17.07	17.10	.6035	.5691	.9373	5.548
.2660	.2819	1.771	.9437	2.853	3.023	.3308	3.542	17.26	17.28	.6027	.5687	.9377	5.541
.2670	.2827	1.776	.9443	2.870	3.039	.3291	3.553	17.45	17.45	.6018	.5683	.9380	5.534
.2680	.2836	1.782	.9449	2.886	3.055	.3274	3.564	17.64	17.67	.6010	.5679	.9383	5.527
.2690	.2845	1.788	.9455	2.904	3.071	.3256	3.575	17.84	17.87	.6002	.5675	.9386	5.520

Table D-1 Cont'd

d/L_o	d/L	$2\pi d/L$	TANH $2\pi d/L$	SINH $2\pi d/L$	COSH $2\pi d/L$	K	$4\pi d/L$	SINH $4\pi d/L$	COSH $4\pi d/L$	n	C_G/C_o	H/H_o'	M
.2700	.2854	1.793	.9461	2.921	3.088	.3239	3.587	18.04	18.07	.5994	.5671	.9390	5.513
.2710	.2863	1.799	.9467	2.938	3.104	.3222	3.598	18.24	18.27	.5986	.5667	.9393	5.506
.2720	.2872	1.804	.9473	2.956	3.120	.3205	3.610	18.46	18.49	.5978	.5663	.9396	5.499
.2730	.2880	1.810	.9478	2.973	3.136	.3189	3.620	18.65	18.67	.5971	.5659	.9400	5.493
.2740	.2889	1.815	.9484	2.990	3.153	.3172	3.631	18.86	18.89	.5963	.5655	.9403	5.486
.2750	.2898	1.821	.9490	3.008	3.170	.3155	3.642	19.07	19.10	.5955	.5651	.9406	5.480
.2760	.2907	1.826	.9495	3.025	3.186	.3139	3.653	19.28	19.30	.5947	.5647	.9410	5.474
.2770	.2916	1.832	.9500	3.043	3.203	.3122	3.664	19.49	19.51	.5940	.5643	.9413	5.468
.2780	.2924	1.837	.9505	3.061	3.220	.3106	3.675	19.71	19.74	.5932	.5639	.9416	5.462
.2790	.2933	1.843	.9511	3.079	3.237	.3089	3.686	19.93	19.96	.5925	.5635	.9420	5.456
.2800	.2942	1.849	.9516	3.097	3.254	.3073	3.697	20.16	20.18	.5917	.5631	.9423	5.450
.2810	.2951	1.854	.9521	3.115	3.272	.3057	3.709	20.39	20.41	.5910	.5627	.9426	5.444
.2820	.2960	1.860	.9526	3.133	3.289	.3040	3.720	20.62	20.64	.5902	.5623	.9430	5.438
.2830	.2969	1.866	.9532	3.152	3.307	.3024	3.731	20.85	20.87	.5895	.5619	.9433	5.432
.2840	.2978	1.871	.9537	3.171	3.325	.3008	3.742	21.09	21.11	.5887	.5615	.9436	5.426
.2850	.2987	1.877	.9542	3.190	3.343	.2992	3.754	21.33	21.35	.5880	.5611	.9440	5.420
.2860	.2996	1.882	.9547	3.209	3.361	.2976	3.765	21.57	21.59	.5873	.5607	.9443	5.414
.2870	.3005	1.888	.9552	3.228	3.379	.2959	3.776	21.82	21.84	.5866	.5603	.9446	5.409
.2880	.3014	1.893	.9557	3.246	3.396	.2944	3.787	22.05	22.07	.5859	.5600	.9449	5.403
.2890	.3022	1.899	.9562	3.264	3.414	.2929	3.798	22.30	22.32	.5852	.5596	.9452	5.397
.2900	.3031	1.905	.9567	3.284	3.433	.2913	3.809	22.54	22.57	.5845	.5592	.9456	5.392
.2910	.3040	1.910	.9572	3.303	3.451	.2898	3.821	22.81	22.83	.5838	.5588	.9459	5.386
.2920	.3049	1.916	.9577	3.323	3.471	.2882	3.832	23.07	23.09	.5831	.5584	.9463	5.380
.2930	.3058	1.922	.9581	3.343	3.490	.2866	3.843	23.33	23.35	.5824	.5580	.9466	5.375
.2940	.3067	1.927	.9585	3.362	3.508	.2851	3.855	23.60	23.62	.5817	.5576	.9469	5.371
.2950	.3076	1.933	.9590	3.382	3.527	.2835	3.866	23.86	23.88	.5810	.5572	.9473	5.366
.2960	.3085	1.938	.9594	3.402	3.546	.2820	3.877	24.12	24.15	.5804	.5568	.9476	5.361
.2970	.3094	1.944	.9599	3.422	3.565	.2805	3.888	24.40	24.42	.5797	.5564	.9480	5.356
.2980	.3103	1.950	.9603	3.442	3.585	.2790	3.900	24.68	24.70	.5790	.5560	.9483	5.351
.2990	.3112	1.955	.9607	3.462	3.604	.2775	3.911	24.96	24.98	.5784	.5556	.9486	5.347
.3000	.3121	1.961	.9611	3.483	3.624	.2760	3.922	25.24	25.26	.5777	.5552	.9490	5.342
.3010	.3130	1.967	.9616	3.503	3.643	.2745	3.933	25.53	25.55	.5771	.5549	.9493	5.337
.3020	.3139	1.972	.9620	3.524	3.663	.2730	3.945	25.82	25.83	.5764	.5545	.9496	5.332
.3030	.3148	1.978	.9624	3.545	3.683	.2715	3.956	26.12	26.14	.5758	.5541	.9499	5.328
.3040	.3157	1.984	.9629	3.566	3.703	.2700	3.968	26.42	26.44	.5751	.5538	.9502	5.323
.3050	.3166	1.989	.9633	3.587	3.724	.2685	3.979	26.72	26.74	.5745	.5534	.9505	5.318
.3060	.3175	1.995	.9637	3.609	3.745	.2670	3.990	27.02	27.04	.5739	.5530	.9509	5.314
.3070	.3184	2.001	.9641	3.630	3.765	.2656	4.002	27.33	27.35	.5732	.5527	.9512	5.309
.3080	.3193	2.007	.9645	3.651	3.786	.2641	4.013	27.65	27.66	.5726	.5523	.9515	5.305
.3090	.3202	2.012	.9649	3.673	3.806	.2627	4.024	27.96	27.98	.5720	.5519	.9518	5.300
.3100	.3211	2.018	.9653	3.694	3.827	.2613	4.036	28.28	28.30	.5714	.5515	.9522	5.296
.3110	.3220	2.023	.9656	3.716	3.848	.2599	4.047	28.60	28.62	.5708	.5511	.9525	5.292
.3120	.3230	2.029	.9660	3.738	3.870	.2584	4.058	28.93	28.95	.5701	.5507	.9528	5.288
.3130	.3239	2.035	.9664	3.760	3.891	.2570	4.070	29.27	29.28	.5695	.5504	.9531	5.284
.3140	.3248	2.041	.9668	3.782	3.912	.2556	4.081	29.60	29.62	.5689	.5500	.9535	5.280
.3150	.3257	2.046	.9672	3.805	3.934	.2542	4.093	29.94	29.96	.5683	.5497	.9538	5.276
.3160	.3266	2.052	.9676	3.828	3.956	.2528	4.104	30.29	30.31	.5678	.5494	.9541	5.272
.3170	.3275	2.058	.9679	3.851	3.978	.2514	4.116	30.64	30.65	.5672	.5490	.9544	5.268
.3180	.3284	2.063	.9682	3.873	4.000	.2500	4.127	30.99	31.00	.5666	.5486	.9547	5.264
.3190	.3294	2.069	.9686	3.896	4.022	.2486	4.139	31.35	31.37	.5660	.5483	.9550	5.260
.3200	.3302	2.075	.9690	3.919	4.045	.2472	4.150	31.71	31.72	.5655	.5479	.9553	5.256
.3210	.3311	2.081	.9693	3.943	4.068	.2459	4.161	32.07	32.08	.5649	.5476	.9556	5.252
.3220	.3321	2.086	.9696	3.966	4.090	.2445	4.173	32.44	32.46	.5643	.5472	.9559	5.249
.3230	.3330	2.092	.9700	3.990	4.114	.2431	4.185	32.83	32.84	.5637	.5468	.9562	5.245
.3240	.3339	2.098	.9703	4.014	4.136	.2418	4.196	33.20	33.22	.5632	.5465	.9565	5.241
.3250	.3349	2.104	.9707	4.038	4.160	.2404	4.208	33.60	33.61	.5627	.5462	.9568	5.237
.3260	.3357	2.110	.9710	4.061	4.183	.2391	4.219	33.97	33.99	.5621	.5458	.9571	5.234
.3270	.3367	2.115	.9713	4.085	4.206	.2378	4.231	34.37	34.38	.5616	.5455	.9574	5.231
.3280	.3376	2.121	.9717	4.110	4.230	.2364	4.242	34.77	34.79	.5610	.5451	.9577	5.227
.3290	.3385	2.127	.9720	4.135	4.254	.2351	4.254	35.18	35.19	.5605	.5448	.9580	5.223

Table D-1 Cont'd

d/L_o	d/L	$2\pi d/L$	TANH $2\pi d/L$	SINH $2\pi d/L$	COSH $2\pi d/L$	K	$4\pi d/L$	SINH $4\pi d/L$	COSH $4\pi d/L$	n	C_G/C_o	H/H_o'	M
.3300	.3394	2.133	.9723	4.159	4.277	.2338	4.265	35.58	35.59	.5599	.5444	.9583	5.220
.3310	.3403	2.138	.9726	4.184	4.301	.2325	4.277	35.99	36.00	.5594	.5441	.9586	5.217
.3320	.3413	2.144	.9729	4.209	4.326	.2312	4.288	36.42	36.43	.5589	.5438	.9589	5.214
.3330	.3422	2.150	.9732	4.234	4.350	.2299	4.300	36.84	36.85	.5584	.5434	.9592	5.210
.3340	.3431	2.156	.9735	4.259	4.375	.2286	4.311	37.25	37.27	.5578	.5431	.9595	5.207
.3350	.3440	2.161	.9738	4.284	4.399	.2273	4.323	37.70	37.72	.5573	.5427	.9598	5.204
.3360	.3449	2.167	.9741	4.310	4.424	.2260	4.335	38.14	38.15	.5568	.5424	.9601	5.201
.3370	.3459	2.173	.9744	4.336	4.450	.2247	4.346	38.59	38.60	.5563	.5421	.9604	5.198
.3380	.3468	2.179	.9747	4.361	4.474	.2235	4.358	39.02	39.04	.5558	.5417	.9607	5.194
.3390	.3477	2.185	.9750	4.388	4.500	.2222	4.369	39.48	39.49	.5553	.5414	.9610	5.191
.3400	.3468	2.190	.9753	4.413	4.525	.2210	4.381	39.95	39.96	.5548	.5411	.9613	5.188
.3410	.3495	2.196	.9756	4.439	4.550	.2198	4.392	40.40	40.41	.5544	.5408	.9615	5.185
.3420	.3504	2.202	.9758	4.466	4.576	.2185	4.404	40.87	40.89	.5539	.5405	.9618	5.182
.3430	.3514	2.208	.9761	4.492	4.602	.2173	4.416	41.36	41.37	.5534	.5402	.9621	5.179
.3440	.3523	2.214	.9764	4.521	4.630	.2160	4.427	41.85	41.84	.5529	.5399	.9623	5.176
.3450	.3532	2.220	.9767	4.547	4.656	.2148	4.439	42.33	42.34	.5524	.5396	.9626	5.173
.3460	.3542	2.225	.9769	4.575	4.682	.2136	4.451	42.83	42.84	.5519	.5392	.9629	5.171
.3470	.3551	2.231	.9772	4.602	4.709	.2124	4.462	43.34	43.35	.5515	.5389	.9632	5.168
.3480	.3560	2.237	.9775	4.629	4.736	.2111	4.474	43.85	43.86	.5510	.5386	.9635	5.165
.3490	.3570	2.243	.9777	4.657	4.763	.2099	4.486	44.37	44.40	.5505	.5383	.9638	5.162
.3500	.3579	2.249	.9780	4.685	4.791	.2087	4.498	44.89	44.80	.5501	.5380	.9640	5.159
.3510	.3588	2.255	.9782	4.713	4.818	.2076	4.509	45.42	45.43	.5496	.5377	.9643	5.157
.3520	.3598	2.260	.9785	4.741	4.845	.2064	4.521	45.95	45.96	.5492	.5374	.9646	5.154
.3530	.3607	2.266	.9787	4.770	4.873	.2052	4.533	46.50	46.51	.5487	.5371	.9648	5.152
.3540	.3616	2.272	.9790	4.798	4.901	.2040	4.544	47.03	47.04	.5483	.5368	.9651	5.149
.3550	.3625	2.278	.9792	4.827	4.929	.2029	4.556	47.59	47.60	.5479	.5365	.9654	5.147
.3560	.3635	2.284	.9795	4.856	4.957	.2017	4.568	48.15	48.16	.5474	.5362	.9657	5.144
.3570	.3644	2.290	.9797	4.885	4.987	.2005	4.579	48.72	48.73	.5470	.5359	.9659	5.141
.3580	.3653	2.296	.9799	4.914	5.015	.1994	4.591	49.29	49.30	.5466	.5356	.9662	5.139
.3590	.3663	2.301	.9801	4.944	5.044	.1983	4.603	49.88	49.89	.5461	.5353	.9665	5.137
.3600	.3672	2.307	.9804	4.974	5.072	.1972	4.615	50.47	50.48	.5457	.5350	.9667	5.134
.3610	.3682	2.313	.9806	5.004	5.103	.1960	4.627	51.08	51.09	.5453	.5347	.9670	5.132
.3620	.3691	2.319	.9808	5.034	5.132	.1949	4.638	51.67	51.67	.5449	.5344	.9673	5.130
.3630	.3700	2.325	.9811	5.063	5.161	.1938	4.650	52.27	52.28	.5445	.5342	.9675	5.127
.3640	.3709	2.331	.9813	5.094	5.191	.1926	4.661	52.89	52.90	.5441	.5339	.9677	5.125
.3650	.3719	2.337	.9815	5.124	5.221	.1915	4.673	53.52	53.53	.5437	.5336	.9680	5.123
.3660	.3728	2.342	.9817	5.155	5.251	.1904	4.685	54.15	54.16	.5433	.5333	.9683	5.121
.3670	.3737	2.348	.9819	5.186	5.281	.1894	4.697	54.78	54.79	.5429	.5330	.9686	5.118
.3680	.3747	2.354	.9821	5.217	5.312	.1883	4.708	55.42	55.43	.5425	.5327	.9688	5.116
.3690	.3756	2.360	.9823	5.248	5.343	.1872	4.720	56.09	56.10	.5421	.5325	.9690	5.114
.3700	.3766	2.366	.9825	5.280	5.374	.1861	4.732	56.76	56.77	.5417	.5322	.9693	5.112
.3710	.3775	2.372	.9827	5.312	5.406	.1850	4.744	57.43	57.44	.5413	.5319	.9696	5.110
.3720	.3785	2.378	.9830	5.345	5.438	.1839	4.756	58.13	58.14	.5409	.5317	.9698	5.107
.3730	.3794	2.384	.9832	5.377	5.469	.1828	4.768	58.82	58.83	.5405	.5314	.9700	5.105
.3740	.3804	2.390	.9834	5.410	5.502	.1818	4.780	59.52	59.53	.5402	.5312	.9702	5.103
.3750	.3813	2.396	.9835	5.443	5.534	.1807	4.792	60.24	60.25	.5398	.5309	.9705	5.101
.3760	.3822	2.402	.9837	5.475	5.566	.1797	4.803	60.95	60.95	.5394	.5306	.9707	5.099
.3770	.3832	2.408	.9839	5.508	5.598	.1786	4.815	61.68	61.68	.5390	.5304	.9709	5.097
.3780	.3841	2.413	.9841	5.541	5.631	.1776	4.827	62.41	62.42	.5387	.5301	.9712	5.095
.3790	.3850	2.419	.9843	5.572	5.661	.1766	4.838	63.13	63.14	.5383	.5299	.9714	5.093
.3800	.3860	2.425	.9845	5.609	5.697	.1756	4.851	63.91	63.91	.5380	.5296	.9717	5.091
.3810	.3869	2.431	.9847	5.643	5.731	.1745	4.862	64.67	64.67	.5376	.5294	.9719	5.090
.3820	.3879	2.437	.9848	5.677	5.765	.1735	4.875	65.45	65.46	.5372	.5291	.9721	5.088
.3830	.3888	2.443	.9850	5.712	5.798	.1725	4.885	66.16	66.17	.5369	.5288	.9724	5.086
.3840	.3898	2.449	.9852	5.746	5.833	.1715	4.898	67.02	67.03	.5365	.5286	.9726	5.084
.3850	.3907	2.455	.9854	5.780	5.866	.1705	4.910	67.80	67.81	.5362	.5284	.9728	5.082
.3860	.3917	2.461	.9855	5.814	5.900	.1695	4.922	68.61	68.62	.5359	.5281	.9730	5.081
.3870	.3926	2.467	.9857	5.850	5.935	.1685	4.934	69.45	69.46	.5355	.5279	.9732	5.079
.3880	.3936	2.473	.9859	5.886	5.970	.1675	4.946	70.28	70.29	.5352	.5276	.9735	5.077
.3890	.3945	2.479	.9860	5.921	6.005	.1665	4.958	71.12	71.13	.5349	.5274	.9737	5.076

d/L_o	d/L	$2\pi d/L$	TANH $2\pi d/L$	SINH $2\pi d/L$	COSH $2\pi d/L$	K	$4\pi d/L$	SINH $4\pi d/L$	COSH $4\pi d/L$	n	C_G/C_o	H/H'_o	M
.3900	.3955	2.485	.9862	5.957	6.040	.1656	4.970	71.97	71.98	.5345	.5271	.9739	5.074
.3910	.3964	2.491	.9864	5.993	6.076	.1646	4.982	72.85	72.86	.5342	.5269	.9741	5.072
.3920	.3974	2.497	.9865	6.029	6.112	.1636	4.993	73.72	73.72	.5339	.5267	.9743	5.071
.3930	.3983	2.503	.9867	6.066	6.148	.1627	5.005	74.58	74.59	.5336	.5265	.9745	5.069
.3940	.3993	2.509	.9869	6.103	6.185	.1617	5.017	75.48	75.49	.5332	.5262	.9748	5.067
.3950	.4002	2.515	.9870	6.140	6.221	.1608	5.029	76.40	76.40	.5329	.5260	.9750	5.066
.3960	.4012	2.521	.9872	6.177	6.258	.1598	5.041	77.31	77.32	.5326	.5258	.9752	5.064
.3970	.4021	2.527	.9873	6.215	6.295	.1589	5.053	78.24	78.24	.5323	.5255	.9754	5.063
.3980	.4031	2.532	.9874	6.252	6.332	.1579	5.065	79.19	79.19	.5320	.5253	.9756	5.062
.3990	.4040	2.538	.9876	6.290	6.369	.1570	5.077	80.13	80.13	.5317	.5251	.9758	5.060
.4000	.4050	2.544	.9877	6.329	6.407	.1561	5.089	81.12	81.12	.5314	.5248	.9761	5.058
.4010	.4059	2.550	.9879	6.367	6.445	.1552	5.101	82.07	82.08	.5311	.5246	.9763	5.056
.4020	.4069	2.556	.9880	6.406	6.483	.1542	5.113	83.06	83.06	.5308	.5244	.9765	5.055
.4030	.4078	2.562	.9882	6.444	6.521	.1533	5.125	84.07	84.07	.5305	.5242	.9766	5.053
.4040	.4088	2.568	.9883	6.484	6.561	.1524	5.137	85.11	85.12	.5302	.5240	.9768	5.052
.4050	.4098	2.575	.9885	6.525	6.601	.1515	5.149	86.14	86.14	.5299	.5238	.9770	5.050
.4060	.4107	2.581	.9886	6.564	6.640	.1506	5.161	87.17	87.17	.5296	.5236	.9772	5.049
.4070	.4116	2.586	.9887	6.603	6.679	.1497	5.173	88.19	88.20	.5293	.5234	.9774	5.048
.4080	.4126	2.592	.9889	6.644	6.718	.1488	5.185	89.28	89.28	.5290	.5232	.9776	5.046
.4090	.4136	2.598	.9890	6.684	6.758	.1480	5.197	90.38	90.39	.5287	.5229	.9778	5.045
.4100	.4145	2.604	.9891	6.725	6.799	.1471	5.209	91.44	91.44	.5285	.5227	.9780	5.044
.4110	.4155	2.610	.9892	6.766	6.839	.1462	5.221	92.54	92.55	.5282	.5225	.9782	5.043
.4120	.4164	2.616	.9894	6.806	6.879	.1454	5.233	93.67	93.67	.5279	.5223	.9784	5.041
.4130	.4174	2.623	.9895	6.849	6.921	.1445	5.245	94.83	94.83	.5277	.5221	.9786	5.040
.4140	.4183	2.629	.9896	6.890	6.963	.1436	5.257	95.95	95.96	.5274	.5219	.9788	5.039
.4150	.4193	2.635	.9898	6.932	7.004	.1428	5.269	97.13	97.13	.5271	.5217	.9790	5.037
.4160	.4203	2.641	.9899	6.974	7.046	.1419	5.281	98.29	98.30	.5269	.5215	.9792	5.036
.4170	.4212	2.647	.9900	7.018	7.088	.1411	5.294	99.52	99.52	.5266	.5213	.9794	5.035
.4180	.4222	2.653	.9901	7.060	7.130	.1403	5.305	100.7	100.7	.5263	.5211	.9795	5.034
.4190	.4231	2.659	.9902	7.102	7.173	.1394	5.317	101.9	101.9	.5261	.5209	.9797	5.033
.4200	.4241	2.665	.9904	7.146	7.215	.1386	5.329	103.1	103.1	.5258	.5208	.9798	5.031
.4210	.4251	2.671	.9905	7.190	7.259	.1378	5.341	104.4	104.4	.5256	.5206	.9800	5.030
.4220	.4260	2.677	.9906	7.234	7.303	.1369	5.353	105.7	105.7	.5253	.5204	.9802	5.029
.4230	.4270	2.683	.9907	7.279	7.349	.1361	5.366	107.0	107.0	.5251	.5202	.9804	5.028
.4240	.4280	2.689	.9908	7.325	7.392	.1353	5.378	108.3	108.3	.5248	.5200	.9806	5.027
.4250	.4289	2.695	.9909	7.371	7.438	.1345	5.390	109.	109.7	.5246	.5198	.9808	5.026
.4260	.4298	2.701	.9910	7.412	7.479	.1337	5.402	110.9	110.9	.5244	.5196	.9810	5.025
.4270	.4308	2.707	.9911	7.457	7.524	.1329	5.414	112.2	112.2	.5241	.5195	.9811	5.024
.4280	.4318	2.713	.9912	7.503	7.570	.1321	5.426	113.6	113.6	.5239	.5193	.9812	5.023
.4290	.4328	2.719	.9913	7.550	7.616	.1313	5.438	115.0	115.0	.5237	.5191	.9814	5.022
.4300	.4337	2.725	.9914	7.595	7.661	.1305	5.450	116.4	116.4	.5234	.5189	.9816	5.021
.4310	.4347	2.731	.9915	7.642	7.707	.1298	5.462	117.8	117.8	.5232	.5187	.9818	5.020
.4320	.4356	2.737	.9916	7.688	7.753	.1290	5.474	119.2	119.3	.5230	.5186	.9819	5.019
.4330	.4366	2.743	.9917	7.735	7.800	.1282	5.486	120.7	120.7	.5227	.5184	.9821	5.018
.4340	.4376	2.749	.9918	7.783	7.847	.1274	5.499	122.2	122.2	.5225	.5182	.9823	5.017
.4350	.4385	2.755	.9919	7.831	7.895	.1267	5.511	123.7	123.7	.5223	.5181	.9824	5.016
.4360	.4395	2.762	.9920	7.880	7.943	.1259	5.523	125.2	125.2	.5221	.5179	.9826	5.015
.4370	.4405	2.768	.9921	7.922	7.991	.1251	5.535	126.7	126.7	.5218	.5177	.9828	5.014
.4380	.4414	2.774	.9922	7.975	8.035	.1244	5.547	128.3	128.3	.5216	.5176	.9829	5.013
.4390	.4424	2.780	.9923	8.026	8.088	.1236	5.560	129.9	129.9	.5214	.5174	.9830	5.012
.4400	.4434	2.786	.9924	8.075	8.136	.1229	5.572	131.4	131.4	.5212	.5172	.9832	5.011
.4410	.4443	2.792	.9925	8.124	8.185	.1222	5.584	133.0	133.0	.5210	.5171	.9833	5.010
.4420	.4453	2.798	.9926	8.175	8.236	.1214	5.596	134.7	134.7	.5208	.5169	.9835	5.009
.4430	.4463	2.804	.9927	8.228	8.285	.1207	5.608	136.3	136.3	.5206	.5168	.9836	5.008
.4440	.4472	2.810	.9928	8.274	8.334	.1200	5.620	137.9	137.9	.5204	.5166	.9838	5.007
.4450	.4482	2.816	.9929	8.326	8.387	.1192	5.632	139.6	139.7	.5202	.5165	.9839	5.006
.4460	.4492	2.822	.9930	8.379	8.438	.1185	5.644	141.4	141.4	.5200	.5163	.9841	5.005
.4470	.4501	2.828	.9930	8.427	8.486	.1178	5.657	143.1	143.1	.5198	.5161	.9843	5.005
.4480	.4511	2.834	.9931	8.481	8.540	.1171	5.669	144.8	144.8	.5196	.5160	.9844	5.004
.4490	.4521	2.840	.9932	8.532	8.590	.1164	5.681	146.6	146.6	.5194	.5158	.9846	5.003

