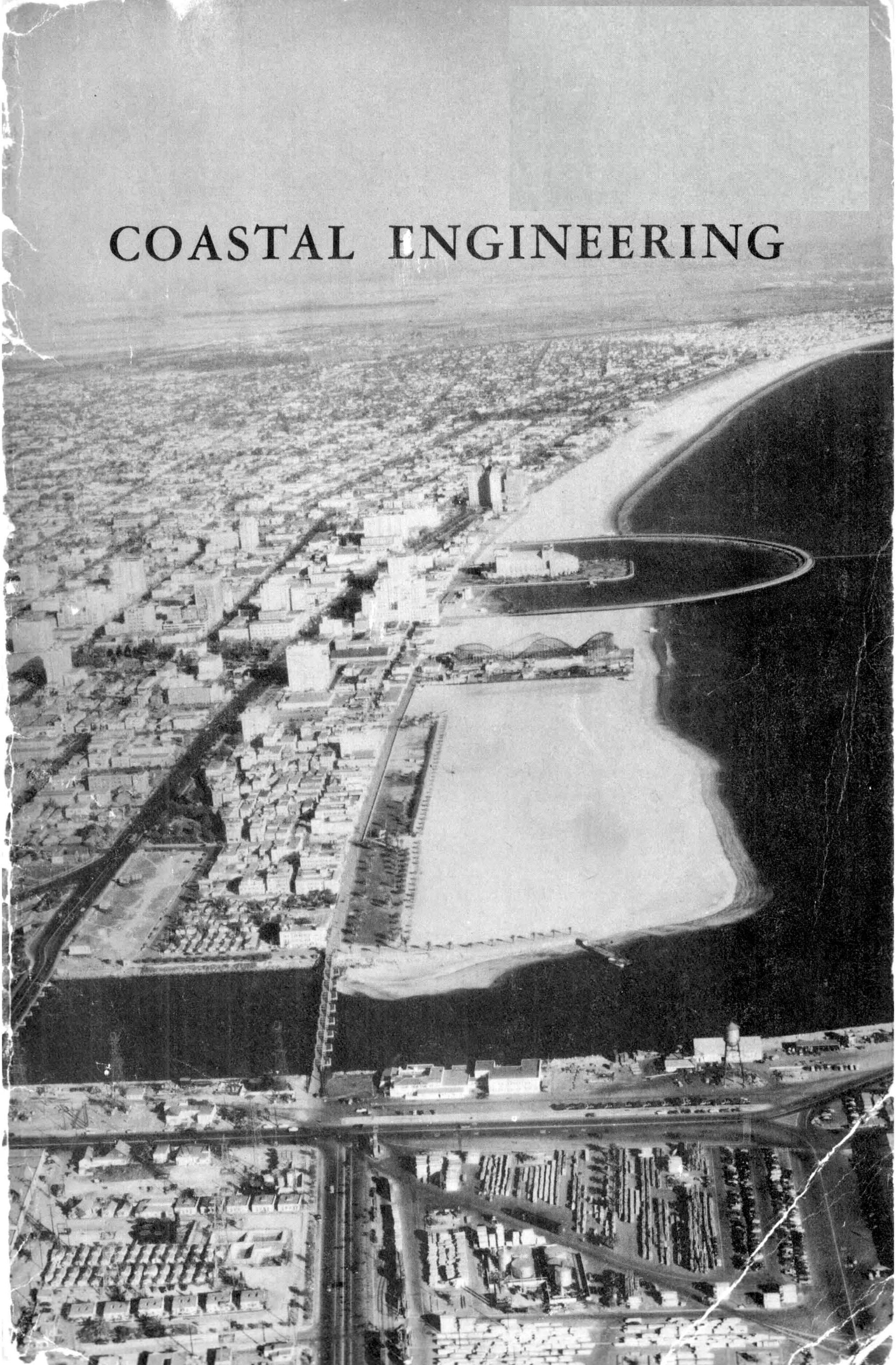
COASTAL ENGINEERING



PROCEEDINGS OF FIRST CONFERENCE

ON

COASTAL ENGINEERING

LONG BEACH, CALIFORNIA

OCTOBER 1950

Edited by

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In Memory of

BORIS A. BAKHMETEFF

who inspired the formation of

The Council on Wave Research

PREFACE

The Conference on Coastal Engineering at Long Beach was conceived originally as a local meeting of engineers and scientists interested in shoreline problems and was sponsored by the University of California. It early became evident that there was widespread interest in the subject and that the program should be planned on a more ambitious scale. The aim was to aid engineers by summarizing the present state of the art and science related to the design and planning of coastal works rather than to present a series of original scientific contributions. Starting from a rather comprehensive outline, invitations were issued to recognized authorities to report on specific phases of the subject, and the authors cooperated splendidly both in their treatment of the subjects assigned and in their avoidance of overlapping other subjects. Although much remains to be done in the way of developing reliable design methods, the series of papers presented at the conference and published in this volume do represent a rather thorough summary of coastal engineering as now practiced.

Engineers engaged in the design of coastal works have had available to them a large number of papers dealing with various phases of the science related to their problems, but proper dealing with design were limited in number and scope. Only a few books on coastal engineering have been published. The quality and scope of the papers and the need for a comprehensive and modern treatment of the subject convinced the sponsors of the conference that publication in a single volume was desirable rather than piecemeal in the scientific and technical journals. The newly-formed Council on Wave Research secured funds to underwrite the publication costs from its parent organization, the Engineering Foundation.

A word about the term "Coastal Engineering" is perhaps in order here. It is not a new or separate branch of engineering and there is no implication intended that a new breed of engineer, and a new society, is in the making. Coastal Engineering is primarily a branch of Civil Engineering which leans heavily on the sciences of oceanography, meteorology, fluid mechanics, electronics, structural mechanics, and others. However, it is also true that the design of coastal works does involve many criteria which are foreign to other phases of civil engineering and the novices in this field should proceed with caution. Along the coastlines of the world, numerous engineering works in various stages of disintegration testify to the futility and wastefulness of disregarding the tremendous destructive forces of the sea. Far worse than the destruction of insubstantial coastal works has been the damage to adjacent shorelines caused by structures planned in ignorance of, and occasionally in disregard of, the shoreline processes operative in the area.

The Council on Wave Research takes this opportunity to thank the authors of the papers and the many others who assisted in organization of the conference and in the preparation of this volume for publication.

Morrough P. O'Brien, Chairman
Council on Wave Research
Engineering Foundation

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SYMBOLS

A = Area.

a' = Length of semi-major axis of orbit of water particle.

$\overline{\ \ }_{av}$ = Subscript "av" refers to average.

B = Breakwater gap width. Also: Width of a channel.

b = Length of wave crest between orthogonals, (perpendicular to the local direction of travel).

b' = Length of semi-minor axis of orbit of water particle.

$\overline{\ \ }_{b}$ = Subscript "b" refers to breaking conditions.

$\overline{\ \ }_{B}$ = Subscript "B" refers to bottom conditions.

C = Wave velocity.

C_D = Coefficient of drag.

C_g = Group velocity.

C_H = Velocity of waves of finite height.

C_i = Instrument calibration constant.

C_M = Coefficient of mass.

D = Decay distance. Also: Diameter.

d = Depth of water, measured from the still-water level to the bottom.

E = Mean total energy of wave per unit length of crest per wave.

E' = Mean total energy of wave per unit area of crest per wave.

E_k = Mean kinetic energy of wave per unit length of crest per wave.

E_p = Mean potential energy of wave per unit length of crest per wave.

e = Wave energy coefficient at the breaker line. Also: Base of Naperian logarithms = 2.718

F = Force. Also: Length of fetch.

F_d = Dynamic force.

F_r = Resistance force.

f = Function of one or more variables, as $f(x,y)$.

g = Acceleration of gravity.

H = Wave height.

H_o' = $K_d\ H_o$

H_D = Height of the average highest one-third of the waves at the end of the decay distance for a specified period of time.

H_F = Height of the average highest one-third of the waves at the end of the fetch for a specified period of time.

i = Beach or breakwater slope. Also: $\sqrt{-1}$.

J = Distance between two underwater contours as used in the orthogonal methods of wave refraction coefficient determination.

k = Roughness coefficient. Also: A constant.

K = Sub-surface pressure response coefficient.

$K_1, K_2, K_3,$ = Coefficients.

K_d = Refraction coefficient.

K' = Diffraction coefficient.

L = Wave length (distance between two successive crests).

L_r = Ratio of linear dimensions, model to prototype.

ℓ = a length.

M = Energy coefficient.

M_z = Total moment about the level, z.

MLLW = Mean Lower Low Water -- a datum.

MSL = Mean Sea Level -- a datum.

m = $2 \pi/L$.

N = Waves steepness factor.

n = Ratio of group velocity to wave velocity.

$\overline{}_o$ = Subscript "o" refers to deep water.

P = Power transmitted by a wave per unit length of crest per wave. Also: probable frequency of suitable wave height.

p = Unit pressure. Also: probable frequency of suitable wind force.

Q = Rate of flow. Also: littoral drift factor.

R = Radius of rolling circle (Gerstner Theory). Also: Distance between contours measured along an orthogonal.

R_1 = Reading of a wave recording instrument.

r = Radius of tracing circle (Gerstner Theory).

$\overline{}_r$ = Subscript "r" refers to a scale ratio.

s = Specific gravity.

$\overline{}_s$ = Subscript "s" refers to surface terms.

S.L.W. = Still (undisturbed) water level.

T = Wave period.

T_D = Average wave period at end of decay.

T_F = Average wave period at end of fetch.

t = time.

t_D = Travel time of waves (from end of fetch to end of decay distance).

t_d = Wind duration, interval of time wind blows at constant velocity in generating waves.

t_ν = Modulus of wave decay due to viscosity.

U = Wind velocity.

$\bar{U}$ = Velocity of mass transport.

U' = Horizontal velocity of motion left after wave motion has been destroyed (Gerstner Theory).

u = Water particle (horizontal component, positive in the direction of wave advance) orbital velocity.

V = Velocity, other than wave.

$\bar{V}$ = Volume.

v = Water particle (vertical component measured positive upwards) orbital velocity.

v_r = Void ratio.

W = Weight.

w = Unit weight. Also: total work performed by all waves of a given period and direction in deep water just offshore during a typical year.

X = Probable frequency of suitable sea condition.

x = Horizontal coordinate (arbitrary origin), positive in direction of wave advance.

Y = Probable frequency of unsuitable swell conditions.

y = Vertical coordinate (arbitrary origin), positive when measured upwards.

Z_s = Surface elevation in diffraction theory.

z = Depth below still-water level for the irrotational theories, or the depth below the mean position of the surface orbit for the trochoidal theory.

α = Angle of wave crest to bottom contours. Also: Angle of wave approach, measured between the shoreline and the line of wave advance.

β = Angular position of the water particle when the maximum horizontal force or moment occurs.

γ = Ratio of unit weights in model and prototype.

Δ = Rock shape factor.

δ = Wave steepness, H/L.

η = Vertical displacement of particle from its mean position.

θ = Angular displacement.

μ = Effective coefficient of fraction. Also: Absolute viscosity.

ν = Kinematic viscosity.

ξ = Horizontal displacement of particle from its mean position.

π = 3.1416.

ρ = Density.

σ = Surface tension.

ϕ = A potential function. Also: an angle.

ψ = Azimuth of direction of wave travel, in the direction of travel.

ω = Angular velocity.

Part 1
BASIC PRINCIPLES OF WAVE MOTION

CHAPTER 1

ORIGIN AND GENERATION OF WAVES

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INTRODUCTION

It is no more possible to speak of <u>the</u> origin of ocean waves than it is to speak of <u>the</u> origin of sound waves, or of electromagnetic waves. Different types of waves exist, often simultaneously, and these differ from one another with respect to their origin and generation.

A convenient classification can be made on the basis of wave period, i.e., the time interval between the passage of successive crests at a fixed point. Tentatively we may use the following major divisions (Fig. 1):

Classification	Period
Capillary waves	less than 0.1 sec.
Ultra-gravity waves	from 0.1 sec. to 1 sec.
Ordinary gravity waves	from 1 sec. to 30 sec.
Infra-gravity waves	from 30 sec.to 5 min.
Long-period waves	from 5 min. to 12 hours
Ordinary tides	12 hours to 24 hours
Trans-tidal waves	24 hours and up

These ranges correspond to bands in the spectrum of electromagnetic waves and altogether constitute the <u>spectrum</u> of ocean waves.

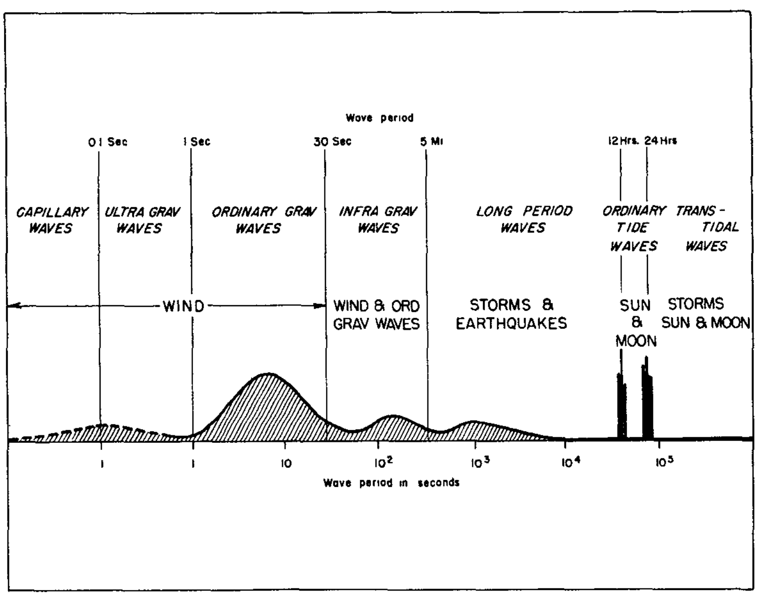

Fig. 1. Tentative classification of ocean waves according to wave period. The forces responsible for various portions of the spectrum are shown. The relative amplitude is indicated by the curve.

1 *Contribution from the Scripps Institution of Oceanography, New Series No. 531. This work represents results of research carried out for the Office of Naval Research, Department of the Navy, and the Beach Erosion Board, Department of the Army, under contract with the University of California.

APPLICATION OF SPECTRUM

Virtually all the energy of the spectrum is contained in two of the period ranges: the ordinary gravity waves, and the ordinary tides. These are the ones that can be noticed with the naked eye, the ones that have affected mankind since the dawn of history. In the minds of this group the term "waves" is virtually synonymous with the ordinary gravity waves, and we shall be concerned chiefly with them.

However, it is not just for academic reasons that the entire spectrum is discussed. There are some engineering applications that are definitely concerned with periods other than those of the ordinary gravity waves or the tides. The reason usually has to do with "resonance." For example, an artificial harbor with a natural period of oscillation of 2 min. may develop troublesome 2-min. surges, and these depend largely on infra-gravity waves, even though these waves are only a few inches high and completely obscured by much higher waves of shorter period. A ship with a natural roll of 5-sec. period is greatly affected by waves of about 5-sec. period, but only slightly affected by a 14-sec. swell, even though the swell may be much higher. The effectiveness of RADAR is limited by "sea clutter" due to reflection and scatter from capillary waves, rather than the higher wind waves. There are other examples, and they will be covered in the following chapters.

THE STUDY OF THE WAVE SPECTRUM

The scientific problem is to study the origin of the entire wave spectrum, and, as a final test of such efforts, to predict it. Once the spectrum is known, it is the problem of the engineer to evaluate its effect on engineering structures, harbors, and many other things. There has been considerable success in predicting tides, and for most practical application these predictions are adequate. There has been moderate success in predicting wind waves and swell. This subject is discussed by R. S. Arthur in Chapter 8. Briefly reviewed below is the little that is known regarding the fundamentals of wave generation, starting with the waves of shortest period.

CAPILLARY WAVES

This portion of the spectrum is affected to a larger extent by surface tension than by gravity. Very roughly we may include waves of periods less than 0.1 sec. The appropriate wave lengths are shorter than one inch. The velocity increases with decreasing period and length, but always exceeds 0.75 ft. per sec.

These waves are caused by wind, but we know nothing of their mode of origin. They are greatly affected by surface-active agents, such as oils and detergents. They are rapidly damped by viscous forces, and apparently require winds above 3 ft. per sec. for their generation. These waves determine largely the optical effects of the surface. One convenient way of studying them is to photograph the glitter of the sun on the water surface. These capillary waves are also largely responsible for what might be termed the inherent roughness of the surface. As such they determine the effectiveness with which the wind can grip the water. This in turn enters in the study of currents, the piling up of water by on-shore winds (storm tides), and also in the generation of longer period waves.

ULTRA-GRAVITY AND ORDINARY GRAVITY WAVES

Again we deal with waves which owe their existence to the wind. Two mechanisms by which the wind may cause these waves to grow are, (a) an excess in pressure on the windward side of waves compared to the pressure on the leeward side, (b) tangential drag on the water surface, assuming waves to have a mass transport according to the irrotational theory. These are the two mechanisms which form the basis of the forecasting method described in Chapter 8. They are of about equal magnitude. There are a number of other possible mechanisms for explaining wave growth, and some of these are now being investigated. All these processes already require the existence of a wave, though possibly one of very small height, in order for the wind to transfer energy to it and to make it grow. However, such a distinction between "origin" and "growth" does not appear to be of practical importance. It seems safe to assume that the impulses received by the sea surface as a consequence of the gustiness of the wind assures a steady supply of very low waves of all length.

The outstanding weakness of present theories concerning the generation of waves is that they do not allow for the variability of the ordinary gravity waves. The theories are concerned only with the higher waves present. They say nothing about the character of the waves twice the period of the higher waves, or half their period.

Consider a rather severe storm, capable of raising a sea for which the higher waves are 20 ft. high and have a period of 10 sec. At the same time there will be present longer waves and shorter waves. The longest waves may have periods up to 25 sec. The height of the 20-sec. waves is perhaps only one foot, and to the naked eye these long waves are obscured by the higher waves present. Owing to the fact that in deep water the velocity of the waves is proportional to their period, these long waves will travel ahead of the heavy swell and may reach land several days prior to the arrival of the highest waves. The practical application of these "forerunners" of the swell in providing a warning of higher waves to come (and possibly the storm itself) has been demonstrated in the Atlantic and Pacific.

Measurements of swell, and forerunners of swell, indicate a remarkably slow attenuation of these waves after they have left the storm area. We are recording in California waves from violent storms in all parts of the Pacific to which we are exposed. As a matter of fact, our heavy summer swell has its origin in the roaring forties of the South Pacific Ocean, 5000 miles and 10 days travel time removed from here. Some allowance for a decrease in wave height and a shift towards the longer periods has been made in existing theories. Measurements at Oceanside, California by the Department of Engineering, University of California, Berkeley, indicate that the present estimates of period increase in swell are in error (Wiegel and Kimberley, 1950).

INFRA-GRAVITY WAVES

Instruments especially adapted for the recording of the long waves have revealed the presence of 2-min. waves that are related to the variability in the incoming ordinary gravity waves. A series of high breakers temporarily raises the water level, a series of low breakers permits the level to fall. This oscillation (surf-beat) appears to be propagated seaward from the surf zone. Just outside the surf zone the amplitude of the surf-beat is 10 percent of that of the mean breakers.

These long waves in their travel seaward are greatly affected by the bottom topography over the entire continental shelf. Isaacs, Williams, and Eckart (1951) have shown that they may even be turned around completely and returned to shore. Calculations show that in depths of several hundred feet the orbital velocities near the bottom associated with the surf beat is likely to be larger than the corresponding velocities of the incoming ordinary gravity waves. A pipe line along the bottom at such depths would be affected principally by such long-period waves. Oscillations in bays and harbors (seiches) may be the result of surf beat.

LONG-PERIOD WAVES

Fairly well defined waves of 15- to 20-min. periods have also been recorded at La Jolla and Oceanside, California. The origin of these waves remains obscure, although a correlation with the meteorological situations is strongly indicated. Some of the remarks made in relation to the surf beat apply here also. These waves are small, hardly more than 2 in. in amplitude.

Waves of somewhat the same period, but of much larger height, result from the sudden displacement of the sea bottom during submarine earthquakes. Popularly these are known as "tidal waves," but since they have nothing to do with the tides, we prefer to use the Japanese word "tsunamis." In Hilo, Hawaii, a tsunami of destructive intensity seems to come about once every 20 years. Protection against such tsunamis is certainly the No. 1 problem of coastal engineering in the Hawaiian area. The construction of a 15 ft. sea wall along the waterfront in Hilo is now being seriously considered. Fortunately the coast of Southern California does not seem to be subject to destructive tsunamis. The largest tsunami waves here that have come to my attention were recorded in the Mission records of San Diego: 2.5 ft.

ORDINARY TIDES

These are caused by the sun and moon. As a whole, the two contribute about equally, since the very much larger mass of the sun is just about compensated for by its larger distance. Most of the energy goes into the semi-diurnal and diurnal periods (the "ordinary" tides), and these are of the same order of magnitude along our coast as the ordinary gravity waves. In most instances, but not always, the effect of the wind on waves of ordinary tide period is relatively small, and the forecast based only on the astronomic factors is satisfactory. It should be added that the forecasts are largely empirical: only the periods of the components are computed from astronomical considerations, whereas the amplitude and phase of each component is determined from tide records at or near the locality for which the forecast is made.

TRANS-TIDAL WAVES

There are various components of solar and lunar tides in this range of periods, but these tend to be outweighed by the meteorologic factors. The situation is therefore reversed from what is found in the ordinary tide range.

In the river Thames non-tidal variations in water level of several days duration have been observed. There seems to be a correlation between such variations and the passage of storms over the Wyville Thompson ridge in the North Atlantic.

There are also seasonal variations in water level related to the variations in ocean current, and hence indirectly to the prevailing winds. It is not generally known that along the coast of Southern California the mean sea level in the summer is 2/3 of a foot above that in the winter.

We could go on, and consider variations in sea level that have taken place in climatic cycles. For example, the oceans are believed to be rising 1/2 foot per century.