Table D-1 Cont'd

d/L_o	d/L	$2\pi d/L$	TANH $2\pi d/L$	SINH $2\pi d/L$	COSH 2 d/L	K	$4\pi d/L$	SINH $4\pi d/L$	COSH $4\pi d/L$	n	C_G/C_o	H/H_o'	M
.4500	.4531	2.847	.9933	8.585	8.643	.1157	5.693	148.4	148.4	.5192	.5157	.9847	5.002
.4510	.4540	2.853	.9934	8.638	8.695	.1150	5.705	150.2	150.2	.5190	.5156	.9848	5.001
.4520	.4550	2.859	.9935	8.693	8.750	.1143	5.717	152.1	152.1	.5188	.5154	.9849	5.000
.4530	.4560	2.865	.9935	8.747	8.804	.1136	5.730	154.0	154.0	.5186	.5152	.9851	5.000
.4540	.4569	2.871	.9936	8.797	8.854	.1129	5.742	155.9	155.9	.5184	.5151	.9852	4.999
.4550	.4579	2.877	.9937	8.853	8.910	.1122	5.754	157.7	157.7	.5182	.5150	.9853	4.998
.4560	.4589	2.883	.9938	8.910	8.965	.1115	5.766	159.7	159.7	.5181	.5148	.9855	4.997
.4570	.4599	2.890	.9938	8.965	9.021	.1109	5.779	161.7	161.7	.5179	.5146	.9857	4.997
.4580	.4608	2.896	.9939	9.016	9.072	.1102	5.791	163.6	163.6	.5177	.5145	.9858	4.996
.4590	.4618	2.902	.9940	9.074	9.129	.1095	5.803	165.6	165.6	.5175	.5144	.9859	4.995
.4600	.4628	2.908	.9941	9.132	9.186	.1089	5.815	167.7	167.7	.5173	.5143	.9860	4.994
.4610	.4637	2.914	.9941	9.183	9.238	.1083	5.827	169.7	169.7	.5172	.5141	.9862	4.994
.4620	.4647	2.920	.9942	9.242	9.296	.1076	5.840	171.8	171.8	.5170	.5140	.9863	4.993
.4630	.4657	2.926	.9943	9.301	9.354	.1069	5.852	173.9	173.9	.5168	.5139	.9864	4.992
.4640	.4666	2.932	.9944	9.353	9.406	.1063	5.864	176.0	176.0	.5167	.5138	.9865	4.991
.4650	.4676	2.938	.9944	9.413	9.466	.1056	5.876	178.2	178.2	.5165	.5136	.9867	4.991
.4660	.4686	2.944	.9945	9.472	9.525	.1050	5.888	180.4	180.4	.5163	.5135	.9868	4.990
.4670	.4695	2.951	.9946	9.533	9.585	.1043	5.900	182.6	182.6	.5162	.5134	.9869	4.989
.4680	.4705	2.957	.9946	9.586	9.638	.1037	5.912	184.8	184.8	.5160	.5132	.9871	4.989
.4690	.4715	2.963	.9947	9.647	9.699	.1031	5.925	187.2	187.2	.5158	.5131	.9872	4.988
.4700	.4725	2.969	.9947	9.709	9.760	.1025	5.937	189.5	189.5	.5157	.5129	.9873	4.988
.4710	.4735	2.975	.9948	9.770	9.821	.1018	5.949	191.8	191.8	.5155	.5128	.9874	4.987
.4720	.4744	2.981	.9949	9.826	9.877	.1012	5.962	194.2	194.2	.5154	.5127	.9875	4.986
.4730	.4754	2.987	.9949	9.888	9.938	.1006	5.974	196.5	196.5	.5152	.5126	.9876	4.986
.4740	.4764	2.993	.9950	9.951	10.00	.1000	5.986	199.0	199.0	.5150	.5125	.9877	4.985
.4750	.4774	2.999	.9951	10.01	10.07	.09942	5.999	201.4	201.4	.5149	.5124	.9878	4.984
.4760	.4783	3.005	.9951	10.07	10.12	.09882	6.011	203.9	203.9	.5147	.5122	.9880	4.984
.4770	.4793	3.012	.9952	10.13	10.18	.09820	6.023	206.5	206.5	.5146	.5121	.9881	4.983
.4780	.4803	3.018	.9952	10.20	10.25	.09759	6.036	209.0	209.0	.5144	.5120	.9882	4.983
.4790	.4813	3.024	.9953	10.26	10.31	.09698	6.048	211.7	211.7	.5143	.5119	.9883	4.982
.4800	.4822	3.030	.9953	10.32	10.37	.09641	6.060	214.2	214.2	.5142	.5117	.9885	4.982
.4810	.4832	3.036	.9954	10.39	10.43	.09583	6.072	216.8	216.8	.5140	.5116	.9886	4.981
.4820	.4842	3.042	.9955	10.45	10.50	.09523	6.085	219.5	219.5	.5139	.5115	.9887	4.980
.4830	.4852	3.049	.9955	10.52	10.57	.09464	6.097	222.2	222.2	.5137	.5114	.9888	4.980
.4840	.4862	3.055	.9956	10.59	10.63	.09405	6.109	225.0	225.0	.5136	.5113	.9889	4.979
.4850	.4871	3.061	.9956	10.65	10.69	.09352	6.121	228.3	228.3	.5134	.5112	.9890	4.979
.4860	.4881	3.067	.9957	10.71	10.76	.09294	6.134	230.6	230.6	.5133	.5111	.9891	4.978
.4870	.4891	3.073	.9957	10.78	10.83	.09236	6.146	233.5	233.5	.5132	.5110	.9892	4.978
.4880	.4901	3.079	.9958	10.85	10.90	.09178	6.159	236.4	236.4	.5130	.5109	.9893	4.977
.4890	.4911	3.086	.9958	10.92	10.96	.09121	6.171	239.6	239.6	.5129	.5107	.9895	4.977
.4900	.4920	3.092	.9959	10.99	11.03	.09064	6.183	242.3	242.3	.5128	.5106	.9896	4.976
.4910	.4930	3.098	.9959	11.05	11.09	.09010	6.195	245.2	245.2	.5126	.5105	.9897	4.976
.4920	.4940	3.104	.9960	11.12	11.16	.08956	6.208	248.3	248.3	.5125	.5104	.9898	4.975
.4930	.4950	3.110	.9960	11.19	11.24	.08901	6.220	251.3	251.3	.5124	.5103	.9899	4.975
.4940	.4960	3.117	.9961	11.26	11.31	.08845	6.232	254.5	254.5	.5122	.5102	.9899	4.974
.4950	.4969	3.122	.9961	11.32	11.37	.08793	6.245	257.6	257.6	.5121	.5101	.9900	4.974
.4960	.4979	3.128	.9962	11.40	11.44	.08741	6.257	260.8	260.8	.5120	.5100	.9901	4.973
.4970	.4989	3.135	.9962	11.47	11.51	.08691	6.269	264.0	264.0	.5119	.5099	.9902	4.973
.4980	.4999	3.141	.9963	11.54	11.59	.08637	6.282	267.3	267.3	.5118	.5098	.9903	4.972
.4990	.5009	3.147	.9963	11.61	11.65	.08584	6.294	270.6	270.6	.5116	.5097	.9904	4.972
.5000	.5018	3.153	.9964	11.68	11.72	.08530	6.306	274.0	274.0	.5115	.5096	.9905	4.971
.5010	.5028	3.159	.9964	11.75	11.80	.08477	6.319	277.5	277.5	.5114	.5095	.9906	4.971
.5020	.5038	3.166	.9964	11.83	11.87	.08424	6.331	280.8	280.8	.5113	.5094	.9907	4.971
.5030	.5048	3.172	.9965	11.91	11.95	.08371	6.343	284.3	284.3	.5112	.5093	.9908	4.970
.5040	.5058	3.178	.9965	11.98	12.02	.08320	6.356	287.9	287.9	.5110	.5092	.9909	4.970
.5050	.5067	3.184	.9966	12.05	12.09	.08270	6.368	291.4	291.4	.5109	.5092	.9909	4.969
.5060	.5077	3.190	.9966	12.12	12.16	.08220	6.380	295.0	295.0	.5108	.5091	.9910	4.969
.5070	.5087	3.196	.9967	12.20	12.24	.08169	6.393	298.7	298.7	.5107	.5090	.9911	4.968
.5080	.5097	3.203	.9967	12.28	12.32	.08119	6.405	302.4	302.4	.5106	.5089	.9912	4.968
.5090	.5107	3.209	.9968	12.35	12.39	.08068	6.417	306.2	306.2	.5105	.5088	.9913	4.967

Table D-1 Cont'd

d/L_o	d/L	$2\pi d/L$	TANH $2\pi d/L$	SINH $2\pi d/L$	COSH $2\pi d/L$	K	$4\pi d/L$	SINH $4\pi d/L$	COSH $4\pi d/L$	n	C_G/C_o	H/H'_o	M
.5100	.5117	3.215	.9968	12.43	12.47	.08022	6.430	310.0	310.0	.5104	.5087	.9914	4.967
.5110	.5126	3.221	.9968	12.50	12.54	.07972	6.442	313.8	313.8	.5103	.5086	.9915	4.967
.5120	.5136	3.227	.9969	12.58	12.62	.07922	6.454	317.7	317.7	.5102	.5086	.9915	4.966
.5130	.5146	3.233	.9969	12.66	12.70	.07873	6.467	321.7	321.7	.5101	.5085	.9916	4.966
.5140	.5156	3.240	.9970	12.74	12.78	.07824	6.479	325.7	325.7	.5100	.5084	.9917	4.965
.5150	.5166	3.246	.9970	12.82	12.86	.07776	6.491	329.7	329.7	.5098	.5083	.9918	4.965
.5160	.5176	3.252	.9970	12.90	12.94	.07729	6.504	333.8	333.8	.5097	.5082	.9919	4.965
.5170	.5185	3.258	.9971	12.98	13.02	.07682	6.516	337.9	337.9	.5096	.5082	.9919	4.964
.5180	.5195	3.264	.9971	13.06	13.10	.07634	6.529	342.2	342.2	.5095	.5081	.9920	4.964
.5190	.5205	3.270	.9971	13.14	13.18	.07587	6.541	346.4	346.4	.5094	.5080	.9921	4.964
.5200	.5215	3.277	.9972	13.22	13.26	.07540	6.553	350.7	350.7	.5093	.5079	.9922	4.963
.5210	.5225	3.283	.9972	13.31	13.35	.07494	6.566	355.1	355.1	.5092	.5078	.9923	4.963
.5220	.5235	3.289	.9972	13.39	13.43	.07449	6.578	359.6	359.6	.5092	.5077	.9924	4.963
.5230	.5244	3.295	.9973	13.47	13.51	.07404	6.590	364.0	364.0	.5091	.5077	.9924	4.962
.5240	.5254	3.301	.9973	13.55	13.59	.07358	6.603	368.5	368.5	.5090	.5076	.9925	4.962
.5250	.5264	3.308	.9973	13.64	13.68	.07312	6.615	373.1	373.1	.5089	.5075	.9926	4.962
.5260	.5274	3.314	.9974	13.73	13.76	.07266	6.628	377.8	377.8	.5088	.5074	.9927	4.961
.5270	.5284	3.320	.9974	13.81	13.85	.07221	6.640	382.5	382.5	.5087	.5074	.9927	4.961
.5280	.5294	3.326	.9974	13.90	13.94	.07177	6.652	387.3	387.3	.5086	.5073	.9928	4.961
.5290	.5304	3.333	.9975	13.99	14.02	.07134	6.665	392.2	392.2	.5085	.5072	.9929	4.960
.5300	.5314	3.339	.9975	14.07	14.10	.07091	6.677	397.0	397.0	.5084	.5071	.9930	4.960
.5310	.5323	3.345	.9975	14.16	14.19	.07047	6.690	402.0	402.0	.5083	.5070	.9931	4.960
.5320	.5333	3.351	.9976	14.25	14.28	.07003	6.702	406.9	406.9	.5082	.5070	.9931	4.959
.5330	.5343	3.357	.9976	14.34	14.37	.06959	6.714	412.0	412.0	.5082	.5069	.9932	4.959
.5340	.5353	3.363	.9976	14.43	14.46	.06915	6.727	417.2	417.2	.5081	.5068	.9933	4.959
.5350	.5363	3.370	.9976	14.52	14.55	.06872	6.739	422.4	422.4	.5080	.5068	.9933	4.959
.5360	.5373	3.376	.9977	14.61	14.64	.06829	6.752	427.7	427.7	.5079	.5067	.9934	4.958
.5370	.5383	3.382	.9977	14.70	14.73	.06787	6.764	433.1	433.1	.5078	.5066	.9935	4.958
.5380	.5393	3.388	.9977	14.79	14.82	.06746	6.776	438.5	438.5	.5077	.5066	.9935	4.958
.5390	.5402	3.394	.9977	14.88	14.91	.06705	6.789	444.0	444.0	.5077	.5065	.9936	4.958
.5400	.5412	3.401	.9978	14.97	15.01	.06664	6.801	449.5	449.5	.5076	.5065	.9936	4.957
.5410	.5422	3.407	.9978	15.07	15.10	.06623	6.814	455.1	455.1	.5075	.5064	.9937	4.957
.5420	.5432	3.413	.9978	15.16	15.19	.06582	6.826	460.7	460.7	.5074	.5063	.9938	4.957
.5430	.5442	3.419	.9979	15.25	15.29	.06542	6.838	466.4	466.4	.5073	.5063	.9938	4.956
.5440	.5452	3.426	.9979	15.35	15.38	.06501	6.851	472.2	472.2	.5073	.5062	.9939	4.956
.5450	.5461	3.432	.9979	15.45	15.48	.06461	6.863	478.1	478.1	.5072	.5061	.9940	4.956
.5460	.5471	3.438	.9979	15.54	15.58	.06420	6.876	484.3	484.3	.5071	.5060	.9941	4.956
.5470	.5481	3.444	.9980	15.64	15.67	.06380	6.888	490.3	490.3	.5070	.5060	.9941	4.955
.5480	.5491	3.450	.9980	15.74	15.77	.06341	6.901	496.4	496.4	.5070	.5059	.9942	4.955
.5490	.5501	3.456	.9980	15.84	15.87	.06302	6.913	502.5	502.5	.5069	.5059	.9942	4.955
.5500	.5511	3.463	.9980	15.94	15.97	.06263	6.925	508.7	508.7	.5068	.5058	.9942	4.955
.5510	.5521	3.469	.9981	16.04	16.07	.06224	6.937	515.0	515.0	.5067	.5058	.9942	4.954
.5520	.5531	3.475	.9981	16.14	16.17	.06186	6.950	521.6	521.6	.5067	.5057	.9943	4.954
.5530	.5541	3.481	.9981	16.24	16.27	.06148	6.962	528.1	528.1	.5066	.5056	.9944	4.954
.5540	.5551	3.488	.9981	16.34	16.37	.06110	6.975	534.8	534.8	.5065	.5056	.9944	4.954
.5550	.5560	3.494	.9982	16.44	16.47	.06073	6.987	541.4	541.4	.5065	.5056	.9945	4.953
.5560	.5570	3.500	.9982	16.54	16.57	.06035	7.000	548.1	548.1	.5064	.5055	.9945	4.953
.5570	.5580	3.506	.9982	16.65	16.68	.05997	7.012	554.9	554.9	.5063	.5054	.9946	4.953
.5580	.5590	3.512	.9982	16.75	16.78	.05960	7.025	562.0	562.0	.5063	.5053	.9947	4.953
.5590	.5600	3.519	.9982	16.85	16.88	.05923	7.037	569.1	569.1	.5062	.5053	.9947	4.953
.5600	.5610	3.525	.9983	16.96	16.99	.05887	7.050	576.1	576.1	.5061	.5053	.9947	4.952
.5610	.5620	3.531	.9983	17.06	17.09	.05850	7.062	583.3	583.3	.5061	.5052	.9948	4.952
.5620	.5630	3.537	.9983	17.17	17.20	.05814	7.074	590.7	590.7	.5060	.5051	.9949	4.952
.5630	.5640	3.543	.9983	17.28	17.31	.05778	7.087	598.0	598.0	.5059	.5051	.9949	4.952
.5640	.5649	3.550	.9984	17.38	17.41	.05743	7.099	605.0	605.0	.5059	.5050	.9950	4.951
.5650	.5659	3.556	.9984	17.49	17.52	.05707	7.112	613.2	613.2	.5058	.5050	.9950	4.951
.5660	.5669	3.562	.9984	17.60	17.63	.05672	7.124	620.8	620.8	.5057	.5049	.9951	4.951
.5670	.5679	3.568	.9984	17.71	17.74	.05637	7.136	628.5	628.5	.5057	.5049	.9951	4.951
.5680	.5689	3.575	.9984	17.82	17.85	.05602	7.149	636.4	636.4	.5056	.5048	.9952	4.951
.5690	.5699	3.581	.9985	17.94	17.97	.05567	7.161	644.3	644.3	.5056	.5048	.9952	4.950

Table D-1 Cont'd

d/L_o	d/L	$2\pi d/L$	TANH $2\pi d/L$	SINH $2\pi d/L$	COSH $2\pi d/L$	K	$4\pi d/L$	SINH $4\pi d/L$	COSH $4\pi d/L$	n	C_G/C_o	H/H'_o	M
.5700	.5709	3.587	.9985	18.05	18.08	.05532	7.174	652.4	652.4	.5055	.5047	.9953	4.950
.5710	.5719	3.593	.9985	18.16	18.19	.05497	7.186	660.5	660.5	.5054	.5047	.9953	4.950
.5720	.5729	3.600	.9985	18.28	18.31	.05463	7.199	668.8	668.8	.5054	.5046	.9954	4.950
.5730	.5738	3.606	.9985	18.39	18.42	.05430	7.211	677.2	677.2	.5053	.5046	.9954	4.950
.5740	.5748	3.612	.9985	18.50	18.53	.05396	7.224	685.6	685.6	.5053	.5045	.9955	4.950
.5750	.5758	3.618	.9986	18.62	18.64	.05363	7.236	694.3	694.3	.5052	.5045	.9955	4.949
.5760	.5768	3.624	.9986	18.73	18.76	.05330	7.249	703.2	703.2	.5052	.5044	.9956	4.949
.5770	.5778	3.630	.9986	18.85	18.88	.05297	7.261	711.9	711.9	.5051	.5044	.9956	4.949
.5780	.5788	3.637	.9986	18.97	19.00	.05264	7.274	720.8	720.8	.5051	.5043	.9957	4.949
.5790	.5798	3.643	.9986	19.09	19.12	.05231	7.286	729.9	729.9	.5050	.5043	.9957	4.949
.5800	.5808	3.649	.9987	19.21	19.24	.05198	7.298	739.0	739.0	.5049	.5043	.9957	4.948
.5810	.5818	3.656	.9987	19.33	19.36	.05166	7.311	748.1	748.1	.5049	.5042	.9958	4.948
.5820	.5828	3.662	.9987	19.45	19.48	.05134	7.323	757.5	757.5	.5048	.5042	.9958	4.948
.5830	.5838	3.668	.9987	19.58	19.60	.05102	7.336	767.0	767.0	.5048	.5041	.9959	4.948
.5840	.5848	3.674	.9987	19.70	19.73	.05070	7.348	776.7	776.7	.5047	.5041	.9959	4.948
.5850	.5858	3.680	.9987	19.81	19.84	.05040	7.361	786.5	786.5	.5047	.5040	.9960	4.948
.5860	.5867	3.686	.9987	19.94	19.96	.05009	7.373	796.4	796.4	.5046	.5040	.9960	4.948
.5870	.5877	3.693	.9988	20.06	20.09	.04978	7.386	806.5	806.5	.5046	.5040	.9960	4.947
.5880	.5887	3.699	.9988	20.19	20.21	.04947	7.398	816.5	816.5	.5045	.5039	.9961	4.947
.5890	.5897	3.705	.9988	20.32	20.34	.04916	7.411	826.7	826.7	.5045	.5039	.9961	4.947
.5900	.5907	3.712	.9988	20.45	20.47	.04885	7.423	837.1	837.1	.5044	.5038	.9962	4.947
.5910	.5917	3.718	.9988	20.57	20.60	.04855	7.436	847.6	847.6	.5044	.5038	.9962	4.947
.5920	.5927	3.724	.9988	20.70	20.73	.04824	7.448	858.2	858.2	.5043	.5037	.9963	4.947
.5930	.5937	3.730	.9989	20.83	20.86	.04794	7.460	868.9	868.9	.5043	.5037	.9963	4.946
.5940	.5947	3.737	.9989	20.97	20.99	.04764	7.473	879.8	879.8	.5043	.5037	.9963	4.946
.5950	.5957	3.743	.9989	21.10	21.12	.04735	7.485	890.8	890.8	.5042	.5036	.9964	4.946
.5960	.5967	3.749	.9989	21.23	21.25	.04706	7.498	901.9	901.9	.5042	.5036	.9964	4.946
.5970	.5977	3.755	.9989	21.35	21.37	.04677	7.510	913.4	913.4	.5041	.5036	.9964	4.946
.5980	.5987	3.761	.9989	21.49	21.51	.04648	7.523	925.0	925.0	.5041	.5035	.9965	4.946
.5990	.5996	3.767	.9989	21.62	21.64	.04619	7.535	936.5	936.5	.5040	.5035	.9965	4.946
.6000	.6006	3.774	.9990	21.76	21.78	.04591	7.548	948.1	948.1	.5040	.5035	.9965	4.945
.6100	.6106	3.836	.9991	23.17	23.19	.04313	7.673	1,074	1,074	.5036	.5031	.9969	4.944
.6200	.6205	3.899	.9992	24.66	24.68	.04052	7.798	1,217	1,217	.5032	.5028	.9972	4.943
.6300	.6305	3.961	.9993	26.25	26.27	.03806	7.923	1,379	1,379	.5029	.5025	.9975	4.942
.6400	.6404	4.024	.9994	27.95	27.97	.03576	8.048	1,527	1,527	.5026	.5023	.9977	4.941
.6500	.6504	4.086	.9994	29.75	29.77	.03359	8.173	1,771	1,771	.5023	.5020	.9980	4.940
.6600	.6603	4.149	.9995	31.68	31.69	.03155	8.298	2,008	2,008	.5021	.5018	.9982	4.940
.6700	.6703	4.212	.9996	33.73	33.74	.02964	8.423	2,275	2,275	.5019	.5017	.9983	4.939
.6800	.6803	4.274	.9996	35.90	35.92	.02784	8.548	2,579	2,579	.5017	.5015	.9985	4.939
.6900	.6902	4.337	.9997	38.23	38.24	.02615	8.674	2,923	2,923	.5015	.5013	.9987	4.938
.7000	.7002	4.400	.9997	40.71	40.72	.02456	8.799	3,314	3,314	.5013	.5012	.9988	4.938
.7100	.7102	4.462	.9997	43.34	43.35	.02307	8.925	3,757	3,757	.5012	.5011	.9989	4.937
.7200	.7202	4.525	.9998	46.14	46.15	.02167	9.050	4,258	4,258	.5011	.5010	.9990	4.937
.7300	.7302	4.588	.9998	49.13	49.14	.02035	9.175	4,828	4.828	.5010	.5009	.9991	4.937
.7400	.7401	4.650	.9998	52.31	52.32	.01911	9.301	5,473	5,473	.5009	.5008	.9992	4.937
,7500	.7501	4.713	.9998	55.70	55.71	.01795	9.426	6,204	6,204	.5008	.5007	.9993	4.936
.7600	.7601	4.776	.9999	59.31	59.31	.01686	9.552	7,034	7,034	.5007	.5006	.9994	4.936
.7700	.7701	4.839	.9999	63.15	63.16	.01583	9.677	7,976	7,976	.5006	.5005	.9995	4.936
.7800	.7801	4.902	.9999	67.24	67.25	.01487	9.803	9,042	9,042	.5005	.5004	.9996	4.936
.7900	.7901	4.964	.9999	71.60	71.60	.01397	9.929	10,250	10,250	.5005	.5004	.9996	4.936
.8000	.8001	5.027	.9999	76.24	76.24	.01312	10.05	11,620	11,620	.5004	.5004	.9996	4.936
.8100	.8101	5.090	.9999	81.18	81.19	.01232	10.18	13,180	13,180	.5004	.5004	.9996	4.936
.8200	.8201	5.153	.9999	86.44	86.44	.01157	10.31	14,940	14,940	.5003	.5003	.9997	4.935
.8300	.8301	5.215	.9999	92.04	92.05	.01086	10.43	17,340	17,340	.5003	.5003	.9997	4.935
.8400	.8400	5.278	1.000	98.00	98.01	.01020	10.56	19,210	19,210	.5003	.5003	.9997	4.935
.8500	.8500	5.341	1.000	104.4	104.4	.009582	10.68	21,780	21,780	.5002	.5002	.9998	4.935
.8600	.8600	5.404	1.000	111.1	111.1	.009000	10.81	24,690	24,690	.5002	.5002	.9998	4.935
.8700	.8700	5.467	1.000	118.3	118.3	.008451	10.93	28,000	28,000	.5002	.5002	.9998	4.935
.8800	.8800	5.529	1.000	126.0	126.0	.007934	11.06	31,750	31,750	.5002	.5002	.9998	4.935
.8900	.8900	5.592	1.000	134.2	134.2	.007454	11.18	36,000	36,000	.5002	.5002	.9998	4.935

d/L_o	d/L	$2\pi d/L$	TANH $2\pi d/L$	SINH $2\pi d/L$	COSH $2\pi d/L$	K	$4\pi d/L$	SINH $4\pi d/L$	COSH $4\pi d/L$	n	C_G/C_o	H/H'_o	M
.9000	.9000	5.655	1.000	142.9	142.9	.007000	11.31	40,810	40,810	.5001	.5001	.9999	4.935
.9100	.9100	5.718	1.000	152.1	152.1	.006574	11.44	46,280	46,280	.5001	.5001	.9999	4.935
.9200	.9200	5.781	1.000	162.0	162.0	.006173	11.56	52,470	52,470	.5001	.5001	.9999	4.935
.9300	.9300	5.844	1.000	172.5	172.5	.005797	11.69	59,500	59,500	.5001	.5001	.9999	4.935
.9400	.9400	5.906	1.000	183.7	183.7	.005445	11.81	67,470	67,470	.5001	.5001	.9999	4.935
.9500	.9500	5.969	1.000	195.6	195.6	.005113	11.94	76,490	76,490	.5001	.5001	.9999	4.935
.9600	.9600	6.032	1.000	203.5	203.5	.004914	12.06	86,740	86,740	.5001	.5001	.9999	4.935
.9700	.9700	6.095	1.000	222.8	222.8	.004489	12.19	98,350	98,350	.5001	.5001	.9999	4.935
.9800	.9800	6.158	1.000	236.1	236.1	.004235	12.32	111,500	111,500	.5001	.5001	.9999	4.935
.9900	.9900	6.220	1.000	251.4	251.4	.003977	12.44	126,500	126,500	.5000	.5000	1.000	4.935
1.000	1.000	6.283	1.000	267.7	267.7	.003735	12.57	143,400	143,400	.5000	.5000	1.000	4.935

TABLE D-2

FUNCTIONS OF d/L FOR EVEN INCREMENTS OF d/L

from 0.0001 to 0.2890

(This covers the region where interpolation of d/L in Table I is inconvenient. Values of d/L of 0.2890 to 1.0000 can be obtained from Table I by interpolation)

d/L	d/Lo	$2\pi d/L$	TANH $2\pi d/L$	SINH $2\pi d/L$	COSH $2\pi d/L$	K	$4\pi d/L$	SINH $4\pi d/L$	COSH $4\pi d/L$	n	C_G/C_o	H/H'_o	M
0	0	0	0	0	1.0000	1.000	0	0	1.000	1.000	0	∞	∞
.0001000	.00000006283	.0006283	.0006283	.0006283	1.0000	1.000	.001257	.001257	1.000	1.000	.0006283	28.21	12,500,000
.0002000	.0000002514	.001257	.001257	.001257	1.0000	1.000	.002513	.002513	1.000	1.000	.001257	19.95	3,125,000
.0003000	.0000005655	.001885	.001885	.001885	1.0000	1.000	.003770	.003770	1.000	1.000	.001885	16.29	1,389,000
.0004000	.000001005	.002513	.002513	.002513	1.0000	1.000	.005027	.005027	1.000	1.000	.002513	14.10	781,300
.0005000	.000001571	.003142	.003142	.003142	1.0000	1.000	.006283	.006283	1.000	1.000	.003142	12.62	500,000
.0006000	.000002262	.003770	.003770	.003770	1.0000	1.000	.007540	.007540	1.000	1.000	.003770	11.52	347,200
.0007000	.000003079	.004398	.004398	.004398	1.0000	1.000	.008796	.008797	1.000	1.000	.004398	10.66	255,100
.0008000	.000004022	.005027	.005027	.005027	1.0000	1.000	.01005	.01005	1.000	1.000	.005026	9.974	195,300
.0009000	.000005090	.005655	.005655	.005655	1.0000	1.000	.01131	.01131	1.000	1.000	.005655	9.403	154,300
.001000	.000006283	.006283	.006283	.006283	1.0000	1.000	.01257	.01257	1.000	1.000	.006283	8.921	125,000
.001100	.000007603	.006912	.006911	.006912	1.0000	1.000	.01382	.01382	1.000	1.000	.006911	8.506	103,300
.001200	.000009048	.007540	.007540	.007540	1.0000	1.000	.01508	.01508	1.000	1.000	.007540	8.144	86,810
.001300	.00001062	.008168	.008168	.008168	1.0000	1.000	.01634	.01634	1.000	1.000	.008168	7.824	73,970
.001400	.00001231	.008796	.008796	.008797	1.0000	1.000	.01759	.01759	1.000	1.000	.008796	7.539	63,780
.001500	.00001414	.009425	.009425	.009425	1.0000	1.000	.01885	.01885	1.000	1.000	.009424	7.284	55,560
.001600	.00001608	.01005	.01005	.01005	1.0001	.9999	.02011	.02011	1.000	1.000	.01005	7.052	48,830
.001700	.00001816	.01068	.01068	.01068	1.0001	.9999	.02136	.02136	1.000	1.000	.01068	6.842	43,260
.001800	.00002036	.01131	.01131	.01131	1.0001	.9999	.02262	.02262	1.000	1.000	.01131	6.649	38,580
.001900	.00002269	.01194	.01194	.01194	1.0001	.9999	.02388	.02388	1.000	1.000	.01194	6.472	34,630

Table D-2 Cont'd

d/L	d/Lo	$2\pi d/L$	TANH $2\pi d/L$	SINH $2\pi d/L$	COSH $2\pi d/L$	K	$4\pi d/L$	SINH $4\pi d/L$	COSH $4\pi d/L$	n	C_G/C_o	H/H_o'	M
.002000	.00002514	.01257	.01257	.01257	1.0001	.9999	.02513	.02514	1.000	.9999	.01257	6.308	31,250
.002100	.00002772	.01319	.01319	.01320	1.0001	.9999	.02639	.02639	1.000	.9999	.01319	6.156	28,350
.002200	.00003040	.01382	.01382	.01382	1.0001	.9999	.02765	.02765	1.000	.9999	.01382	6.015	25,830
.002300	.00003324	.01445	.01445	.01445	1.0001	.9999	.02890	.02891	1.000	.9999	.01445	5.882	23,630
.002400	.00003619	.01508	.01508	.01508	1.0001	.9999	.03016	.03016	1.000	.9999	.01508	5.759	21,700
.002500	.00003928	.01571	.01571	.01571	1.0001	.9999	.03142	.03142	1.000	.9999	.01571	5.642	20,000
.002600	.00004248	.01634	.01633	.01634	1.0001	.9999	.03267	.03268	1.001	.9999	.01633	5.533	18,490
.002700	.00004579	.01696	.01696	.01697	1.0001	.9999	.03393	.03394	1.001	.9999	.01696	5.429	17,150
.002800	.00004925	.01759	.01759	.01759	1.0002	.9998	.03519	.03519	1.001	.9999	.01759	5.332	15,950
.002900	.00005284	.01822	.01822	.01822	1.0002	.9998	.03644	.03645	1.001	.9999	.01822	5.239	14,870
.003000	.00005652	.01885	.01885	.01885	1.0002	.9998	.03770	.03771	1.001	.9999	.01885	5.151	13,890
.003100	.00006039	.01948	.01948	.01948	1.0002	.9998	.03896	.03897	1.001	.9999	.01947	5.067	13,010
.003200	.00006435	.02011	.02010	.02011	1.0002	.9998	.04021	.04022	1.001	.9999	.02010	4.987	12,210
.003300	.00006841	.02073	.02073	.02073	1.0002	.9998	.04147	.04148	1.001	.9999	.02073	4.911	11,480
.003400	.00007262	.02136	.02136	.02136	1.0002	.9998	.04273	.04274	1.001	.9998	.02136	4.838	10,820
.003500	.00007697	.02199	.02199	.02199	1.0002	.9998	.04398	.04399	1.001	.9998	.02199	4.769	10,210
.003600	.00008140	.02262	.02262	.02262	1.0003	.9997	.04524	.04525	1.001	.9998	.02261	4.702	9,648
.003700	.00008599	.02325	.02324	.02325	1.0003	.9997	.04650	.04652	1.001	.9998	.02324	4.638	9,134
.003800	.00009071	.02388	.02387	.02388	1.0003	.9997	.04775	.04777	1.001	.9998	.02387	4.577	8,660
.033900	.00009551	.02450	.02450	.02451	1.0003	.9997	.04901	.04903	1.001	.9998	.02449	4.518	8,221
.004000	.0001005	.02513	.02513	.02513	1.0003	.9997	.05027	.05029	1.001	.9998	.02511	4.462	7,815
.004100	.0001056	.02576	.02576	.02576	1.0003	.9997	.05152	.05154	1.001	.9998	.02574	4.407	7,439
.004200	.0001108	.02639	.02638	.02639	1.0003	.9997	.05278	.05280	1.001	.9998	.02637	4.354	7,090
.004300	.0001161	.02702	.02701	.02702	1.0004	.9996	.05404	.05406	1.001	.9998	.02700	4.303	6,764
.004400	.0001216	.02765	.02764	.02765	1.0004	.9996	.05529	.05531	1.002	.9997	.02763	4.254	6,460
.004500	.0001272	.02827	.02827	.02828	1.0004	.9996	.05655	.05658	1.002	.9997	.02825	4.207	6,176
.004600	.0001329	.02890	.02889	.02890	1.0004	.9996	.05781	.05784	1.002	.9997	.02888	4.161	5,911
.004700	.0001387	.02953	.02952	.02953	1.0004	.9996	.05906	.05909	1.002	.9997	.02951	4.116	5,662
.004800	.0001447	.03016	.03015	.03016	1.0005	.9995	.06032	.06035	1.002	.9997	.03014	4.073	5,429
.004900	.0001508	.03079	.03078	.03079	1.0005	.9995	.06158	.06161	1.002	.9997	.03076	4.032	5,209
.005000	.0001570	.03142	.03141	.03143	1.0005	.9995	.06283	.06287	1.002	.9997	.03139	3.991	5,003
.005100	.0001634	.03204	.03203	.03205	1.0005	.9995	.06409	.06413	1.002	.9997	.03202	3.951	4,809
.005200	.0001698	.03267	.03266	.03268	1.0005	.9995	.06535	.06539	1.002	.9996	.03265	3.913	4,626
.005300	.0001764	.03330	.03329	.03331	1.0005	.9995	.06660	.06665	1.002	.9996	.03328	3.876	4,453
.005400	.0001832	.03393	.03392	.03394	1.0006	.9994	.06786	.06791	1.002	.9996	.03391	3.840	4,290
.005500	.0001900	.03456	.03455	.03457	1.0006	.9994	.06911	.06916	1.002	.9996	.03454	3.805	4,135
.005600	.0001970	.03519	.03517	.03520	1.0006	.9994	.07037	.07042	1.002	.9996	.03517	3.771	3,989
.005700	.0002041	.03581	.03580	.03582	1.0006	.9994	.07163	.07169	1.003	.9996	.03579	3.738	3,851
.005800	.0002112	.03644	.03642	.03645	1.0007	.9993	.07288	.07294	1.003	.9996	.03641	3.706	3,719
.005900	.0002186	.03707	.03705	.03708	1.0007	.9993	.07414	.07420	1.003	.9995	.03703	3.675	3,594
.006000	.0002261	.03770	.03768	.03771	1.0007	.9993	.07540	.07547	1.003	.9995	.03766	3.644	3,475
.006100	.0002337	.03833	.03831	.03834	1.0007	.9993	.07665	.07672	1.003	.9995	.03829	3.614	3,363
.006200	.0002414	.03896	.03894	.03897	1.0008	.9992	.07791	.07798	1.003	.9995	.03892	3.584	3,255
.006300	.0002492	.03958	.03956	.03959	1.0008	.9992	.07917	.07925	1.003	.9995	.03954	3.556	3,153
.006400	.0002570	.04021	.04019	.04022	1.0008	.9992	.08042	.08050	1.003	.9995	.04017	3.528	3,055
.006500	.0002653	.04084	.04082	.04085	1.0008	.9992	.08168	.08177	1.003	.9994	.04080	3.501	2,962
.006600	.0002735	.04147	.04144	.04148	1.0009	.9991	.08294	.08303	1.003	.9994	.04142	3.475	2,873
.006700	.0002819	.04210	.04207	.04211	1.0009	.9991	.08419	.08428	1.004	.9994	.04204	3.449	2,788
.006800	.0002904	.04273	.04270	.04274	1.0009	.9991	.08545	.08555	1.004	.9994	.04267	3.423	2,707
.006900	.0002990	.04335	.04333	.04336	1.0009	.9991	.08671	.08681	1.004	.9994	.04330	3.398	2,629
.007000	.0003077	.04398	.04395	.04399	1.0010	.9990	.08796	.08807	1.004	.9994	.04392	3.374	2,554
.007100	.0003165	.04461	.04458	.04462	1.0010	.9990	.08922	.08933	1.004	.9993	.04455	3.350	2,483
.007200	.0003254	.04524	.04521	.04525	1.0010	.9989	.09048	.09060	1.004	.9993	.04518	3.327	2,415
.007300	.0003346	.04587	.04584	.04589	1.0011	.9989	.09173	.09185	1.004	.9993	.04581	3.304	2,349
.007400	.0003439	.04650	.04646	.04652	1.0011	.9989	.09299	.09312	1.004	.9993	.04644	3.281	2,286
.007500	.0003532	.04712	.04709	.04714	1.0011	.9989	.09425	.09438	1.004	.9993	.04706	3.260	2,226
.007600	.0003627	.04775	.04772	.04777	1.0011	.9989	.09550	.09565	1.005	.9992	.04768	3.238	2,167
.007700	.0003722	.04838	.04834	.04840	1.0012	.9988	.09676	.09681	1.005	.9992	.04830	3.217	2,112
.007800	.0003820	.04901	.04897	.04903	1.0012	.9988	.09802	.09817	1.005	.9992	.04893	3.197	2,058
.007900	.0003918	.04964	.04960	.04966	1.0012	.9988	.09927	.09943	1.005	.9992	.04956	3.176	2,006

Table D-2 Cont'd

d/L	d/Lo	2πd/L	TANH 2πd/L	SINH 2πd/L	COSH 2πd/L	K	4πd/L	SINH 4πd/L	COSH 4πd/L	n	C_G/C_o	H/H'_o	M
.008000	.0004018	.05027	.05022	.05029	1.0013	.9987	.1005	.1007	1.005	.9992	.05018	3.157	1,956
.008100	.0004118	.05089	.05085	.05091	1.0013	.9987	.1018	.1020	1.005	.9991	.05080	3.137	1,909
.008200	.0004221	.05152	.05147	.05154	1.0013	.9987	.1030	.1032	1.005	.9991	.05142	3.118	1,862
.008300	.0004324	.05215	.05210	.05217	1.0014	.9986	.1043	.1045	1.005	.9991	.05205	3.099	1,818
.008400	.0004429	.05278	.05273	.05280	1.0014	.9986	.1056	.1058	1.006	.9991	.05268	3.081	1,775
.008500	.0004536	.05341	.05336	.05343	1.0014	.9986	.1068	.1070	1.006	.9991	.05331	3.062	1,733
.008600	.0004644	.05404	.05398	.05406	1.0015	.9985	.1081	.1083	1.006	.9990	.05394	3.044	1,693
.008700	.0004751	.05466	.05461	.05469	1.0015	.9985	.1093	.1095	1.006	.9990	.05456	3.027	1,655
.008800	.0004860	.05529	.05524	.05533	1.0015	.9985	.1106	.1108	1.006	.9990	.05518	3.010	1,617
.008900	.0004972	.05592	.05586	.05595	1.0016	.9984	.1118	.1121	1.006	.9990	.05580	2.993	1,581
.009000	.0005084	.05655	.05649	.05658	1.0016	.9984	.1131	.1133	1.006	.9989	.05643	2.977	1,546
.009100	.0005198	.05718	.05712	.05721	1.0016	.9984	.1144	.1146	1.006	.9989	.05706	2.960	1,513
.009200	.0005312	.05781	.05774	.05784	1.0017	.9983	.1156	.1158	1.007	.9989	.05768	2.944	1,480
.009300	.0005427	.05843	.05836	.05846	1.0017	.9983	.1169	.1171	1.007	.9989	.05830	2.929	1,449
.009400	.0005545	.05906	.05899	.05909	1.0017	.9983	.1181	.1184	1.007	.9988	.05892	2.913	1,418
.009500	.0005664	.05969	.05962	.05973	1.0018	.9982	.1194	.1196	1.007	.9988	.05955	2.898	1,388
.009600	.0005784	.06032	.06025	.06036	1.0018	.9982	.1206	.1209	1.007	.9988	.06018	2.882	1,360
.009700	.0005905	.06095	.06087	.06099	1.0019	.9981	.1219	.1222	1.007	.9988	.06080	2.867	1,332
.009800	.0006027	.06158	.06150	.06162	1.0019	.9981	.1232	.1235	1.008	.9987	.06142	2.853	1,305
.009900	.0006150	.06220	.06212	.06224	1.0019	.9981	.1244	.1247	1.008	.9987	.06204	2.839	1,279
.01000	.0006275	.06283	.06275	.06287	1.0020	.9980	.1257	.1260	1.0079	.9987	.06267	2.825	1,253
.01100	.0007591	.06912	.06901	.06917	1.0024	.9976	.1382	.1387	1.0096	.9984	.06890	2.694	1,036
.01200	.0009031	.07540	.07526	.07547	1.0028	.9972	.1508	.1513	1.0114	.9981	.07511	2.580	871.0
.01300	.001060	.08168	.08150	.08177	1.0033	.9967	.1634	.1641	1.0134	.9978	.08131	2.480	742.9
.01400	.001228	.08795	.08774	.08808	1.0039	.9961	.1759	.1768	1.0155	.9974	.08751	2.389	641.1
.01500	.001410	.09425	.09397	.09439	1.0044	.9956	.1885	.1896	1.0178	.9970	.09369	2.310	558.9
.01600	.001603	.1005	.1002	.1007	1.0051	.9949	.2011	.2024	1.0203	.9966	.09986	2.238	491.6
.01700	.001809	.1068	.1064	.1070	1.0057	.9943	.2136	.2153	1.0229	.9962	.1060	2.172	435.8
.01800	.002027	.1131	.1126	.1133	1.0064	.9936	.2262	.2281	1.0257	.9958	.1121	2.112	389.1
.01900	.002258	.1194	.1188	.1197	1.0071	.9929	.2388	.2410	1.0286	.9953	.1183	2.056	349.5
.02000	.002500	.1257	.1250	.1260	1.008	.9922	.2513	.2540	1.032	.9947	.1244	2.005	315.8
.02100	.002755	.1320	.1312	.1323	1.009	.9914	.2639	.2669	1.035	.9942	.1305	1.958	286.8
.02200	.003022	.1382	.1374	.1387	1.010	.9905	.2765	.2800	1.038	.9937	.1365	1.915	261.5
.02300	.003301	.1445	.1435	.1450	1.011	.9896	.2890	.2931	1.042	.9931	.1425	1.873	239.6
.02400	.003592	.1508	.1497	.1514	1.011	.9887	.3016	.3062	1.046	.9925	.1485	1.834	220.3
.02500	.003895	.1571	.1558	.1577	1.012	.9878	.3142	.3194	1.050	.9919	.1545	1.799	203.3
.02600	.004210	.1634	.1619	.1641	1.013	.9868	.3267	.3326	1.054	.9912	.1605	1.765	188.2
.02700	.004537	.1697	.1680	.1705	1.014	.9858	.3393	.3458	1.058	.9905	.1665	1.733	174.8
.02800	.004876	.1759	.1741	.1768	1.016	.9847	.3519	.3592	1.063	.9898	.1724	1.703	162.7
.02900	.005226	.1822	.1802	.1832	1.017	.9836	.3644	.3725	1.067	.9891	.1783	1.675	151.9
.03000	.005589	.1885	.1863	.1896	1.018	.9825	.3770	.3860	1.072	.9884	.1841	1.648	142.2
.03100	.005963	.1948	.1924	.1960	1.019	.9813	.3896	.3995	1.077	.9876	.1900	1.622	133.4
.03200	.006347	.2011	.1984	.2024	1.020	.9801	.4021	.4131	1.082	.9868	.1958	1.598	125.4
.03300	.006746	.2073	.2044	.2088	1.022	.9789	.4147	.4267	1.087	.9860	.2016	1.575	118.1
.03400	.007155	.2136	.2104	.2153	1.023	.9776	.4273	.4404	1.093	.9851	.2073	1.553	111.4
.03500	.007575	.2199	.2164	.2217	1.024	.9763	.4398	.4541	1.098	.9843	.2130	1.532	105.3
.03600	.008007	.2262	.2224	.2281	1.026	.9749	.4524	.4680	1.104	.9834	.2187	1.512	99.75
.03700	.008450	.2325	.2284	.2346	1.027	.9736	.4650	.4819	1.110	.9824	.2244	1.493	94.61
.03800	.008905	.2388	.2343	.2410	1.029	.9722	.4775	.4959	1.116	.9815	.2300	1.475	89.88
.03900	.009370	.2450	.2403	.2527	1.030	.9708	.4901	.5099	1.123	.9805	.2356	1.457	85.50
.04000	.009847	.2513	.2462	.2540	1.032	.9693	.5027	.5241	1.129	.9795	.2411	1.440	81.43
.04100	.01033	.2576	.2521	.2605	1.033	.9677	.5152	.5383	1.136	.9785	.2467	1.424	77.67
.04200	.01083	.2639	.2579	.2670	1.035	.9662	.5278	.5526	1.143	.9775	.2521	1.408	74.17
.04300	.01134	.2702	.2638	.2735	1.037	.9646	.5404	.5670	1.150	.9765	.2576	1.393	70.91
.04400	.01186	.2765	.2696	.2800	1.039	.9630	.5529	.5815	1.157	.9754	.2630	1.379	67.88
.04500	.01239	.2827	.2754	.2865	1.040	.9613	.5655	.5961	1.164	.9743	.2684	1.365	65.05
.04600	.01294	.2890	.2812	.2931	1.042	.9596	.5781	.6108	1.172	.9732	.2737	1.352	62.39
.04700	.01349	.2953	.2870	.2996	1.044	.9579	.5906	.6256	1.180	.9721	.2790	1.339	59.91
.04800	.01405	.3016	.2928	.3062	1.046	.9562	.6032	.6404	1.188	.9709	.2843	1.326	57.57
.04900	.01463	.3079	.2985	.3128	1.048	.9544	.6158	.6554	1.196	.9697	.2895	1.314	55.38

Table D-2 Cont'd

d/L	d/Lo	2πd/L	TANH 2πd/L	SINH 2πd/L	COSH 2πd/L	K	4πd/L	SINH 4πd/L	COSH 4πd/L	n	C_G/C_o	H/H_o'	M
.05000	.01521	.3142	.3042	.3194	1.050	.9526	.6283	.6705	1.204	.9685	.2947	1.303	53.32
.05100	.01580	.3204	.3099	.3260	1.052	.9508	.6409	.6857	1.213	.9673	.2998	1.291	51.38
.05200	.01641	.3267	.3156	.3326	1.054	.9489	.6535	.7010	1.221	.9661	.3049	1.281	49.55
.05300	.01702	.3330	.3212	.3392	1.056	.9470	.6660	.7164	1.230	.9649	.3099	1.270	47.82
.05400	.01765	.3393	.3269	.3458	1.058	.9451	.6786	.7319	1.239	.9636	.3149	1.260	46.19
.05500	.01829	.3456	.3325	.3525	1.060	.9431	.6912	.7475	1.249	.9623	.3199	1.250	44.65
.05600	.01893	.3519	.3380	.3592	1.063	.9411	.7037	.7633	1.258	.9610	.3248	1.241	43.19
.05700	.01958	.3581	.3436	.3658	1.065	.9391	.7163	.7791	1.268	.9597	.3297	1.231	41.80
.05800	.02025	.3644	.3491	.3726	1.067	.9371	.7289	.7951	1.278	.9583	.3346	1.222	40.49
.05900	.02092	.3707	.3546	.3793	1.070	.9350	.7414	.8112	1.288	.9570	.3394	1.214	39.24
.06000	.02161	3770	.3601	.3860	1.072	.9329	.7540	.8275	1.298	.9556	.3441	1.205	38.06
.06100	.02230	.3833	.3656	.3927	1.074	.9308	.7666	.8439	1.308	.9542	.3488	1.197	36.93
.06200	.02300	.3896	.3710	.3995	1.077	.9286	.7791	.8604	1.319	.9528	.3534	1.189	35.86
.06300	.02371	.3958	.3764	.4062	1.079	.9265	.7917	.8770	1.330	.9514	.3581	1.182	34.83
.06400	.02444	.4021	.3818	.4130	1.082	.9243	.8043	.8938	1.341	.9499	.3626	1.174	33.86
.06500	.02516	.4084	.3871	.4199	1.085	.9220	.8168	.9107	1.353	.9484	.3672	1.167	32.93
.06600	.02590	.4147	.3925	.4267	1.087	.9198	.8294	.9278	1.364	.9470	.3716	1.160	32.04
.06700	.02665	.4210	.3978	.4335	1.090	.9175	.8419	.9450	1.376	.9455	.3761	1.153	31.19
.06800	.02739	.4273	.4030	.4404	1.093	.9152	.8545	.9624	1.388	.9440	.3804	1.147	30.38
.06900	.02817	.4335	.4083	.4473	1.095	.9128	.8671	.9799	1.400	.9424	.3848	1.140	29.61
.07000	.02895	.4398	.4135	.4541	1.098	.9105	.8796	.9976	1.412	.9409	.3891	1.134	28.86
.07100	.02973	.4461	.4187	.4611	1.101	.9081	.8922	1.015	1.425	.9393	.3933	1.128	28.15
.07200	.03052	.4524	.4239	.4680	1.104	.9057	.9048	1.033	1.438	.9378	.3975	1.122	27.47
.07300	.03132	.4587	.4290	.4749	1.107	.9033	.9173	1.052	1.451	.9362	.4016	1.116	26.81
.07400	.03213	.4650	.4341	.4819	1.110	.9008	.9299	1.070	1.464	.9346	.4057	1.110	26.18
.07500	.03294	.4712	.4392	.4889	1.113	.8984	.9425	1.088	1.478	.9330	.4098	1.105	25.58
.07600	.03377	.4775	.4443	.4958	1.116	.8959	.9551	1.107	1.492	.9314	.4138	1.099	25.00
.07700	.03460	.4838	.4493	.5029	1.119	.8934	.9676	1.126	1.506	.9298	.4177	1.094	24.45
.07800	.03543	.4901	.4542	.5100	1.123	.8909	.9802	1.145	1.520	.9281	.4216	1.089	23.92
.07900	.03628	.4964	.4593	.5170	1.126	.8883	.9927	1.164	1.534	.9264	.4255	1.084	23.40
.08000	.03714	.5027	.4642	.5241	1.129	.8857	1.005	1.183	1.549	.9248	.4293	1.079	22.90
.08100	.03799	.5089	.4691	.5312	1.132	.8831	1.018	1.203	1.564	.9231	.4330	1.075	22.42
.08200	.03887	.5152	.4740	.5383	1.136	.8805	1.030	1.223	1.580	.9214	.4367	1.070	21.96
.08300	.03975	.5215	.4789	.5455	1.139	.8779	1.043	1.243	1.595	.9197	.4404	1.066	21.52
.08400	.04063	.5278	.4837	.5526	1.143	.8752	1.056	1.263	1.611	.9179	.4440	1.061	21.09
.08500	.04152	.5341	.4885	.5598	1.146	.8726	1.068	1.283	1.627	.9162	.4476	1.057	20.68
.08600	.04242	.5404	.4933	.5670	1.150	.8699	1.081	1.304	1.643	.9145	.4511	1.053	20.28
.08700	.04333	.5466	.4980	.5743	1.153	.8672	1.093	1.324	1.660	.9127	.4545	1.049	19.90
.08800	.04424	.5529	.5027	.5815	1.157	.8645	1.106	1.346	1.676	.9109	.4579	1.045	19.53
.08900	.04516	.5592	.5074	.5888	1.160	.8617	1.118	1.367	1.693	.9092	.4613	1.041	19.17
.09000	.04608	.5655	.5120	.5961	1.164	.8590	1.131	1.388	1.711	.9074	.4646	1.037	18.82
.09100	.04702	.5718	.5167	.6034	1.168	.8562	1.144	1.410	1.728	.9056	.4679	1.034	18.49
.09200	.04796	.5781	.5213	.6108	1.172	.8534	1.156	1.431	1.746	.9038	.4711	1.030	18.16
.09300	.04890	.5843	.5258	.6182	1.176	.8506	1.169	1.453	1.764	.9020	.4743	1.027	17.85
.09400	.04985	.5906	.5303	.6256	1.180	.8478	1.181	1.476	1.783	.9002	.4774	1.023	17.55
.09500	.05081	.5969	.5348	.6330	1.184	.8450	1.194	1.498	1.801	.8984	.4805	1.020	17.26
.09600	.05177	.6032	.5393	.6404	1.188	.8421	1.206	1.521	1.820	.8966	.4835	1.017	16.97
.09700	.05275	.6095	.5438	.6479	1.192	.8392	1.219	1.544	1.840	.8947	.4865	1.014	16.69
.09800	.05372	.6158	.5482	.6554	1.196	.8364	1.232	1.567	1.859	.8929	.4894	1.011	16.42
.09900	.05470	.6220	.5526	.6629	1.200	.8335	1.244	1.591	1.879	.8910	.4923	1.008	16.16
.1000	.05569	.6283	.5569	.6705	1.204	.8306	1.257	1.615	1.899	.8892	.4952	1.005	15.91
.1010	.05668	.6346	.5612	.6781	1.208	.8277	1.269	1.638	1.920	.8873	.4980	1.002	15.67
.1020	.05768	.6409	.5655	.6857	1.213	.8247	1.282	1.663	1.940	.8854	.5007	.9993	15.43
.1030	.05869	.6472	.5698	.6933	1.217	.8218	1.294	1.687	1.961	.8836	.5034	.9966	15.20
.1040	.05970	.6535	.5740	.7010	1.221	.8189	1.307	1.712	1.983	.8817	.5061	.9940	14.98
.1050	.06071	.6597	.5782	.7087	1.226	.8159	1.319	1.737	2.004	.8798	.5087	.9914	14.76
.1060	.06173	.6660	.5824	.7164	1.230	.8129	1.332	1.762	2.026	.8779	.5113	.9891	14.55
.1070	.06276	.6723	.5865	.7241	1.235	.8100	1.345	1.788	2.049	.8760	.5138	.9865	14.35
.1080	.06378	.6786	.5906	.7319	1.239	.8070	1.357	1.814	2.071	.8741	.5163	.9841	14.15
.1090	.06482	.6849	.5947	.7397	1.244	.8040	1.370	1.840	2.094	.8722	.5187	.9818	13.95