CONCLUSIONS

Any shore installation is, therefore, under the influence of waves ranging in periods from milliseconds to years. The underlying causes of these waves are the winds, directly or indirectly; the sun and moon, and submarine earthquakes. By far the largest part of the spectrum reflects the action of the wind. The wind is most effective in raising waves whose periods lie between 5 sec. and 15 sec. These can be predicted, in a manner. The fact that shorter and longer waves are of smaller height does not mean that they are of no practical consequence. In 10 to 20 years we may be able to predict the entire spectrum of wind generated waves.

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CHAPTER 2

ELEMENTS OF WAVE THEORY

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INTRODUCTION

The first known mathematical solution for finite height, periodic waves of stable form was developed by Gerstner (1802). From equations that were developed, Gerstner (1802) arrived at the conclusion that the surface curve was trochoidal in form. Froude (1862) and Rankine (1863) developed the theory but in the opposite manner, i.e., they started with the assumption of a trochoidal form and then developed their equations from this curve. The theory was developed for waves in water of infinite depth with the orbits of the water particles being circular, decreasing in geometrical progression as the distance below the water surface increased in arithmetical progression. Recent experiments (Wiegel, 1950) have shown that the surface profile, represented by the trochoidal equations (as well as the first few terms of Stokes' theory), closely approximates the actual profiles for waves traveling over a horizontal bottom. However the theory necessitates molecular rotation of the particles, while the manner in which waves are formed by conservative forces necessitates irrotational motion.

The first satisfactory treatment of two dimensional wave motion in water of arbitrary depth was given by La Place (1776) for waves of small amplitude. Airy (1845) developed an irrotational theory for waves traveling over a horizontal bottom in any depth of water. This theory was developed for waves of very small height. Airy (1845) showed that the velocity of propagation of the wave form was dependent upon the wave length as well as upon the water depth.

Stokes (1847) presented an approximate solution for waves of finite height which satisfied the boundary conditions of waves in water of uniform depth and, in addition, required irrotational motion. The series was to the third approximation for finite depths, or to the fifth approximation for infinite depths, but there was no proof of their convergence. The most interesting features of the solution, apart from the irrotational motion, were, first, the dependency of the wave velocity upon wave height as well as upon wave length and water depth and, second, the fact that orbital motion of the particles was open rather than closed, indicating a mass transport in the direction of wave travel. Experiments (Mitchim, 1940) have shown both of these findings to be correct.

Levi-Civita (1925) proved that Stokes' series was convergent for "deep-water" waves and Struik (1926) proved that it was convergent for "shallow-water" waves.

Reynolds (1877) and Rayleigh (1877) worked on the problem of the difference between the energy transmission velocity of a wave group and the velocity of the wave form. They concluded that the energy of the group of waves was propagated with a velocity less than that of the individual waves. In deep water, the "group" velocity was found to be one-half the wave velocity.

The problem of the maximum steepness (the ratio of the wave height to its length) that a wave could attain without breaking was worked on by Stokes (1847), Michell (1893), and Havelock (1918). Their conclusions were in close agreement. A crest angle of 120 degrees, or a steepness of $H/L = 0.142$,* was found to be the theoretical limit.

Recently, many field and laboratory studies, as well as analytical studies, have been made. These observations, together with the mathematical studies, lead

*See list of symbols.

to the conclusion that Stokes' irrotational theory represents the natural phenomena more closely than the other theories.

Waves in nature vary considerably in height and period over a relatively short length of time at any point of observation. In a generating area, the wave characteristics show the maximum variability; however, even after the waves have passed into a region of relative calm, considerable variations in wave characteristics exist. In theoretical problems, such variability cannot be treated mathematically and certain idealized conditions must be assumed. Accordingly, the first step in the analysis of oscillatory waves is to study the behavior of single wave trains of uniform period and amplitude as they progress in water of constant depth. Present-day wave theory deals with periodic waves of stable form in which all elements of the wave profile advance with the same velocity relative to the undisturbed water. Complete development of the various analyses of Lamb (1932), Stokes (1847), Gerstner (1802), and others are not presented herein as they are readily available in the original references.

WAVES OF SMALL AMPLITUDE

If waves are of small amplitude compared to their length and to the depth of the water, the wave profile closely approximates a sine curve. The equation for motion (Lamb, 1932), considering both gravity and surface tension, is:

$$C^2 = (gL/2\pi + 2\pi\sigma/\rho L) \tanh 2\pi d/L \tag{1}$$

For water deeper than one-half the wave length, tanh $2\pi d/L$ is almost equal to 1 and the equation reduces to:

$$C_o^2 = gL_o/2\pi + 2\pi\sigma/\rho L_o \tag{2}$$

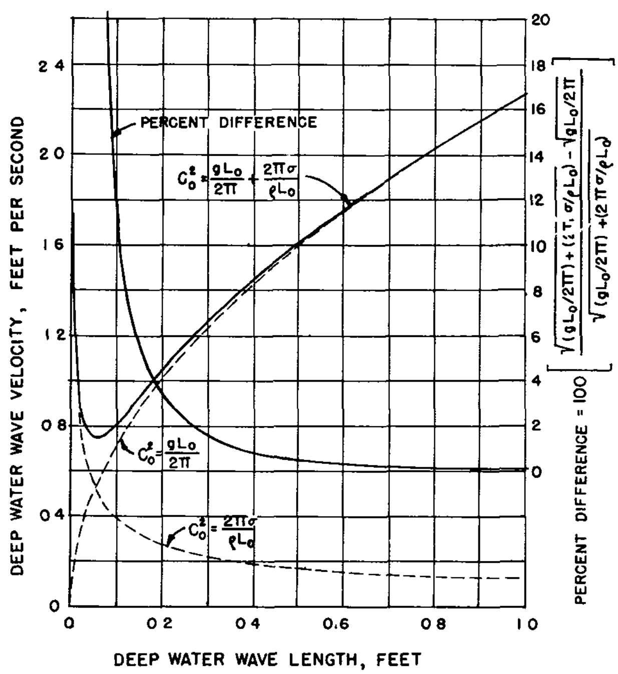


Fig. 1. Effect of surface tension on deep water wave velocity in fresh water at 70°F.

The relative effects on velocity of the gravity and the surface tension components for deep-water waves are presented in Fig. 1. Experimental data by Chinn (1949) and Kaplan (1950) verifies the equation. It can be seen that for any wave over a foot in length, the effect of surface tension may be neglected. In practice, these small waves are usually called ripples as distinguished from the longer waves.

Neglecting the effect of surface tension, the equation for velocity of propagation of gravity waves (Airy, 1845; Lamb, 1932)

$$C^2 = (gL/2\pi) \tanh 2\pi d/L \tag{3}$$

and for "deep-water:"

$$C_o^2 = gL_o/2\pi \tag{4a}$$

or, in English units:

$$C_o^2 = 5.12\, L_o \tag{4b}$$

Since the relationship between length, period and velocity of all periodic wave phenomena is defined by:

$$L = CT \tag{5}$$

it follows that:

$$L_o = gT^2/2\pi \tag{6a}$$

or, in English units for "deep-water.:"

$$L_o = 5.12\ T^2 \tag{6b}$$

Actually, there is no abrupt change from "deep" to "shallow" water. The effect of depth of water on the wave characteristics is gradual, and waves in any finite depth of water are affected by the depth. The depths for which the simplified equations are no longer applicable depends upon the degree of accuracy desired in calculations. The custom has developed, for most engineering studies, to call water which is deeper than one-half the wave length "deep-water" and water which is less than half the wave length "shallow water."

At the other extreme, "very shallow water," tanh $2\pi d/L$ approaches the value of $2\pi d/L$ and equation 3 becomes:

$$C^2 = gd \tag{7}$$

the well-known equation for very long low waves This equation also holds for waves of finite height. Fig. 2 shows the relationships between wave period, wave velocity, and depth of water. Fig 3 shows the relationships of wave period, wave length and depth of water

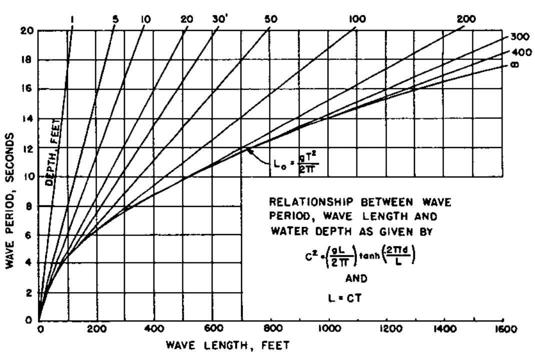


Fig. 2

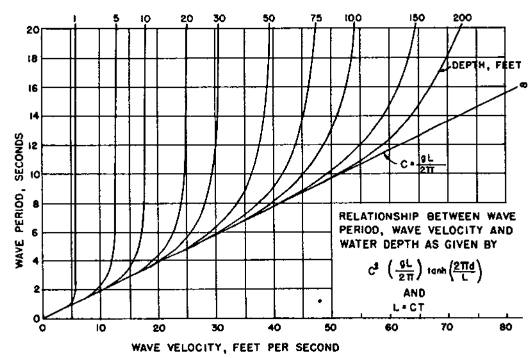


Fig. 3

Surface Profile. The surface curve for waves of small amplitude as given by this theory is the sinusoidal equation:

$$y = (H/2)\cos 2\pi(t/L - x/L) \tag{8}$$

Orbital Motion. The motion of the individual particle is elliptical (Fig. 4a).

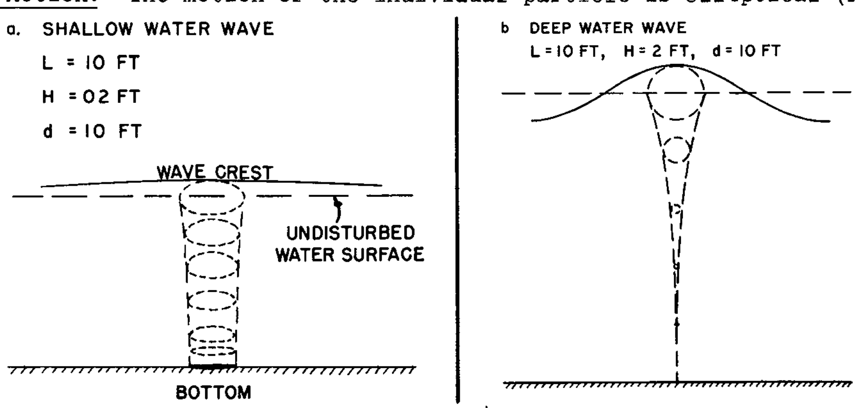


Fig. 4

The horizontal and vertical displacements from its mean position, a distance z (measured negatively downward) below the still-water surface, are:

$$\xi = \tfrac{1}{2}H \frac{\cosh 2\pi(d + z)/L}{\sinh 2\pi d/L} \cos 2\pi(x/L - t/T) \qquad (9a)$$

$$\eta = \tfrac{1}{2}H \frac{\sinh 2\pi(d + z)/L}{\sinh 2\pi d/L} \sin 2\pi(x/L - t/T) \qquad (9b)$$

From these equations it can be seen that the semi-orbital amplitudes of the subsurface particle's motions are:

$$a' = \tfrac{1}{2}H \frac{\cosh 2\pi(d + z)/L}{\sinh 2\pi d/L} \qquad (10a)$$

$$b' = \tfrac{1}{2}H \frac{\sinh 2\pi(d + z)/L}{\sinh 2\pi d/L} \qquad (10b)$$

with the ratio of the orbital amplitudes (Fig. 5) being:

$$\frac{b'}{a'} = \tanh 2\pi(d + z)/L \qquad (11)$$

Recent experiments (Morison, 1948) have verified these equations (Figs. 6 and 7) except that, in addition, there is some mass transport. The full amplitude of the orbital motion at the surface ($2a_s'$ and $2b_s'$) may be expressed as:

$$2a_s' = H \coth 2\pi d/L \qquad (12a)$$

$$2b_s' = H \qquad (12b)$$

When the equations are converted into their exponential form, it is found that as the water depth approaches infinity:

$$a' \longrightarrow \tfrac{1}{2}He^{2\pi z/L} \qquad (13a)$$

$$b' \longrightarrow \tfrac{1}{2}He^{2\pi z/L} \qquad (13b)$$

However, the horizontal and vertical semi-amplitudes approach these limiting values at different rates with respect to z (Fig 5). So, although the orbital motion near the surface becomes nearly circular in shape very rapidly as the depth of water increases, the orbital paths become flatter and flatter with increasing distance below the surface until, at the bottom, the vertical motion is zero and so the particle moves back and forth with a purely horizontal motion. Only when the water depth becomes "infinite" are all the particle paths circular. Fig. 8a shows the vertical amplitude of oscillation for various depths and wave lengths, and Fig. 8b shows the horizontal amplitude of oscillation for various depths and wave lengths.

By differentiating the horizontal and vertical orbital displacements with respect to time, the horizontal and vertical components of the water particle velocities occupying an average position at a distance z below the center of the surface particle (this neglect of second order quantities appears to be allowable) are found to be:

$$u_z = \frac{\partial \xi}{\partial t} = \frac{\pi H \cosh 2\pi(d + z)/L}{T \sinh 2\pi d/L} \sin 2\pi(x/L - t/T) \qquad (14a)$$

$$v_z = \frac{\partial \eta}{\partial t} = \frac{-\pi H \sinh 2\pi(d + z)/L}{T \sinh 2\pi d/L} \cos 2\pi(x/L - t/T) \qquad (14b)$$

with the average velocities over one-half their cycle being:

$$(u_z)\text{ave.} = \pm \frac{2H \cosh 2\pi(d + z)/L}{T \sinh 2\pi d/L} \qquad (15a)$$

$$(vz)\text{ave.} = \pm \frac{2H \sinh 2\pi(d + z)/L}{T \sinh 2\pi d/L} \qquad (15b)$$

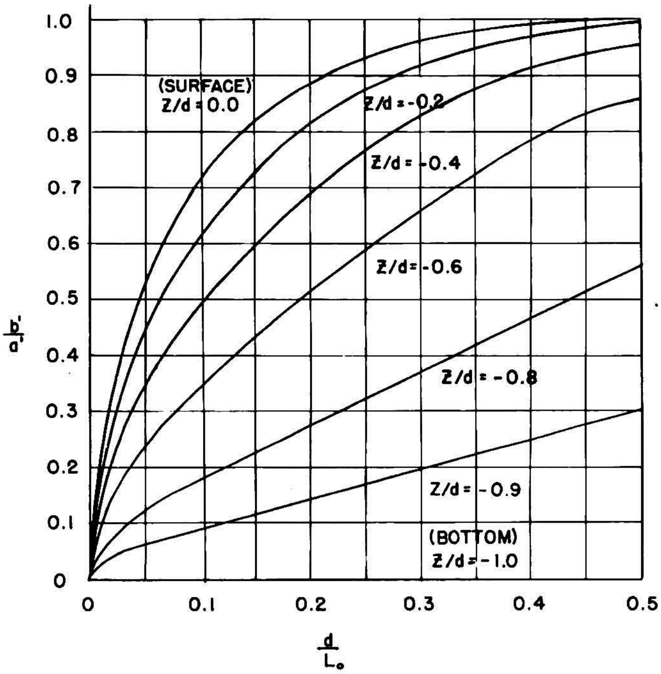


Fig. 5

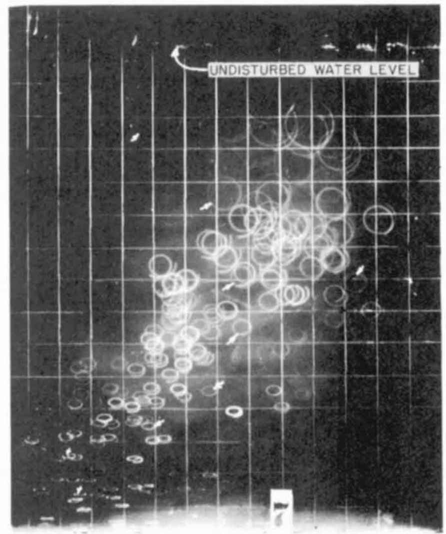
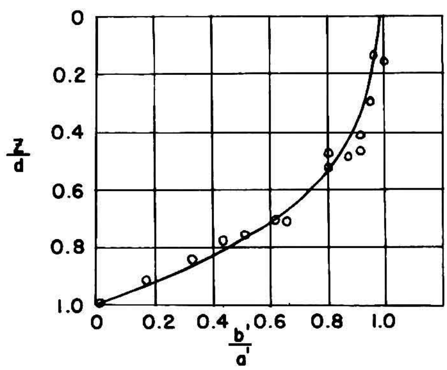
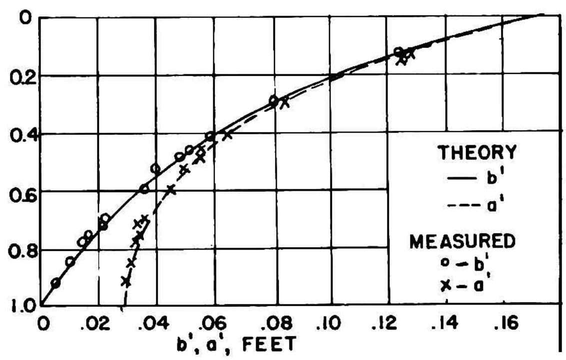


Fig. 6. Photograph of water particle orbits for a wave with the following dimensions. d = 2.50 feet, H = 0.339 feet, L = 6.42 feet, T = 1 12 seconds and d/L_o = 0.39

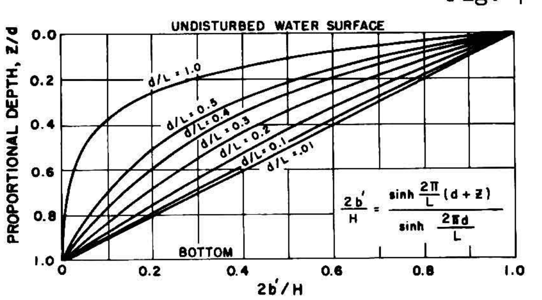


a. Comparison of the ratios of measured orbit axes with theory for d/L_o=0.39

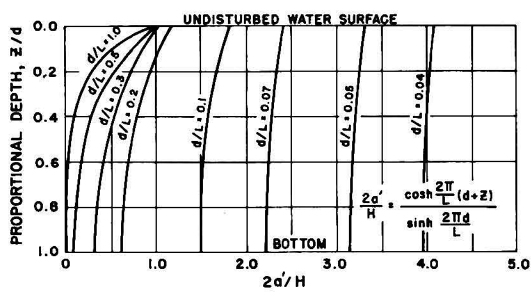


b. Comparison of measured orbit semi-major and semi-minor axes with theory: d/L_o= 0.39

Fig. 7



a. Vertical amplitude of oscillation for proportional depth related to fraction of wave height.



b. Horizontal amplitude of oscillation for proportional depth related to fraction of wave height.

Fig. 8

and their maximum orbital velocities being,

$$(u_z)_{max.} = \frac{\pi H \cosh 2\pi(d + z)/L}{T \sinh 2\pi d/L} \tag{16a}$$

$$(v_z)_{max.} = \frac{\pi H \sinh 2\pi(d + z)/L}{T \sinh 2\pi d/L} \tag{16b}$$

An example of the manner in which the maximum horizontal component of orbital velocity varies with the period is shown in Fig. 9. These values are for a particle on the ocean bottom.

Energy of Waves. The kinetic energy per unit width (along the crest) for a wave is the summation of the kinetic energy of the particles in motion. For a wave of sinusoidal form in deep water, this is given by,

$$E_k = wL_oH_o^2/16 \tag{17}$$

The potential energy per unit width for a wave is computed from the elevation or depression of the water from the undisturbed level and is given by,

$$E_p = wL_oH_o^2/16 \tag{18}$$

It can be seen that half of the energy of a wave is kinetic and half potential. The total energy is expressed by,

$$E = wL_oH_o^2/8 \tag{19a}$$

which, when combined with Equation 6a, gives,

$$E = wgT^2H_o^2/16\pi \tag{19b}$$

Effect of Viscosity. The effect of viscous damping of water waves of small amplitude of sinusoidal form has been studied mathematically by Lamb (1932) for waves in deep-water and by Hough (1896) for waves in any depth of water (assuming the bottom to be perfectly smooth). The modulus of decay, t_ν, (the time necessary for the wave height to be reduced in the ratio of e : 1) is given by,

$$t_\nu = L^2/8\pi^2\nu \tag{20}$$

It can be seen from Fig. 10 that extremely short (capillary) waves die out rapidly but that the damping is very small for waves of any appreciable length.

Sub-Surface Pressures. With the development of the pressure type wave recorder (Folsom, 1949; Isaacs and Wiegel, 1950), it became necessary to utilize the equations for pressure at any point beneath the water surface. The solution (Lamb, 1932) for an incompressible, nonviscous fluid is,

$$K = [\cosh 2\pi dL(1 - z/d)]/\cosh 2\pi d/L \tag{21}$$

where K, the sub-surface pressure response factor, is the ratio of the pressure at any depth below the water surface and the pressure at the surface. The ratio of the distance below the surface to the water depth is known as the proportional depth. This can be represented in dimensionless form as shown in Fig. 11. Tabulated values have been published by the Beach Erosion Board (Wiegel, 1948). Experiments (Folsom, 1947) show that this approximates the case for waves of finite height. However, for waves of finite height, the measured pressures were about ten percent lower than the theory (for very small waves) predicts.

WAVES OF FINITE AMPLITUDE

Experiments (Beach Erosion Board, 1941; Morison, 1951; Wiegel, 1950) have shown that the equations for waves of small amplitude continue to be valid, as far

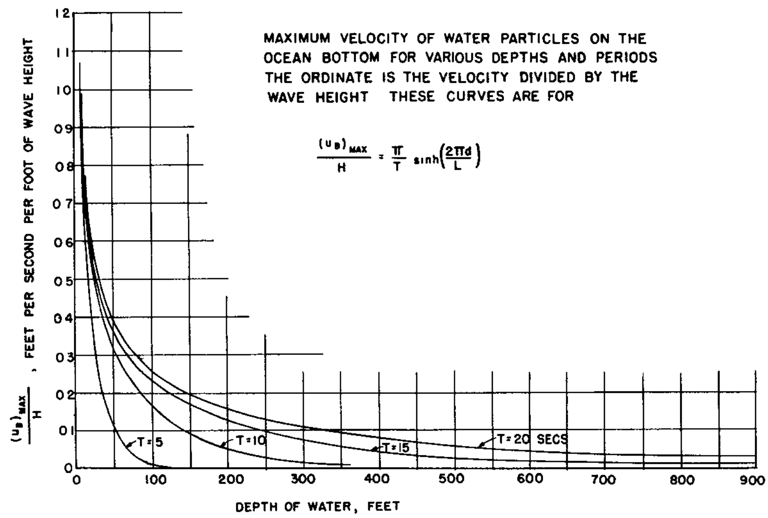
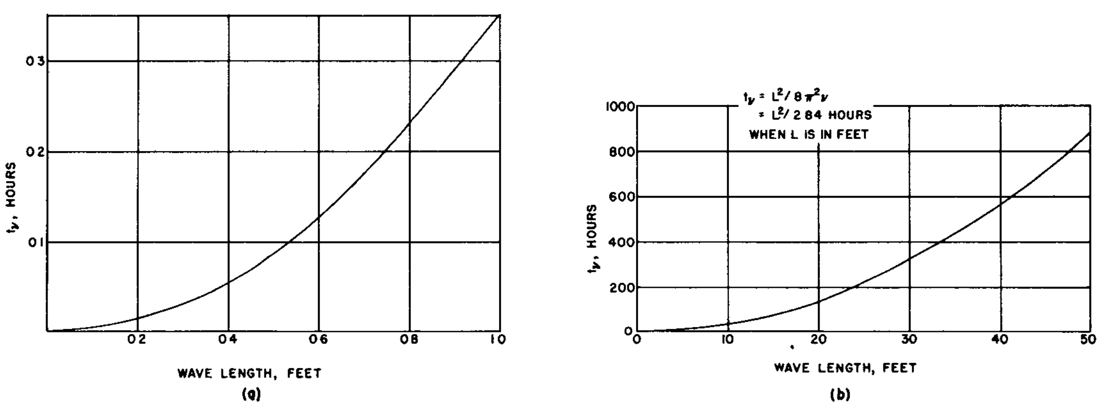


Fig. 9



Relationship between modulus of decay due to viscous damping and wave length.
Fig. 10

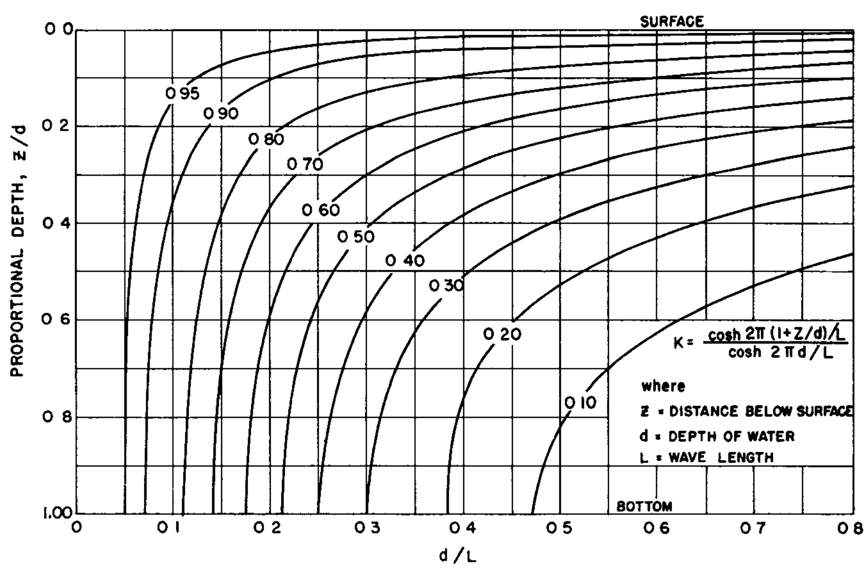


Fig. 11. Pressure response factor.

as engineering applications are concerned, for waves of appreciable height. It has also been observed that the very long, low ocean swell from distant storms are approximately sinusoidal in deep-water. However, for waves of greater height, theory indicates that certain corrections are necessary.

Two theories have been developed for waves of finite height. The first theory, developed by Gerstner (1802) and later by Froude (1862) and Rankine (1863), is known as the trochoidal theory. This theory has been used widely by naval architects and engineers in their studies. The second theory, developed principally by Stokes (1847) and later by Struik (1926) and Levi-Civita (1925), is more difficult to apply but it predicts certain results that have been experimentally verified which are not predicted by the trochoidal theory.

Trochoidal Theory - Infinite Water Depth. The trochoidal theory (Gerstner, 1802), the first theory to be developed for waves of finite height, is often used for engineering calculations. One reason for its use is the ease with which the equations may be used. It appears to represent the actual wave profiles as well as actually satisfying the pressure conditions at the surface and the continuity conditions. However, it requires rotation of the particles and does not predict any mass transport in the direction of wave propagation, while observations (Mitchim, 1940; Beach Erosion Board, 1941) show that there is mass transport. This theory, developed for waves in water of infinite depth, has been well presented by Gaillard (1935).

The equations of the surface profile (Fig. 12a) are,

$$x = R\theta - r \sin \theta \qquad (22a)$$

$$y = R - r \cos \theta \qquad (22b)$$

It can be seen that the wave length, L_o, is equal to $2\pi R$, while the wave height, H_o, is equal to $2r_s$, where r_s is the value of r for the surface orbit. In order to plot the equation of wave shape in dimensionless form with the origin of the coordinates at the crest and the vertical dimension measured negatively, downward, these equations may be transformed to:

$$x'/L_o = 1 - [(\text{rad } \theta/2\pi) - (H_o/2L_o)\sin\theta] \qquad (22c)$$

$$y'/H_o = 1/2(1 - \cos\theta) \qquad (22d)$$

where, x' and y' are measured from the wave crest. These have been plotted in Fig. 13 with H_o/L_o as the parameter. It can be seen that as H_o/L_o approaches zero, the curve approaches a sine wave and the surface is nearly that as developed in the irrotational theory for waves of very small amplitude.

The positions of the crest and trough relative to the undisturbed water level are,

$$\text{Height of crest} = H_o - [r_s - (r_s^2/2R)] = 1/2H_o + \pi H_o^2/4L_o \qquad (23a)$$

$$\text{Depth of trough} = 2\pi R[r_s - (r_s^2/2R)]/L_o = 1/2H_o - \pi H_o^2/4L_o \qquad (23b)$$

Thus, the crest is more than half the wave height above the undisturbed water level, while the trough is less than half the wave height below this level. Experiments performed by the Beach Erosion Board (1941) verify these relationships (Fig. 14). It should be noted that they verify the results of the theory of Stokes (1847) as well.

The paths described by the water particles during one cycle are circles with the radii decreasing exponentially with depth (Fig. 12b). This is expressed as,

$$a' = b' = r_s\, e^{2\pi z/L_o} = 1/2H_o e^{2\pi z/L_o} \qquad (24)$$

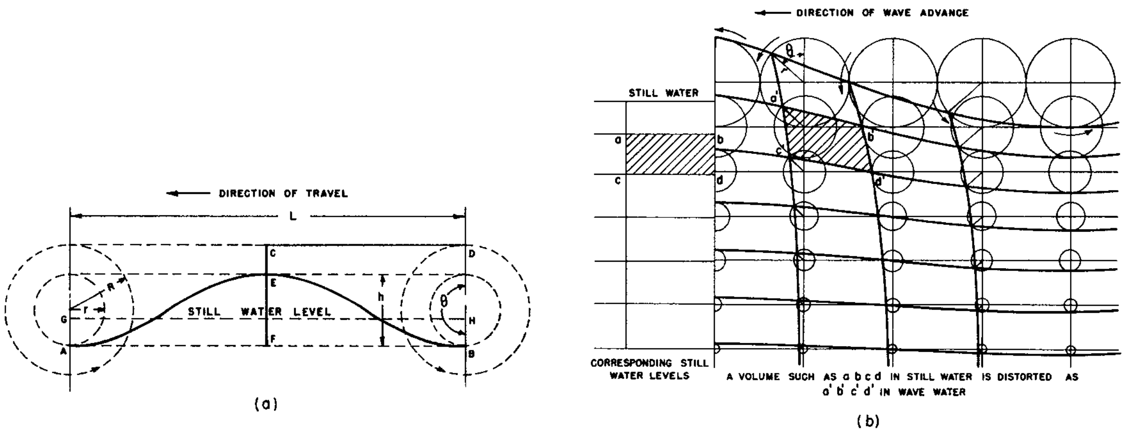


Fig. 12. Trochoidal Wave.

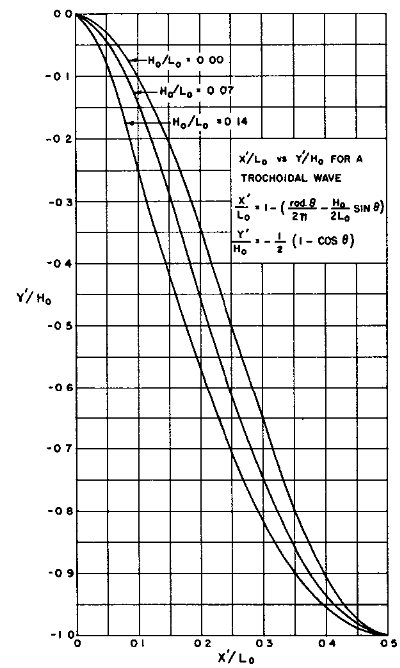
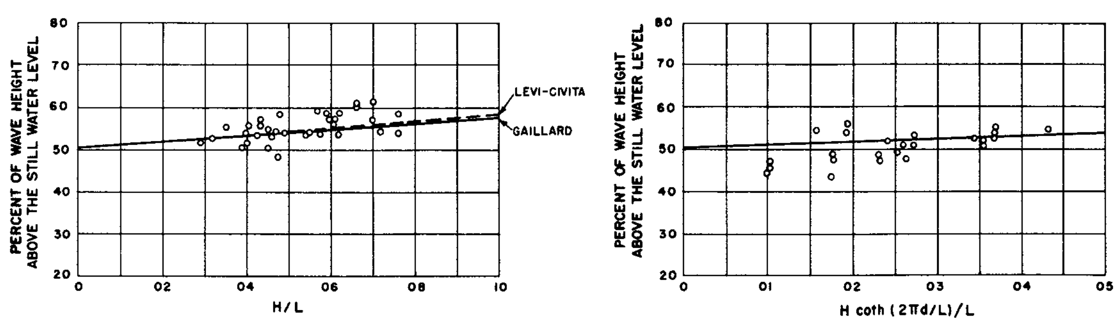


Fig. 13.



(a) DEEP WATER CONDITIONS – BEACH EROSION BOARD

(b) SHALLOW WATER CONDITIONS – UNIVERSITY OF CALIFORNIA

Fig. 14. Percent of wave height above the still water level for water of uniform depth.

The energy of the wave is equally divided between kinetic and potential, with the total energy being,

$$E = wL_oH_o^2[1 - 1/2(\pi H_o/L_o)^2]/8 \tag{25}$$

Trochoidal Theory - Finite Depth. The trochoidal theory as extended to water of finite depth has been presented by Gaillard (1935) and is widely used. There appears to be no published mathematical work which substantiates the conclusions presented by Gaillard (1935). Perhaps the facts that (a) the wave velocity, orbital velocities and wave shapes as represented in the trochoidal theory were the same as those in the theory of Airy (1845) for waves in deep-water, and (b) other equations of the trochoidal theory reduced to those of Airy (1845) for small amplitudes led Gaillard (1935) to examine the similarities between equations from a reduced (elliptical) trochoidal theory and the Airy (1845) theory for waves in finite depth. The equations of wave velocity, and orbital velocities and shapes as obtained from the reduced trochoidal theory are the same as those of Airy (1845) for shallow-water waves and for small amplitudes. Other reduced trochoidal equations are almost identical to those of Airy (1845). However, the reduced tr choid theory does not satisfy either the conditions of continuity or dynamical equilibrium except at the trough and crest (Gaillard, 1935) and hence, this theory, although widely used, is not sound.

Gaillard (1935) states that a shallow-water wave differs from a wave in very deep water in that the particle paths are elliptical rather than circular, with the eccentricity of the ellipses depending upon the ratio of the wave length to the depth of water. For a particular length of wave, the eccentricity increases with decreasing water depth so that, in very shallow water, its particle paths are nearly horizontal lines; while the orbits decrease in size with increasing distance below the undisturbed water level with the vertical axes decreasing at a more rapid rate than the horizontal axes until, at the bottom, the vertical motion is zero and the particle moves in a horizontal line. The angular velocity is not constant, but greatest in the vicinity of the trough and crest. It should be noted that this theory predicts that the velocity at the crest of the orbit is the same as the velocity at the bottom of the orbit. Recent experiments performed in the wave channel at the University of California, Berkeley, show that this is not true. The actual crest velocities are greater than the trough velocities.

The following equations, describing the reduced trochoidal surface, were developed and presented by Gaillard (1935) (Fig. 15),

$$x = R\theta - a'\sin\theta \tag{26a}$$

$$y = b'\cos\theta \tag{26b}$$

The velocity of propagation is,

$$C^2 = gLb_s'/2\pi a_s' = gL(\tanh 2\pi d/L)/2\pi \tag{3}$$

The equations for the semi-axes of the orbits are,

$$b' = 1/2H[\cosh 2\pi(d + z)/L]/\sinh 2\pi d/L \tag{10a}$$

$$a' = 1/2H[\sinh 2\pi(d + z)/L]/\sinh 2\pi d/L \tag{10b}$$

and the ratio of the semi-axes is,

$$\frac{b'}{a'} = \tanh 2\pi(d + z)/L \tag{11}$$

The total energy of the wave, which is one-half kinetic and one-half potential, is

$$E = wLH^2(1 - MH^2/L^2)/8 \tag{27}$$

where M, the energy coefficient, is

$$M = \pi^2/(2 \tanh^2 2\pi d/L) \tag{28}$$

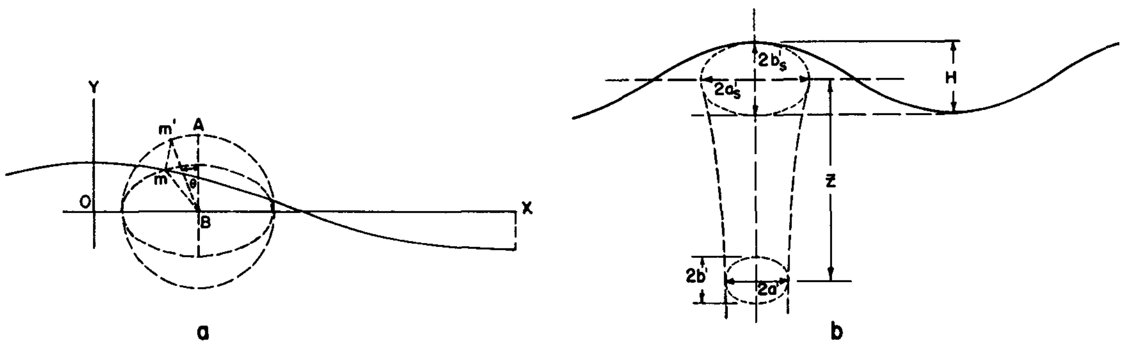


Fig. 15. Shallow water wave, trochoidal theory.

The equations for the shape of the surface profile may be written in a dimensionless form,

$$x/L = [\mathrm{rad}(\arccos 2y/H)/2\pi] - [H \sin(\arccos 2y/H)/2 \tanh 2\pi d/L] \tag{29}$$

This, together with the equation for the displacement of the crest and trough from the undisturbed water level (y_{swl}),

$$y_{swl}/H = 1/2 - (\pi H/4L) \tanh 2\pi d/L \tag{30}$$

allows the plotting, in dimensionless form, of the wave profile, or, as $x/L = t/T$, the variation of surface elevation with time. Experiments (Wiegel, 1950) have shown that actual waves are very closely trochoidal in shape (Fig. 16). It should be pointed out that these profiles (i.e., for these values of d/L) are very nearly the same as given to the third approximation by Stokes (1847). If the equations for the trochoid are expanded into a series, it can be seen that to the third term it is the same as Stokes' equation as well.

<u>Trochoidal Theory - Rotation.</u> Stokes (1847) has shown that the trochoidal theory necessitates rotation and derives the following expression,

$$\text{Vorticity} = 2\omega = \frac{\partial v}{\partial x} - \frac{\partial u}{\partial y}$$

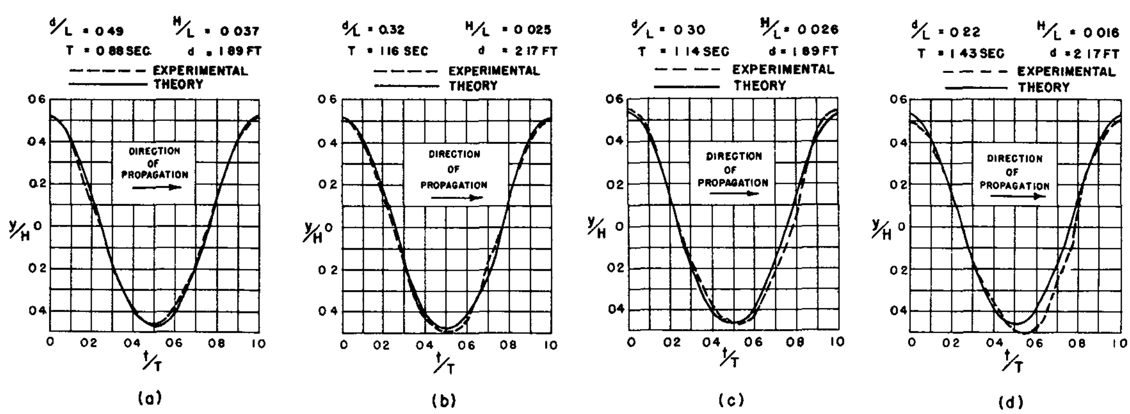


Fig. 16. Comparison of experimental elevation-time curves with trochoidal theory.

$$\omega = \frac{-(2\pi^3 H^2/L^3)Ce^{4\pi z/L}}{1-(\pi H/L)^2 e^{4\pi z/L}}$$

and

$$U' = -(\pi H_o/L_o)^2 \, C_o \, e^{4\pi z/L} \tag{31}$$

where U' is the horizontal velocity remaining after wave motion has been destroyed. According to Stokes (1847), "It appears then that in order that it should be possible to excite these waves in deep-water previously free from wave disturbance, by means of pressures applied to the surface, a preparation must be laid in the shape of a horizontal velocity decreasing from the surface downward according to the value $e^{4\pi z/L}$, .. "

Irrotational Theory. The irrotational theory for waves of finite height in water of uniform depth was developed by Stokes (1847), Rayleigh (1877), Struik (1926), and Levi-Civita (1925). Experimental evidence substantiates the conclusion that this is the theory which most nearly represents actual wave motion.

Stokes (1847) found, to the second approximation, that the velocity of wave propagation is independent of wave height and is the same as the theories of Airy (1845) and Gerstner (1802) (Equation 3). However, to the third approximation,

$$C_H^2 = \frac{gL}{2\pi} \tanh 2\pi d/L \left\{ 1 + \left(\frac{\pi H}{L}\right)^2 \left[\frac{2(\cosh 4\pi d/L)^2 + 2(\cosh 4\pi d/L) + 5}{8(\sinh 2\pi d/L)^4} \right] \right\} \tag{32}$$

which, for deep-water conditions, reduces to:

$$(C_H)_o = \sqrt{\frac{gL_o}{2\pi}\left[1 + \left(\frac{\pi H_o}{L_o}\right)^2\right]} \tag{33}$$

Fig. 17 shows experimental values compared with theoretical values (Morison, 1951). Other experimental work (Beach Erosion Board, 1941) shows approximately the same results. It appears that the experimental error is of the same order of magnitude as the difference between the equations corrected for height and the equations for waves of small amplitude. Because of this, the more simple equation for waves of small amplitude can be used for most engineering calculations.

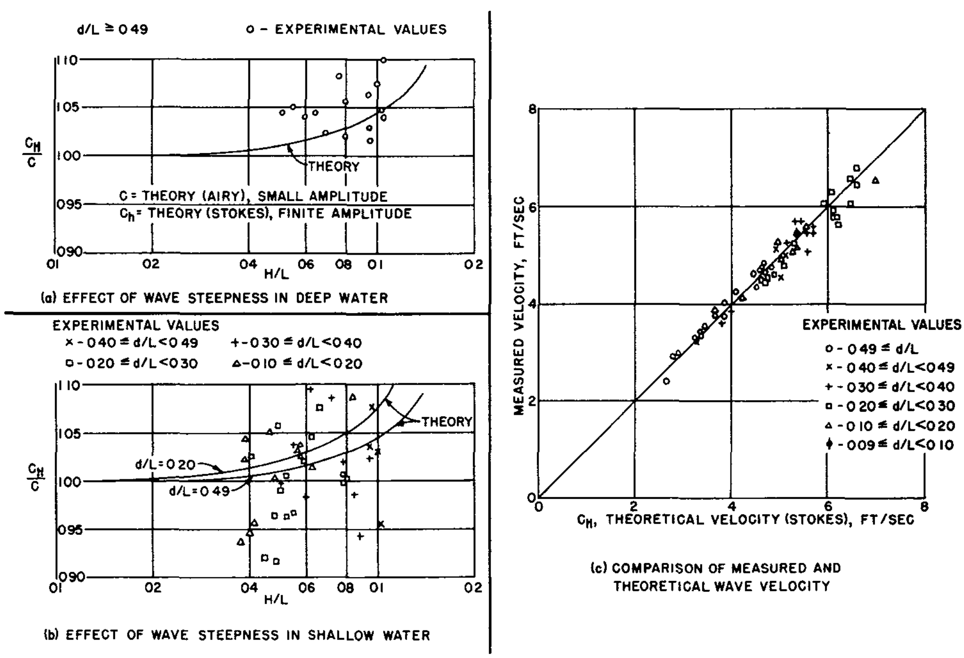


Fig. 17. Velocity of waves of finite height.