Table D-2 Cont'd

d/L	d/Lo	$2\pi d/L$	TANH $2\pi d/L$	SINH $2\pi d/L$	COSH $2\pi d/L$	K	$4\pi d/L$	SINH $4\pi d/L$	COSH $4\pi d/L$	n	C_G/C_o	H/H_o'	M
.1100	.06586	.6912	.5987	.7475	1.249	.8010	1.382	1.867	2.118	.8703	.5211	.9797	13.77
.1110	.06690	.6974	.6027	.7554	1.253	.7980	1.395	1.893	2.141	.8684	.5234	.9775	13.58
.1120	.06795	.7037	.6067	.7633	1.258	.7949	1.407	1.920	2.165	.8665	.5257	.9753	13.41
.1130	.06901	.7100	.6107	.7712	1.263	.7919	1.420	1.948	2.189	.8645	.5279	.9731	13.23
.1140	.07006	.7163	.6146	.7791	1.268	.7888	1.433	1.975	2.214	.8626	.5301	.9711	13.06
.1150	.07113	.7226	.6185	.7871	1.273	.7858	1.445	2.003	2.239	.8607	.5323	.9691	12.90
.1160	.07220	.7289	.6224	.7951	1.278	.7827	1.458	2.032	2.264	.8587	.5344	.9672	12.74
.1170	.07327	.7351	.6262	.8032	1.283	.7797	1.470	2.060	2.290	.8568	.5365	.9654	12.59
.1180	.07434	.7414	.6300	.8112	1.288	.7766	1.483	2.089	2.316	.8549	.5386	.9635	12.43
.1190	.07542	.7477	.6338	.8193	1.293	.7735	1.495	2.118	2.343	.8529	.5406	.9617	12.29
.1200	.07650	.7540	.6375	.8275	1.298	.7704	1.508	2.148	2.369	.8510	.5425	.9600	12.14
.1210	.07759	.7603	.6412	.8357	1.303	.7673	1.521	2.178	2.397	.8491	.5444	.9583	12.00
.1220	.07868	.7666	.6449	.8439	1.309	.7642	1.533	2.208	2.424	.8471	.5463	.9567	11.87
.1230	.07978	.7728	.6486	.8521	1.314	.7612	1.546	2.239	2.452	.8452	.5482	.9551	11.73
.1240	.08085	.7791	.6520	.8604	1.319	.7581	1.558	2.270	2.480	.8432	.5500	.9535	11.61
.1250	.08198	.7854	.6558	.8687	1.325	.7549	1.571	2.301	2.509	.8413	.5517	.9520	11.48
.1260	.08308	.7917	.6594	.8770	1.330	.7518	1.583	2.333	2.538	.8393	.5534	.9505	11.35
.1270	.08419	.7980	.6629	.8854	1.336	.7487	1.596	2.365	2.568	.8374	.5551	.9490	11.23
.1280	.08530	.8043	.6664	.8938	1.341	.7456	1.609	2.398	2.598	.8354	.5568	.9476	11.11
.1290	.08642	.8105	.6699	.9022	1.347	.7424	1.621	2.430	2.628	.8335	.5584	.9463	11.00
.1300	.08753	.8168	.6733	.9107	1.353	.7393	1.634	2.464	2.659	.8316	.5599	.9450	10.89
.1310	.08866	.8231	.6768	.9192	1.358	.7362	1.646	2.497	2.690	.8296	.5614	.9437	10.78
.1320	.08978	.8294	.6801	.9278	1.364	.7331	1.659	2.531	2.722	.8277	.5629	.9424	10.67
.1330	.09091	.8357	.6835	.9364	1.370	.7299	1.671	2.566	2.754	.8257	.5644	.9412	10.56
.1340	.09204	.8420	.6868	.9450	1.376	.7268	1.684	2.600	2.786	.8238	.5658	.9401	10.46
.1350	.09317	.8482	.6902	.9537	1.382	.7237	1.696	2.636	2.819	.8218	.5672	.9389	10.36
.1360	.09431	.8545	.6934	.9624	1.388	.7205	1.709	2.671	2.852	.8199	.5685	.9378	10.26
.1370	.09544	.8608	.6967	.9711	1.394	.7174	1.722	2.707	2.886	.8179	.5698	.9367	10.17
.1380	.09659	.8671	.6999	.9799	1.400	.7142	1.734	2.744	2.920	.8160	.5711	.9357	10.07
.1390	.09773	.8734	.7031	.9887	1.406	.7111	1.747	2.781	2.955	.8141	.5724	.9347	9.983
.1400	.09888	.8797	.7063	.9976	1.412	.7080	1.759	2.818	2.990	.8121	.5736	.9337	9.894
.1410	.1000	.8859	.7094	1.006	1.419	.7048	1.772	2.856	3.026	.8102	.5748	.9327	9.806
.1420	.1012	.8922	.7125	1.015	1.425	.7017	1.784	2.894	3.062	.8083	.5759	.9318	9.721
.1430	.1023	.8985	.7156	1.024	1.432	.6985	1.797	2.933	3.099	.8064	.5770	.9309	9.638
.1440	.1035	.9048	.7186	1.033	1.438	.6954	1.810	2.972	3.136	.8044	.5781	.9300	9.556
.1450	.1046	.9111	.7216	1.042	1.445	.6923	1.822	3.012	3.173	.8025	.5791	.9292	9.476
.1460	.1058	.9174	.7247	1.052	1.451	.6891	1.835	3.052	3.211	.8006	.5801	.9284	9.398
.1470	.1070	.9236	.7276	1.061	1.458	.6860	1.847	3.092	3.250	.7987	.5811	.9276	9.321
.1480	.1081	.9299	.7306	1.070	1.464	.6829	1.860	3.133	3.289	.7968	.5821	.9268	9.246
.1490	.1093	.9362	.7335	1.079	1.471	.6797	1.872	3.175	3.329	.7949	.5830	.9261	9.173
.1500	.1105	.9425	.7364	1.088	1.478	.6766	1.885	3.217	3.369	.7930	.5839	.9254	9.101
.1510	.1116	.9488	.7392	1.098	1.485	.6734	1.898	3.260	3.410	.7911	.5848	.9247	9.031
.1520	.1128	.9551	.7421	1.107	1.492	.6703	1.910	3.303	3.451	.7892	.5856	.9240	8.962
.1530	.1140	.9613	.7449	1.116	1.499	.6672	1.923	3.346	3.493	.7873	.5864	.9234	8.894
.1540	.1151	.9676	.7477	1.126	1.506	.6641	1.935	3.391	3.535	.7854	.5872	.9228	8.828
.1550	.1163	.9739	.7504	1.135	1.513	.6610	1.948	3.435	3.578	.7835	.5880	.9222	8.763
.1560	.1175	.9802	.7531	1.145	1.520	.6579	1.960	3.481	3.621	.7816	.5887	.9216	8.700
.1570	.1187	.9865	.7558	1.154	1.527	.6547	1.973	3.526	3.665	.7797	.5893	.9211	8.638
.1580	.1199	.9928	.7585	1.164	1.535	.6516	1.985	3.573	3.710	.7779	.5900	.9205	8.577
.1590	.1210	.9990	.7612	1.174	1.542	.6485	1.998	3.620	3.755	.7760	.5907	.9200	8.517
.1600	.1222	1.005	.7638	1.183	1.549	.6454	2.011	3.667	3.801	.7741	.5913	.9196	8.459
.1610	.1234	1.012	.7664	1.193	1.557	.6423	2.023	3.715	3.847	.7723	.5919	.9191	8.401
.1620	.1246	1.018	.7690	1.203	1.564	.6392	2.036	3.764	3.894	.7704	.5925	.9186	8.345
.1630	.1258	1.024	.7716	1.213	1.572	.6361	2.048	3.813	3.942	.7686	.5930	.9182	8.290
.1640	.1270	1.030	.7741	1.223	1.580	.6331	2.061	3.863	3.990	.7667	.5935	.9179	8.236
.1650	.1281	1.037	.7766	1.233	1.587	.6300	2.073	3.913	4.039	.7649	.5940	.9175	8.183
.1660	.1293	1.043	.7791	1.243	1.595	.6269	2.086	3.964	4.088	.7631	.5945	.9171	8.131
.1670	.1305	1.049	.7815	1.253	1.603	.6239	2.099	4.016	4.138	.7613	.5950	.9167	8.079
.1680	.1317	1.056	.7840	1.263	1.611	.6208	2.111	4.068	4.189	.7595	.5954	.9164	8.029
.1690	.1329	1.062	.7864	1.273	1.619	.6177	2.124	4.121	4.241	.7576	.5958	.9161	7.980

Table D-2 Cont'd

d/L	d/Lo	$2\pi d/L$	TANH $2\pi d/L$	SINH $2\pi d/L$	COSH $2\pi d/L$	K	$4\pi d/L$	SINH $4\pi d/L$	COSH $4\pi d/L$	n	C_G/C_o	H/H_o'	M
.1700	.1341	1.068	.7887	1.283	1.627	.6147	2.136	4.175	4.293	.7558	.5962	.9158	7.932
.1710	.1353	1.074	.7911	1.293	1.635	.6117	2.149	4.229	4.346	.7540	.5965	.9155	7.885
.1720	.1365	1.081	.7935	1.304	1.643	.6086	2.161	4.284	4.399	.7523	.5969	.9153	7.838
.1730	.1377	1.087	.7958	1.314	1.651	.6056	2.174	4.340	4.454	.7505	.5972	.9150	7.793
.1740	.1389	1.093	.7981	1.325	1.660	.6026	2.187	4.396	4.508	.7487	.5975	.9148	7.748
.1750	.1401	1.100	.8004	1.335	1.668	.5995	2.199	4.453	4.564	.7469	.5978	.9146	7.704
.1760	.1413	1.106	.8026	1.345	1.676	.5965	2.212	4.511	4.620	.7451	.5980	.9144	7.661
.1770	.1425	1.112	.8048	1.356	1.685	.5935	2.224	4.569	4.677	.7434	.5983	.9142	7.619
.1780	.1437	1.118	.8070	1.367	1.693	.5905	2.237	4.628	4.735	.7416	.5985	.9140	7.577
.1790	.1449	1.125	.8092	1.377	1.702	.5875	2.249	4.688	4.793	.7399	.5987	.9138	7.536
.1800	.1460	1.131	.8114	1.388	1.711	.5845	2.262	4.749	4.853	.7382	.5989	.9137	7.496
.1810	.1472	1.137	.8135	1.399	1.720	.5816	2.275	4.810	4.918	.7364	.5991	.9136	7.457
.1820	.1484	1.144	.8156	1.410	1.728	.5786	2.287	4.872	4.974	.7347	.5992	.9135	7.419
.1830	.1496	1.150	.8177	1.420	1.737	.5757	2.300	4.935	5.035	.7330	.5993	.9134	7.381
.1840	.1508	1.156	.8198	1.431	1.746	.5727	2.312	4.999	5.098	.7313	.5995	.9133	7.343
.1850	.1520	1.162	.8218	1.442	1.755	.5697	2.325	5.063	5.161	.7296	.5996	.9132	7.307
.1860	.1532	1.169	.8239	1.454	1.764	.5668	2.337	5.129	5.225	.7279	.5997	.9131	7.271
.1870	.1544	1.175	.8259	1.465	1.773	.5639	2.350	5.195	5.290	.7262	.5997	.9131	7.235
.1880	.1556	1.181	.8278	1.476	1.783	.5610	2.362	5.262	5.356	.7245	.5998	.9131	7.201
.1890	.1568	1.188	.8298	1.487	1.792	.5581	2.375	5.329	5.422	.7228	.5998	.9130	7.167
.1900	.1580	1.194	.8318	1.498	1.801	.5551	2.388	5.398	5.490	.7212	.5998	.9130	7.133
.1910	.1592	1.200	.8337	1.510	1.811	.5522	2.400	5.467	5.558	.7195	.5998	.9130	7.100
.1920	.1604	1.206	.8356	1.521	1.820	.5493	2.413	5.538	5.625	.7179	.5998	.9130	7.068
.1930	.1616	1.213	.8375	1.533	1.830	.5465	2.425	5.609	5.697	.7162	.5998	.9130	7.036
.1940	.1628	1.219	.8393	1.544	1.840	.5436	2.438	5.681	5.768	.7146	.5998	.9131	7.005
.1950	.1640	1.225	.8412	1.556	1.849	.5408	2.450	5.754	5.840	.7129	.5997	.9131	6.974
.1960	.1652	1.232	.8430	1.567	1.859	.5379	2.463	5.827	5.913	.7113	.5997	.9131	6.944
.1970	.1664	1.238	.8448	1.579	1.869	.5350	2.476	5.902	5.988	.7097	.5996	.9132	6.914
.1980	.1676	1.244	.8466	1.591	1.879	.5322	2.488	5.978	6.061	.7081	.5995	.9133	6.885
.1990	.1688	1.250	.8484	1.603	1.889	.5294	2.501	6.055	6.137	.7065	.5994	.9133	6.856
.2000	.1700	1.257	.8501	1.614	1.899	.5266	2.513	6.132	6.213	.7049	.5993	.9134	6.828
.2010	.1712	1.263	.8519	1.626	1.909	.5238	2.526	6.211	6.291	.7033	.5992	.9135	6.801
.2020	.1724	1.269	.8535	1.638	1.920	.5210	2.538	6.290	6.369	.7018	.5990	.9137	6.774
.2030	.1736	1.276	.8552	1.651	1.930	.5182	2.551	6.371	6.449	.7002	.5988	.9138	6.747
.2040	.1748	1.282	.8570	1.663	1.940	.5154	2.564	6.452	6.529	.6987	.5987	.9139	6.720
.2050	.1760	1.288	.8586	1.675	1.951	.5127	2.576	6.535	6.611	.6971	.5986	.9140	6.694
.2060	.1772	1.294	.8602	1.687	1.961	.5099	2.589	6.619	6.694	.6956	.5984	.9141	6.669
.2070	.1784	1.301	.8619	1.700	1.972	.5071	2.601	6.703	6.777	.6941	.5982	.9142	6.644
.2080	.1796	1.307	.8635	1.712	1.983	.5044	2.614	6.789	6.862	.6925	.5980	.9144	6.619
.2090	.1808	1.313	.8651	1.725	1.994	.5016	2.626	6.876	6.948	.6910	.5978	.9146	6.594
.2100	.1820	1.320	.8667	1.737	2.004	.4989	2.639	6.963	7.035	.6895	.5976	.9147	6.570
.2110	.1832	1.326	.8682	1.750	2.015	.4962	2.652	7.052	7.123	.6880	.5973	.9149	6.547
.2120	.7844	1.332	.8697	1.762	2.026	.4935	2.664	7.143	7.219	.6865	.5971	.9151	6.524
.2130	.1856	1.338	.8713	1.775	2.037	.4908	2.677	7.234	7.302	.6850	.5969	.9153	6.501
.2140	.1868	1.345	.8728	1.788	2.049	.4881	2.689	7.326	7.394	.6835	.5966	.9155	6.479
.2150	.1880	1.351	.8743	1.801	2.060	.4854	2.702	7.420	7.487	.6821	.5963	.9157	6.457
.2160	.1892	1.357	.8757	1.814	2.071	.4828	2.714	7.514	7.580	.6806	.5960	.9159	6.435
.2170	.1904	1.364	.8772	1.827	2.083	.4801	2.727	7.610	7.675	.6792	.5958	.9161	6.413
.2180	.1915	1.370	.8786	1.840	2.094	.4775	2.739	7.707	7.772	.6777	.5955	.9164	6.393
.2190	.1927	1.376	.8801	1.853	2.106	.4749	2.752	7.805	7.869	.6763	.5952	.9166	6.372
.2200	.1939	1.382	.8815	1.867	2.118	.4722	2.765	7.905	7.968	.6749	.5949	.9168	6.351
.2210	.1951	1.389	.8829	1.880	2.129	.4696	2.777	8.006	8.068	.6735	.5946	.9170	6.331
.2220	.1963	1.395	.8842	1.893	2.141	.4670	2.790	8.108	8.169	.6720	.5943	.9173	6.312
.2230	.1975	1.401	.8856	1.907	2.153	.4644	2.802	8.211	8.272	.6706	.5939	.9175	6.292
.2240	.1987	1.407	.8869	1.920	2.165	.4619	2.815	8.316	8.375	.6692	.5936	.9178	6.273
.2250	.1999	1.414	.8883	1.934	2.177	.4593	2.827	8.422	8.481	.6679	.5933	.9181	6.254
.2260	.2011	1.420	.8896	1.948	2.189	.4567	2.840	8.529	8.587	.6665	.5929	.9183	6.236
.2270	.2022	1.426	.8909	1.962	2.202	.4542	2.853	8.637	8.695	.6651	.5925	.9186	6.218
.2280	.2034	1.433	.8922	1.975	2.214	.4516	2.865	8.756	8.800	.6637	.5921	.9189	6.200
.2290	.2046	1.439	.8935	1.989	2.227	.4491	2.878	8.859	8.915	.6624	.5918	.9191	6.182

Table D-2 Cont'd

d/L	d/Lo	$2\pi d/L$	TANH $2\pi d/L$	SINH $2\pi d/L$	COSH $2\pi d/L$	K	$4\pi d/L$	SINH $4\pi d/L$	COSH $4\pi d/L$	n	C_G/C_o	H/H_o'	M
.2300	.2058	1.445	.8947	2.003	2.239	.4466	2.890	8.971	9.027	.6611	.5915	.9194	6.165
.2310	.2070	1.451	.8960	2.017	2.252	.4441	2.903	9.085	9.140	.6597	.5911	.9197	6.148
.2320	.2082	1.458	.8972	2.032	2.264	.4416	2.915	9.201	9.255	.6584	.5907	.9200	6.131
.2330	.2093	1.464	.8984	2.046	2.277	.4391	2.928	9.318	9.372	.6571	.5904	.9203	6.114
.2340	.2105	1.470	.8996	2.060	2.290	.4366	2.941	9.437	9.489	.6558	.5900	.9206	6.097
.2350	.2117	1.477	.9008	2.075	2.303	.4342	2.953	9.557	9.609	.6545	.5896	.9209	6.081
.2360	.2129	1.483	.9020	2.089	2.316	.4318	2.966	9.678	9.730	.6532	.5892	.9212	6.066
.2370	.2141	1.489	.9032	2.104	2.329	.4293	2.978	9.801	9.852	.6519	.5888	.9215	6.050
.2380	.2152	1.495	.9043	2.118	2.343	.4269	2.991	9.926	9.976	.6507	.5884	.9218	6.034
.2390	.2164	1.502	.9055	2.133	2.356	.4244	3.003	10.05	10.10	.6494	.5880	.9221	6.019
.2400	.2176	1.508	.9066	2.148	2.370	.4220	3.016	10.18	10.23	.6481	.5876	.9225	6.004
.2410	.2188	1.514	.9077	2.163	2.383	.4196	3.029	10.31	10.36	.6469	.5872	.9228	5.990
.2420	.2199	1.521	.9088	2.178	2.397	.4172	3.041	10.44	10.49	.6456	.5868	.9231	5.976
.2430	.2211	1.527	.9099	2.193	2.410	.4149	3.054	10.57	10.62	.6444	.5863	.9234	5.961
.2440	.2223	1.533	.9110	2.208	2.424	.4125	3.066	10.71	10.75	.6432	.5859	.9238	5.947
.2450	.2234	1.539	.9120	2.224	2.438	.4101	3.079	10.84	10.89	.6420	.5855	.9241	5.933
.2460	.2246	1.546	.9131	2.239	2.452	.4078	3.091	10.98	11.03	.6408	.5851	.9244	5.919
.2470	.2258	1.552	.9141	2.255	2.466	.4055	3.104	11.12	11.17	.6396	.5846	.9248	5.906
.2480	.2270	1.558	.9151	2.270	2.480	.4032	3.116	11.26	11.31	.6384	.5842	.9251	5.893
.2490	.2281	1.565	.9162	2.286	2.495	.4008	3.129	11.40	11.45	.6372	.5838	.9255	5.880
.2500	.2293	1.571	.9172	2.301	2.509	.3985	3.142	11.55	11.59	.6360	.5833	.9258	5.867
.2510	.2305	1.577	.9182	2.317	2.524	.3962	3.154	11.70	11.74	.6348	.5829	.9262	5.854
.2520	.2316	1.583	.9191	2.333	2.538	.3940	3.167	11.84	11.89	.6337	.5824	.9265	5.841
.2530	.2328	1.590	.9201	2.349	2.553	.3917	3.179	11.99	12.04	.6325	.5820	.9269	5.829
.2540	.2339	1.596	.9210	2.365	2.568	.3894	3.192	12.15	12.19	.6314	.5815	.9273	5.817
.2550	.2351	1.602	.9220	2.381	2.583	.3872	3.204	12.30	12.34	.6303	.5811	.9276	5.805
.2560	.2363	1.609	.9229	2.398	2.598	.3849	3.217	12.46	12.50	.6291	.5807	.9280	5.793
.2570	.2374	1.615	.9239	2.414	2.613	.3827	3.230	12.61	12.65	.6280	.5802	.9283	5.782
.2580	.2386	1.621	.9248	2.430	2.628	.3805	3.242	12.77	12.81	.6269	.5797	.9287	5.770
.2590	.2398	1.627	.9257	2.447	2.643	.3783	3.255	12.94	12.98	.6258	.5793	.9291	5.759
.2600	.2409	1.634	.9266	2.464	2.659	.3761	3.267	13.10	13.14	.6247	.5788	.9294	5.748
.2610	.2421	1.640	.9275	2.480	2.674	.3739	3.280	13.27	13.31	.6236	.5784	.9298	5.737
.2620	.2432	1.646	.9283	2.497	2.690	.3717	3.292	13.44	13.47	.6225	.5779	.9301	5.726
.2630	.2444	1.653	.9292	2.514	2.706	.3696	3.305	13.61	13.64	.6215	.5775	.9305	5.716
.2640	.2455	1.659	.9301	2.531	2.722	.3674	3.318	13.78	13.81	.6204	.5770	.9309	5.705
.2650	.2467	1.665	.9309	2.548	2.737	.3653	3.330	13.95	13.99	.6193	.5765	.9313	5.695
.2660	.2478	1.671	.9317	2.566	2.754	.3632	3.343	14.13	14.17	.6183	.5761	.9316	5.685
.2670	.2490	1.678	.9326	2.583	2.770	.3610	3.355	14.31	14.34	.6172	.5756	.9320	5.675
.2680	.2501	1.684	.9334	2.600	2.786	.3589	3.368	14.49	14.53	.6162	.5752	.9324	5.665
.2690	.2513	1.690	.9342	2.618	2.803	.3568	3.380	14.67	14.71	.6152	.5747	.9328	5.655
.2700	.2524	1.697	.9350	2.636	2.819	.3547	3.393	14.86	14.89	.6142	.5742	.9331	5.645
.2710	.2536	1.703	.9357	2.653	2.835	.3527	3.405	15.05	15.08	.6132	.5737	.9335	5.636
.2720	.2547	1.709	.9365	2.671	2.852	.3506	3.418	15.24	15.27	.6122	.5733	.9339	5.627
.2730	.2559	1.715	.9373	2.689	2.869	.3485	3.431	15.43	15.46	.6112	.5728	.9343	5.617
.2740	.2570	1.722	.9381	2.707	2.886	.3465	3.443	15.63	15.66	.6102	.5724	.9346	5.608
.2750	.2582	1.728	.9388	2.726	2.903	.3444	3.456	15.83	15.86	.6092	.5719	.9350	5.599
.2760	.2593	1.734	.9396	2.744	2.920	.3424	3.468	16.03	16.06	.6082	.5714	.9354	5.590
.2770	.2605	1.740	.9403	2.762	2.938	.3404	3.481	16.23	16.26	.6072	.5710	.9358	5.582
.2780	.2616	1.747	.9410	2.781	2.955	.3384	3.493	16.43	16.47	.6063	.5705	.9362	5.573
.2790	.2627	1.753	.9417	2.799	2.973	.3364	3.506	16.64	16.67	.6053	.5701	.9366	5.565
.2800	.2639	1.759	.9424	2.818	2.990	.3344	3.519	16.85	16.88	.6044	.5696	.9369	5.556
.2810	.2650	1.766	.9431	2.837	3.008	.3324	3.531	17.07	17.10	.6035	.5691	.9373	5.548
.2820	.2662	1.772	.9438	2.856	3.026	.3305	3.544	17.28	17.31	.6025	.5687	.9377	5.540
.2830	.2673	1.778	.9445	2.875	3.044	.3285	3.556	17.50	17.53	.6016	.5682	.9381	5.532
.2840	.2684	1.784	.9452	2.894	3.062	.3266	3.569	17.72	17.75	.6007	.5677	.9384	5.524
.2850	.2696	1.791	.9458	2.913	3.080	.3247	3.581	17.95	17.98	.5998	.5673	.9388	5.516
.2860	.2707	1.797	.9465	2.933	3.099	.3227	3.594	18.18	18.20	.5989	.5668	.9392	5.509
.2870	.2718	1.803	.9472	2.952	3.117	.3208	3.607	18.40	18.43	.5980	.5664	.9396	5.501
.2880	.2730	1.810	.9478	2.972	3.136	.3189	3.619	18.64	18.67	.5971	.5659	.9400	5.493
.2890	.2741	1.816	.9484	2.992	3.154	.3170	3.632	18.88	18.90	.5962	.5654	.9404	5.486