The equation for the wave profile, to the third approximation, is:

$$y = a \cos 2\pi x/L + (\pi a^2/L)(\cos 4\pi x/L)\left[\frac{(e^{2\pi d/L}+e^{-2\pi d/L})(e^{4\pi d/L}+e^{-4\pi d/L}+4)}{(e^{2\pi d/L}-e^{-2\pi d/L})^4}\right] +$$

$$(\pi^2 a^3/L^2)(\cos 6\pi x/L)\left[\frac{(e^{12\pi d/L}+e^{-12\pi d/L})+14(e^{8\pi d/L}+e^{-8\pi d/L})+19(e^{4\pi d/L}+e^{-4\pi d/L})+32}{(e^{2\pi d/L}-e^{-2\pi d/L})^6}\right] \tag{34}$$

The equations for the horizontal and vertical components of orbital velocities are (according to verbal communication from R. A. Fuchs, Institute of Engineering Research, University of California, Berkeley).

$$\frac{d\xi}{dt} = -\frac{\pi H}{L}\cdot C\left[\frac{\cosh 2\pi(d+z+\eta)/L}{\sinh 2\pi d/L}\right]\cos[2\pi(x+\xi-Ct)/L] +$$

$$\frac{3}{4}\left(\frac{\pi H}{L}\right)^2 C\left[\frac{\cosh 4\pi(d+z-\eta)/L}{(\sinh 2\pi d/L)^4}\right]\cos[4\pi(x+\xi-Ct)/L] \tag{35a}$$

which, upon expanding, substituting and neglecting terms of third order or higher, becomes,

$$\frac{d\xi}{dt} = \frac{-\pi H}{L}\cdot C\left[\frac{\cosh 2\pi(d+z)/L}{\sinh 2\pi d/L}\right]\cos\,[2\pi(x-Ct)/L] + \left(\frac{\pi H}{L}\right)^2\cdot\frac{C}{(\sinh 2\pi d/L)^2}\cdot$$

$$\left[-\frac{1}{2}+\frac{3}{4}\,\frac{\cosh\frac{4\pi}{L}(d+z)}{(\sinh 2\pi d/L)^2}\right]\cos\,[4\pi(x-Ct)/L] + \left(\frac{\pi H}{L}\right)^2\frac{C}{2}\left[\frac{\cosh 4\pi(d+z)/L}{(\sinh 2\pi d/L)^2}\right] \tag{35b}$$

and,

$$\frac{d\eta}{dt} = \frac{\pi H}{L}\cdot C\left[\frac{\sinh 2\pi(d+z+\eta)/L}{\sinh 2\pi d/L}\right](\sin 2\pi(x+\xi-Ct)/L) -$$

$$\frac{3}{4}\left(\frac{\pi H}{L}\right)^2 C\left[\frac{\sinh 4\pi(d+z+\eta)}{(\sinh 2\pi d/L)^4}\right](\sin 4\pi(x+\xi-Ct)/L) \tag{35c}$$

which, upon expanding, substituting and neglecting terms of third order or higher, becomes,

$$\frac{d\eta}{dt} = \frac{\pi H}{L}\cdot C\frac{\sinh 2\pi(d+z)/L}{\sinh 2\pi d/L}(\sin 2\pi(x-Ct)/L) - \frac{3}{4}\left(\frac{\pi H}{L}\right)^2 C\left[\frac{\sinh 4\pi(d+z)/L}{(\sinh 2\pi d/L)^4}\right](\sin 4\pi(x-Ct)/L) \tag{35d}$$

The equations for particle displacement about their undisturbed positions are,

$$\xi = \frac{H}{2}\,\frac{\cosh 2\pi(d+z)/L}{\sinh 2\pi d/L}\,(\sin 2\pi(x-Ct)/L) -$$

$$\frac{\pi H^2}{4L(\sinh 2\pi d/L)^2}\left[-\frac{1}{2}+\frac{3}{4}\,\frac{\cosh 4\pi(d+z)/L}{(\sinh 2\pi d/L)^2}\right](\sin 4\pi(x-Ct)/L) + \left(\frac{\pi H}{L}\right)^2\cdot\frac{C}{2}\cdot\frac{\cosh 4\pi(d+z)/L}{(\sinh 2\pi d/L)^2}\cdot t \tag{36a}$$

and,

$$\eta = \frac{H}{2}\,\frac{\sinh 2\pi(d+z)/L}{\sinh 2\pi d/L}(\cos 2\pi(x-Ct)/L) - \frac{3}{16}\cdot\frac{\pi H^2}{L}\,\frac{\sinh 4\pi(d+z)/L}{(\sinh 2\pi d/L)^4}\,(\cos 4\pi(x-Ct)/L) \tag{36b}$$

Thus, the particle orbit lies a little above an ellipse at the crest and is a little flatter than an ellipse at the trough while, at the same time, the particle is moving forward (i.e., mass transport). This is shown in Fig. 18.

These equations show the most interesting result of the theory of Stokes (1847). That is, by not neglecting the effect of height (the velocity of a particle depends not only upon its mean position, but also upon its displacement from its mean position) it is shown that the particle velocity is greater in its forward

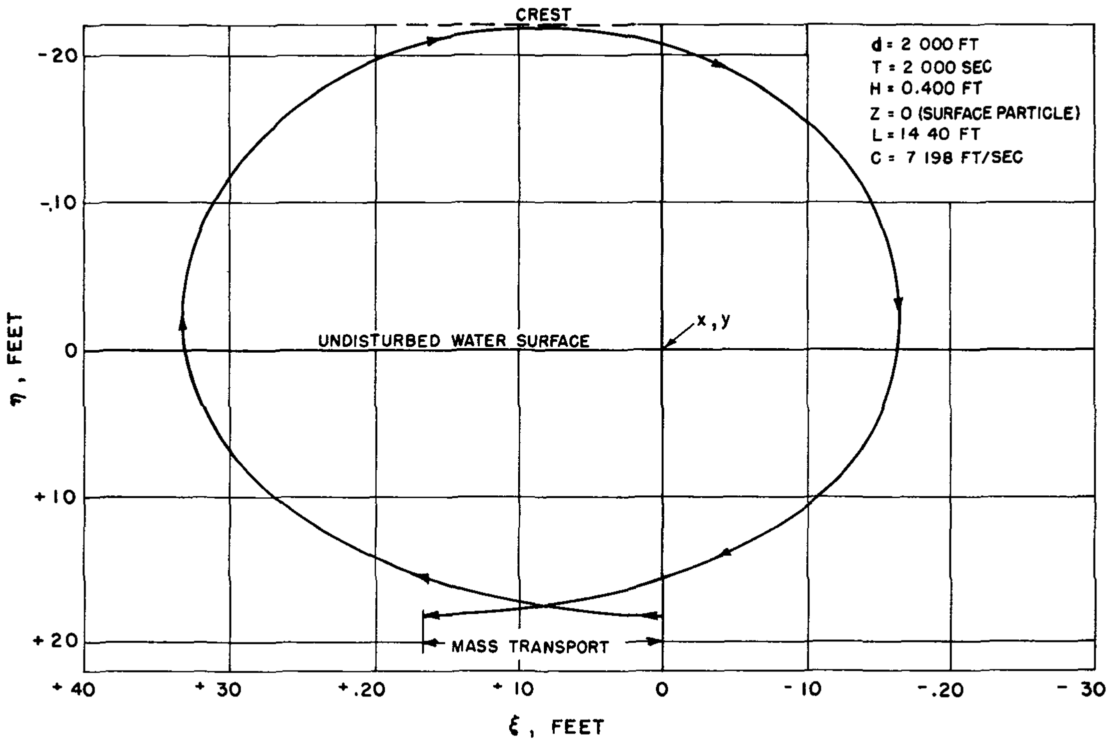


Fig. 18
Theoretical orbit of surface particle - Stokes' irrotational theory, second order.

movement (with the crest) than in its backward movement (with the trough). Laboratory experiments performed at the University of California, Berkeley, confirm this conclusion. This results in the fact that the forward motions of the particles are not altogether compensated by their backward motions. Hence, in addition to their orbital motion, there is a progressive motion in the direction of propagation of the waves. The orbits are open, not closed (Figs. 6 and 18). This motion has become known as "mass transport" and is given to the second approximation by

$$\bar{U} = 1/2(\pi H/L)^2 \; C\left[\frac{\cosh 4\pi(d+z)/L}{\sinh^2 2\pi d/L}\right] \tag{37a}$$

For deep-water, this becomes,

$$\bar{U}_o = (\pi H_o/L_o)^2 \; C_o e^{4\pi z/L} \tag{37b}$$

which is identical with the equation expressing the horizontal velocity remaining (due to rotation) after wave motion has been destroyed in the rotational trochoidal theory (Equation 31). In other words, in order for a wave of finite height to exist, it is necessary for this additional velocity to exist. In the trochoidal theory, it is in the form of molecular rotation (which is not substantiated by observations) of particles moving in a closed orbit, while, in the irrotational theory, it results from particles moving in an open orbit (which is substantiated by observations (Beach Erosion Board, 1941; Mitchim, 1940; Morison, 1948).

Maximum Theoretical Wave Steepness. Stokes (1847) came to the conclusion that for any wave whose crest angle was greater than 120°, the series would cease to be convergent and hence the wave form would become discontinuous. However, the possibility of a wave existing with a crest angle equal to 120° was not shown until later. Michell (1893) found the theoretical limit was H/L = 0.14 and Havelock (1918) found it to be 0.1418.

PROPAGATION OF A FINITE WAVE TRAIN THROUGH AN UNDISTURBED MEDIA

In nature, an infinitely long series of waves does not exist; rather a train consisting of a finite number of waves, which are formed by winds in a storm area, travels on the ocean surface. These "wave groups" travel at a different velocity than that of the individual waves. Rather simple examples of wave groups are waves generated at the bow of a ship and the waves generated in a wave tank by operating the wave generator for only a few strokes (Beach Erosion Board, 1942). In these cases, it can be seen that the lead wave in the group decreases in height as it progresses, the potential energy being transformed into kinetic energy as the wave form induces corresponding velocities in the previously undisturbed water. The wave finally disappears while, at the same time, a new wave begins to appear at the rear of the group as the velocity pattern left behind is such that the flow converges towards one section and diverges from another section, forming the crest and trough.

The velocity with which the wave group travels (Lamb, 1932) is given by,

$$C_g = 1/2C[1+(4\pi d/L)/\sinh 4\pi d/L] \tag{38a}$$

for waves of very small amplitude in any depth of water. The group velocity, as related to deep-water velocity, has been presented in Fig. 6, Chapter 3. For deep-water, Equation 38a becomes,

$$(C_g)_o = 1/2C_o \tag{38b}$$

Reynolds (1877), for waves in infinite depth of water, and Rayleigh (1877), for waves in finite depth, developed equations for the transmission of energy by a wave group. In recent literature, the equations have been interpreted to mean that either (a) all the energy advances with group velocity or (b) half the energy advances with the wave-front velocity. However, as Rayleigh (1877) pointed out, for deep-water conditions:

> "It appears that the energy propagated across any point, when a train of waves is passing, is only one-half of the energy necessary to supply the waves which pass in the same time, so that if the train of waves be limited, it is impossible that its front can be propagated with the full velocity of the waves .... because this would imply the acquisition of more energy than can in fact be supplied."

Reynolds (1877) states:

> "So that after the waves have advanced through two wave-lengths the distribution of the energy will have advanced one, or the speed of the groups is one-half that of the waves."

From the mathematical arguments of these two investigators, it appears that the energy travels at the group velocity.

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CHAPTER 3

THE TRANSFORMATION OF WAVES IN SHALLOW WATER

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The purpose of this paper is to summarize existing knowledge of the processes involved in the movement of progressive oscillatory waves through shallow water and the fundamental principles controlling these processes. Variations in wave characteristics and their physical significance will be discussed as well as agreement between theory and observation. The application of available knowledge to engineering problems is treated in the following chapters.

ENERGY CONSIDERATIONS

Development of the theory of generation of progressive oscillatory waves by wind action and the basic theory of such wave motion has made it possible to formulate a reasonable concept of the energy budget of a wave at any stage of its life. From these sources it can be shown that the total energy available in a wave at the instant it enters shallow water of depth approximately equal to one-half the wave length is

$$E = \frac{wL_oH_o^2}{8} \left(1 - 4.93 \frac{H_o^2}{L_o^2}\right) \tag{1}$$

where w is the unit weight of water, L_o is the wave length, and H_o is the wave height.

It can be shown further that, of this amount of energy, one-half is transmitted forward with the wave form into regions of calm. Study by Sverdrup and Munk (1947), and others (Beach Erosion Board, 1942; Rossby, 1947), has lead to the conclusion that the potential energy of the wave, representing one-half the total energy, is transmitted forward with the wave velocity, leading to the following expression for the power transmitted per unit of crest width in deep water,

$$P = \frac{E}{2T} = \frac{wH_o^2}{16} \left(1 - 4.93 \frac{H_o^2}{L_o^2}\right) \sqrt{\frac{gL_o}{2\pi}} \tag{2}$$

where T is the wave period, and g is the acceleration due to gravity.

As the wave moves into shallow water the orbits of the water particles become ellipses instead of circles, with the eccentricity of the ellipses depending upon the ratio of the wave length to the depth of water. Under these conditions basic theory leads to the expression for total energy per wave length in shallow water

$$E_s = \frac{wLH^2}{8} \left\{1 - 19.74 \left(\frac{a_s'}{L}\right)^2\right\} \tag{3}$$

The ratio of the semi-major axis of the water particle orbit to the wave length, a_s'/L, is a function of both H/L and d/L, and consequently equation 3 can be given the form

$$E_s = \frac{wLH^2}{8} \left(1 - M \frac{H^2}{L^2}\right) \tag{3a}$$

where M is a function of d/L, that has a limiting value when d/L = 0.5 of 4.93, thus agreeing with equation 1 (see equation 28, Chapter 2).

The power transmitted per unit of crest width then is,

$$P = \frac{E}{2T} = \frac{wH^2}{16}\left(1 - M\frac{H^2}{L^2}\right)\left(\sqrt{\frac{gL}{2\pi}\tanh\frac{2\pi d}{L}}\right) \tag{4}$$

These equations are based on the assumption that waves are trochoidal and of small height.

Another approach to the energy problem, found in Lamb (1932), results in the finding that the total energy of a progressive wave system of amplitude $H_o/2$ is equal to the work required to raise a stratum of water $H_o/2$ in thickness through a height equal to $H_o/4$. In this case the total energy available per unit surface area is:

$$E' = \frac{1}{8} wH_o^2 \tag{5}$$

where H_o is the total wave height. The power corresponding per unit of crest width is found to be:

$$P = 1/8 \; wH_o^2\sqrt{\frac{gL_o}{2\pi}} \tag{6}$$

This expression is valid for deep-water waves of small steepness, and may, with sufficient accuracy, be applied to deep-water waves of great steepness. Rayleigh (1877) has shown that the expression for power in any depth of water, per unit crest width is:

$$P = n \cdot E \cdot C \tag{7}$$

where C is the wave velocity. In this equation "n" is the ratio between group velocity and wave velocity and has the value,

$$n = 1/2\left[1 + \frac{4\pi d/L}{\sinh\frac{4\pi d}{L}}\right] \tag{7a}$$

The value of n approaches 1/2 in deep water, and unity in shallow water. Thus, the mean power per unit wave crest width, in any depth of water is

$$P = \frac{n}{8} wH^2 \sqrt{\frac{gL}{2\pi}} \tag{8}$$

It must be noted that these expressions define a conservative wave system in which no energy is added to or subtracted from the wave energy; in other words, the use of these expressions in the study of shallow water wave motion involves the assumption that no energy is lost or added in divergence or convergence of the wave, no energy is lost due to bottom friction or viscous effects, and no energy is added or lost by wind action during the passage of the wave through shallow water. However, experiment and observation show that the expressions are sufficiently accurate for most purposes of an engineering nature. In the discussions to follow it will be assumed that the above limitations are admissible unless otherwise stated.

VARIABILITY OF WAVE CHARACTERISTICS

Experience in the study of waves in shallow water has shown the wave characteristics of most importance to wave action problems to be: the wave height, H; the wave length, L; the wave period, T; the wave steepness, H/L; the wave velocity, C; and the wave energy, E, or wave power, P. The variability in shallow water of each of these characteristics will be discussed separately. However, since we are concerned primarily in this paper with the physical significance of the variability, some prior description of observed behavior of waves in shallow water may be desirable. The description will be limited to swell, ignoring local wind waves, since present theory was developed largely on the study of swell and the case may be presented somewhat more simply.

The classic description of wave behavior in shallow water states that as a depth equal to L/2 is reached the wave "feels" the bottom and is retarded. To an observer probably the first noticeable effect is a bending or refraction of the wave, closely followed by peaking of the wave crest. On a shoal shore partial breaking of the wave occurs at this time, the wave then reforming and partially breaking one or several times as it moves inshore, finally breaking in a single plunge or spilling over itself for an appreciable time. On a steeply sloping shore partial breaking seldom occurs and the wave breaks usually in a single plunge. In either case as the wave reaches relatively shoal water the crest appears to be an intumescence or mound, quite accented, while the trough appears attenuated and relatively flat. On the Pacific Coast, characterized by conditions of deep water close to shore, the waves appear to be long-crested, crest lengths of a mile or more being not unusual, whereas on the shoal Atlantic Coast crest lengths of several hundred feet are more the rule.

The wave changes form radically when it breaks, and apparently regardless of its prior characteristics, assumes the appearance of a miniature tidal bore, or a traveling hydraulic jump, as it rushes foaming up the beach to its limit of uprush. The recession or back-wash of the wave from its upper limit of travel on the beach resembles simple sheet flow without any characteristic wave appearance.

The theory of wave behavior in shallow water is valid only for wave travel from deep water to the point of breaking. Little is known of the transformations involved in breaking waves and much study of the feature is required.

From the physical point of view the entire transformation of a wave in shallow water must be due in the first instance to modification of the deep water flow pattern. This may occur, so far as is known, only by the imposition of boundary conditions at the bottom and the free surface. Since the free surface is not changed in moving from deep to shallow water, all the transformation must be due to bottom effects. Observation in wave tanks leads to the conclusion that the principal effect of the bottom is to reduce the vertical movements of the water particles in wave action, and since it is assumed that the wave energy is conserved, it follows that the orbits of the water particles must be flattened from circles to ellipses, as observation verifies does occur. Munk (1948) has prepared a diagram illustrative of this process (Fig. 1). The theoretical values are shown on Figs. 2 and 3 which have been extracted from a publication of the Beach Erosion Board (1942). It will be noted that the water particle motion is no longer symmetrical as it was in deep water, since the horizontal movement and therefore the velocities exceed the vertical velocities. In relatively shallow water the vertical movement at the bottom ceases altogether and the orbital motion there is simply a back and forth motion.

The influence of the modification of the particle flow pattern on wave velocity is denoted by the term $\tanh 2\pi d/L$ in the wave velocity expression

$$C^2 = \frac{gL}{2\pi} \tanh \frac{2\pi d}{L} \tag{9}$$

This expression is subject theoretically to two limitations of possible practical importance. First, the equation is derived under the assumption of infinitely small wave steepness. Present rigorous theoretical determination indicates a small correction for wave steepness in the direction that steeper waves travel at higher velocities. Second, the equation is derived under the assumption that the depth is constant, yet it is applied to the case of sloping bottoms.

Many experiments have been made in wave tanks and some observations made in nature to determine the agreement between theory and observation. The results are summarized on Fig. 4, showing the effects of initial steepness based on observations at Scripps Pier (Scripps Institution of Oceanography, 1944), and Fig. 5, showing effects of steepness and bottom slope as determined from laboratory studies (Wiegel, 1950). The variability of wave velocity with depth, expressed as the

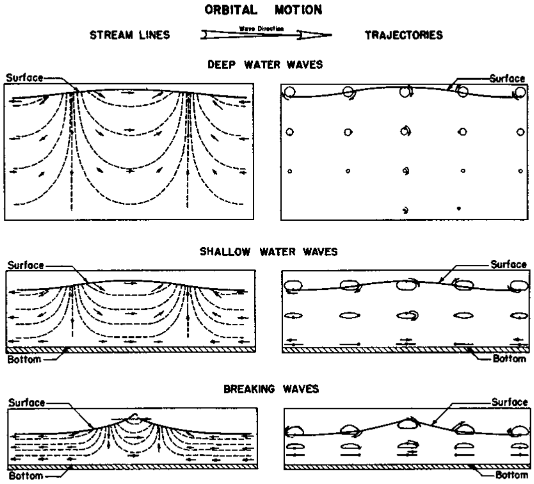

Fig. 1
Orbital motion in progressive surface waves

The dashed lines in the figures to the left are the streamlines, i.e., lines everywhere parallel to the flow. The direction and length of the arrows indicate direction and velocity of orbital motion. The figures to the right show the particle trajectories, i.e., the paths described by fixed particles of water.

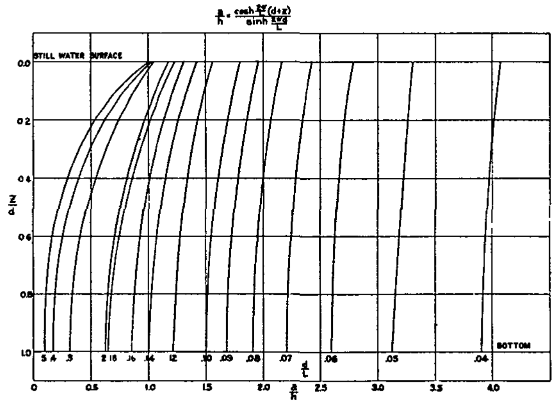

Fig. 2. Horizontal amplitude of oscillation for proportional depth related to fraction of wave height.

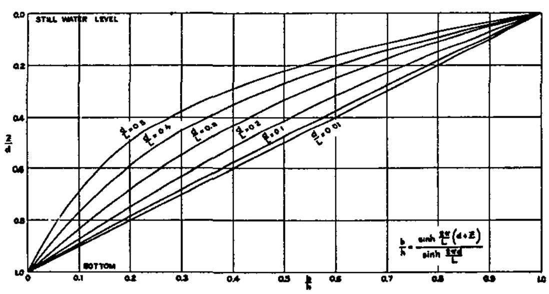

Fig. 3. Vertical amplitude of oscillation for proportional depth related to fraction of wave height.

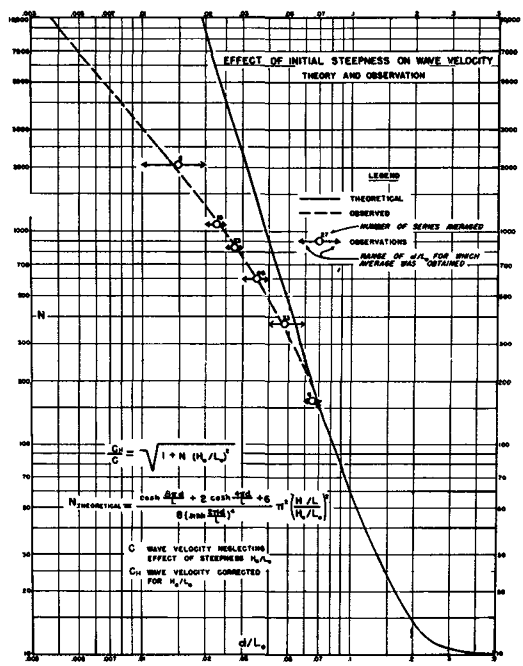

Fig. 4.

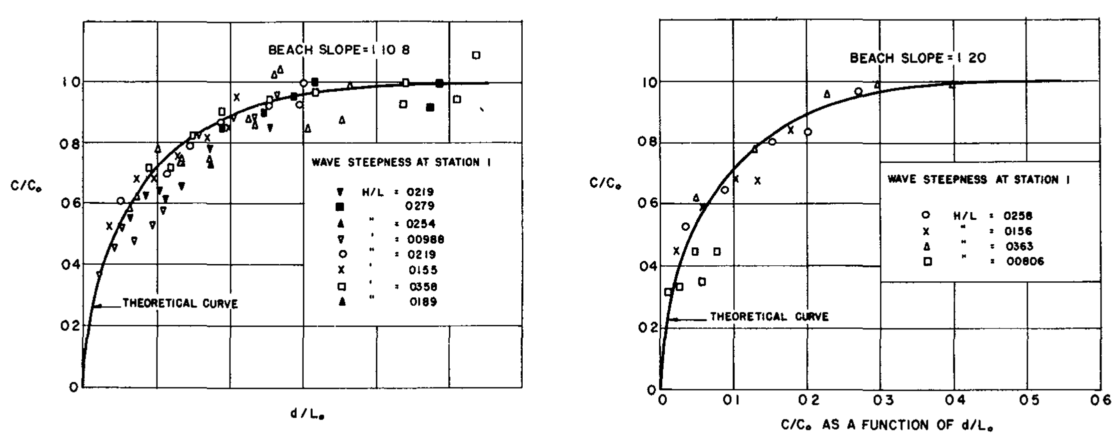

Fig. 5. Variability of wave velocity with water depth.

ratio of the velocity in shallow water to the velocity in deep water is shown as curve C/C_o in Fig. 6 (Hydrographic Office, 1944).

It will be remembered that all wave action satisfies the basic requirement that the wave length is equal to the product of the wave velocity and the wave period. Observation confirms the assumption made in the development of theory that the wave period is constant for the wave travel distances and times concerned in wave travel through shallow water. In these circumstances it is apparent that,

$$T = L_o/C_o = L/C$$

and $$L/L_o = C/C_o \tag{10}$$

It follows therefore that the wave length varies in shallow water to the same extent as the wave velocity. Since the wave length and velocity both decrease with decrease in depth, conservation of energy of the wave requires that the wave height increase It is assumed as a basic premise that the power transmitted per unit width of wave crest remains constant at all points along the path of travel of the wave from deep water to the point of breaking. Physically this means that the wave energy entering a given zone equals the energy leaving the zone; therefore, no energy is either accumulated or destroyed Referring to prior discussion of energy considerations it will be recalled that in deep water one-half the total energy, the potential energy, advances with the wave form at the wave velocity; whereas, in shallow water a larger portion of the energy advances, the increase being derived from the wave kinetic energy. The fractional addition is measured by the term n in Equation 7. The physical significance of this behavior lies in the fact that, whereas the potential energy is periodic and advances in phase with the wave form, the kinetic energy in a deep-water wave is evenly distributed, non-periodic, and thus independent of the position or velocity of the wave. However, in shallow water the particle orbits are deformed and the kinetic energy is no longer evenly distributed; in the ultimate it becomes periodic and, like the potential energy, advances in phase with the surface deformation.

If we denote deep-water conditions by the subscript o, we may write

$$P_o = P = n_o E_o C_o = nEC$$

and, by substitution of appropriate values it can be shown that

$$H/H_o = \sqrt{\frac{1}{2n} C_o/C} \tag{11}$$

The variability in height of a wave traveling through shoal water without refraction can be computed from this relation and is shown as curve H/H'_o on Fig. 6. It will be noted that the wave height decreases to about 91% of its deep water value, then increases rapidly as the wave moves into lesser and lesser depths. The reason for this behavior lies in the relative values of n and C_o/C, n increases with decreasing depth thus reducing wave height, while C_o/C increases with decreasing depth and increases wave height. As the depth becomes less than about 0.06 times the wave length, the increase in height due to velocity variation overbalances the decrease due to the variation in n and the wave height increases beyond its deep-water value (Suquet, 1949).

Theoretical studies at the University of California (Putnam, 1949; Putnam and Johnson, 1949) indicate that the assumption of constant power transmission may be inadmissible for shallow areas of flat slope. Fig. 7 shows the results of some of these studies, and indicates that reductions of as much as 30% in wave heights predicted on the basis of frictionless theory may occur.

In the usual case waves will not travel through shoal water without refraction, and correction of wave height must be made to take account of refraction effects (Groen and Weenik, 1950). These effects will be discussed in detail in Chapter 4 and, therefore, will not be treated here. For the purpose of completeness Fig. 8 is presented to indicate the magnitude of the corrections required (Hydrographic Office, 1944). In this figure the angle α_o is the angle between the wave crest and the shore line when the wave is in deep water, and K_d is the cor-

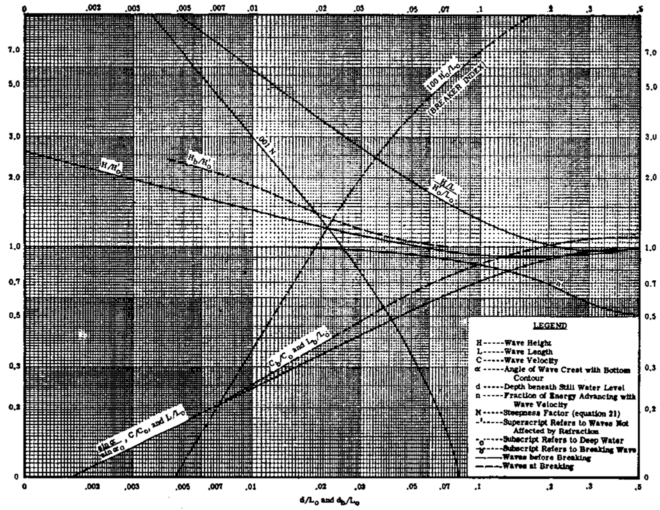

Fig. 6. Waves in shallow water. Change in height and length from deep water to point of breaking.

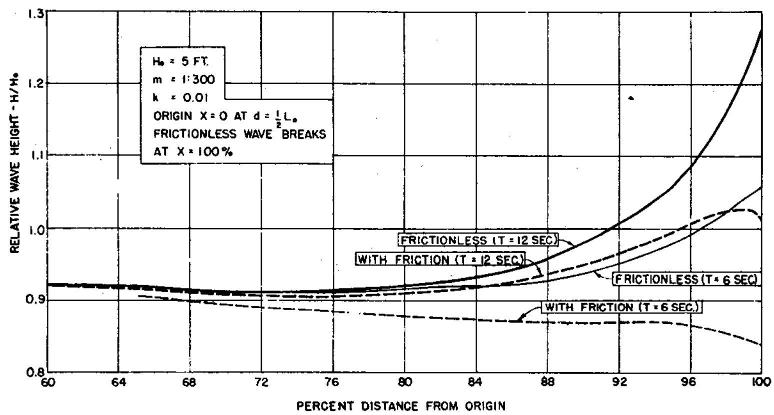

Fig. 7. Effect of bottom friction on wave height.

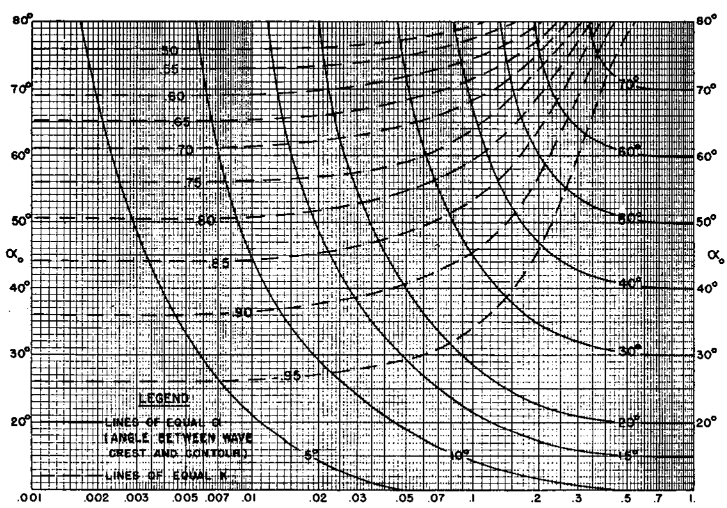

Fig. 8. Change in wave direction and height due to refraction on beaches with straight, parallel depth contours.

rection factor to be applied to values of H obtained from Fig. 6. Since the wave height and length vary differently with wave travel through shallow water, then the wave steepness must vary. Denoting steepness by $H/L = \delta$ we can from previous relations set

$$\delta/\delta_o = H/L \big/ H_o/L_o = H/H_o \cdot C_o/C \qquad (12)$$

This relation is shown on Fig. 6 as curve $(H/L)/(H_o/L_o)$. It will be noted that the steepness first decreases slightly, then increases very rapidly. The sudden, sharp increase in steepness is one of the most easily noticed features of wave travel in shallow water and is striking to even a casual observer noticing long swell on a beach. Steepness increases until the wave breaks. Studies by Michell (1893), Stokes (1847), Havelock (1918), and others on the problem of the greatest height attainable by an oscillatory wave of permanent form lead to the conclusion that the minimum included angle at the crest is 120°, and that the maximum height has a value of 0.1418 of the wave length. No experimental confirmation of this theory is yet available, however, large numbers of wave observations, made under a wide variety of conditions, have not included any reliable data showing heights exceeding the L/7 limit predicted by theory.

An additional feature of wave motion in shallow water that is usually of secondary importance is the variability in the percentage of wave height above and below still water. Experimental results obtained by Wiegel (1950) are shown on Fig. 9. These data show reasonably good agreement with theory and it may be considered that confirmation has been established insofar as most applications are concerned.

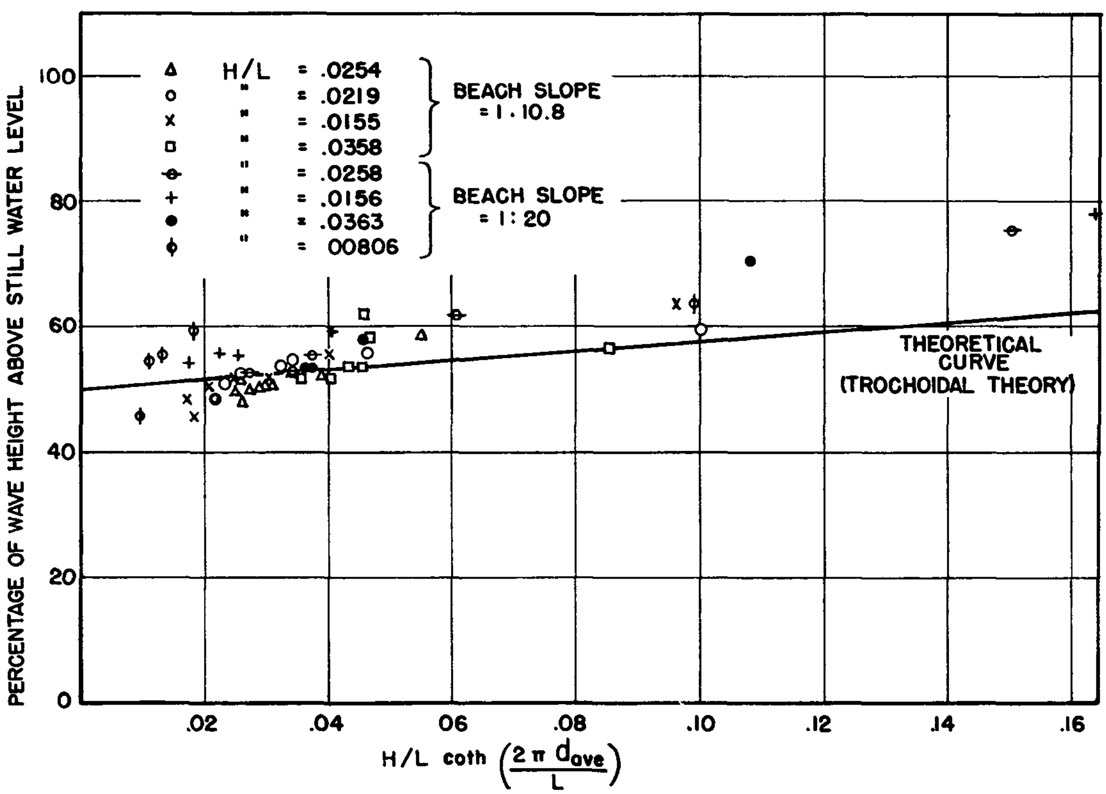

Fig. 9. Comparison between experimental and theoretical percentages of wave height above still water level.

BREAKERS AND SURF

At the outset it might be stated that there is no satisfactory theory dealing with breakers, i.e., individual waves that are breaking, or with surf, which is the collection of individually breaking waves. Theory indicates and observation confirms that a wave will break, i.e., lose its form, when the crest angle approximates 120°, or the steepness reaches about 1/7. However, waves break, in the sense of losing their form, under conditions other than these critical conditions. It is well known, for example, that waves traveling over a flat slope will partially break at the crest, reform as apparently stable waves, then break again, and so on, finally breaking completely. It is equally well known that wind frequently causes waves to break partially, or perhaps simply blows off the crests of the waves.

The preceding discussion of the variability of wave characteristics shows that as a wave approaches very shallow water the crests tend to "hump" and become separated by long, flat troughs. In appearance this pattern resembles a series of solitary waves and this resemblance suggested the possibility of application of solitary wave theory to the problem of breaking of oscillatory waves.

Present theoretical knowledge of breaking oscillatory waves is based on solitary wave theory; however, a somewhat different approach was suggested recently by Stoker (1949), based on the theory of compressible flow of a gas and the analogy between a discontinuous shock wave in a compressible gas to a breaker in water.

In theory, a solitary wave consists of a single crest lying in its entirety above the still-water level, and extending infinitely to still water before and behind the crest (Keller, 1949). Application of the solitary wave theory to breakers and surf represents, therefore, a departure from the phenomenon described by theory; however, since the major portion of the energy of a solitary wave is confined to a narrow zone about the crest, the application may be admissible. Further, the character of the motion of the water particles is quite different.

No attempt will be made here to develop the solitary wave theory as applied to the breaker and surf problem, this having been done by Munk (1949a) and Keller (1949). Only the developed theory will be discussed, the reader being referred to the published works for details.

The extreme crest angle at breaking appears to be one of several elements entering the breaker problem. Solitary wave theory as well as oscillatory wave theory are in agreement that a wave must break when the extreme crest angle is 120°. For the solitary wave it can be shown that the corresponding ratio of wave height to depth has the critical value of 0.7813. This means that when the wave height reaches a value of 0.7813 times the depth the wave will break. The frequently useful inverse ratio is 1.28; i.e., waves will break in water of depths equal to 1.28 times the wave height.

A second criterion for breaking is that the velocity of the water particles at the very crest is equal to the wave velocity. This is the apriori condition assumed to exist when developing the theory with regard to extreme crest angle. It should be regarded as a distinct criterion, however, since, for example, the action of wind on the wave crest may cause equality of particle and wave velocity regardless of crest angle, or there may be rapid increases in wave steepness caused by sudden changes in depth.

It will be obvious that the theoretical treatment is based on the concept that breaking of a wave is a kinematic problem, involving only the relation between particle velocities and wave velocities. The simple criterion for breaking is that the particle velocity at the crest must exceed the wave velocity. However, it is possible to show mathematically that another criterion exists; namely, both the equations of continuity and of equal pressure must be satisfied if oscillatory motion of the water particles is to exist. This criterion controls the breaking of waves in areas of appreciable depth variation, and probably is the criterion controlling intermittent breaking and re-forming of the wave.

Munk (1949a) has established several relationships useful in a practical way. From energy considerations he shows that the ratio of wave height at breaking to deep-water wave height is

$$H_b/H_o = \frac{1}{3.3\sqrt[3]{H_o/L_o}} \qquad (13)$$

The agreement of theory and observation is shown by Fig. 10.

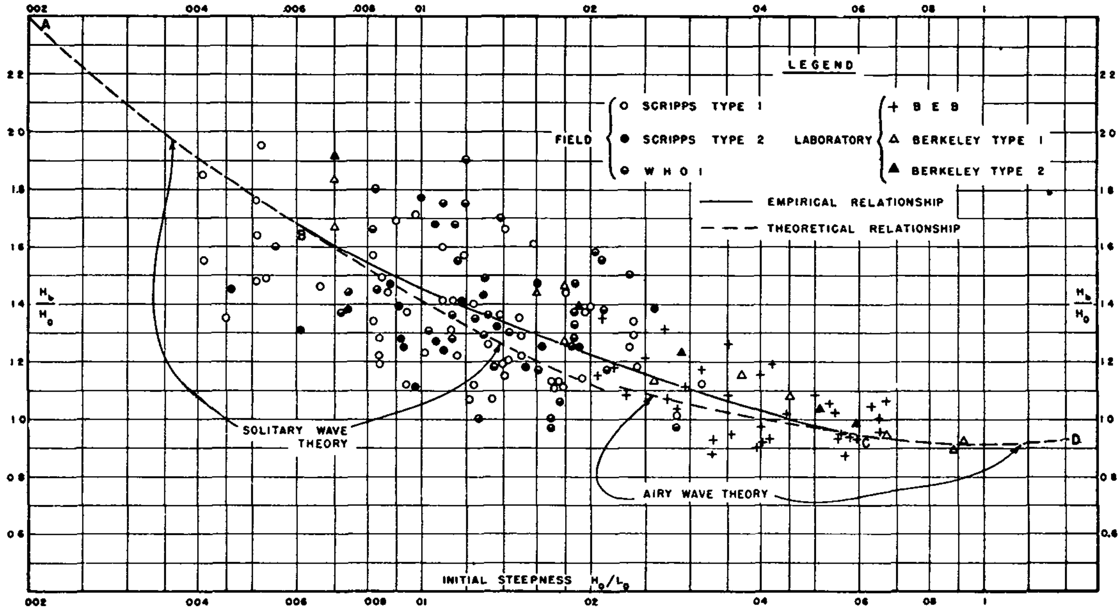

Fig. 10. Comparison between predicted and observed breaker heights.

The relationship between water depth at the breaking point and the wave height at breaking has been mentioned earlier, the relationship is

$$d_b = 1.28\ H_b \tag{14}$$

Observations show a wide scatter of values of the ratio d_b/H_b, probably due in large part to variation of actual conditions from the conditions assumed in development of the theory. The observations are considered sufficiently reliable to lead to the conclusion that although theory shows fair agreement with observation on the average, the theory is deficient in not being able to account for the wide scatter of observation.

The form of the wave at breaking is not predicted acceptably by the solitary wave theory.

The elevation of the mean position of the water line at the point of breaking above mean sea level in deep water is given by

$$y \approx \frac{13.7}{g} \left(\frac{H_b}{T}\right)^2 \tag{15}$$

Observations in wave tanks are in good agreement with theory.

Fig. 6 shows curves indicating the relationships discussed herein for various values of the applicable water depth-wave length ratio. The curve marked "Breaker Index" is an empirical curve showing the relative depth at which a wave of a given steepness will break (Hydrographic Office, 1944).

EFFECT OF CURRENTS

The effect of currents on wave motion in shallow water has been studied by Arthur (1950) in connection with the determination of the refraction of shallow water waves moving in any given distribution of current and depth. The method evolved is applicable for the case of shallow water waves whose velocity is given to a satisfactory degree of approximation by the expression

$$C^2 = gd$$

The solution is based on Fermat's Principle in optical refraction by which the rays representing least time of travel are obtained, and is analogous to the problem of minimal flight path in aerial navigation.

If θ be the azimuth between the relative wave velocity and an arbitrary direction, V_c is the component in the direction of wave velocity of the horizontal current, C is the wave velocity, and n is the tangent to the wave crest, then the solution is given by:

$$\partial\theta/\partial t = - \frac{\partial(V_c + C)}{\partial n} \tag{16}$$

The physical significance of this expression is that the rate of change of direction of wave travel is numerically equal to the variation of shear along the crest in a scalar field of the relative wave speed and the current component in the direction of wave speed. The negative sign shows that the direction of wave advance turns toward the direction of negative shear along the crest.

The application of this solution to various problems involves considerable cumbersome computation, however, graphical solution probably is possible. Experimental confirmation of the theory is not yet available.

REFLECTION AND SURF BEAT

The transformations undergone by waves running through shallow water discussed previously have not included consideration of possible reflection phenomena caused by the shoaling bottom (Cochrane and Arthur, 1948). There is in fact little in the physical appearance of such waves to suggest that reflection occurs until the bottom slope becomes relatively steep. Reflected waves have been observed frequently from slopes that are steep, for example, breakwater side slopes of 1 on 2, and from vertical faces where the clapotis phenomenon is striking. The author is not aware that the problem has received much attention from the theoretical point of view, however, some empirical work has been done at the Beach Erosion Board (Caldwell, 1949).