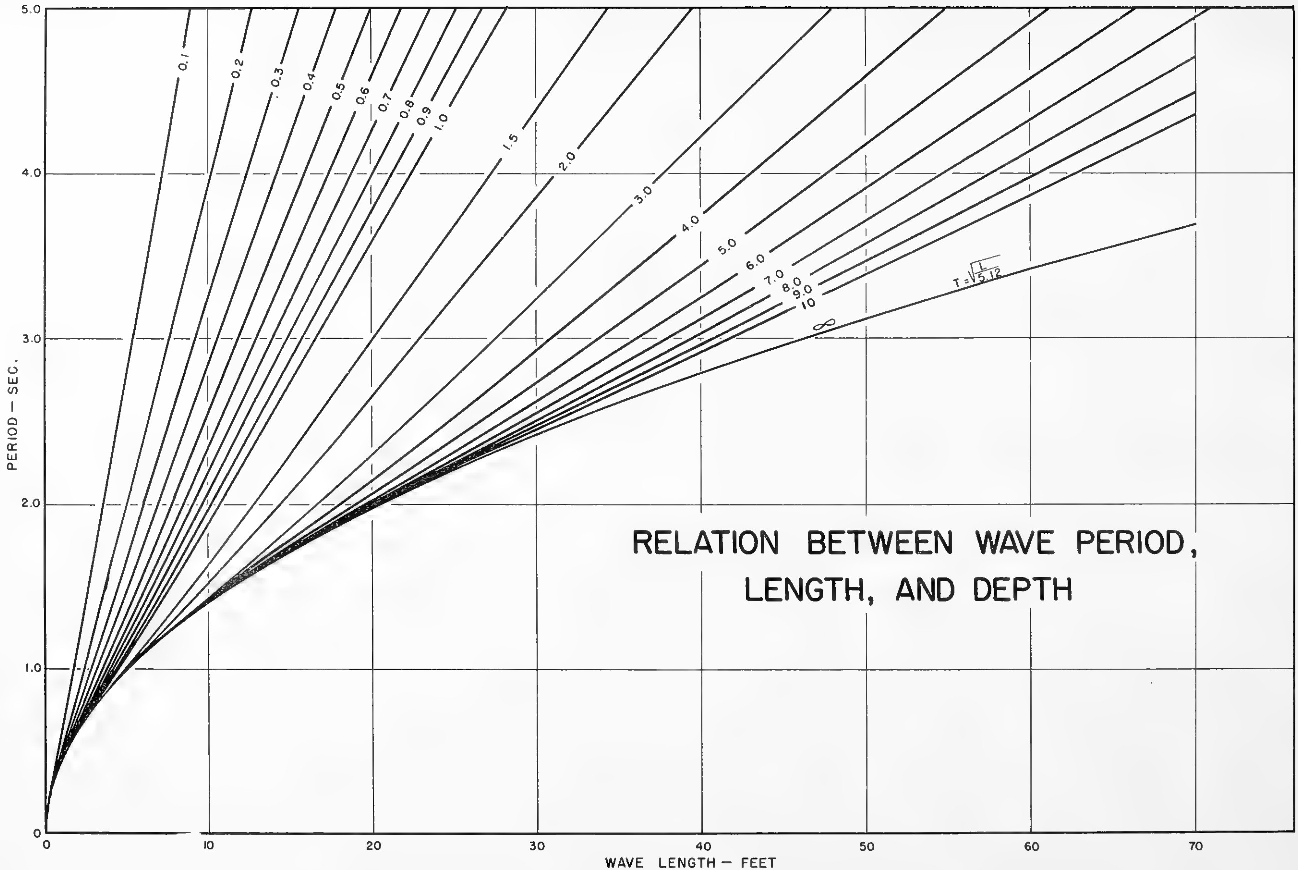
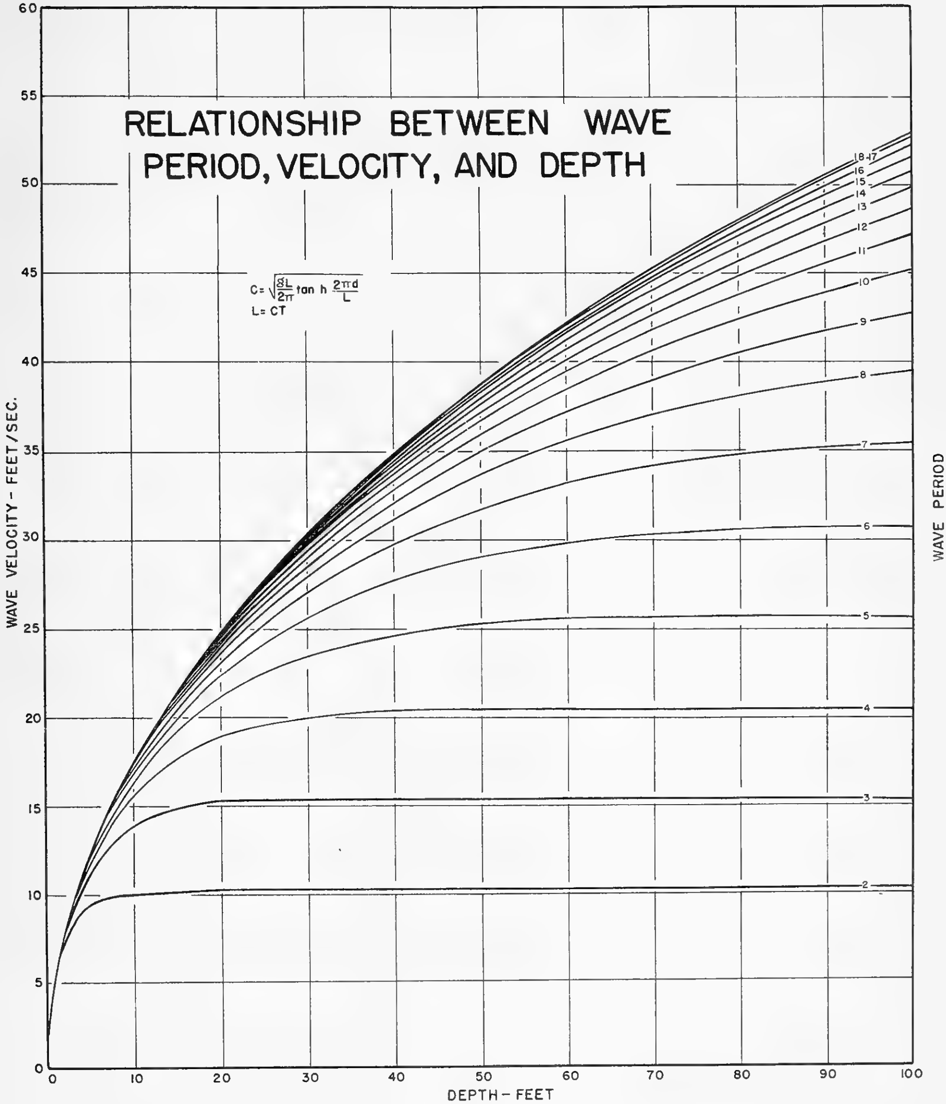

RELATION BETWEEN WAVE PERIOD, LENGTH, AND DEPTH
PERIOD — SEC.
WAVE LENGTH — FEET
0
1.0
2.0
3.0
4.0
5.0
0
10
20
30
40
50
60
70
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.5
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10
∞
$T=\sqrt{\frac{L}{5.12}}$

PLATE D-1



RELATIONSHIP BETWEEN WAVE PERIOD, VELOCITY, AND DEPTH
$C=\sqrt{\frac{gL}{2\pi}\tanh\frac{2\pi d}{L}}$
$L=CT$
WAVE VELOCITY - FEET/SEC.
DEPTH - FEET
WAVE PERIOD
0
5
10
15
20
25
30
35
40
45
50
55
60
0
10
20
30
40
50
60
70
80
90
100
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
18-17

PLATE D-2

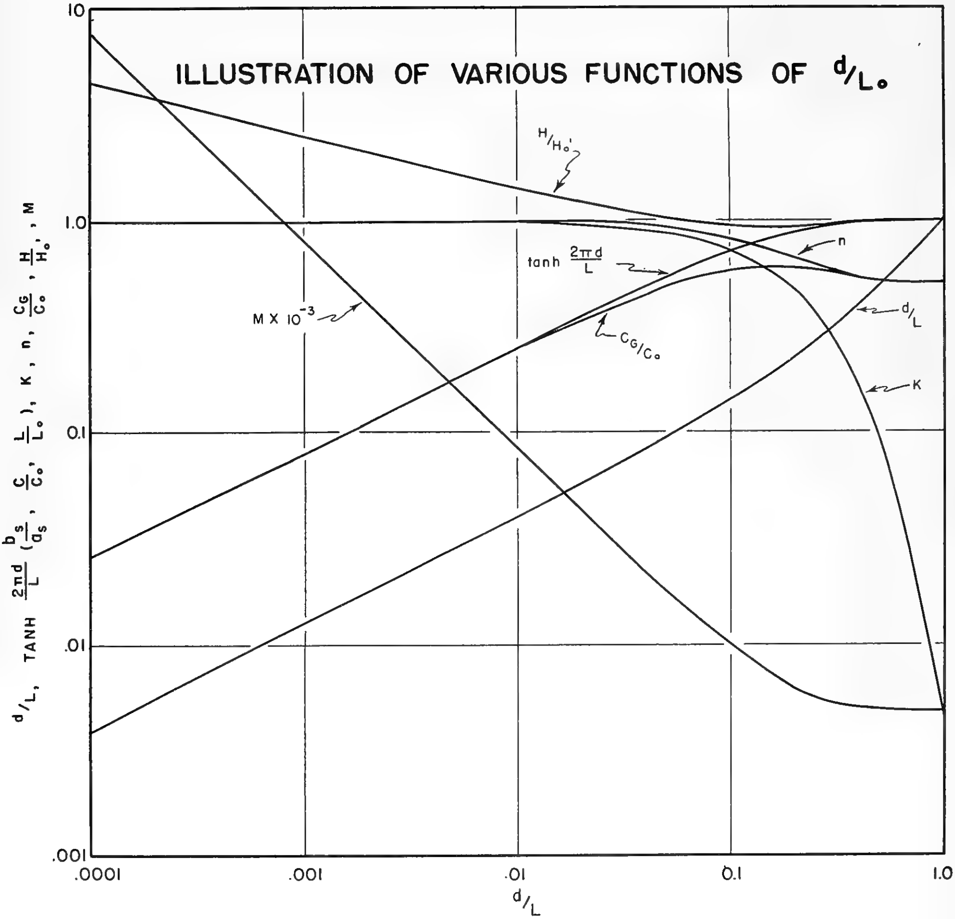


ILLUSTRATION OF VARIOUS FUNCTIONS OF d/Lo
H/Ho'
n
tanh 2πd/L
M X 10^-3
d/L
Cg/Co
K
10
1.0
0.1
.01
.001
.0001
.001
.01
0.1
1.0
d/L, TANH 2πd/L (bs/as, C/Co, L/Lo), K, n, Cg/Co, H/Ho', M
d/L

PLATE D-3

Table D-3 - Beaufort Scale of Wind Velocities

Scale	Description	Terms used in Weather Bureau Forecasts	(Statute Miles per hour)	(knots)
0	Calm		0 - 3	0 - 2.6
1	Light Air	Light	8	6.9
2	Light Breeze		13	11.3
3	Gentle Breeze	Gentle	18	15.6
4	Moderate Breeze	Moderate	23	20
5	Fresh Breeze	Fresh	28	24.3
6	Strong Breeze	Strong	34	29.5
7	Moderate Gale		40	34.7
8	Fresh Gale	Gale	48	41.6
9	Strong Gale		56	48.6
10	Whole Gale	Whole Gale	65	56.4
11	Storm		75	65.1
12	Hurricane	Hurricane	90 and over	78.1 and over

Table D-4 - Scale Showing State of Sea

Scale	Description	Height of Wave feet
0	Calm	0 - 1
1	Smooth	1 - 2
2	Slight	2 - 3
3	Moderate	3 - 5
4	Rough	5 - 8
5	Very rough	8 - 12
6	High	12 - 20
7	Very high	20 - 40
8	Precipitous	40 up
9	Confused	Record chief direction

Table D-5 - Douglas Sea Scale Showing Character of Sea Swell

Scale	Description
0	None
1	Low - short or average
2	Low - long
3	Moderate - short
4	Moderate - average
5	Moderate - long
6	Heavy - short
7	Heavy - average
8	Heavy - long
9	Confused

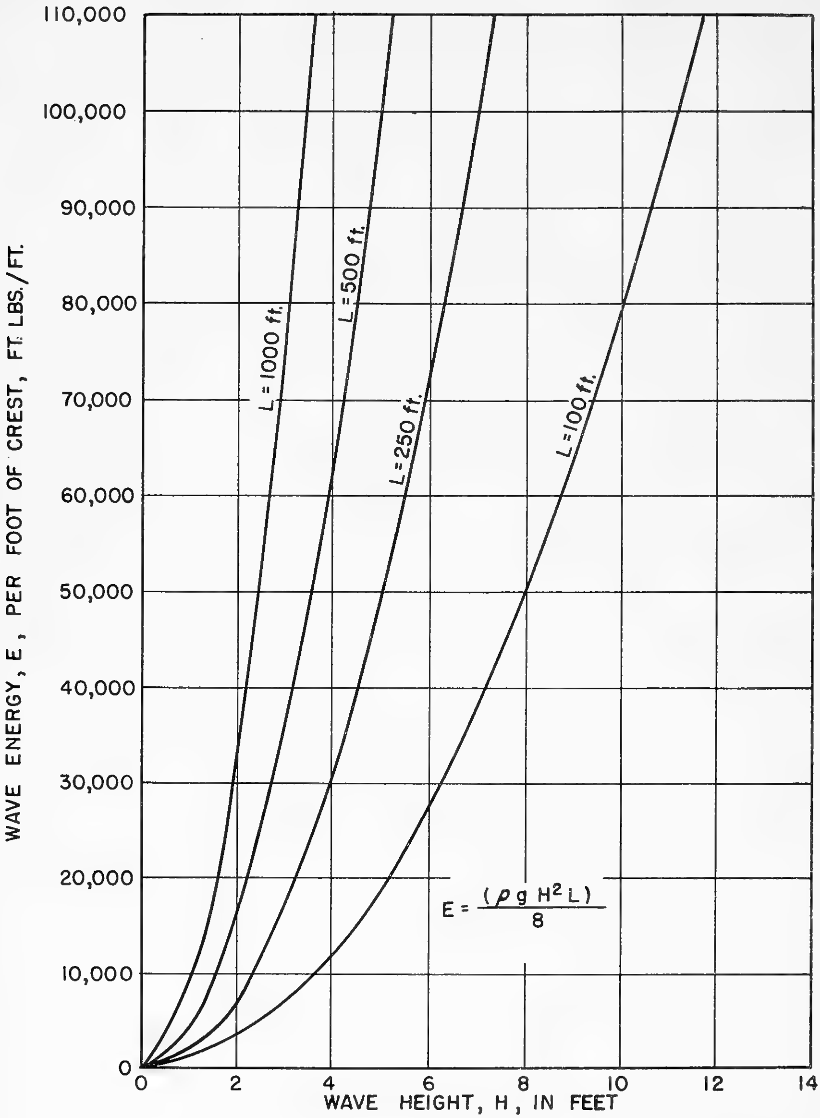


PLATE D-4 RELATIONSHIP BETWEEN WAVE ENERGY, WAVE LENGTH, AND WAVE HEIGHT

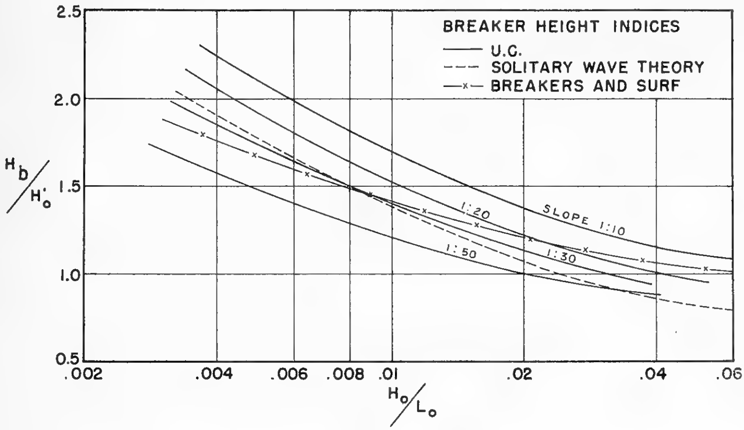


PLATE D-5

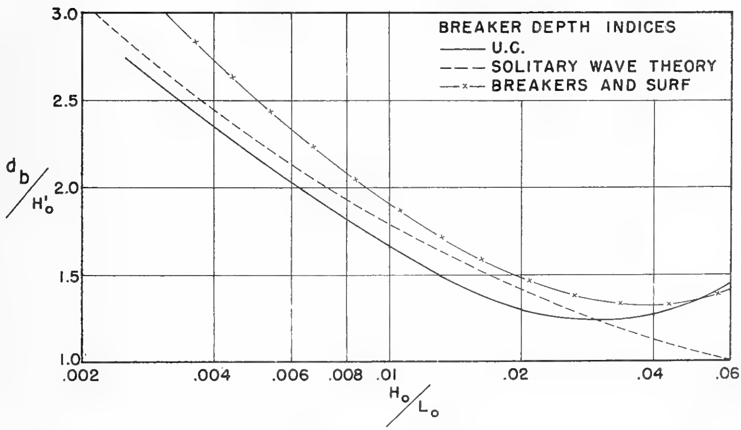


PLATE D-6

Table D-6 Deep water wave length (L_o) and velocity (C_o) as a function of wave period (T).

T (Seconds)	C_o (Ft./Sec)	C_o (Knots)	L_o (Feet)	T (Seconds)	C_o (Ft./Sec)	C_o (Knots)	L_o (Feet)	T (Seconds)	C_o (Ft./Sec)	C_o (Knots)	L_o (Feet)	T (Seconds)	C_o (Ft./Sec)	C_o (Knots)	L_o (Feet)
3.0	15.4	9.1	46.1	7.6	38.9	23.9	296	12.2	62.4	37.0	762	16.8	86.0	50.9	1444
3.1	15.9	9.4	49.2	7.7	39.4	23.3	304	12.3	63.0	37.3	775	16.9	86.5	51.2	1461
3.2	16.4	9.7	52.4	7.8	39.9	23.6	312	12.4	63.5	37.6	787	17.0	87.0	51.5	1479
3.3	16.9	10.0	55.8	7.9	40.4	23.9	320	12.5	64.0	37.9	800	17.1	87.5	51.8	1496
3.4	17.4	10.3	59.2	8.0	40.9	24.2	328	12.6	64.5	38.2	813	17.2	88.0	52.1	1514
3.5	17.9	10.6	62.7	8.1	41.4	24.5	336	12.7	65.0	38.5	826	17.3	88.5	52.4	1531
3.6	18.4	10.9	66.4	8.2	42.0	24.8	344	12.8	65.5	38.8	839	17.4	89.0	52.7	1549
3.7	18.9	11.2	70.1	8.3	42.5	25.1	353	12.9	66.0	39.1	852	17.5	89.6	53.0	1567
3.8	19.4	11.5	73.9	8.4	43.0	25.4	361	13.0	66.5	39.4	865	17.6	90.1	53.3	1585
3.9	20.0	11.8	77.9	8.5	43.5	25.7	370	13.1	67.0	39.7	879	17.7	90.6	53.6	1603
4.0	20.5	12.1	81.9	8.6	44.0	26.1	379	13.2	67.6	40.0	892	17.8	91.1	53.9	1621
4.1	21.0	12.4	86.1	8.7	44.5	26.4	388	13.3	68.1	40.3	906	17.9	91.6	54.2	1639
4.2	21.5	12.7	90.3	8.8	45.0	26.7	397	13.4	68.6	40.6	919	18.0	92.1	54.5	1658
4.3	22.0	13.0	94.7	8.9	45.6	27.0	406	13.5	69.1	40.9	933	18.1	92.6	54.8	1677
4.4	22.5	13.3	99.1	9.0	46.1	27.3	415	13.6	69.6	41.2	947	18.2	93.1	55.1	1695
4.5	23.0	13.6	104	9.1	46.6	27.6	424	13.7	70.1	41.5	961	18.3	93.6	55.4	1714
4.6	23.5	13.9	108	9.2	47.1	27.9	433	13.8	70.6	41.8	975	18.4	94.2	55.8	1732
4.7	24.0	14.2	113	9.3	47.6	28.2	442	13.9	71.1	42.1	989	18.5	94.7	56.1	1751
4.8	24.6	14.5	118	9.4	48.1	29.5	452	14.0	71.6	42.4	1004	18.6	95.2	56.4	1770
4.9	25.1	14.8	123	9.5	48.6	28.8	461	14.1	72.2	42.7	1018	18.7	95.7	56.7	1789
5.0	25.6	15.2	128	9.6	49.1	29.1	471	14.2	72.7	43.0	1032	18.8	96.2	57.0	1809
5.1	26.1	15.5	133	9.7	49.6	29.4	491	14.3	73.2	43.3	1047	18.9	96.7	57.3	1828
5.2	26.6	15.8	138	9.8	50.2	29.7	491	14.4	73.7	43.6	1062	19.0	97.2	57.6	1847
5.3	27.1	16.1	144	9.9	50.7	30.0	502	14.5	74.2	43.9	1076	19.1	97.8	57.9	1867
5.4	27.6	16.4	149	10.0	51.2	30.3	512	14.6	74.7	44.2	1091	19.2	98.3	58.2	1886
5.5	28.1	16.7	155	10.1	51.7	30.6	522	14.7	75.2	44.5	1106	19.3	98.8	58.5	1906
5.6	28.7	17.0	161	10.2	52.2	30.9	533	14.8	75.7	44.8	1121	19.4	99.3	58.8	1926
5.7	29.2	17.3	166	10.3	52.7	31.2	543	14.9	76.2	45.1	1137	19.5	99.8	59.1	1946
5.8	29.7	17.6	172	10.4	53.2	31.5	554	15.0	76.8	45.4	1152	19.6	100.3	59.4	1966
5.9	30.2	17.9	178	10.5	53.7	31.8	564	15.1	77.3	45.8	1167	19.7	100.8	59.7	1986
6.0	30.7	18.2	184	10.6	54.2	32.1	575	15.2	77.8	46.1	1183	19.8	101.3	60.0	2006
6.1	31.2	18.5	191	10.7	54.8	32.4	586	15.3	78.3	46.4	1199	19.9	101.8	60.3	2027
6.2	31.7	18.8	197	10.8	55.3	32.7	597	15.4	78.8	46.7	1214	20.0	102.4	60.6	2047
6.3	32.2	19.1	203	10.9	55.8	33.0	608	15.5	79.3	47.0	1230	21.0	107.5	63.6	2257
6.4	32.8	19.4	210	11.0	56.3	33.3	620	15.6	79.8	47.3	1246	22.0	112.6	66.7	2477
6.5	33.3	19.7	216	11.1	56.8	33.6	631	15.7	80.4	47.6	1263	23.0	117.7	69.7	2707
6.6	33.8	20.0	223	11.2	57.3	53.9	642	15.8	80.9	47.9	1277	24.0	122.8	72.7	2948
6.7	34.3	20.3	230	11.3	57.8	34.2	654	15.9	81.4	48.2	1293	25.0	128.0	75.7	3199
6.8	34.8	20.6	237	11.4	58.3	34.5	665	16.0	81.9	48.5	1310	26.0	133.1	78.8	3460
6.9	35.3	20.9	244	11.5	58.9	34.8	677	16.1	82.4	48.8	1326				
7.0	35.8	21.2	251	11.6	59.4	35.1	689	16.2	82.9	49.1	1343				
7.1	36.3	21.5	258	11.7	59.9	35.4	701	16.3	83.4	49.4	1359				
7.2	36.8	21.8	265	11.8	60.4	35.8	713	16.4	83.9	49.7	1376				
7.3	37.4	22.1	273	11.9	60.9	36.1	725	16.5	84.4	50.0	1393				
7.4	37.9	22.4	280	12.0	61.4	36.4	737	16.6	85.0	50.3	1410				
7.5	38.4	22.7	288	12.1	61.9	36.7	750	16.7	85.5	50.6	1427				

$$L_o = \frac{gT^2}{2} = 5.12\ T^2$$

$$C_o = \frac{gT}{2} = 5.12\ T$$

TABLE D-7 - Values used for plotting orthogonals

Wave period (T) = 5 seconds
Deep water wave length (L_o) = 128 feet

Depth	$\Delta L/L_{av}$	C_d/C_s*	C_s/C_d*
(fathoms) 10			
	0.003	1.004	0.996
9			
	0.007	1.007	0.993
8			
	0.011	1.012	0.988
7			
	0.019	1.020	0.980
6			
	0.032	1.034	0.968
5			
	0.052	1.055	0.948
4			
	0.085	1.090	0.917
3			
	0.145	1.158	0.864
2			
	0.290	1.341	0.746
1			

Wave period (T) = 6 seconds
Deep water wave length = (L_o) = 184.2 ft.

Depth	$\Delta L/L_{av}$	C_d/C_s*	C_s/C_d*
(fathoms) 15			
	0.005	1.005	0.995
13			
	0.011	1.011	0.989
11			
	0.009	1.009	0.991
10			
	0.014	1.013	0.987
9			
	0.019	1.019	0.981
8			
	0.026	1.026	0.974
7			
	0.037	1.037	0.964
6			
	0.050	1.052	0.950
5			
	0.074	1.075	0.930
4			
	0.093	1.111	0.900
3			
	0.154	1.180	0.847
2			
	0.330	1.363	0.734
1			

Wave period (T) = 7 seconds
Deep water wave length (L_o) = 251 feet

Depth	$\Delta L/L_{av}$	C_d/C_s*	C_s/C_d*
(fathoms) 21			
	0.006	1.006	0.994
17.5			
	0.010	1.010	0.990
15			
	0.014	1.015	0.985
13			
	0.025	1.025	0.976
11			
	0.018	1.018	0.982
10			
	0.023	1.023	0.977
9			
	0.029	1.030	0.971
8			
	0.037	1.039	0.964
7			
	0.047	1.049	0.953
6			
	0.064	1.065	0.939
5			
	0.082	1.086	0.921
4			
	0.117	1.123	0.890
3			
	0.175	1.192	0.839
2			
	0.310	1.378	0.726
1			

Wave period (T) = 8 seconds
Deep water wave length (L_o) = 327 feet

Depth	$\Delta L/L_{av}$	C_d/C_s*	C_s/C_d*
(fathoms) 27			
	0.006	1.015	0.986
20			
	0.013	1.013	0.987
17.5			
	0.021	1.021	0.979
15			
	0.013	1.012	0.988
14			
	0.014	1.014	0.986
13			
	0.016	1.017	0.983
12			
	0.020	1.021	0.980
11			
	0.025	1.025	0.976
10			
	0.031	1.031	0.970
9			
	0.038	1.037	0.964
8			
	0.046	1.046	0.956
7			
	0.056	1.059	0.946
6			
	0.069	1.069	0.933
5			
	0.090	1.095	0.914
4			
	0.124	1.131	0.844
3			
	0.183	1.199	0.834
2			
	0.30	1.389	0.720
1			

* Note:- C_d is the wave velocity in deeper water; C_s is the wave velocity in shallower water. When an orthogonal is being drawn from deep to shallow water, $C_d/C_s = C_1/C_2$. When an orthogonal is being drawn from shallow to deep water, $C_s/C_d = C_1/C_2$.