Tests made by Caldwell (1949) of the reflection of solitary waves from impermeable slopes, varying in inclination from 6° to 90°, i.e., from about 1 on 10 to vertical show that in this range the reflected energy varies widely from a minimum of about 8% of the incident energy for the 1 on 10 slope to 100% for the vertical face. Observations by Munk (1949b) at the Scripps Pier shoreface, having an average slope of about 1 on 50 from the shore to 40 foot depth, lead him to conclude that the shore acts as a radiating line source returning approximately one per cent of the incoming wave energy in the form of long period waves. Munk (1949b) attributes the returning energy to the variability of water transport into the surf zone, and terms the phenomenon "surf beat."

Gridel (1946) in discussing the application of the methods of physical optics to the study of oscillatory wave motion in shallow water, concludes, in agreement with Miche (1944), that whenever wave refraction occurs, then wave reflection must occur also, thus leading to the conclusion that the energy content of a wave traveling through shallow water is not constant, but continually decreasing.

It is not possible at present to state which of these various situations represents the truth; it can be stated that, in common with most of the problems of wave transformation in shallow water discussed previously, the need and opportunity for much additional theoretical work is great.

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CHAPTER 4

REFRACTION AND DIFFRACTION DIAGRAMS

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SCOPE

The use of diagrams to indicate the effects of refraction and diffraction of ordinary wind waves and swell in offshore areas is by no means an innovation in coastal and harbor engineering. Refraction diagrams in particular have been used in various forms by engineers in the United States and in Europe for more than a decade. The principles and procedures for constructing refraction and diffraction diagrams have been developed by academic research and investigation. The purposes of this paper are (1) to review briefly these principles and procedures, and (2) to describe their practical application by the Corps of Engineers, Los Angeles District, Department of the Army.

REFRACTION DIAGRAMS

PRINCIPLES AND PROCEDURES OF DIAGRAM CONSTRUCTION

Applicable formulas. The basic principles underlying the construction of all types of refraction diagrams are expressed by Snell's law and by the formula for wave velocity in shallow water. Snell's law states that where the bottom contours are parallel the sine of the angle between the wave crest and the bottom contour is proportional to the velocity of wave propagation. This law generally is expressed by the formula:

$$\frac{\sin \alpha_1}{\sin \alpha_2} = \frac{C_1}{C_2} \tag{1}$$

where α_1, α_2 = angles between the wave crest and the bottom contour at two successive points along the orthogonal to the wave crests,

and C_1, C_2 = the corresponding velocities of wave propagation at the points where α_1 and α_2 are measured.

The formula for wave velocity is:

$$C^2 = \frac{g}{2\pi} L \tanh 2\pi \frac{d}{L}, \tag{2}$$

where d = depth,

C = wave velocity at depth, d,

L = wave length at depth, d,

g = acceleration of gravity.

Equation 2 contains three variables; d, C, and L. Because L = CT and T is a constant, equation 2 may be expressed in terms of the two variables, d, and C. The solution of the resulting equation is complex and time consuming. However, the University of California has reduced the equation to tabular form. The tables have been published by the Beach Erosion Board (Wiegel, 1948).

Currents of tributary streams. Because ocean currents and currents of tributary streams have a negligible effect on the velocity of wave propagation in the southern California area, their effect on refraction-diagram construction is not considered in detail in this paper. Where a refraction diagram is drawn across a line of current discontinuity (i.e., at the ocean mouth of a large river), the diagram must conform to applicable procedures. These procedures are described elsewhere (Johnson, 1947).

Wave-crest method. The first method of diagram construction is the wave-crest method. Successive positions of the wave crest are drawn by plotting the wave advance from point to point along the crest. The distance of wave advance is determined by the average depth and by the average velocity of propagation for the selected point on the wave crest over a given interval of advance, which is usually expressed as a multiple of the wave length. Graduated acetate scales, which show the actual forward displacement distance for any given depth, are used for this purpose. This type of scale is illustrated in available publications (Hydrographic Office, 1944; Johnson, O'Brien, and Isaacs, 1948). The scales shown were prepared for the dimensionless unit $\frac{d}{L_o}$. The value of $\frac{d}{L_o}$ must be determined separately for each plotted point because the value is based on the map scale and on the wave length in deep water. In the practical application of this procedure, the work may be greatly simplified by preparing a graduated acetate scale for each map scale used and for each wave period diagrammed. On each of these acetate scales the intervals of wave advance are given in some multiple of the wave length and are expressed in the same unit (i.e., feet, fathoms, or meters) as that of the bottom contours. After the full series of wave-crest positions from deep water to shore are plotted, orthogonals to these lines are drawn at any desired interval to determine wave convergence or divergence.

Crestless method. The second method of refraction-diagram construction omits the plotting of the successive wave-crest positions. Each orthogonal is plotted directly by determining its shoreward deflection as it crosses successive bottom contours. The determination of this deflection involves a relationship derived from Snell's law and expressed by the formula

$$\Delta\alpha = \frac{R}{J}\frac{\Delta L}{L_{av}} \sin\alpha$$

where $\Delta\alpha$ = the angular change in direction of the orthogonal,

R = the segment of the orthogonal over which the angular change occurs,

ΔL = the change in wave length as the wave crosses the R segment,

J = the distance between the bottom contours passing through each end of the R segment,

L_{av} = the average wave length as the wave crosses the R segment,

and α = the average angle between the wave crest and the bottom contours along the R segment.

This formula contains an approximation that renders its solution invalid for values of α greater than about 13°. By use of the relationship $\frac{R}{J} = \sec\alpha$ the above formula may be converted into

$$\Delta\alpha = \frac{\Delta L}{L_{av}} \tan\alpha$$

This converted formula is simpler than the original formula when α is less than about 80°. However, when α is more than about 80°, $\tan\alpha$ begins to approach infinity, and certain approximations are no longer valid. The first formula must then be used.

Before using the crestless method of diagram construction, a table showing the values of $\frac{\Delta L}{L_{av}}$ at various depths down to about half the deep-water wave length must be prepared. Two types of plotting aids especially designed for crestless orthogonal projection are shown in the publication of Johnson, O'Brien, and Isaacs (1948), which also gives explicit instructions for their use. One is essentially a protractor containing a simple nomograph for determining values of $\Delta\alpha$; the other, which is designed for use with a drafting arm, measures the angle α directly, indicates the value of $\Delta\alpha$ in the same step, and greatly expedites the plotting process.

Base hydrography. Regardless of the method used, a detailed, fairly large-scale map of the bottom topography is essential. Because some published charts of the Hydrographic Office are small in scale or lack sufficient sounding coverage for diagramming of the desired accuracy, bromide prints of the original field sheets on which the hydrographic surveys are plotted sometimes are used. These prints, which are obtained from the Hydrographic Office in Washington, D.C., contain a wealth of detail not shown on the published charts. The bromide prints, which generally show the areas contiguous to shore in fairly large scale, are especially useful in diagram construction. In the wave-crest method of refraction-diagram construction, drawings of bottom contours are not required; the average depths are determined by inspection of the soundings. In the crestless method, bottom contouring is required. The publication of Johnson, O'Brien, and Isaacs (1948) describes how the contours must be idealized, (i.e., modified to eliminate minor irregularities).

PRACTICAL APPLICATION BY THE LOS ANGELES DISTRICT

General. In the practical application of diagram construction by the Los Angeles District, the wave-crest method generally is used where (1) relatively flat banks and gentle slopes occur in the underwater terrain, and (2) the exact alinement of the bottom contours is difficult to determine. The crestless method generally is used where other conditions of bottom topography occur.

Inaccuracies may occur in both methods of diagram construction where exceptionally rough terrain occurs. Submerged reefs and scarps may cause a wave to break and re-form in several smaller waves instead of moving along sharp turns, as indicated on the diagram. These irregularities, especially such irregularities as projections of ledge rock near the surface, may also cause diffraction or flowing of wavy energy along the crest. This results in a wave pattern that does not conform to the pattern indicated by a diagram of pure refraction. Further investigation is required to determine those limits of bottom steepness beyond which the theoretical refraction pattern does not conform to the actual wave behavior.

An important limitation of the refraction principles is that they are applicable to oscillatory waves only. When the wave breaks or begins to spill, the refraction theory no longer applies. Consequently, in drawing a refraction diagram, the breaker depth for the height and period of the wave diagrammed should be determined; diagram construction should end at that limiting depth. The tide stage, which assumes considerable importance near the shore line, must be considered in diagram construction to determine the effects of waves on shore structures.

Advantages and disadvantages of wave-crest method. The advantages of the wave-crest method of diagram construction include the following: (1) the diagram can be drawn without contour lines over the base hydrography; (2) the completed diagram shows successive positions of the wave crest; and (3) the operators quickly attain skill in the application of the easily understood principles and procedures for the diagram's construction.

The disadvantages of the wave-crest method include the following: (1) a tendency to smooth out the wave-crest position on the diagram frequently results in the error of depicting a wave convergence instead of the actual crest severance with resultant crossing of the disrupted wave elements; (2) the selected interval of wave-crest advance is usually too large in areas of very shallow water and rugged bottom terrain; (3) the pattern of successive wave crests, which is the first step in the two-step process of developing orthogonals, is a rarely required refinement in diagram analysis; and (4) the practical use of the universal scale graduated in terms of the unit $\frac{d}{Lo}$ requires a separate acetate scale for each map scale used and for each wave period diagrammed.

Advantages and disadvantages of crestless method. The advantages of the crestless method of diagram construction include the following: (1) the orthogonals are developed in a single operation; (2) the points of crest severance and the areas of crossing wave trains are clearly indicated on the diagram; (3) the dimensionless protractors for diagram construction are used with equal facility for hydrography drawn to any scale and for all ranges of wave periods; (4) the limitation of $\Delta\alpha$ to a maximum of 13° provides a criterion that regulates the interval of advance and assures accurate operations; (5) the wave-crest position can be readily determined wherever required by drawing a curve perpendicular to the orthogonals; and (6) this method is the most efficient method when the purpose of the study is to determine the wave direction at a given point and a single orthogonal may suffice (i.e., the diagramming of a relatively broad area necessary for the development of a single orthogonal by the wave-crest method is not required).

The disadvantages of the crestless method include the following: (1) the depiction of successive wave-crest positions is eliminated; (2) an operator's faulty judgment in idealizing contours may result in errors in the diagram; and (3) the inexperienced operator must be closely supervised because the principles and procedures for the diagram's construction are somewhat difficult to understand. (A common error, especially where the orthogonal is nearly normal to the contours, is that of turning the orthogonal in the wrong direction.)

Diagrammatic comparison of the two methods. A diagrammatic comparison of the two methods was made by superimposing diagrams, one drawn by the wave-crest method and the other by the crestless method, over the same bottom contours (Fig. 1). The diagram in which the wave-crest method was used indicates incorrectly the development of an exceptionally strong wave convergence. The diagram in which the crestless method was used indicates correctly crest severance and a pattern of crossing wave trains.

Special techniques of diagram construction. Operators who are not familiar with the basic theories of the methods they are using may experience difficulty in constructing diagrams where irregular bottom contours occur. As previously stated, one disadvantage in using the wave-crest method is that errors may occur in the diagram if the selected interval of wave-crest advance is too large. These errors may be eliminated by transferring the diagram to larger-scale charts of the near shore area and by shortening the selected interval of wave-crest advance in that area. This type of transfer is shown in Fig. 4, where a small-scale drawing shows refraction over the broad continental shelf opposite Redondo Beach and a large-scale drawing shows refraction over the head of Redondo submarine canyon. The refraction over the head of Redondo submarine canyon causes convergence at the shore line between the Redondo Beach breakwater and Horseshoe Pier. This effect could not be shown to advantage on the small-scale drawing.

The inexperienced operator using the crestless method over irregular contours may experience difficulty in fitting the length of the R segment of the orthogonal to the contour pattern at each step. This must be done to prevent overshooting the limitations imposed by the approximations used in this method of diagram construction. For example, if the R length is too great, the orthogonal may advance into a region where the alinement of contours is considerably different from the contour alinement at the starting point. Thus, the R segment often must be shortened to such an extent that its distal end is between two of the plotted contours. As a result, a new contour line must be interpolated locally to pass through the desired point. The experienced operator who is able to judge the effects of the changing angles of incidence may be able to adjust his orthogonal alinement to the true position in a single long step instead of the several short steps that the inexperienced operator must take. To develop this technique, the inexperienced operator must first draw the orthogonals by short steps and repeat them with longer steps that are so adjusted as to obtain conformity. The ability to make shortcuts can be developed only through experience.

A technique used with considerable effectiveness in obtaining accurate results consists of requiring the operator at each step to draw a short dash at the distal end of the R segment and a small circle at the point of direction change. Constructing a diagram in this way enables the supervisor to easily check any part of the diagram.

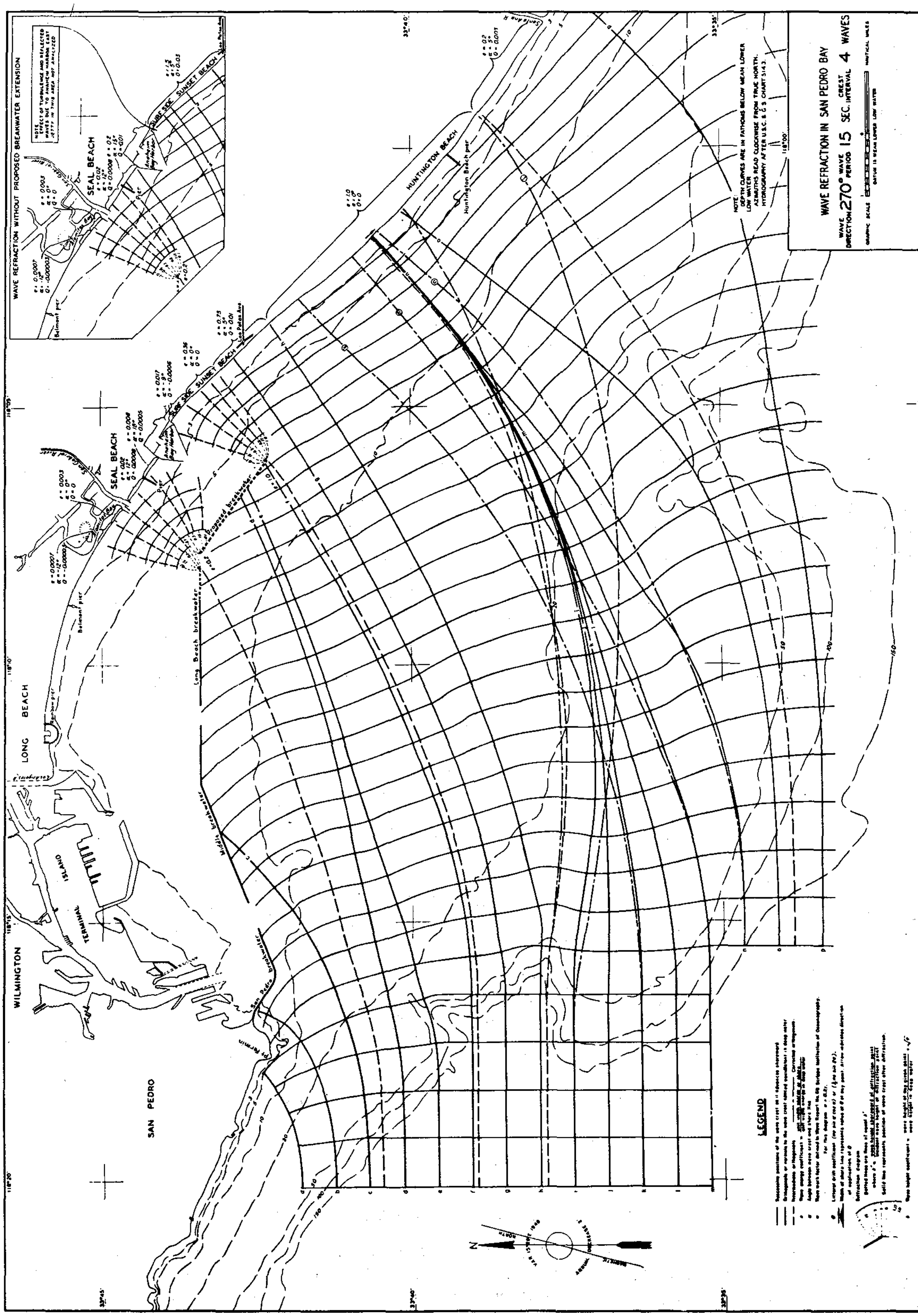

WAVE REFRACTION IN SAN PEDRO BAY
WAVE DIRECTION 270° WAVE PERIOD 15 SEC. CREST INTERVAL 4 WAVES
WAVE REFRACTION WITHOUT PROPOSED BREAKWATER EXTENSION
HUNTINGTON BEACH
SEAL BEACH
SUNSET BEACH
LONG BEACH
WILMINGTON
SAN PEDRO
TERMINAL
ISLAND
Long Beach breakwater
Middle breakwater
San Pedro breakwater
Proposed breakwater
Belmont pier
Huntington Beach pier
LEGEND

Fig. 1

The training of a new operator should include explaining that the orthogonal itself is a curve that becomes tangent to his work lines at the beginning and end of each R segment. The work may be compared to that of the highway or railroad surveyor plotting a traverse in a series of tangents and then filling in the curves to meet BCs and ECs established through consideration of the terrain and minimum radii. The short dashes that mark the points of tangency and the small circles that mark the points of intersection point up the comparison.

Because of the approximations of the method, the greater the difference in alinement between the true orthogonal and the work line, the less accurate the work will be. Where the value of $\Delta\alpha$ is greater than 13°, the angle at which the orthogonal curve crosses the contour at the median point of the R segment may be considerably different from the angle at which the back tangent crosses that contour and the approximation that assumes these two angles to be identical is not valid. If the contours are sharply curved or irregular, the approximation limits may be violated unwittingly by crossing several differently skewed contours in a single step. The only sure method of obtaining accuracy under changing conditions is to shorten the interval of advance at each step by interpolating additional contours if necessary.

Another major difficulty encountered by the inexperienced operator is the change from the use of the simplified formula involving the tangent of α to the method involving the secant of the angle and the R/J ratio. The transition between these two methods occurs where α exceeds 80° and the orthogonal tends to follow along the contours. Here again, if the operator places a short dash across the orthogonal at each point of tangency and a small circle at each angle point, the transition is made without confusion. One should remember that when using the R/J method, the maximum curvature of the orthogonal is being indicated for the value of $\Delta L/L_{av}$ concerned. The orthogonal will soon begin to cross contours, plotted or interpolated, at an angle less than 80°; continuation of the R/J method at this point results in serious error. Where the contour spacing changes, the length of the R segment must be correspondingly changed or again the limits of approximation are violated. Operators of the Los Angeles District avoid these errors by "boxing in" the R-J segments, as shown in Fig. 2.

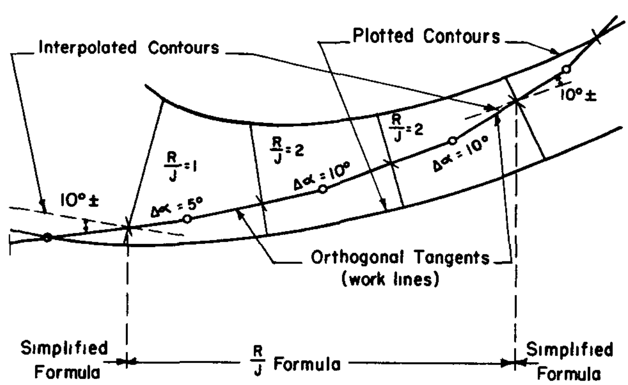


Fig. 2
Orthogonal plotted by R/J formula.

The inexperienced operator often expends time and effort unnecessarily by using too small an interval of advance although, because of the uniformity of the contours, he could use a larger interval without sacrificing accuracy. This is especially true where the orthogonals cross the contours approximately at right angles. The danger of error lies not so much in progressing with steps that are too long as in turning in the wrong direction. If the operator remembers that the orthogonal always turns toward the shallow side, he will avoid turning in the wrong direction, which is one of the most common sources of error in diagramming with the crestless method.

PRACTICAL APPLICATION OF DIAGRAM PROCEDURES TO COASTAL ENGINEERING

General. During the past decade, oceanographers and meteorologists have often collaborated in their studies to determine the relationship between storms at sea and subsequent wave action within a wide radius of the storm centers. By using the refraction-diagram procedure previously described, the effect of offshore wave conditions on nearshore areas has been fairly well determined (Munk and Traylor, 1947). Aerial photographs confirm the accuracy of the wave-forecasting and refraction-diagramming techniques. In nearly all cases, the conformity of the theoretical diagram to the actual photographed wave pattern was sufficiently close to dispel any doubts regarding the accuracy of refraction diagrams. The obvious conclusion was apparent: the refraction diagram combined with wave forecasts and hindcasts was a useful tool to the coastal engineer in many ways.

Problems of convergence. An important application of the refraction theory to coastal engineering practice is the determination of zones of wave convergence to be avoided in determining safe sites for proposed shore structures and to be considered in planning to protect existing structures. Studies of the beach-erosion problem at Redondo Beach indicated that existing structures could be protected by a proposed breakwater extension. This new structure, which would be offshore beyond the point of convergence, would intercept waves before they reached destructive heights of convergence.

The Standard Oil Company's oil-loading wharf at El Segundo, California is a structure that was built before application of the refraction theory to coastal engineering practice. As a result, it was unwittingly located in a convergence zone, and huge waves are not uncommon in the vicinity of the structure. A wave gage installed at the end of the wharf recorded waves in excess of 20 ft.; at the time the record was made, even higher waves were observed breaking beyond the wharf in water more than 35 ft. deep, although the concurrent wave action at other points along the coast was nominal. The average wave period was about 20 seconds at that time. Wave-refraction diagrams drawn for this exceptionally long-period swell disclosed a convergence caused by a broad offshore ridge lying between the Redondo and Santa Monica submarine canyons. Fig. 3 shows a diagram, drawn by the crestless method, of this convergence by 18-second waves from the west.

Utilization of wave hindcasts. The designer of a marine structure is interested in the direction of approach of the strongest waves anticipated at the site of the structure. Observations at the shore line may indicate whether the largest waves come from upcoast or down coast of the normal to shore, but give little information concerning the wave regimen offshore. The determination of predominant wave characteristics beyond the breaker zone often is important in designing structures such as piers, jetties, breakwaters, and groins. This can be done by, (1) analyzing a comprehensive series of aerial photographs made during a representative time interval, or (2) constructing and analyzing refraction diagrams of some of the more recurrent deep-water storm waves that wave hindcasts, based on historical weather maps, indicate are common in the area. The refraction-diagram method generally is the least expensive and the most efficient method.

Wave-hindcasting, which is a specialized field of science in itself, is described by R. S. Arthur in Chapter 8. However, the method of utilizing the hindcasts in refraction-diffraction analysis should be described briefly. The results of hindcast analysis generally are presented in tables and graphs that indicate, (1) the duration of waves of various significant heights and directions, and (2) the work expended by those waves over a given time interval with a breakdown by significant wave periods. The data shown in these tables are generally obtained for a number of deep-water stations offshore. If an island screen occurs between the line of offshore stations and the coast (as is the case in the southern California area), refraction studies must be made to indicate the effect of the island screen on those waves of each different period and direction that affect the study.

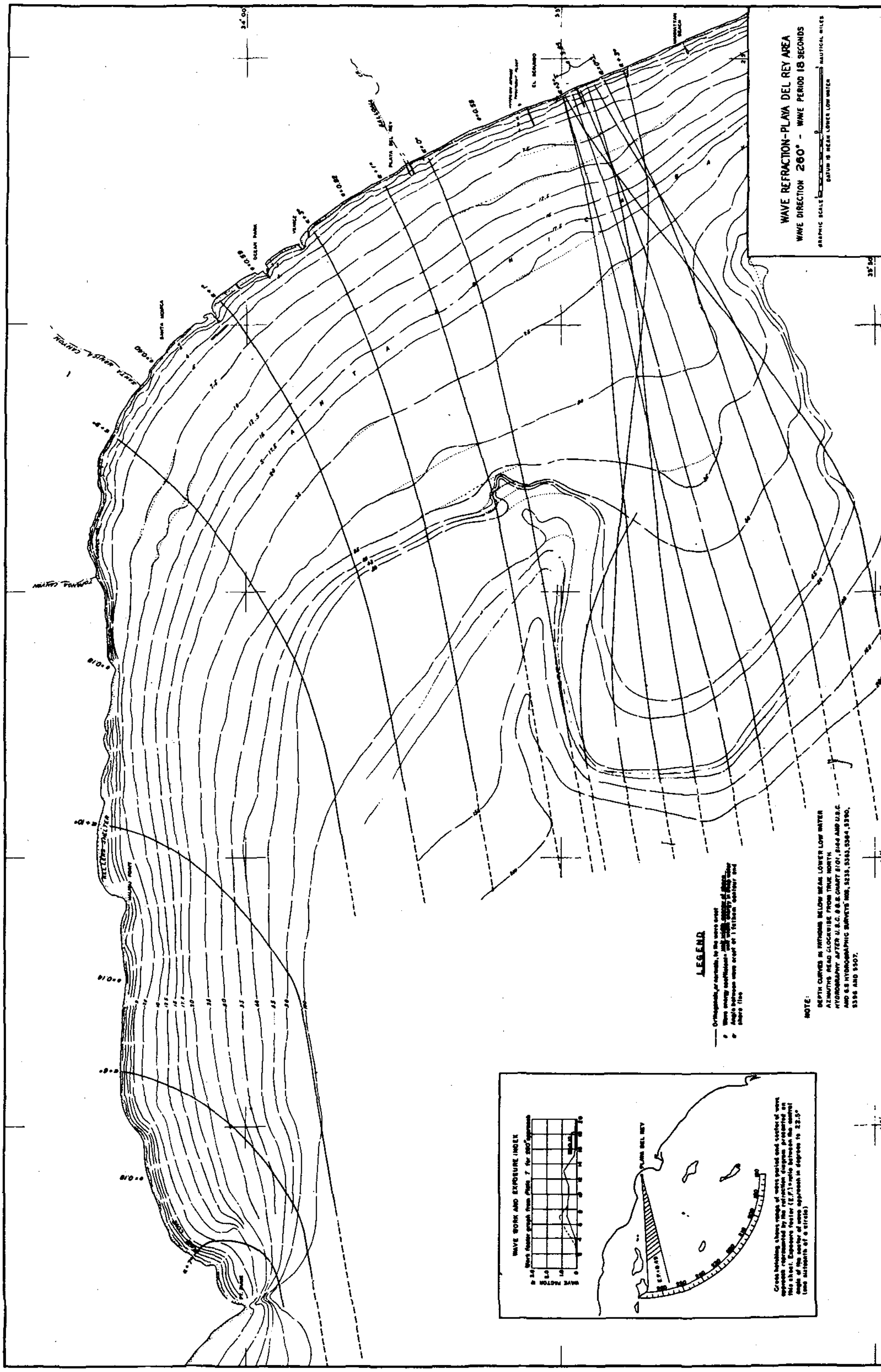

WAVE REFRACTION–PLAYA DEL REY AREA
WAVE DIRECTION 260° - WAVE PERIOD 18 SECONDS
LEGEND
NOTE:
WAVE WORK AND EXPOSURE INDEX
PLAYA DEL REY

Fig. 3

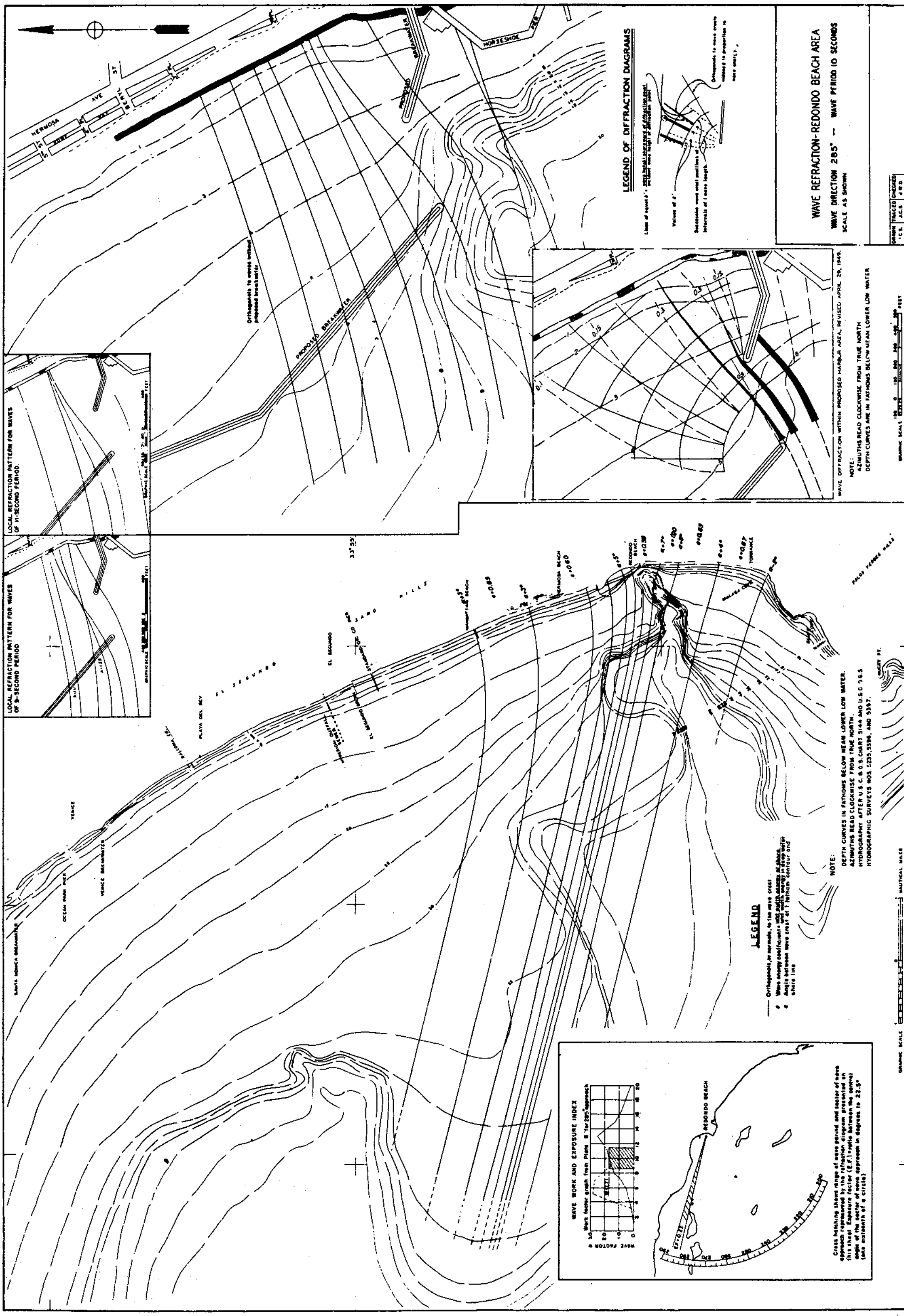

WAVE REFRACTION-REDONDO BEACH AREA
WAVE DIRECTION 285° — WAVE PERIOD 10 SECONDS
SCALE AS SHOWN
LEGEND OF DIFFRACTION DIAGRAMS
WAVE DIFFRACTION WITHIN PROPOSED HARBOR AREA
LOCAL REFRACTION PATTERN FOR WAVES OF 11-SECOND PERIOD
LOCAL REFRACTION PATTERN FOR WAVES OF 8-SECOND PERIOD
PROPOSED BREAKWATER
HERMOSA AVE
HORSESHOE PIER
REDONDO BEACH
HERMOSA BEACH
MANHATTAN BEACH
EL SEGUNDO
PLAYA DEL REY
VENICE
OCEAN PARK PIER
VENICE BREAKWATER
SANTA MONICA BREAKWATER
TORRANCE
PALOS VERDES HILLS
LEGEND
NOTE:
WAVE WORK AND EXPOSURE INDEX

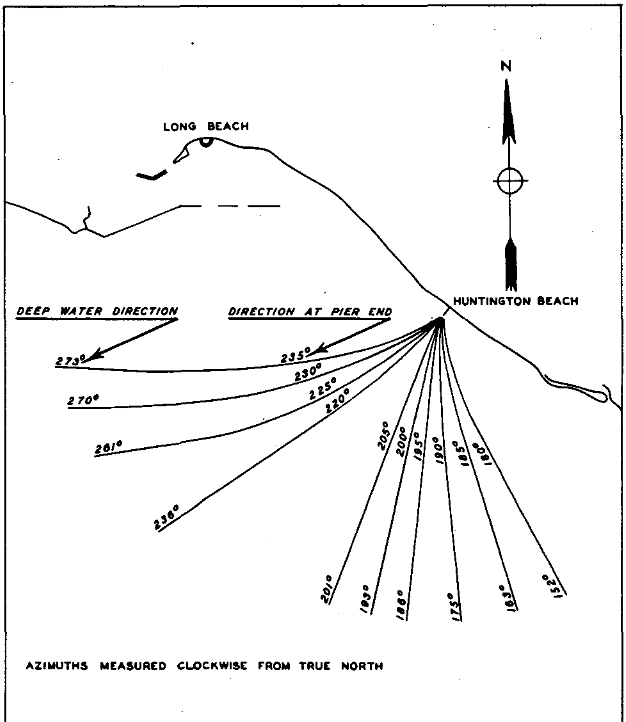
Fig. 4

To complete the analysis, refraction diagrams are made to show the effect of the continental shelf on those wave segments that penetrate the screen.

Selection of wave types for study. Obviously, drawing a refraction diagram for every possible wave period and direction is impracticable. The practicable alternative is the study of a limited number of wave types, each of which may be considered representative of all waves within certain limits of direction and period. All waves within a sector of 10° to 20° at the offshore station and within a period range of 2 or 3 seconds generally can be represented on a single diagram. Where islands and mainland-coast headlands cut off waves from certain directions, the diagram coverage may be limited to the sectors of unobstructed approach. Although the width of these sectors may vary for different wave periods, for practical purposes the sector widths for the predominant wave period are considered representative of all periods.

Effect of island screens on waves. When a wave with crest length assumed to be infinite approaches an island, a central segment of the crest converges toward the island shore, which absorbs or reflects the wave energy. Adjacent intermediate segments on both sides of the central segment are refracted by the insular shelf into the island lee where they cross and continue in their diverse directions. Unrefracted segments along the outer sides of the intermediate segments continue shoreward in an unchanged direction. In analyzing the effect of the island screen, the effects of refraction on the intermediate segments of each wave crest are disregarded. The unrefracted sectors approaching a mainland station are assumed to include the intermediate segments even though they have been diverted away from the station. The reasoning behind this anomalous procedure is as follows: For every intermediate wave segment refracted away from the mainland station, part of a wave crest from another direction is probably diverted toward that station by refraction over the same insular shelf.

Presentation of analytic data. Insets on Figs. 3 and 4 show the offshore direction sector represented by the refraction diagrams shown on these figures. Wave-work diagrams for the 22-1/2° sectors covered by each refraction diagram are also given for the nearest offshore station of a wave hindcast study. These wave-work diagrams are also shown on Figs. 3 and 4. The exposure factor EF is the ratio of the sector width covered by the refraction diagram to the sector width represented by the work diagram. On Fig. 4, the period range represented by the refraction diagram is shown by cross-hatching on the work diagram. The angle between the wave crest and the contour at breaker depth is given on all diagrams for each orthogonal. The convergence or divergence of the orthogonals is indicated by the energy coefficient e, which is the ratio of the spacing between orthogonals in deep water to the spacing between orthogonals at the breaker line.



FAN-TYPE REFRACTION DIAGRAM
WAVE PERIOD - 12 SECONDS

Fig. 5

Pin-pointing the analysis. As a result of analyzing a complete series of refraction diagrams for a given shore segment, information is obtained on (1) the probable direction of littoral drift along the shore, (2) the zones of convergence for each type of wave, and (3) the cumulative duration of each type of wave train over a period of time. This type of general study may suggest the advisability of a more detailed study of waves within a limited period range or direction sector where slight changes in period and direction produce major changes in the surf zone. The marked effects of one-second changes in wave periods are shown on Fig. 5. A detailed study of the effect of changes in period in the Santa Barbara Channel approach indicated that only waves of 11 seconds and less could reach the Redondo Beach area through that corridor. Longer-period waves were diverted toward shore or away from the Redondo Beach area by banks and shelves at the east end of the Santa Barbara Channel.

Application to groin design. Where a system of groins is required to stabilize a beach affected by a strong littoral drift, the alinement of the orthogonals of the predominant waves will indicate the proper orientation of each groin axis. The fillets of sand trapped by the groins generally assume a shore alinement perpendicular to the orthogonals (i.e., parallel to the breaker crest). By determining the probable limit of seaward advance of the shore line on the up-drift side of each groin, the length of each fillet along the shore may readily be predicted. As a result, the proper location for the root of the next groin on the up-drift side may be determined. By utilizing this method of groin-field design, each groin is alined properly according to the predominant approach direction and the groin spacing is adjusted to utilize to maximum advantage the sand-trapping effect of each structure.

Fan-type diagram. The deep-water direction of approach of any wave may be determined, when its period and its direction at a point near shore are known, by plotting an orthogonal by the crestless method of diagram construction from that point seaward into deep water. A number of these orthogonals plotted for waves of the same period but of different directions at the near-shore point (origin) produces a fan-type diagram that has many uses. For example, at Huntington Beach an automatic wave recorder with a device intended to measure wave direction was established at the end of a pier. The deep-water direction of the recorded waves could readily be determined by a series of fan-type refraction diagrams covering the probable range of wave period and near-shore direction. Fig. 5 shows one of the diagrams of this series drawn for waves of 12-second period.

DIFFRACTION

PRINCIPLES AND PROCEDURES OF DIAGRAM CONSTRUCTION

General. The term diffraction, as used in this paper, may be defined as the phenomenon in which the propagation of water waves continues into a sheltered region formed by a breakwater or similar barrier that interrupts part of an otherwise regular wave train.

Putnam and Arthur (1948) state that Penney and Price (1944) showed that Sommerfeld's solution of the optical diffraction problem, described by Bateman (1944), is also a solution of the water-wave diffraction problem. The article by Putnam and Arthur (1948) describes the development of a simplified solution to the water-wave diffraction problem and gives data on the verification of both the complete and the simplified theoretical solutions by experiments with deep-water waves. These experiments, which were made for the water area affected by the tip of a single breakwater, did not attempt to verify the theoretical diffracted wave-crest pattern. More recently Blue and Johnson (1949) reported on experiments with diffraction at a breakwater gap. The diffracted wave pattern and comparative wave heights were investigated for both deep- and shallow-water waves.

Diffraction equation. Putnam and Arthur (1948) found that the expression for the surface elevation, Z_s, is

$$Z_s = (k_1 k_2 C/g)\, e^{-ik_2 Ct} \cosh k_2 d \,.\, F(x,y),$$

where C = incident wave velocity,

d = depth of water (assumed uniform),

t = time,

y = horizontal distance in direction of incident wave travel,

x = horizontal distance perpendicular to y,

e = base of natural logarithms = 2.7183,

i = $\sqrt{-1}$,

g = acceleration of gravity,

and k_1, k_2 = constants.

The only factor changed by diffraction is $F(x,y)$. Thus, the modulus of $F(x,y)$ determines the wave height, and the argument of $F(x,y)$ determines the wave pattern. Putnam and Arthur (1948) presented graphs of the modulus and argument of $F(x,y)$ based on tabulated values of the Fresnel integrals.

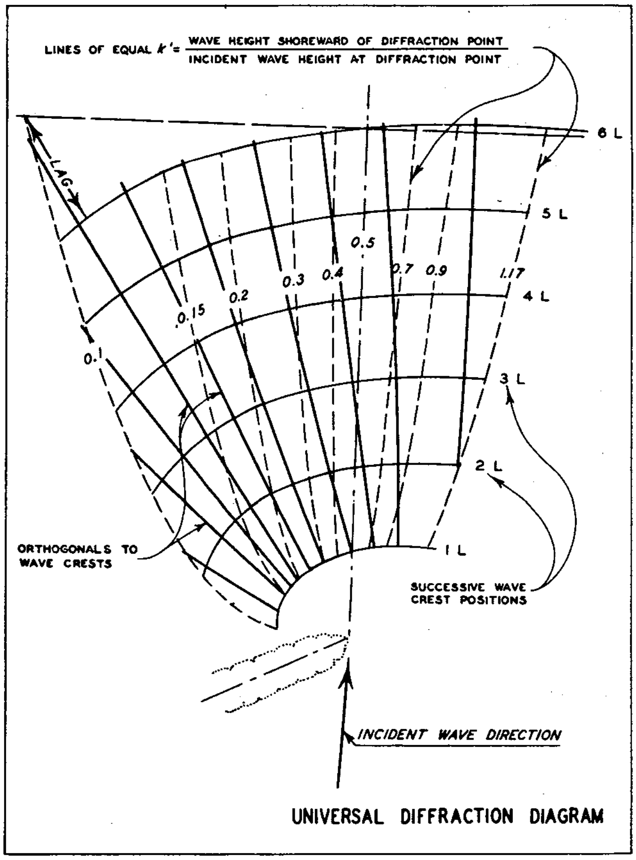


Fig. 6

The complete solution of the equation is somewhat complex because some of the terms affect the diffracted wave height and crest pattern only under unusual conditions or beyond the region in which the wave analyst is generally interested in diffraction effects. Consequently, Putnam and Arthur (1948) developed a simplified equation, which omitted those terms, as follows:

$$K' = F(x,y) = e^{-2\pi i\, y/L} \cdot f(u),$$

where

$$u^2 = (4/L\ (\sqrt{x^2 + y^2} - y),$$

$$K' = \text{diffraction coefficient} = \frac{\text{diffracted wave height at point } (x,y)}{\text{incident wave height at the breakwater}}$$

(the choice of K' to represent this function is dictated by its analogy to the K_d factor of refraction theory),

L = wave length at point of diffraction.

<u>Diffraction diagram.</u> In solving this equation for various values of x and y, lines of equal K' are determined that take the form of parabolas centering on the y axis and passing through the point of diffraction. The line for $K' = 0.5$ is the y axis itself or the orthogonal to the incident wave projected past the point of diffraction. The area on the sheltered side of the y axis is within the geometric shadow of the breakwater, and the area on the other side is outside the geometric shadow. Although this diagram applies only to sharp-cornered breakwater tips, the use of rounded tips similar to those found in rubble-mound construction did not produce large differences between experiment and theory. The angle between the diffraction axis and the breakwater axis theoretically has no effect on the dif-

fraction pattern. The position of the diffracted wave crest is established from values of crest lag, which are determined from the argument of f(u), and which are constant along any line of equal K'. Fig. 6 shows a diagram of wave diffraction carried to the wave-crest position six wave lengths beyond the point of diffraction. The diagram shows both the lines of equal K' and the successive crest positions at even wave-length intervals.

Experimental verification. The experiments conducted by Putnam and Arthur (1948) showed generally close agreement between the experimental and theoretical values of K'. This agreement, which generally was closer for the simplified solution than for the completed solution, was most markedly shown near the diffraction axis and within the geometric shadow. Outside the geometric shadow, experimental heights were less than the theoretical heights.

The results of experiments by Blue and Johnson (1949) generally corroborated those by Putnam and Arthur (1948) regarding the values of K', and disclosed fair conformity of experimental wave patterns to theoretical patterns for both deep- and shallow-water waves. The experimental wave patterns, which generally were ahead of their theoretical positions along the gap center line, showed a more sharply convex crest shape than the theoretical patterns. Irregularities in the experimental wave patterns evidently were related to wave steepness, the steeper the incident wave, the greater being the velocity increase along the diffraction axis and outside the geometric shadow and the more irregular being the diffraction pattern. The theoretical diffraction pattern generally is within the limits of accuracy required for investigations of harbor design, especially for long-period wind waves and swell. Within the geometric shadow the theoretical values of wave height determined from the calculated values of K' normally are within about 10 percent of the actual value. Because the theoretical heights of waves generally exceed the actual heights and the design of structures inside the harbor is based on the theoretical heights, maximum protection is provided.