Table D-7 Cont'd

Wave period (T) = 9 seconds Deep water wave length (L_o) = 415 feet			
Depth	$\Delta L/L_{av}$	C_d/C_s*	C_s/C_d*
(fathoms) 35			
	0.011	1.005	0.995
30			
	0.012	1.011	0.989
25			
	0.025	1.026	0.975
20			
	0.022	1.022	0.978
17.5			
	0.033	1.032	0.969
15			
	0.015	1.017	0.983
14			
	0.019	1.019	0.981
13			
	0.022	1.022	0.978
12			
	0.025	1.026	0.975
11			
	0.029	1.031	0.970
10			
	0.035	1.036	0.966
9			
	0.043	1.042	0.959
8			
	0.048	1.051	0.952
7			
	0.064	1.062	0.941
6			
	0.071	1.077	0.929
5			
	0.098	1.100	0.909
4			
	0.136	1.136	0.881
3			
	0.196	1.206	0.829
2			
	0.350	1.390	0.720
1			

Wave period (T) = 10 seconds Deep water wave length (L_o) = 512 feet			
Depth	$\Delta L/L_{av}$	C_d/C_s*	C_s/C_d*
(fathoms) 43			
	0.010	1.007	0.993
35			
	0.010	1.011	0.989
30			
	0.020	1.021	0.979
25			
	0.040	1.040	0.962
20			
	0.029	1.031	0.970
17.5			
	0.041	1.042	0.960
15			
	0.021	1.020	0.980
14			
	0.023	1.023	0.978
13			
	0.026	1.026	0.975
12			
	0.029	1.030	0.971
11			
	0.034	1.034	0.967
10			
	0.039	1.040	0.962
9			
	0.045	1.046	0.956
8			
	0.054	1.055	0.948
7			
	0.063	1.066	0.938
6			
	0.078	1.081	0.925
5			
	0.100	1.103	0.906
4			
	0.132	1.140	0.878
3			
	0.189	1.211	0.826
2			
	0.326	1.397	0.716
1			

Wave period (T) = 11 seconds Deep water wave length (L_o) = 620 feet			
Depth	$\Delta L/L_{av}$	C_d/C_s*	C_s/C_d*
(fathoms) 50			
	0.01	1.010	0.990
40			
	0.01	1.012	0.989
35			
	0.02	1.019	0.982
30			
	0.03	1.032	0.969
25			
	0.05	1.053	0.950
20			
	0.04	1.037	0.964
17.5			
	0.05	1.049	0.953
15			
	0.020	1.023	0.978
14			
	0.027	1.026	0.975
13			
	0.029	1.029	0.972
12			
	0.032	1.033	0.968
11			
	0.038	1.037	0.964
10			
	0.044	1.043	0.959
9			
	0.050	1.049	0.954
8			
	0.058	1.058	0.947
7			
	0.070	1.068	0.936
6			
	0.025	1.084	0.923
5			
	0.105	0.105	0.905
4			
	0.137	1.143	0.875
3			
	0.203	1.208	0.825
2			
	0.325	1.400	0.714
1			

Wave period (T) = 12 seconds Deep water wave length (L_o) = 737 feet			
Depth	$\Delta L/L_{av}$	C_d/C_s*	C_s/C_d*
(fathoms) 61			
	0.007	1.007	0.994
50			
	0.010	1.007	0.993
45			
	0.008	1.011	0.989
40			
	0.012	1.018	0.983
35			
	0.032	1.027	0.974
30			
	0.040	1.042	0.960
25			
	0.065	1.062	0.941
20			
	0.044	1.043	0.959
17.5			
	0.051	1.054	0.949
15			
	0.024	1.025	0.976
14			
	0.027	1.028	0.973
13			
	0.031	1.031	0.970
12			
	0.035	1.034	0.967
11			
	0.039	1.039	0.962
10			
	0.044	1.044	0.958
9			
	0.049	1.051	0.951
8			
	0.057	1.059	0.944
7			
	0.068	1.070	0.935
6			
	0.083	1.086	0.921
5			
	0.105	1.108	0.904
4			
	0.135	1.148	0.871
3			
	0.20	1.210	0.829
2			
	0.33	1.406	0.709
1			

Table D-7 Cont'd

Wave period (T) = 13 seconds
Deep water wave length (L_o) = 865 feet

Depth (fathoms)	$\Delta L/L_{av}$	C_d/C_s*	C_s/C_d*
72			
	0.006	1.006	0.994
60			
	0.013	1.013	0.987
50			
	0.012	1.011	0.989
45			
	0.016	1.017	0.984
40			
	0.024	1.024	0.977
35			
	0.033	1.034	0.967
30			
	0.048	1.049	0.953
25			
	0.070	1.072	0.933
20			
	0.047	1.047	0.955
17.5			
	0.055	1.058	0.945
15			
	0.027	1.027	0.974
14			
	0.029	1.029	0.972
13			
	0.032	1.033	0.968
12			

Depth (fathoms)	$\Delta L/L_{av}$	C_d/C_s*	C_s/C_d*
12			
	0.036	1.036	0.966
11			
	0.040	1.041	0.961
10			
	0.045	1.046	0.956
9			
	0.051	1.052	0.950
8			
	0.059	1.061	0.942
7			
	0.070	1.071	0.933
6			
	0.085	1.088	0.919
5			
	0.103	1.109	0.902
4			
	0.135	1.147	0.872
3			
	0.200	1.213	0.824
2			
	0.341	1.410	0.709
1			

Wave period (T) = 14 seconds
Deep water wave length (L_o) = 1,003.5 feet

Depth (fathoms)	$\Delta L/L_{av}$	C_d/C_s*	C_s/C_d*
85			
	0.006	1.007	0.993
70			
	0.030	1.030	0.970
50			
	0.038	1.038	0.963
40			
	0.029	1.030	0.971
35			
	0.040	1.041	0.961
30			
	0.055	1.056	0.947
25			
	0.075	1.078	0.928
20			
	0.049	1.050	0.952
17.5			
	0.059	1.061	0.942
15			
	0.028	1.029	0.972
14			
	0.030	1.031	0.970
13			

Depth (fathoms)	$\Delta L/L_{av}$	C_d/C_s*	C_s/C_d*
13			
	0.033	1.033	0.967
12			
	0.037	1.037	0.964
11			
	0.041	1.042	0.960
10			
	0.046	1.047	0.955
9			
	0.052	1.054	0.949
8			
	0.058	1.062	0.942
7			
	0.070	1.073	0.932
6			
	0.085	1.089	0.918
5			
	0.105	1.109	0.902
4			
	0.144	1.148	0.871
3			
	0.188	1.217	0.822
2			
	0.338	1.405	0.712
1			

Table D-7 Cont'd

Wave period (T) = 15 seconds
Deep water wave length (L_o) = 1,152 feet

Depth (fathoms)	L/L_{av}	C_d/C_s*	C_s/C_d*	Depth (fathoms)	L/L_{av}	C_d/C_s*	C_s/C_d*
96				12			
	0.060	1.060	0.944		0.038	1.038	0.963
50				11			
	0.047	1.048	0.954		0.043	1.043	0.959
40				10			
	0.080	1.083	0.923		0.047	1.048	0.954
30				9			
	0.059	1.061	0.942		0.053	1.054	0.948
25				8			
	0.080	1.084	0.923		0.061	1.063	0.941
20				7			
	0.050	1.053	0.949		0.073	1.076	0.930
17.5				6			
	0.063	1.064	0.940		0.080	1.089	0.918
15				5			
	0.029	1.029	0.972		0.106	1.112	0.899
14				4			
	0.031	1.032	0.969		0.138	1.148	0.871
13				3			
	0.034	1.035	0.966		0.197	1.218	0.821
12				2			
					0.336	1.405	0.712
				1			

Wave period (T) = 16 seconds
Deep water wave length (L_o) = 1,310 feet

Depth (fathoms)	$\Delta L/L_{av}$	C_d/C_s*	C_s/C_d*	Depth (fathoms)	$\Delta L/L_{av}$	C_d/C_s*	C_s/C_d*
109				12			
	0.028	1.028	0.973		0.038	1.039	0.962
70				11			
	0.055	1.057	0.946		0.042	1.044	0.958
50				10			
	0.055	1.056	0.947		0.046	1.049	0.953
40				9			
	0.038	1.039	0.962		0.053	1.056	0.947
35				8			
	0.049	1.050	0.952		0.063	1.063	0.941
30				7			
	0.065	1.066	0.938		0.074	1.075	0.930
25				6			
	0.084	1.088	0.919		0.088	1.090	0.917
20				5			
	0.051	1.055	0.948		0.108	1.114	0.898
17.5				4			
	0.067	1.066	0.938		0.133	1.149	0.870
15				3			
	0.030	1.030	0.971		0.200	1.215	0.823
14				2			
	0.033	1.032	0.969		0.344	1.407	0.711
13				1			
	0.035	1.035	0.966				
12							

Table D-7 Cont'd

Wave period (T) = 17 seconds. Deep water wave length (L_o) = 1,480 feet			
Depth	L/L_{av}	C_d/C_s*	C_s/C_d*
(fathoms) 123			
	0.007	1.008	0.992
100			
	0.036	1.037	0.965
70			
	0.067	1.070	0.935
50			
	0.060	1.063	0.941
40			
	0.044	1.043	0.959
35			
	0.052	1.054	0.949
30			
	0.065	1.069	0.935
25			
	0.091	1.092	0.916
20			
	0.055	1.057	0.946
17.5			
	0.065	1.068	0.937
15			
	0.077	1.083	0.923
12.5			
	0.102	1.106	0.904
10			
	0.135	1.142	0.876
7.5			
	0.195	1.211	0.826
5			
	0.112	1.115	0.897
4			
	0.140	1.147	0.872
3			
	0.189	1.222	0.819
2			
	0.339	1.400	0.714
1			

Wave period (T) = 18 seconds. Deep water wave length (L_o) = 1,659 ft.			
Depth	L/L_{av}	C_d/C_s*	C_s/C_d*
(fathoms) 138			
	0.015	1.016	0.984
100			
	0.047	1.048	0.954
70			
	0.079	1.081	0.925
50			
	0.067	1.069	0.935
40			
	0.099	1.106	0.904
30			
	0.069	1.073	0.932
25			
	0.095	1.095	0.913
20			
	0.125	1.132	0.884
15			
	0.181	1.199	0.834
10			
	0.106	1.111	0.900
8			
	0.126	1.145	0.873
6			
	0.117	1.091	0.917
5			
	0.10	1.112	0.899
4			
	0.14	1.155	0.866
3			
	0.20	1.219	0.820
2			
	0.73	1.409	0.710
1			

Plates D-7 to D-11 incl. - Graphic determination of the weight of cap stone in terms of wave height, side slope and slope coefficient K'.

These curves are graphic solutions of the Iribarren formula as modified by Hudson for the determination of the side and end slopes and the weight of cap stone, at those slopes, required to withstand various wave heights. This modified formula is

$$W = \frac{K' \, Y_w \, S^3 \, S_r \, U^3 \, H^3}{(U \cos\alpha - \sin\alpha)^3 \, (S_r - S_f)^3}$$

where

- W = weight of stone in pounds
- K' = slope coefficient - see plate D-12
- Y_w = specific weight of fresh water
- S_f = specific gravity of the water (in sea water $S_f = 1.03$)
- S_r = specific gravity of the stone
- U = effective coefficient - stone on stone = 1.09
- H = wave height at the structure
- = the angle the sea side slope makes with the horizontal

In fresh water the formula reduces to

$$W = \frac{88.3K' \, S_r \, H^3}{(U \cos\alpha - \sin\alpha)^3 \, (S_r - 1)^3}$$

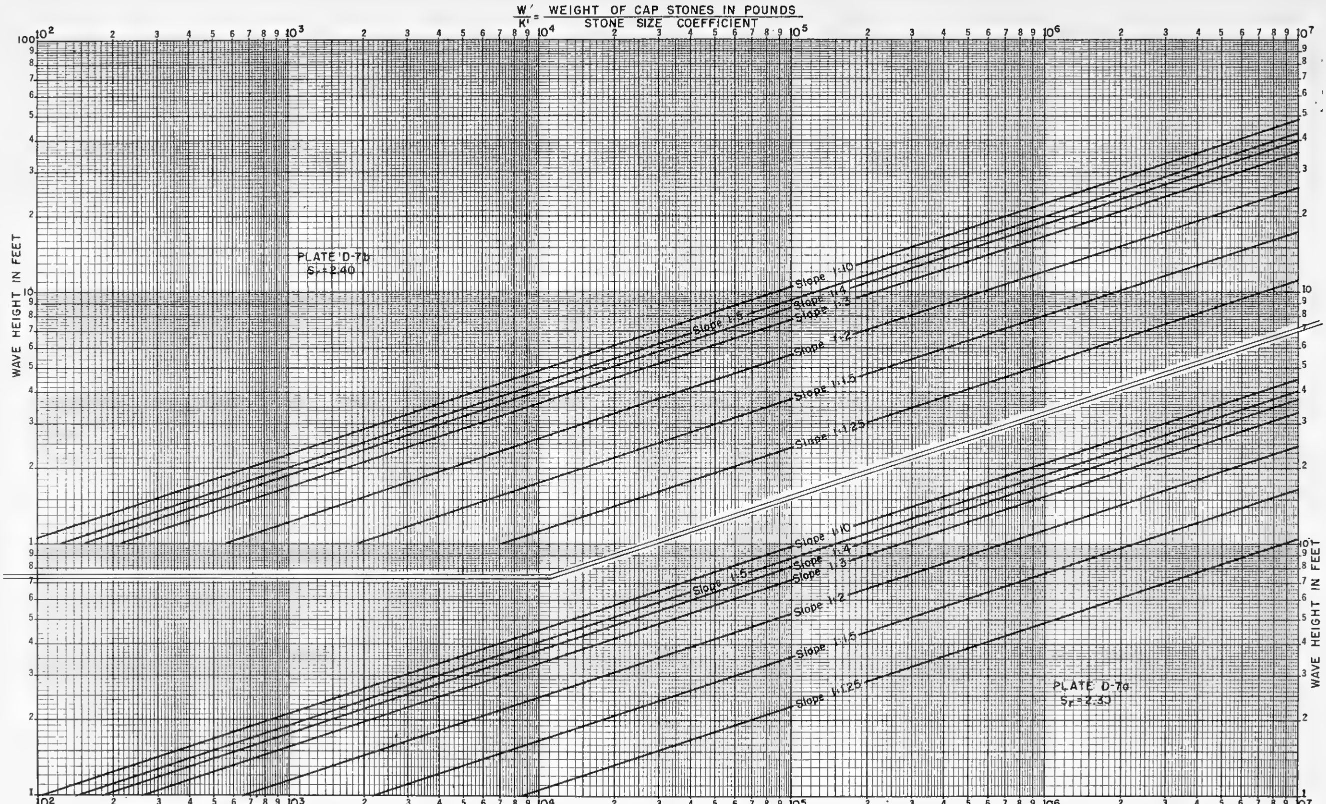
In ocean water it reduces to

$$W = \frac{80.7 \, K' \, S_r \, H^3}{(U \cos\alpha - \sin\alpha)^3 \, (S_r - 1.03)^3}$$

The curves of plate D-12 show the variation of K' in the basic equation with d/L and α.

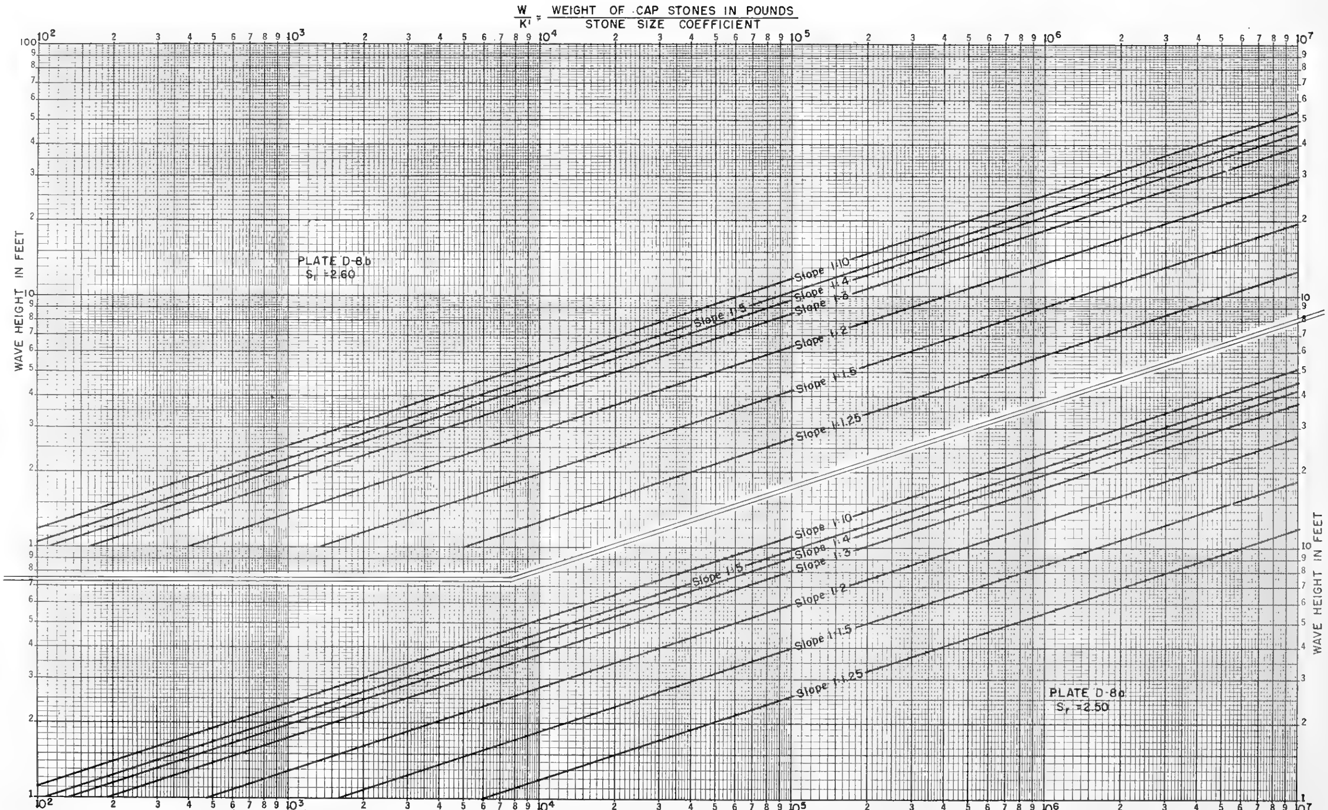
An example of the use of these curves follows:

Given a breakwater founded in 30 feet of sea water under waves with a height of 10 feet and a length of 200 feet at the structure composed of quarry stone having a specific gravity of 2.65, from Plate D-9, W/K' for a side slope of 1 on 1.5 is 1.25×10^6. From Plate D-12 with d/L = 0.15 and a slope of 1 on 1.5, K' equals 0.018. Therefore the size cap stone required is $1.25 \times 10^6 \times 0.018 = 22{,}500$ pounds or 11.2 tons.



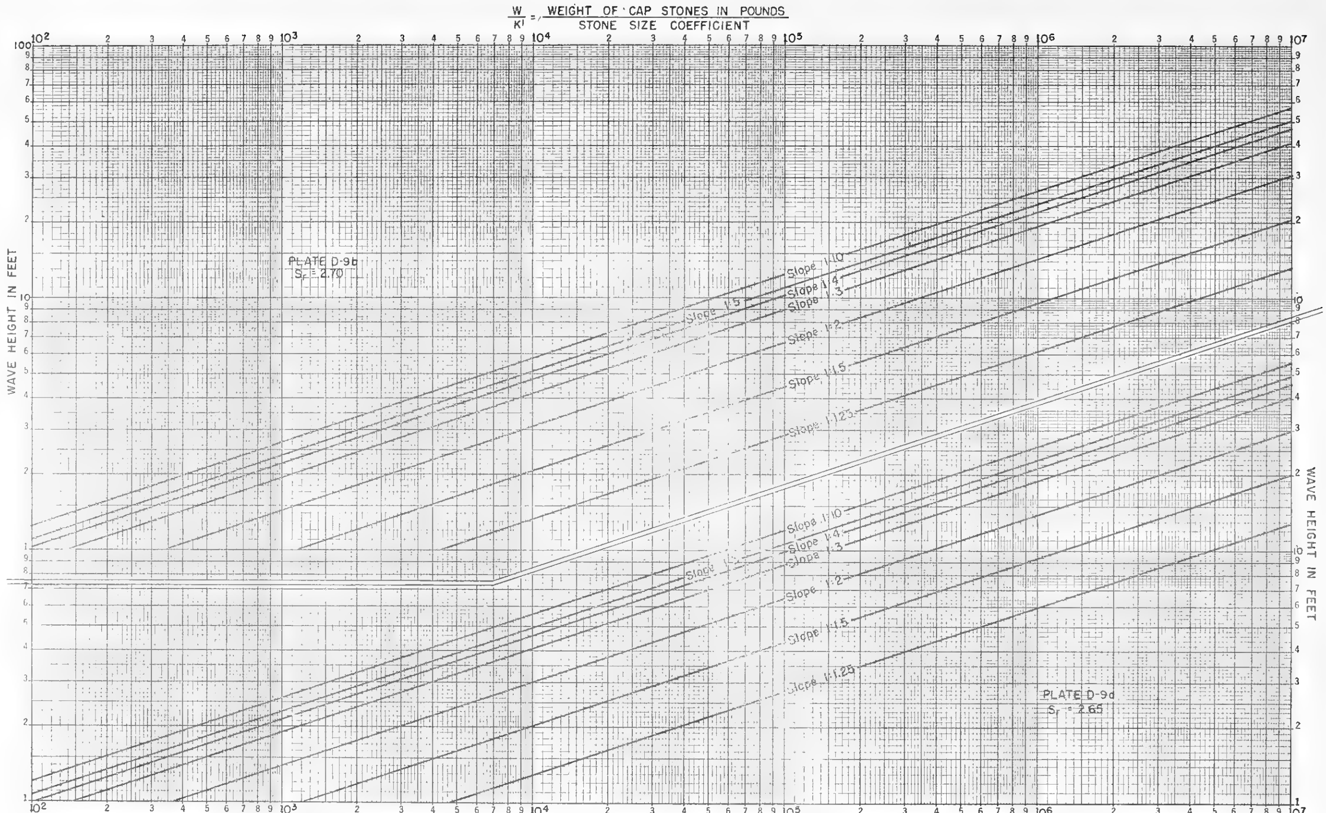
$\frac{\text{STONE WEIGHT}}{\text{STONE SIZE COEFFICIENT}}$ VS WAVE HEIGHT FOR VARIOUS VALUES OF BREAKWATER SLOPE (S_r = 2.30 & S_r = 2.40)

PLATE D-7



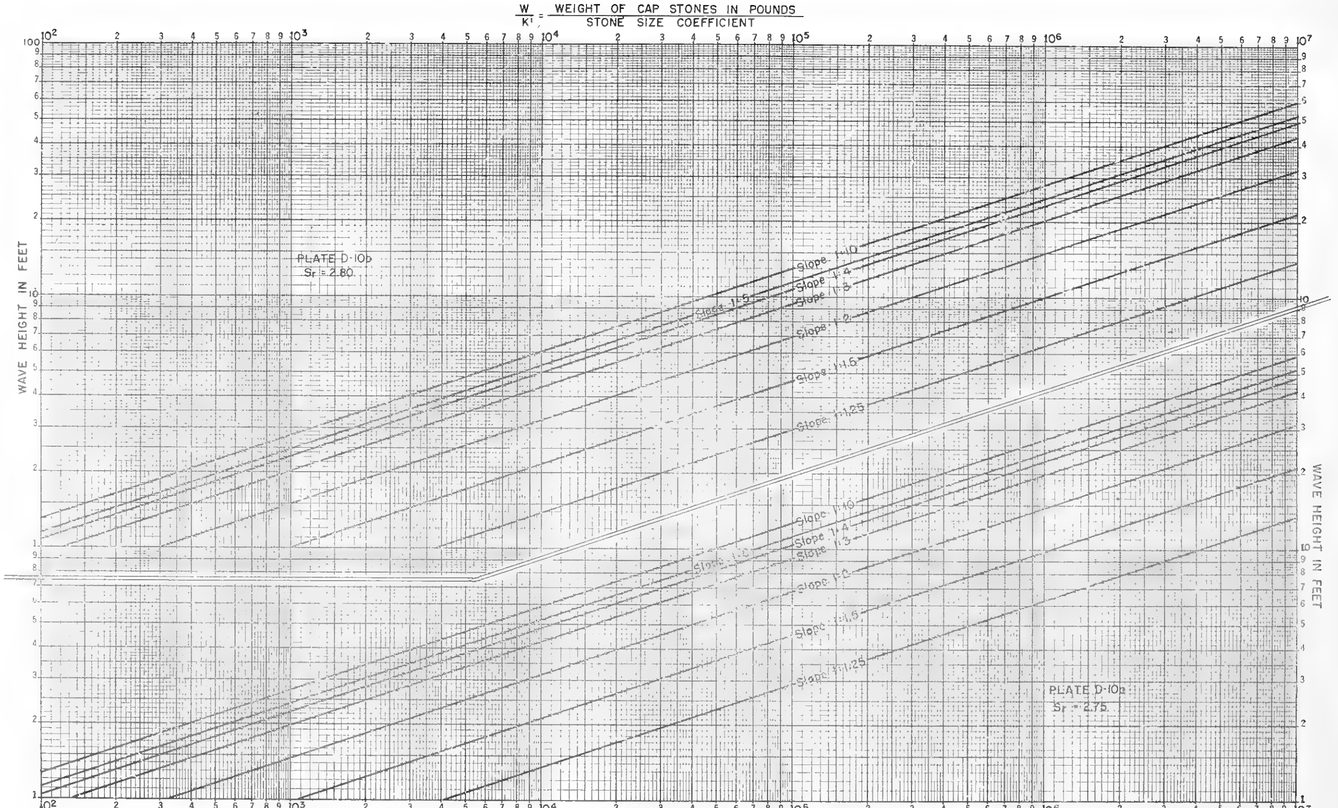
$\frac{\text{STONE WEIGHT}}{\text{STONE SIZE COEFFICIENT}}$ VS WAVE HEIGHT FOR VARIOUS VALUES OF BREAKWATER SLOPE (S_r =2.50 & S_r =2.60)

PLATE D-8



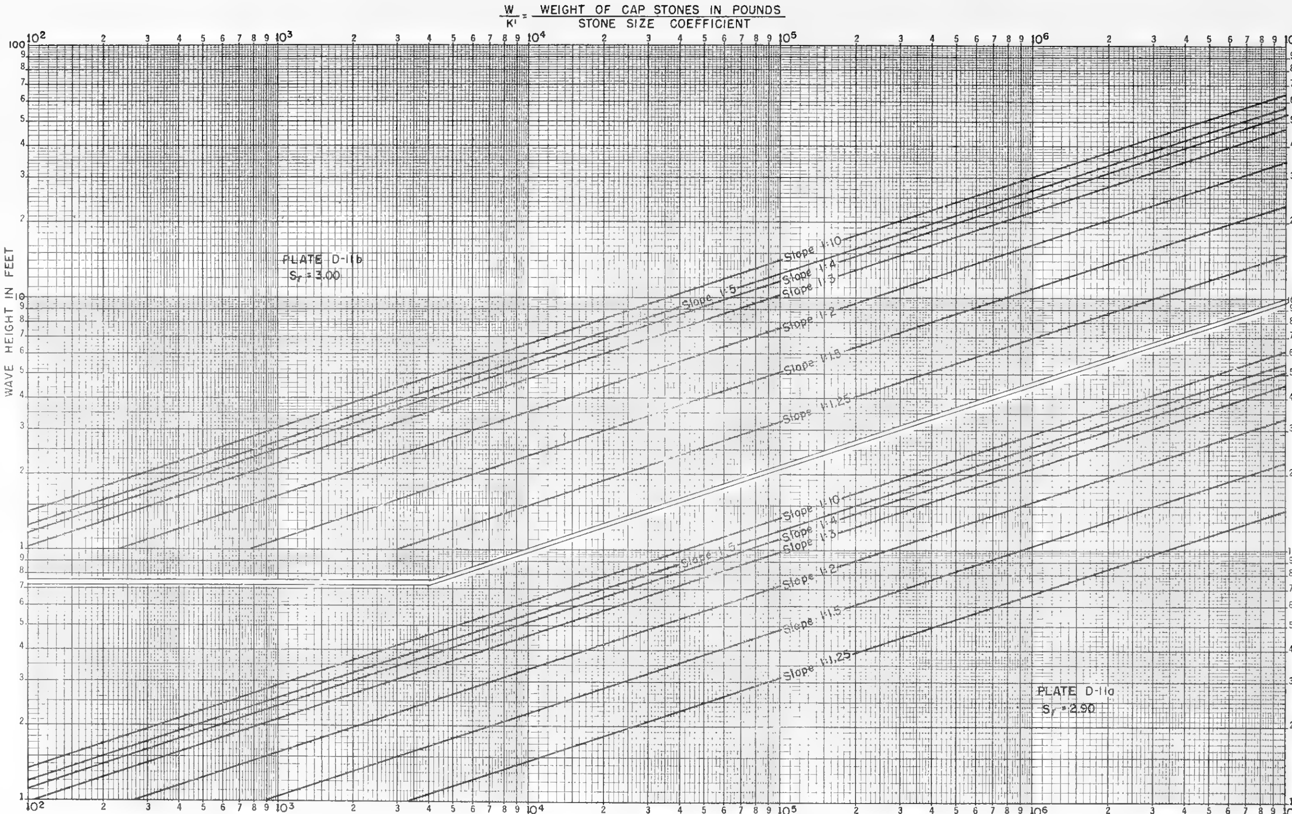
STONE WEIGHT / STONE SIZE COEFFICIENT VS WAVE HEIGHT FOR VARIOUS VALUES OF BREAKWATER SLOPE. (S_r = 2.65 & S_r = 2.70)

PLATE D-9



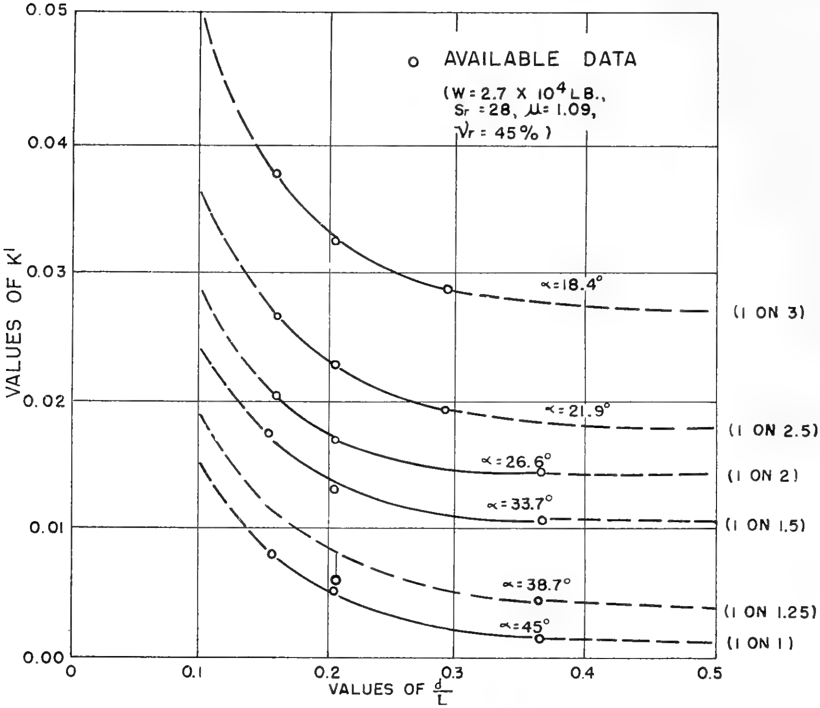
$\frac{\text{STONE WEIGHT}}{\text{STONE SIZE COEFFICIENT}}$ VS WAVE HEIGHT FOR VARIOUS VALUES OF BREAKWATER SLOPE ($S_r = 2.75$ & $S_r = 2.80$)

PLATE D-10



STONE WEIGHT / STONE SIZE COEFFICIENT VS WAVE HEIGHT FOR VARIOUS VALUES OF BREAKWATER SLOPE (S_r = 2.90 & S_r = 3.00)

PLATE D-11



DETERMINATION OF K' (FOR RUBBLE MOUND STRUCTURES) IN TERMS OF d/L AND α

PLATE D-12

APPENDIX E

MISCELLANEOUS DERIVATIONS

APPENDIX E

MISCELLANEOUS DERIVATIONS

I. Refraction Diagrams; derivation of method and design of template.

A. Derivation. The parametric equations for an orthogonal given by Arthur, Munk and Isaacs are:

$$\frac{dx}{dt} = C \cos\beta ; \frac{dy}{dt} = C \sin\beta ; \frac{d\beta}{dt} = \sin\beta \frac{\partial C}{\partial x} - \cos\beta \frac{\partial C}{\partial y} \qquad \text{(E-1)}$$

These may be solved by simple separation of variables for a velocity field which is a function of y alone. (Here $\beta = 90^{o} - \alpha$ where α = the angle between a tangent to a contour and a normal to an orthogonal, see Figure E-1). In particular, for a field which varies linearly with y, $C = C_o (1 - ay)$, the solutions for x and y are

$$x = \frac{1}{a \sin\alpha_o} (\cos\alpha - \cos\alpha_o)$$

$$y = \frac{1}{a \sin\alpha_o} (\sin\alpha_o - \sin\alpha) \qquad \text{(E-2)}$$

which are the parametric equations of a circle of radius

$$r = \sqrt{\frac{\sin^2 (2\alpha_o) + 4}{a \sin (2\alpha_o)}} \text{ and center at} \qquad \text{(E-3)}$$

$$x = - \frac{2}{a \sin (2\alpha_o)} ; y = \frac{1}{a} \qquad \text{(E-4)}$$

The solution for y may be put in the form

$$\sin\alpha = \sin (\alpha_o - \Delta\alpha) = (1 - \frac{\Delta C}{C_o}) \sin\alpha_o \qquad \text{(E-5)}$$

$$= \frac{C}{C_o} \sin\alpha_o \quad \text{which is Snell's law.}$$

From these, exact values of $\Delta\alpha$ and x at any point in the field may be found.

B. Template Design. Referring to Figure E-1, if from the point of intersection P of the mid-contour with an incoming orthogonal, a perpendicular is dropped to an arbitrary point R, then the line $\overline{RQ}$ perpendicular to the tangent to the mid-contour = $\sin\alpha_1 \times \overline{PR}$ (angle RPQ = α_1). If another line from R equal in length to $C_1/C_2 \times \overline{PR}$ (where C_1 and C_2 are the velocities at contours 1 and 2 respectively) is drawn to intersect the tangent to the mid-contour, the following relationships hold:

$$\sin\,(R\;S\;P) = \frac{\sin\alpha_1 \times \overline{PR}}{C_1/C_2 \times \overline{PR}} = C_2/C_1 \sin\alpha_1 \tag{E-6}$$

but $\quad C_2/C_1 \sin\alpha_1 = \sin\alpha_2;$

by Snell's law: $(R\;S\;P) = \alpha_2$ (E-7)

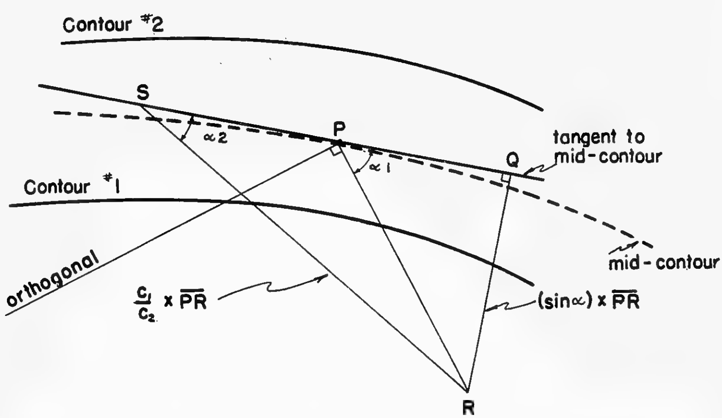


FIGURE E-1 Application of Snell's Law to template construction

The template constructed on these principles is illustrated in Figure 12 of the text. The point labelled "turning point" corresponds to R, and the line labelled "orthogonal" corresponds to the incoming orthogonal.

II. Diffraction

A. Waves passing a single breakwater. - The general equation for progressive irrotational waves of small amplitude may be written:

$$\eta = \frac{AikC}{g} \cosh\,(kd) \cdot e^{ikCt} \cdot F(x,y) \tag{E-8}$$

where L = the wave length

$k = \frac{2\pi}{L}$

C = the wave velocity

$\frac{AkC}{g} \times e^{ikCt} \times \cosh(kd)$ = the maximum amplitude of wave motion.

η = the surface elevation at time t, given by the real part of equation (1)

For waves travelling in the direction of the positive y axis with no barrier present,

$$F(x,y) = e^{-iky} \tag{E-9}$$

With a single rigid barrier present, Putnam and Arthur give as a solution

$$F(x,y) = e^{-iky} \cdot f(u_1) + e^{iky} \cdot g(u_2) \tag{E-10}$$

where $f(u_1)$, and $g(u_2)$ are given by

$$f(u_1) = \frac{1}{\sqrt{2}} e^{i\pi/4} \int_{-\infty}^{u_1} e^{-i\pi v^2/2} \, dv \tag{E-11}$$

$$g(u_2) = \frac{1}{\sqrt{2}} e^{i\pi/4} \int_{-\infty}^{u_2} e^{-i\pi v^2/2} \, dv \tag{E-12}$$

u_1 and u_2 being given by,

$$u_1 = \sqrt{\frac{4(r-y)}{L}} \tag{E-13}$$

$$u_2 = \sqrt{\frac{4(r+y)}{L}} \tag{E-14}$$

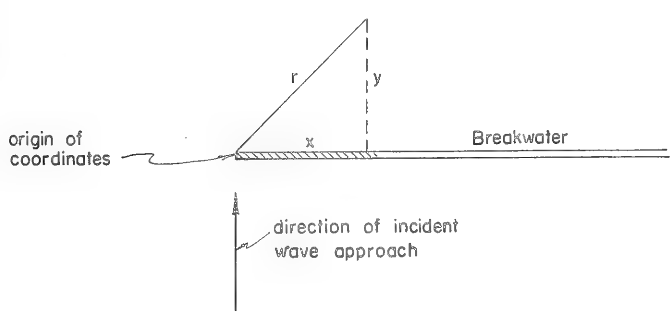


FIGURE E-2 Diffraction coordinates for a single breakwater

In Figure E-2 for both x and y positive (protected region) the signs of the upper limit of the integrals in equations (E-11) and (E-12) are negative. For x positive and y negative both limits are positive. For x negative and y positive the upper limit of the integral in (E-11) is positive and that in E-12 is negative. As a simplified solution, Putnam and Arthur give

$$F(x,y) = e^{-iky} f(u) \qquad (u = u_1) \tag{E-15}$$

Using the simplified solution and comparing equations (E-9)and (E-15) it can be seen that the modulus of f(u) (written $|f\ (u)|$) determines the relative height of waves with a barrier present to those without a barrier. That is, the ratio

$$\frac{\text{diffracted wave height}}{\text{incident wave height}} = K' = \left|f\ (u)\right| \tag{E-16}$$

Also from equation (E-9) and E-15) diffracted waves differ in phase from undiffracted waves by the argument of f(u) (written arg f (u)). i.e. equation (E-15) may be written

$$F(x,y) = e^{-ik(y-\frac{\arg f(u)}{K})} \cdot |f(u)| \tag{E-17}$$

Both the modulus and argument of f(u) may be determined from tabulated values of the Fresnel integrals C(u) and S(u) since

$$C(u) - iS(u) = \int_0^u e^{-i\pi v^2/2}\, dv \qquad \text{for } u > 0 \tag{E-18}$$

and it can be shown that for $u < 0$, $f(-u) = 1-f(u)$ (E-19)

The modulus and argument of f(u) are plotted on Figure E-3 as a function of u. Equation (E-13) may be solved for X/L.

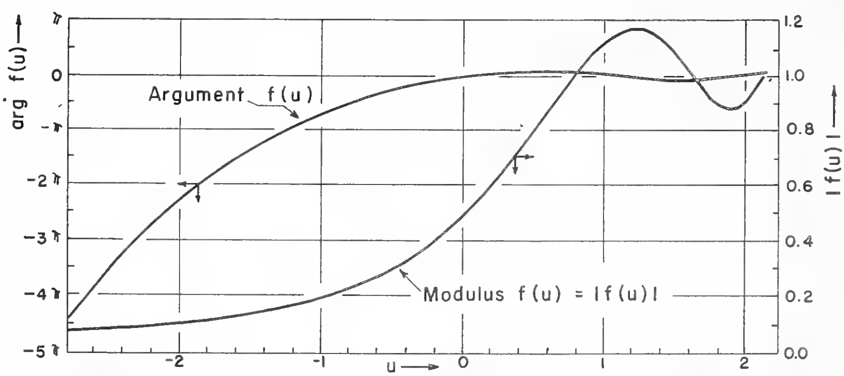


FIGURE E-3 FUNCTIONS OF u: ARGUMENT f(u) AND MODULUS f(u).

$$X/L = (\mp)\ 0.707\ u \sqrt{Y/L + 0.125\ u^2} \tag{E-20}$$

A diffraction diagram consisting of lines of equal wave height reduction (K') and wave crest advance positions may be drawn from computations similar to those shown in Table (E-1). Values of K' are chosen (including the maximum and minimum at the points of reversal of the curve $|f(u)| = K'$ vs. u), and are entered in Column 1. From Figure E-3 corresponding values of u are found and entered in Column 2. Columns 3 and 4 are computed as indicated. With columns 3 and 4, for every value of Y/L in the heading of columns 5 through 12 (these values represent distances in wave lengths leeward of the end of the breakwater) corresponding values of X/L are computed with equation E-20 and are entered in the table. Curves of constant K' may be drawn from these X/L and Y/L values of columns 5 through 12. (See Figure 19 of text).

Along these curves, since u is constant, arg f(u) will also be constant; which means that the lines of constant K' may be considered to be lines of constant phase lag. The amount of crest lag in percent of wave length along any of these lines is given by

$$\text{crest lag} = \frac{\arg f(u)}{2\pi} \tag{E-21}$$

in which u is taken to correspond to each value of K', and arg f(u) is taken from Figure E-3, and entered in column 13. With arg f(u), the crest lag is computed from equation (E-21) and entered in column 14.

Since the wave crest lag is constant along any one line of K', crest positions along these lines after diffraction may be plotted as on Figure 19 by marking points on them, from and normal to the undiffracted position of any wave crest, a distance equal to the calculated values of crest lag. Positive values of crest lag represent lag and negative values represent lead of the new position of wave crest. All linear dimensions on the graph, Figure 19, are divided by L the incident wave length. Note that Figure 19 being dimensionless may be used for all wave conditions and for all size drawings by increasing or decreasing the scale of the diagram to correspond to the scale of the drawing.

Equation (E-15) may be written in the following forms by use of the relationship (E-19). When the upper limit in equation (E-11) is positive;

$$F(x,y) = e^{-iky} - f \quad \text{for } x \leq 0 \quad \text{and} \begin{Bmatrix} x \geq 0 \\ y \leq 0 \end{Bmatrix} \tag{E-22}$$

When the upper limit in equation E-11 is negative;

$$F(x,y) = f \quad \text{for} \begin{Bmatrix} x \geq 0 \\ y \leq 0 \end{Bmatrix} \tag{E-23}$$

where $f = e^{-iky} f(-u)$

Table E-1 - Calculation of diffraction coefficients and crest lag for a single breakwater.

K'	u	0.707 u	$0.125\ u^2$	Values of x/L for y/L = (a): 1	2	3	4	5	6	7	8	arg. f(u)	(b) crest lag / L = arg. f(u)/2
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
0.1	−2.25	−1.59	0.635	2.03	2.58	3.03	3.42	3.78	4.10	4.40	4.67	−8.80	1.4
0.15	−1.44	−1.018	0.259	1.14	1.53	1.84	2.10	2.34	2.55	2.74	2.93	−3.77	0.60
0.2	−1.02	−0.721	0.130	0.77	1.05	1.28	1.46	1.64	1.79	1.93	2.06	−2.05	0.33
0.3	−0.528	−0.373	0.035	0.38	0.53	0.65	0.75	0.84	0.92	0.99	1.06	−0.82	0.13
0.4	−0.225	−0.159	0.006	0.16	0.23	0.28	0.32	0.36	0.39	0.42	0.45	−0.38	0.06
0.5	0	0	0	0	0	0	0	0	0	0	0	0	0
0.6	0.184	0.130	0.0042	−0.130	−0.184	−0.227	−0.260	−0.29	−0.318	−0.344	−0.368	0.19	−0.03
0.7	0.341	0.241	0.0145	−0.243	−0.349	−0.420	−0.485	−0.54	−0.591	−0.689	−0.683	0.31	−0.05
0.8	0.486	0.343	0.0295	−0.348	−0.488	−0.597	−0.69	−0.77	−0.84	−0.91	−0.97	0.38	−0.06
0.9	0.631	0.446	0.0498	−0.456	−0.637	−0.776	−0.895	−1.00	−1.10	−1.18	−1.26	0.31	−0.05
1.0	0.719	0.551	0.076	−0.576	−0.804	−0.976	−1.12	−1.24	−1.36	−1.46	−1.56	0.31	−0.05
1.17	1.218	0.861	0.186	−0.94	−1.26	−1.525	−1.75	−1.96	−2.14	−2.31	−2.44	0	0
1.0	1.610	1.138	0.324	−1.30	−1.72	−2.06	−2.35	−2.66	−2.86	−3.08	−3.28	−0.19	0.03
0.88	1.878	1.328	0.441	−1.56	−2.03	−2.42	−2.75	−3.10	−3.38	−3.62	−3.86	0	0
1.0	2.124	1.502	0.564	−1.73	−2.24	−2.64	−3.00	−3.54	−3.85	−4.13	−4.40	0.09	−0.02

(a) $x/L = (\mp)\ 0.707\ u - \sqrt{y/L + 0.125\ u^2}$; x/L positive for $u < 0$; x/L negative for $u > 0$

(b) negative values of crest lag indicate crest lead.

Equation (E-24)

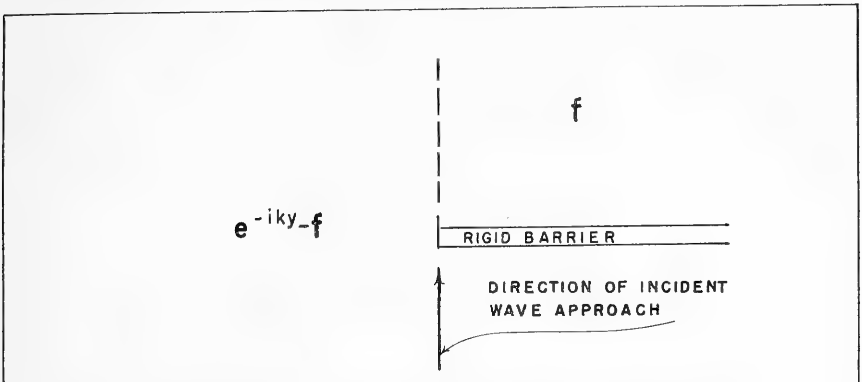


FIGURE E-4 GRAPHIC REPRESENTATION OF THE AREAS OF APPLICATION OF EQUATIONS (E-15) & (E-16)

Equation (E-25)

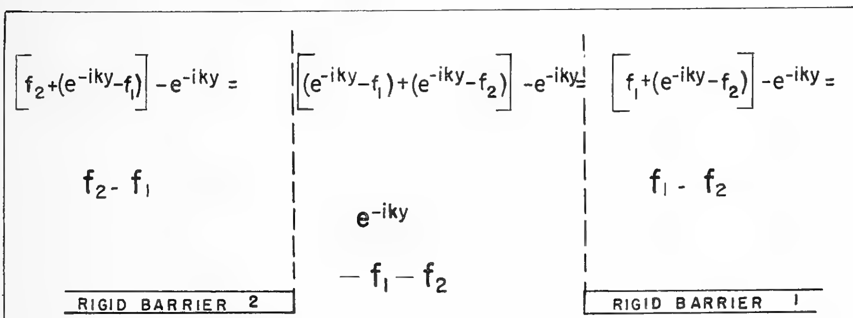


FIGURE E-5 GRAPHIC REPRESENTATION OF THE AREAS OF APPLICATION OF THE SOLUTIONS TO THE DIFFRACTION PROBLEM AT A BREAKWATER GAP.

The simplified solution for a non-reflecting type barrier is the same. Graphically these solutions are shown on Figure E-4. Now, x does not appear directly in the solution in the form (E-24). As can be seen from (E-24), only the regions of effect of the barrier are delineated without reference to any coordinate system.

Waves Passing a Gap - Gap Width Less Than 5 Wave Lengths. - Blue has shown that a simplified solution of the gap diffraction problem may be found by adding together the separate solutions due to each arm of the breakwater, and subtracting e^{-iky}. In this manner we may graph the gap solution as on Figure E-5, the factors in brackets indicating the effects of the individual arms. This is the simplified solution. The complete solution contains additional terms computed from relationships similar to those in equation (E-12).

Calculations and Computations. - The computations for the gap problem are somewhat more complex than those for a single breakwater. If we establish the following coordinate system, (Figure E-6)

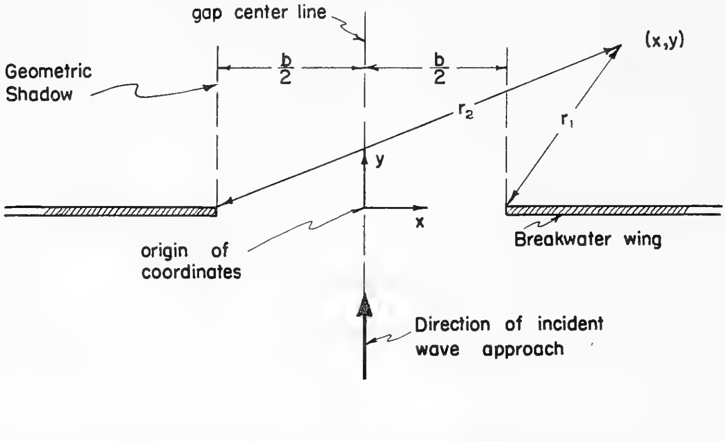


FIGURE E-6 Diffraction coordinates at a breakwater gap equation (E-25) may be written

$$F(x,y) = e^{-iky} - f_1 - f_2 \quad \text{for } 0 \leqq x \leqq b/2$$

and (E-26)

$$F(x,y) = f_1 - f_2 \quad \text{for } x \geqq b/2$$

(the solution for $x<0$ will be the mirror image of that for $x>0$).

Computations may be made by use of equation E-23 and Figure E-7, which show real and imaginary parts of $f(-u)$. Writing $f(-u)$ as

$$f(-u) = s + iw \tag{E-27}$$

equation (19) may be written as

$$f(x,y) = e^{-iky}\ (1-s_1-s_2) + i(-w_1-w_2) \text{ for } 0 \leqq x \leqq b/2 \tag{E-28}$$

$$f(x,y) = e^{-iky}\ (s_1-s_2) + i(w_1-w_2) \text{ for } x \geqq b/2$$

s_1 and w_1 correspond to u_1 which is defined by

$$u_1^{\ 2} = \frac{4(r_1-y)}{L} = 4\left[\sqrt{\left(\frac{x-b/2}{L}\right)^2 + \left(\frac{y}{L}\right)^2} - \frac{y}{L}\right] \tag{E-29}$$

and s_2 and w_2 correspond to u_2 which is defined by

$$u_2^{\ 2} = \frac{4(r_2-y)}{L} = 4\left[\sqrt{\left(\frac{x + b/2}{L}\right)^2 + \left(\frac{y}{L}\right)^2} - \frac{y}{L}\right] \tag{E-30}$$

If equation (E-26) is written

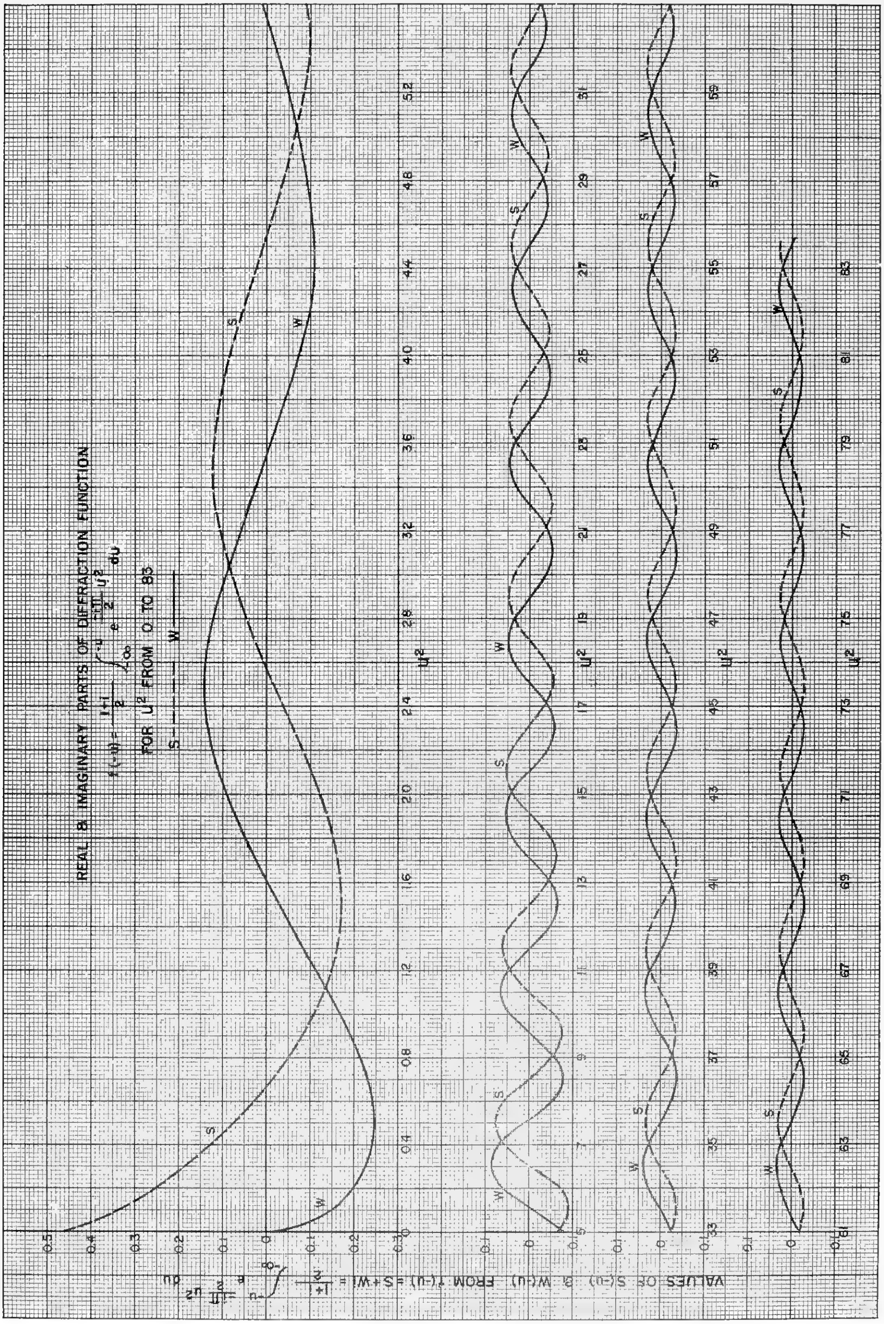
$$F(x,y) = e^{-iky}\ (s + iW) \tag{E-31}$$

where S and W represent the sums of the real and imaginary parts respectively, of equation (E-28) comparison with equation (E-9) shows that a diffraction coefficient K' may be defined as

$$K' = \frac{|F(x,y)| \text{ for diffracted wave}}{|F(x,y)| \text{ for incident wave}} = |F(x,y)| \text{ for diffracted wave} \tag{E-32}$$

which is equal to

$$K' = \sqrt{S^2 + W^2} \tag{E-33}$$



REAL & IMAGINARY PARTS OF DIFFRACTION FUNCTION
$f(-u) = \frac{1+i}{2} \int_{\infty}^{-u} e^{\frac{-i\pi}{2} u^2} du$
FOR U^2 FROM 0 TO 83
S - - - - W ——
VALUES OF S(-u) & W(-u) FROM f(-u) = S + Wi
U^2

FIGURE E-7

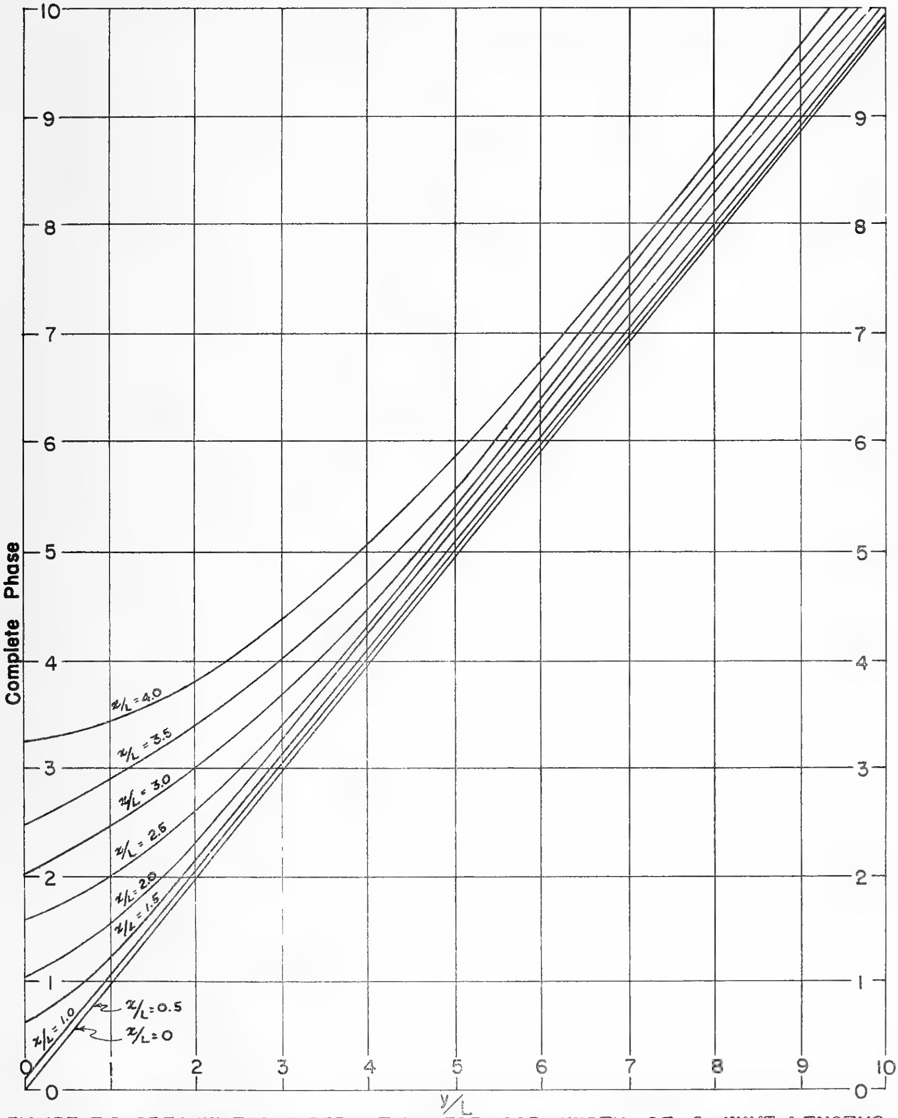


FIGURE E-8 BREAKWATER DIFFRACTION FOR GAP WIDTH OF 2 WAVE LENGTHS

The amount of crest lag in percent of wave length at any point is given by

$$\text{crest lag} = \frac{\arg\,(S + iW)}{2\pi} = \tan^{-1}\left(\frac{W}{S}\right) \qquad \text{(E-34)}$$

Diffraction Coefficients. - The manner in which lines of equal diffraction coefficients are found is illustrated in Table E-2. These computations are for diffraction effects where the gap width is twice the wave length. For a particular value of X/L and various values of Y/L, u_1^2 and u_2^2 are calculated by use of equations E-29 and E-30. With these values s_1, w_1, s_2 and w_2 are found from Figure E-7 and when summed in the manner determined by equation E-28, give the (S) and (W) values of equation E-14. Diffraction coefficients, K', are then calculated by use of this equation for each value of X/L and Y/L. These values are plotted as illustrated on Figure 21 of the text. Contour lines of equal diffraction coefficient, K', may then be drawn.

Wave Crest Positions. - Equation (E-34), being a tangent function, contains no indication of the position of a wave crest other than giving the amount of lag or lead (phase difference) of a diffracted wave crest over an undiffracted one. (Positive values of phase difference indicate a crest lag, and negative values, a crest lead). There is no way of telling from the solution of equation (E-27) alone to which undiffracted wave crest this lag or lead applies.

This may be determined however, through the construction of a graph of Y/L vs. complete phase Figure E-8 for various values of X/L. ("Complete phase" indicates the actual distance in wave cycles of a wave crest from the gap.) A 45° line is first sketched in. The complete phases along the line for X/L = 0 will lie just below this 45° line and successive curves for X/L = 0.5, 1.0, 1.5 etc. will lie above and approximately parallel to each preceding curve.

From equation (E-34), values of $\tan^{-1}\frac{W}{S}$ and $\dfrac{\tan^{-1}\frac{W}{S}}{360} = \dfrac{\text{Phase difference(PD)}}{360}$

are calculated. For each integral $\frac{Y}{L}$ value, these PD/360' values are added to or subtracted from that value of integral phase which will bring the actual complete phase line to the desired position; slightly above the curve for the next smaller value of X/L. For example; with X/L = 2.5 and Y/L = 2; $PD/360^{\circ} = +0.380$ which is subtracted from 3, and for Y/L = 3; $PD/360^{\circ} = -0.386$ which is added to 3 to give complete phase values of 2.62 and 3.39 respectively.

Points for the wave pattern are computed by noting from the curves shown in Figure E-8 the values of Y/L at the points where the lines of constant X/L itersect the line of integral phase. These values are tabulated in Table E-3. Wave patterns now may be drawn as curves joining the points having the same integral phases. The patterns for the gap width of 2L are shown in Figure 21 of the text, together with the contours of equal diffraction coefficients.

Table E-2. - Calculation of diffraction coefficients and phases for 2 breakwaters; gap width 2 wave lengths

y/L	u_1^2	s_1	w_1	u_2^2	s_2	w_2	S $1-(s_1+s_2)$	W $-(w_1+w_2)$	S^2	W^2	S^2+W^2	K	W/S	$\tan^{-1}W/S$ (degrees)	$\frac{\tan^{-1}W/S}{360}$	Complete phase
								x/L = 0								
0	4.0	+0.085	-0.07	4.0	+0.085	-0.07	+0.83	+0.14	0.689	0.020	0.709	0.84	+0.169	+9.6	+0.0267	
1	1.66	-0.16	+0.02	1.66	-0.16	+0.02	+1.32	-0.04	1.742	0.002	1.744	1.32	-0.030	-1.7	-0.0047	1.01
2	0.944	-0.09	-0.18	0.944	-0.09	-0.18	+1.18	+0.36	1.392	0.030	1.522	1.23	+0.305	+17.0	+0.047	1.95
3	0.65	0.00	-0.24	0.65	+0.00	-0.24	+1.00	+0.48	1.0	0.230	1.230	1.11	+0.48	+25.6	+0.071	2.93
4	0.493	+0.08	-0.245	0.493	+0.08	-0.245	+0.84	+0.49	0.706	0.240	0.946	0.97	+0.583	+30.2	+0.084	3.92
5	0.396	+0.13	-0.24	0.396	+0.13	-0.24	+0.74	+0.48	0.548	0.230	0.778	0.88	+0.649	+33.0	+0.092	4.91
6	0.328	+0.17	-0.23	0.328	+0.17	-0.23	+0.66	+0.46	0.436	0.212	0.648	0.81	+0.698	+34.9	+0.097	5.90
7	0.284	+0.19	-0.22	0.284	+0.19	-0.22	+0.62	+0.44	0.384	0.194	0.578	0.76	+0.710	+35.4	+0.098	6.90
8	0.248	+0.22	-0.215	0.248	+0.22	-0.215	+0.56	+0.43	0.314	0.185	0.499	0.71	+0.768	+37.5	+0.104	7.90
9	0.220	+0.24	-0.205	0.220	+0.24	-0.205	+0.52	+0.41	0.270	0.168	0.48	0.66	+0.788	+38.3	+0.106	8.89
10	0.199	+0.25	-0.20	0.199	+0.25	-0.20	+0.50	+0.40	0.250	0.160	0.410	0.64	+0.80	+38.7	+0.108	0.89
								x/L = 0.5								
0	2.0	-0.12	+0.10	6.0	-0.07	+0.065	+1.19	-0.165	1.416	0.027	1.443	1.20	-0.139	-7.9	-0.022	0.02
1	0.472	+0.09	-0.245	3.21	+0.11	+0.06	+0.80	+0.185	0.640	0.034	0.674	0.82	+0.232	+13.1	+0.0364	0.96
2	0.248	+0.22	-0.22	2.0	-0.12	+0.10	+0.90	+0.12	0.810	0.014	0.824	0.91	+0.134	+ 7.7	+0.0214	1.98
3	0.164	+0.28	-0.18	1.42	-0.17	-0.05	+0.89	+0.23	0.792	0.053	0.845	0.92	+0.259	+14.5	+0.0403	2.96
4	0.124	+0.31	-0.17	1.09	-0.13	-0.145	+0.82	+0.31	0.672	0.096	0.772	0.88	+0.378	+20.7	+0.0584	3.94
5	0.100	+0.33	-0.15	0.88	-0.07	-0.20	+0.74	+0.35	0.548	0.123	0.671	0.82	+0.473	+25.3	+0.0704	4.93
6	0.084	+0.35	-0.14	0.75	-0.03	-0.22	+0.72	+0.36	0.518	0.130	0.648	0.80	+0.500	+26.6	+0.0739	5.93
7	0.072	+0.37	-0.13	0.636	+0.01	-0.235	+0.62	+0.365	0.384	0.133	0.517	0.72	+0.588	+30.5	+0.0847	6.92
8	0.064	+0.38	-0.12	0.551	+0.06	-0.24	+0.56	+0.36	0.314	0.130	0.444	0.67	+0.643	+32.8	+0.0910	7.91
9	0.056	+0.39	-0.11	0.504	+0.08	-0.245	+0.53	+0.355	0.281	0.126	0.407	0.64	+0.670	+33.8	+0.0940	8.91
10	0.052	+0.40	-0.10	0.448	+0.10	-0.245	+0.50	+0.345	0.250	0.119	0.369	0.61	+0.690	+34.6	+0.0961	9.90
								x/L = 1.								
0	0	0.5	0	8.0	+0.06	-0.05	+0.44	+0.05	0.194	0.003	0.197	0.44	+0.144	+6.5	+0.018	
1	0	0.5	0	4.95	-0.06	-0.08	+0.56	+0.08	0.314	0.006	0.320	0.57	+0.143	+8.3	+0.023	0.98
2	0	0.5	0	3.31	+0.11	+0.05	+0.39	-0.05	0.152	0.003	0.155	0.39	-0.128	-7.3	-0.020	2.02
3	0	0.5	0	2.42	-0.03	+0.14	+0.53	-0.14	0.281	0.020	0.301	0.55	-0.264	-14.8	-0.041	3.04
4	0	0.5	0	1.89	-0.14	+0.07	+0.64	-0.07	0.410	0.005	0.415	0.64	-0.109	- 6.2	-0.017	4.02
5	0	0.5	0	1.54	-0.17	-0.02	+0.67	+0.02	0.449	0.0004	0.449	0.67	+0.030	+ 1.7	+0.0047	5.00
6	0	0.5	0	1.30	-0.16	-0.09	+0.66	+0.09	0.436	0.008	0.444	0.67	+0.136	+7.8	+0.021	5.98
7	0	0.5	0	1.12	-0.13	-0.13	+0.63	+0.13	0.397	0.017	0.414	0.64	+0.206	+11.6	+0.032	6.97
8	0	0.5	0	0.985	-0.10	-0.17	+0.60	+0.17	0.360	0.029	0.389	0.62	+0.283	+15.8	+0.044	7.96
9	0	0.5	0	0.875	-0.07	-0.20	+0.57	+0.20	0.325	0.040	0.365	0.60	+0.351	+19.3	+0.054	8.95
10	0	0.5	0	0.792	-0.04	-0.21	+0.54	+0.21	0.292	0.044	0.336	0.58	+0.389	+21.3	+0.059	9.94

Table E-2 (cont'd)

y/L	U_1^2	s_1	w_1	u_2^2	s_2	w_2	S (s_1-s_2)	W (w_1-w_2)	S^2	W^2	S^2+W^2	K	W/S	$\tan^{-1}W/S$ (degrees)	$\frac{\tan^{-1}W/S}{360}$	Complete phase
								x/L ≐ 1.5								
0	2.0	-0.12	+0.10	10.0	-0.055	+0.05	-0.065	+0.05	0.0042	0.0025	0.0067	0.09	-0.782	+142.0	+0.395	0.60
1	0.472	+0.090	-0.245	6.78	+0.04	+0.08	+0.05	-0.325	0.0025	0.1056	0.1081	0.33	-6.50	-81.3	-0.226	1.23
2	0.248	+0.22	-0.20	4.81	+0.04	+0.09	+0.26	-0.13	0.0676	0.0169	0.0845	0.29	-0.50	-26.6	-0.074	2.07
3	0.164	+0.028	-0.18	3.62	+0.12	-0.01	+0.16	-0.17	0.0256	0.0289	0.0545	0.23	-1.06	-46.7	-0.129	3.13
4	0.124	+0.32	-0.17	2.87	+0.06	+0.12	+0.26	-0.29	0.0676	0.0841	0.1517	0.39	-1.11	-48.0	-0.133	4.13
5	0.100	+0.33	-0.15	2.36	-0.05	+0.14	+0.38	-0.29	0.1444	0.0841	0.2285	0.48	-0.673	-37.3	-0.104	5.10
6	0.084	+0.35	-0.14	2.00	-0.12	+0.10	+0.47	-0.24	0.2289	0.0576	0.2785	0.53	-0.511	-27.1	-0.080	6.08
7	0.072	+0.37	-0.13	1.73	-0.16	+0.04	+0.53	-0.17	0.2809	0.0289	0.3098	0.56	-0.321	-17.8	-0.049	7.05
8	0.064	+0.38	-0.12	1.53	-0.17	-0.02	+0.55	-0.10	0.3025	0.0100	0.3125	0.56	-0.182	-10.3	-0.029	8.03
9	0.056	+0.39	-0.11	1.36	-0.16	-0.07	+0.55	-0.04	0.3025	0.0016	0.3041	0.55	-0.073	- 4.2	-0.017	9.02
10	0.052	+0.40	-0.10	1.23	-0.15	-0.11	+0.55	+0.01	0.3025	0.0001	0.3026	0.55	+0.018	+1.0	-	10.00
								x/L = 2								
0	4.0	+0.085	-0.07	12.0	+0.04	-0.04	+0.45	-0.03	0.002	0.001	0.003	0.055	-0.666	-33.7	-0.094	1.09
1	1.66	-0.16	+0.02	8.65	-0.01	-0.07	-0.15	+0.09	0.023	0.008	0.031	0.176	-0.600	+149.0	+0.414	1.59
2	0.944	-0.09	-0.18	6.42	-0.01	+0.09	-0.08	-0.27	0.006	0.073	0.079	0.281	+3.38	-106.5	-0.296	2.30
3	0.64	+0.01	-0.24	4.97	-0.06	-0.08	+0.07	-0.16	0.005	0.026	0.031	0.176	-2.29	-66.4	-0.184	3.18
4	0.493	+0.08	-0.245	4.0	+0.085	-0.07	-0.01	-0.175	0.000	0.031	0.031	0.176	+17.5	-93.3	-0.258	4.26
5	0.396	+0.13	-0.24	3.32	+0.12	+0.04	+0.01	-0.28	0.000	0.078	0.078	0.279	-28.0	-88.0	-0.244	5.24
6	0.328	+0.17	-0.23	2.83	+0.05	+0.12	+0.12	-0.35	0.014	0.123	0.137	0.370	-2.92	-71.1	-0.199	6.20
7	0.284	+0.19	-0.22	2.46	-0.02	+0.14	+0.21	-0.36	0.44	0.130	0.174	0.417	-1.71	-59.7	-0.166	7.17
8	0.248	+0.22	-0.215	2.17	-0.09	+0.12	+0.31	-0.335	0.096	0.112	0.208	0.456	-1.08	-47.2	-0.131	8.13
9	0.220	+0.24	-0.205	1.95	-0.13	+0.09	+0.37	-0.295	0.137	0.087	0.224	0.473	-0.798	-38.6	-0.107	9.11
10	0.199	+0.25	-0.20	1.76	-0.15	+0.04	+0.40	-0.24	0.160	0.058	0.218	0.467	-0.60	-31.0	-0.086	10.09
								x/L = 2.5								
0	6.0	-0.07	+0.065	14.0	-0.04	+0.04	-0.03	+0.025	0.0009	0.0063	0.0072	0.085	-0.83	+140.2	+0.39	0.61
1	3.21	+0.11	+0.06	10.58	+0.01	+0.07	+0.10	-0.01	0.01	0.0001	0.0101	0.1005	-0.10	- 5.7	-0.058	1.06
2	2.0	-0.12	+0.10	8.12	+0.05	-0.06	-0.17	+0.16	0.0289	0.0256	0.0545	0.233	-0.94	+136.8	+0.380	2.62
3	1.42	-0.17	-0.05	6.44	-0.01	+0.09	-0.16	-0.14	0.0256	0.196	0.0452	0.213	+0.87	-139.0	-0.386	3.39
4	1.09	-0.13	-0.145	5.25	-0.09	-0.04	-0.04	-0.105	0.0016	0.0110	0.0126	0.112	+2.63	-110.8	-0.308	4.31
5	0.88	-0.07	-0.20	4.42	+0.02	-0.10	-0.09	-0.10	0.0081	0.01	0.0181	0.135	+1.11	-132.0	-0.367	5.37
6	0.75	-0.03	-0.22	3.78	+0.11	-0.04	-0.14	-0.18	0.0196	0.0324	0.0520	0.226	+1.29	-127.8	-0.355	5.37
7	0.636	+0.01	-0.235	3.30	+0.12	+0.05	-0.11	-0.285	0.0121	0.0812	0.0933	0.305	+2.59	-111.3	-0.309	6.36
8	0.551	+0.06	-0.24	2.93	+0.07	+0.11	-0.01	-0.35	0.0001	0.1225	0.1225	0.351	+35.0	-91.6	-0.255	7.31
9	0.504	+0.08	-0.245	2.62	+0.01	+0.14	+ 0.07	-0.385	0.0049	0.1482	0.1531	0.392	- 5.50	-79.7	-0.222	9.22
10	0.448	+0.10	-0.245	2.38	-0.04	+0.14	+ 0.14	-0.385	0.0196	0.1482	0.1678	0.41	- 2.75	-70.0	-0.195	10.20

Table E-2 (cont'd)

y/L	u_1^2	s_1	w_1	u_2^2	s_2	w_2	S (s_1-s_2)	W (w_1-w_2)	S^2	W^2	S^2+W^2	K	W/S	$\tan^{-1}W/S$ (degrees)	$\frac{\tan^{-1}W/S}{360}$	Complete phase
								x/L =3								
0	8.0	+0.06	-0.05	16.0	+0.04	-0.04	+0.02	-0.01	0.000	0.000	0.01	0.10	-0.5	-26.6	-0.074	2.07
1	4.95	-0.06	-0.08	12.5	+0.0	-0.06	-0.06	-0.02	0.004	0.000	0.004	0.063	+0.333	-161.6	-0.449	2.45
2	3.31	+0.11	+0.05	9.9	-0.06	+0.04	+0.17	+0.01	0.029	0.000	0.029	0.17	+0.059	+3.4	+0.009	2.99
3	2.42	-0.03	+0.14	8.0	+0.06	-0.05	-0.09	+0.19	0.008	0.036	0.044	0.21	-2.11	+115.4	+0.321	3.68
4	1.89	-0.14	+0.07	6.62	+0.02	+0.09	-0.16	-0.02	0.26	0.000	0.026	0.16	+0.125	-172.6	-0.480	4.48
5	1.54	-0.17	-0.02	5.61	-0.09	+0.01	-0.08	-0.03	0.006	0.001	0.007	0.084	+0.38	-159.2	-0.444	5.44
6	1.30	-0.16	-0.29	4.85	-0.04	-0.09	-0.12	+0.0	0.014	0.000	0.014	0.12	+0	+180.0	+0.50	6.50
7	1.12	-0.13	-0.13	4.25	+0.05	-0.10	-0.18	-0.03	0.032	0.001	0.033	0.18	+0.17	-170.4	-0.474	7.41
8	0.985	-0.10	-0.17	3.78	+0.11	-0.04	-0.21	-0.13	0.044	0.017	0.061	0.25	+0.618	-148.3	-0.413	8.41
9	0.875	-0.07	-0.20	3.39	+0.12	+0.03	-0.19	-0.23	0.036	0.053	0.089	0.30	+1.21	-129.5	-0.359	9.36
10	0.792	-0.04	-0.21	3.08	+0.09	+0.08	-0.13	-0.29	0.017	0.084	0.101	0.32	+2.23	-114.2	-0.317	10.32
								x/L = 3.5								
0	10.0	-0.555	+0.05	18.0	-0.035	+0.035	-0.02	-0.015	0.0004	0.0002	0.0006	0.024	+0.75	-143.1	-0.449	2.45
1	6.78	+0.04	+0.08	14.4	-0.01	+0.06	+0.05	+0.02	0.0025	0.0004	0.0029	0.054	+0.40	+26.6	+0.061	2.94
2	4.81	-0.04	-0.09	11.70	+0.06	-0.02	-0.10	-0.07	0.010	0.0049	0.0149	0.039	+0.07	-145.0	-0.404	3.40
3	3.62	+0.12	-0.01	9.63	-0.07	+0.01	+0.19	-0.02	0.0361	0.0004	0.0365	0.191	-0.105	6.0	-0.017	4.02
4	2.87	+0.06	+0.12	8.08	+0.05	-0.06	+0.01	+0.18	0.0001	0.0324	0.0325	0.080	+18.0	+86.7	+0.241	4.76
5	2.38	-0.05	+0.14	6.90	+0.05	+0.07	-0.10	+0.07	0.01	0.0049	0.0149	0.122	-0.70	+142.1	+0.404	5.60
6	2.00	-0.12	-0.10	6.00	-0.07	+0.06	-0.05	+0.04	0.0025	0.0016	0.0041	0.064	-0.80	+141.3	+0.393	6.61
7	1.73	-0.16	-0.04	5.29	-0.09	-0.04	-0.07	+0.08	0.0049	0.0064	0.0113	0.106	-1.14	+131.3	+0.365	7.64
8	1.53	-0.17	-0.02	4.72	-0.03	-0.10	-0.14	+0.08	0.0256	0.0064	0.0320	0.179	-0.572	+150.2	+0.417	8.58
9	1.36	-0.16	-0.07	4.25	+0.05	-0.10	-0.21	+0.03	0.0441	0.0009	0.0450	0.212	-0.143	+171.8	+0.478	9.52
10	1.23	-0.15	-0.11	3.86	+0.10	-0.05	-0.25	-0.06	0.0625	0.0036	0.0661	0.257	+0.240	-166.5	-0.463	10.46
								x/L = 4.0								
0	12.0	+0.04	-0.04	20.0	+0.04	-0.035	0	-0.005	0	0.000025	0.000025	0.005		+90.0	+0.25	3.25
1	8.65	-0.01	-0.07	16.35	+0.02	-0.05	-0.03	-0.02	0.0009	0.0004	0.0013	0.036	+0.666	-146.3	-0.407	3.41
2	6.42	-0.01	+0.09	13.54	-0.06	0	+0.05	+0.09	0.0025	0.0081	0.0106	0.103	+1.8	+ 61.0	+0.169	3.82
3	4.97	-0.06	-0.08	11.30	+0.06	+0.02	-0.12	-0.10	0.0144	0.01	0.0244	0.156	+0.832	-140.1	-0.390	4.39
4	4.0	+0.087	-0.07	9.60	-0.07	-0.01	+0.155	-0.08	0.0240	0.0064	0.0304	0.174	-0.516	- 12.7	-0.076	0.08
5	3.32	+0.12	+0.04	8.28	+0.03	-0.07	+0.09	+0.11	0.0081	0.0121	0.0202	0.142	+1.22	+ 53.2	+0.141	5.86
6	2.83	+0.05	+0.12	7.24	+0.07	+0.04	-0.02	+0.08	0.0004	0.0064	0.0068	0.083	-4.0	+104.0	+0.289	6.71
7	2.46	-0.02	+0.14	6.41	-0.01	+0.09	-0.01	+0.05	0.0001	0.0025	0.0026	0.051	-5.1	+101.3	+0.280	7.72
8	2.17	-0.09	+0.12	5.73	-0.09	+0.03	0	+0.09	0	0.0081	0.0081	0.09		+ 90.0	+0.25	8.75
9	1.95	-0.13	+0.09	5.18	-0.09	-0.05	-0.04	+0.14	0.0016	0.0196	0.0202	0.142	-3.5	+108.2	+0.301	9.76
10	1.76	-0.15	+0.04	4.72	-0.03	-0.10	-0.12	+0.14	0.0144	0.0196	0.0340	0.185	-1.16	+130.8	+0.364	10.64

Table E-3. - Plotting values of y/L and x/L for diffraction diagram for two breakwaters; gap width, 2 wave lengths.

Comp. phase	Values of x/L								
	0	0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0
	(Values of y/L)								
0	0	0	0						
1	1	1	0.95	0.68					
2	2.06	2.2	2.00	1.92	1.62	0.95			
3	3.08	3.04	2.96	2.88	2.72	2.50	2.0	1.42	
4	4.08	4.05	3.98	3.87	3.74	3.62	3.40	2.96	2.32
5	5.10	5.07	5.00	4.90	4.75	4.65	4.55	4.30	3.88
6	6.10	6.07	6.01	5.92	5.80	5.65	5.52	5.38	5.15
7	7.10	7.08	7.03	6.95	6.82	6.68	6.52	6.36	6.27
8	8.10	8.08	8.05	7.95	7.78	7.74	7.57	7.41	7.28
9	9.10	9.09	9.05	8.97	8.88	8.76	8.62	8.95	8.25
10				10.00	9.90	9.80	9.69	9.50	9.34