APPLICATION TO COASTAL ENGINEERING

Artificial harbors. An obvious application of diffraction-diagram construction to coastal engineering is the determination of wave heights and crest patterns within a proposed or existing breakwater-protected harbor for any given characteristics of an incident wave train. Because the degree of variation in diffraction patterns is the same in all directions for all changes in wave length, the same diagram may be used for any combination of map scale and wave period by enlarging or reducing the diagram to fit the incident wave length. A diffraction diagram of any harbor of uniform depth may be quickly drawn by (1) determining the incident wave length, (2) marking off a series of wave lengths along the prolongation of the orthogonal through the breakwater tip, (3) enlarging or reducing the diffraction diagram until the wave lengths of the diagram and the harbor map are the same, and (4) tracing the diagram (with its origin at the breakwater tip and its axis coinciding with the prolonged orthogonal) onto the harbor map.

Breakwater gaps. Some breakwaters have gaps at intervals along their axes to facilitate navigation and to prevent traffic congestion. A determination of the wave pattern and the distribution of wave energy within these harbors in the vicinity of each gap is useful. The findings of Blue and Johnson (1949) for waves with incident direction normal to the breakwater axis indicate that a fairly accurate approximation of the diffracted wave pattern and of the values of K' may be obtained by drawing two mirror-image diagrams with the axis of each diagram passing through one of the two breakwater tips. Considerable judgment must be used in connecting the wave-crest positions between the two axes because the agreement between experiment and theory is poorest in that area. The practice of the Los Angeles District is to round the crest positions between the diffraction axes so that they blend smoothly into the convex curves of the crest positions within the geometric shadows. As a result, the wave crests at the gap center line are somewhat ahead of their theoretical positions. Values of K_d between the two axes are then determined by the refraction-diagram procedure instead of by theoretical computations using the diffraction formula.

Breakwater gaps with oblique wave approach. Where the direction of incident wave propagation is oblique to a straight line connecting the two breakwater tips, the diagram-construction procedure is similar to that described in the preceding paragraph up to the point of connecting the two mirror-image diagrams. The incident wave crest reaches one breakwater tip and there begins to diffract before it reaches the other breakwater tip. Unless the wave crest reaches the second breakwater tip an exact multiple of the wave period later, the two diffraction diagrams will be out of phase. One of these diagrams must then be modified by interpolating crest positions that are in synchronization with the other diagram. These synchronized crest positions then can be connected with the crest positions of the other diagram as described in the preceding paragraph. A diagram of diffraction at a breakwater gap with oblique incident waves is shown in Fig. 1.

Converging breakwaters. Interior harbors often are designed with converging breakwaters for protection of the entrance to the inner harbor. The principle behind this type of entrance protection is that the diffraction of waves passing through the gap between the outer ends of the breakwaters reduces the wave height progressively from the gap to the harbor entrance. A pervious dike or a barrier beach generally is provided to absorb the wave energy on each side of the entrance axis between the landward ends of the converging jetties and the entrance to the interior harbor. Here again, the diffraction diagram may be used to predict the effectiveness of this type of construction in reducing wave heights. The application of diffraction and refraction diagramming principles to the problems of converging breakwaters is the same as the application of these principles to the problems of breakwater gaps.

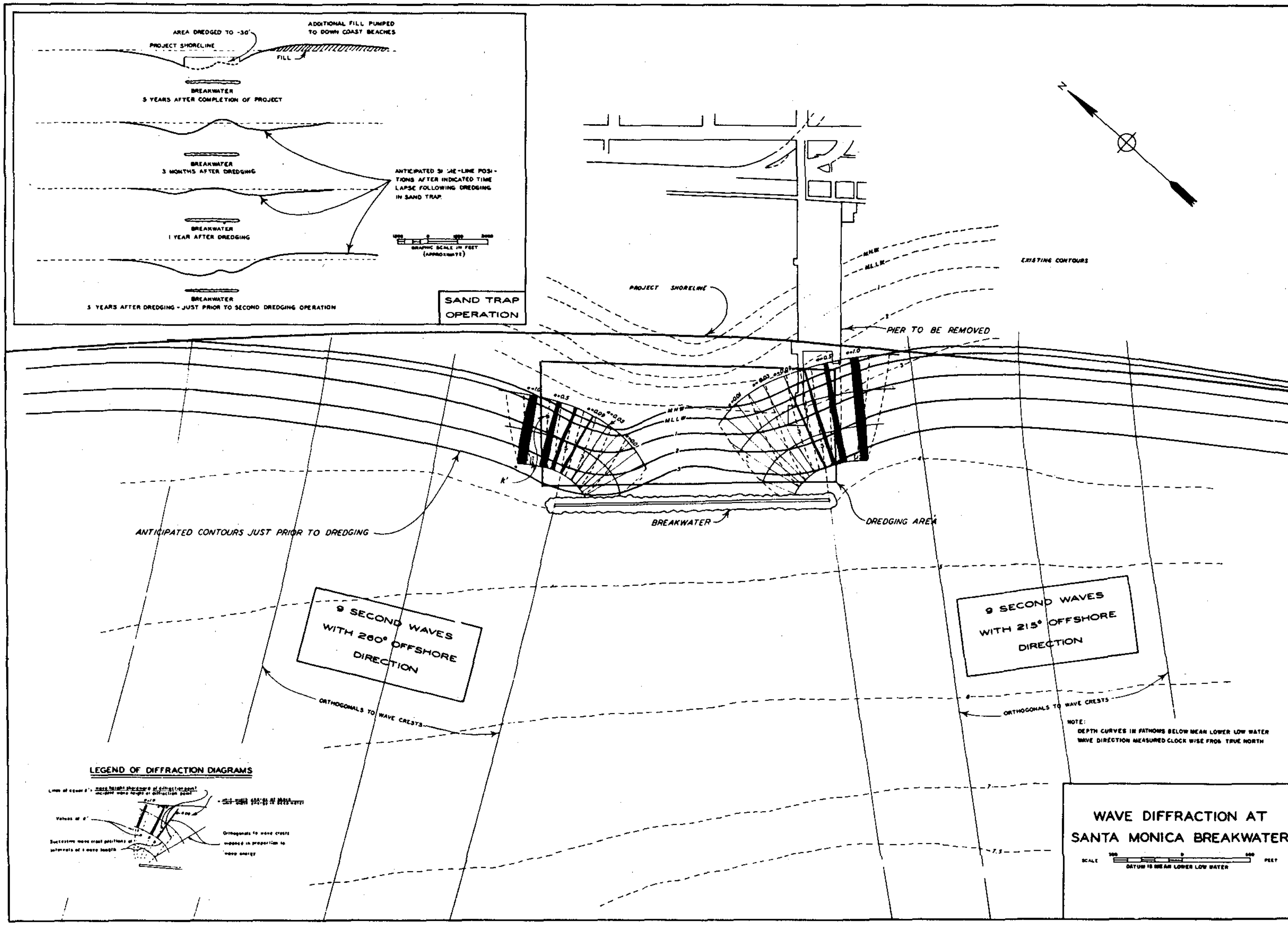
REFRACTION-DIFFRACTION COMBINED

PRINCIPLES AND PROCEDURES OF DIAGRAM CONSTRUCTION

General. If the bottom contours near a breakwater are not normal to the wave direction, refraction as well as diffraction must be considered; after the first few wave lengths shoreward from the barrier, the effects of refraction may be more significant than those of diffraction. Thus, the breakwater problem usually combines the problems of (1) refraction over the continental shelf to the point of diffraction at the breakwater, (2) diffraction for some distance beyond the breakwater, and (3) refraction between that point and shore.

Abrupt-change diagram. In considering the effects of various offshore wave directions and periods near the breakwater, the direction and length of each wave to be considered must be determined by a refraction diagram for the area between deep water and the diffraction point. Shoreward from the diffraction point, diffraction and refraction occur concurrently. However, for practical purposes and in the absence of experimental data on the combined effects of these two phenomena, the Los Angeles District uses a pure diffraction diagram only for a short distance beyond the point of diffraction, and thence reverts abruptly to pure refraction theory by extending the orthogonals to shore by refraction principles. This may be done by (1) the wave-crest method -- to determine the successive positions of the wave crest beyond the last position revealed by the diffraction theory or (2) the crestless method (by starting each orthogonal normal to the most shoreward wave crest of the diffraction diagram) -- to determine the divergence of orthogonals without delineating the wave-crest positions. By use of the lines of equal K', refraction coefficients may be carried from the offshore area through the diffraction area and into shore.

Modified diffraction diagram. If the orthogonal drawn through the breakwater tip by refraction procedures curves sharply, greater accuracy can be obtained by using the orthogonal as the diffraction axis and by varying each successive wave-crest position of the diffraction diagram to conform to that axis. The diffraction diagram thus modified should not be carried more than two or three wave lengths beyond the diffraction point before changing to refraction procedures. Otherwise, the wave pattern would be distorted.



SAND TRAP OPERATION
AREA DREDGED TO -30'
ADDITIONAL FILL PUMPED TO DOWN COAST BEACHES
PROJECT SHORELINE
FILL
BREAKWATER
5 YEARS AFTER COMPLETION OF PROJECT
3 MONTHS AFTER DREDGING
1 YEAR AFTER DREDGING
5 YEARS AFTER DREDGING - JUST PRIOR TO SECOND DREDGING OPERATION
EXISTING CONTOURS
PROJECT SHORELINE
PIER TO BE REMOVED
MHW
MLLW
BREAKWATER
DREDGING AREA
ANTICIPATED CONTOURS JUST PRIOR TO DREDGING
9 SECOND WAVES WITH 260° OFFSHORE DIRECTION
9 SECOND WAVES WITH 215° OFFSHORE DIRECTION
ORTHOGONALS TO WAVE CRESTS
NOTE:
DEPTH CURVES IN FATHOMS BELOW MEAN LOWER LOW WATER
WAVE DIRECTION MEASURED CLOCK WISE FROM TRUE NORTH
LEGEND OF DIFFRACTION DIAGRAMS
WAVE DIFFRACTION AT SANTA MONICA BREAKWATER
DATUM IS MEAN LOWER LOW WATER

Fig. 7

Weighted-line orthogonal diagram. In studying the wave characteristics near an offshore obstruction or in the lee of a breakwater, information often is required regarding the wave height and the energy per unit length of wave crest at all points in the area diagrammed. This may be done by widening the orthogonals in such a way that the line width is proportional to either of these two values. Orthogonals thus widened are called weighted-line orthogonals. By using the square root of the energy coefficient in weighting the orthogonals, the values of K_d and K' (i.e., wave heights) can be represented. Figs. 4 and 7 show diagrams of diffraction and refraction with orthogonals weighted in proportion to the wave energy per unit crest length from a point seaward of this point of diffraction to shore. A diagram of this type is particularly useful in showing the distribution of wave energy in such a manner that the layman can quickly grasp the implication of the diagram.

PRACTICAL APPLICATION BY THE LOS ANGELES DISTRICT

Sand-trap characteristics of breakwaters. Offshore breakwaters tend to trap littoral drift through the reduction of wave energy in their lee. This tendency is markedly shown at Santa Monica, California, where the breakwater, originally built to form a sheltered area for harbor use, has trapped several million cubic yards of littoral material since its construction in 1934. The beach in its lee has advanced seaward more than 800 feet, considerably reducing the effective harbor area. A proposal has been made to utilize the breakwater's sand-trapping tendencies by operating a dredge in its protected lee to distribute the trapped material along adjacent beaches as required. Fig. 7 shows how refraction-diffraction diagrams have been used to predict successive positions of the shore line and offshore contours as the fill continues to advance seaward year after year despite dredging operations. The rate of littoral drift had been determined previously by computing the accretion rate.

The weighted orthogonals indicate that the energy distribution in the breakwater lee drops off rapidly. A cursory examination of the diagram would indicate that the large quantities of sand that have reached the sheltered area of the breakwater lee could not have been transported by the negligible amount of wave energy indicated. However, much of this sand has been transported by storm waves that often in the southern California area have ten times the energy of normal waves. Under storm conditions waves along the diffraction parabola K' = 0.3 are as high as normal waves in the unprotected area, and adjacent parts of the wave crest are also proportionately higher. The wave energy, which assumes sizeable proportions, is directed toward the center of the breakwater shadow. The transporting capacities of the diffracted wave elements are fully utilized -- littoral drift is carried from outside both parts of the breakwater toward the center of the breakwater lee.

Upcoast and down coast from the sheltered area, the shore line tends to assume a position generally normal to the direction of wave approach. Although the exact position of the shore line cannot be predicted by theoretical analysis, a general consideration of the applied forces will indicate the probable shore alinement that would result from any sustained wave action of constant period and direction. The inset in Fig. 7 shows several anticipated positions of the shore line after the proposed dredging operation in the sand-trap area.

Littoral drift opposite an offshore breakwater tip. Fig. 1 shows a diagram that indicates the use of refraction and diffraction diagrams in studying the effects on the adjacent shore line of an existing breakwater and a proposed breakwater extension in San Pedro Bay. Sand movement along the beach is strongly affected by the offshore breakwater as well as by the shore-connected structures in this region. Studies that included the use of wave hindcasts and refraction-diffraction diagrams have been made of the anomalous behavior of the beach and surf in the affected area. As a result, corrective action is being planned.

Development of natural harbors. Natural harbors and sheltered anchorage areas protected by jutting headlands or reefs often present problems of both refraction and diffraction. Because natural obstacles to wave propagation are seldom so sharply defined or so abrupt as artificial barriers, the combined effects of refraction and diffraction result in a complicated pattern of wave behavior. The Los Angeles District has not yet had occasion to make a refraction-diffraction study of a natural harbor. Such a study could be made to compare the effectiveness of two or more harbor sites, each having partial natural protection, under various conditions of wave attack. The study probably would indicate methods of improving a natural shelter to form a safe harbor, possibly by extending a reef or headland artificially. The actual method of diagram construction used would be based on local conditions, but the principles and procedures presented in this discussion would provide a basis for a reliable analysis.

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CHAPTER 5

NEARSHORE CIRCULATION*

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INTRODUCTION

Studies of nearshore circulation were initiated at Scripps Institution during World War II. A method of estimating the velocity of longshore currents from known wave conditions on straight beaches with parallel contours was devised by Munk and Traylor (1945) and later revised by Putnam, Munk and Traylor (1949). Their methods were based on energy and momentum considerations which were applied to the following two types of observations: (1) field observations of longshore currents along the straight beach at Oceanside, California made by Munk and Traylor (1945), and (2) laboratory measurements conducted at the Department of Engineering, University of California.

In 1945 a program of field observations was initiated to study the nearshore currents in relation to a variety of coastal types and submarine configurations. Operations extending over a period of one year involved measurement of currents inside the breakers at 63 stations from the United States -- Mexican boundary to Newport, California (Shepard, 1950) (Fig. 1). The observations were repeated approximately every 12 days. Subsequently the shallow waters adjacent to individual beaches representative of various types of environment have been studied intensively. This work has included a beach with adjacent submerged ridge and canyon topography (Shepard and Inman, 1950), two straight beaches with parallel bottom contours, and one beach at the head of a crescentic bay. In addition the effects of jetties, piers, and points have been investigated. During this work currents inside and outside the breaker zone were investigated. Most of the observations were made in southern California, but studies along many other coasts of the United States and in the Hawaiian Islands indicate that the results have a general application.

All devices used in these investigations were of a free drifting type, the velocity being determined by the distance of travel in a given period of time. Currents outside of the surf zone were measured by surface floats, dye, and triplanes (current crosses) submerged at a variety of depths. Locations were obtained by multiple sextant angles from accompanying boats. Inside the breaker line bottom currents were measured by volley balls given slight negative buoyancy. Surface currents were observed by free floating kelp and dye. Details of the methods are given by Shepard and Inman (1950). Wave recordings were obtained simultaneously with many of the measurements. Angles of wave approach were determined in part by the transit-sighting bar method devised by Forrest (1950).

TERMINOLOGY AND GENERAL PRINCIPLES OF CIRCULATION

Observations of nearshore circulation show that there are certain basic principles which apply to most environments, including in varying degree straight beaches with parallel offshore contours and beaches adjoined by irregular submarine topography. There appear to be at least two interrelated current systems (Fig. 2):

1. The coastal currents which flow roughly parallel to the shore, and constitute a relatively uniform drift in the deeper water adjacent to the surf zone. These currents may be tidal currents, transient wind-driven currents, or currents associated with the distribution of mass in local waters.

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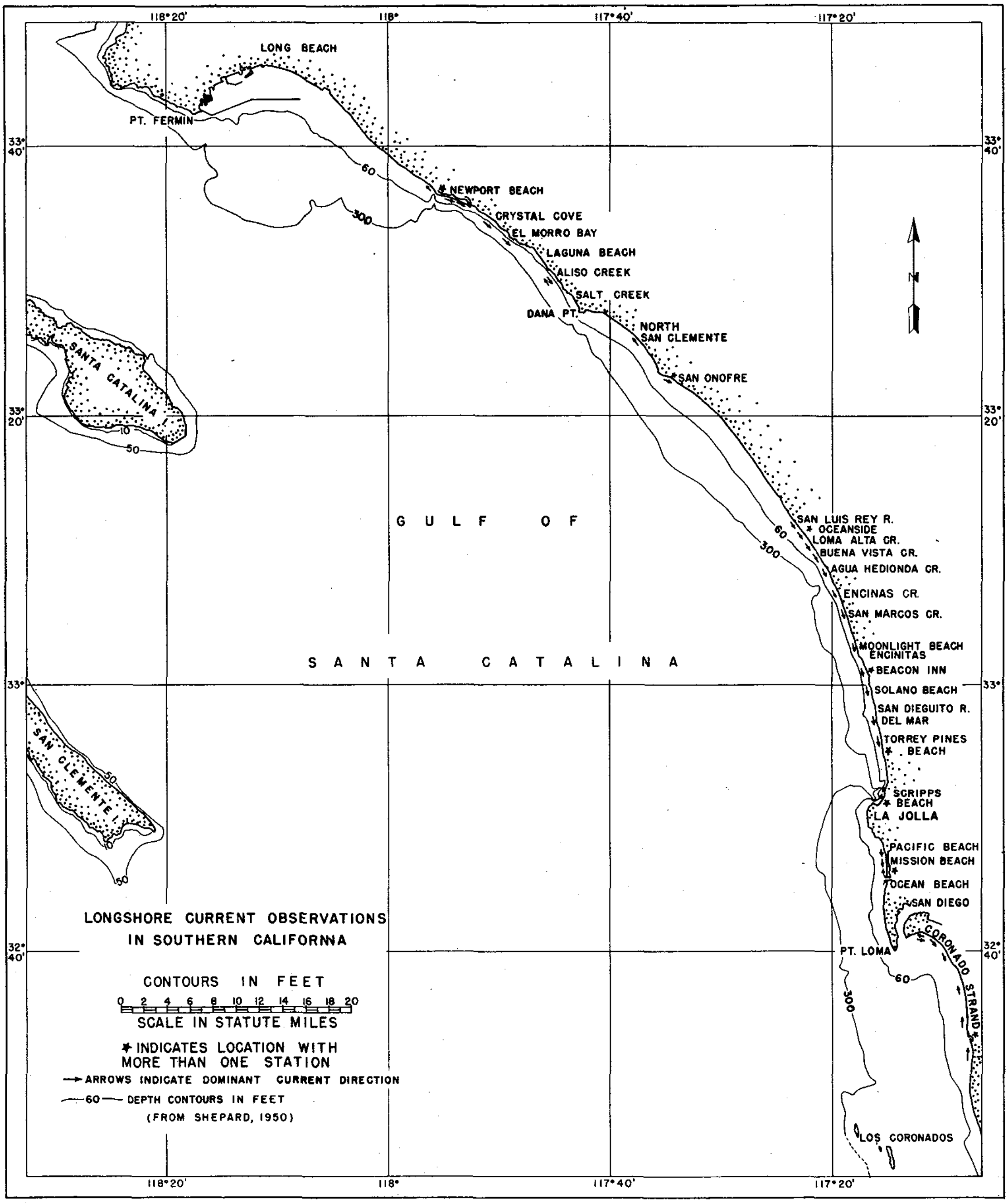

Fig. 1
Showing areas where nearshore current measurements have been made in southern Calif.

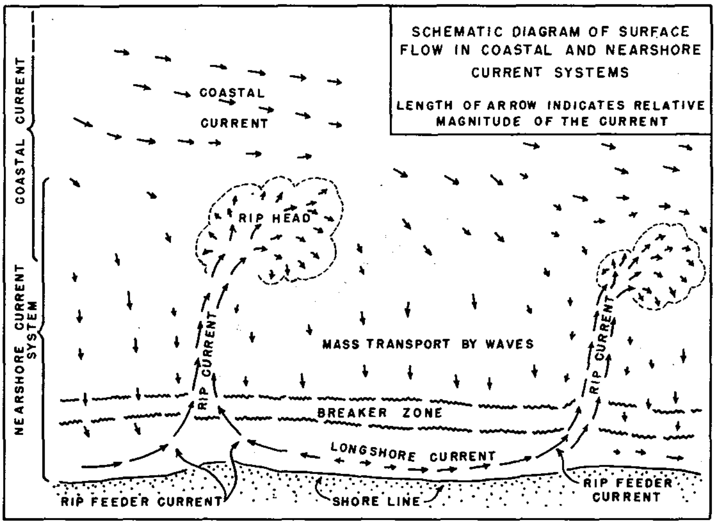

Fig. 2

2. A nearshore system which may be superimposed on the inner portion of the coastal current or in the absence of a coastal current may exist independently. The nearshore system is associated with wave action in and near the breaker zone and consists of: (a) shoreward mass transport of water due to wave motion, which carries water through the breaker zone in the direction of wave propagation, (b) movement of this water parallel to the coast as longshore currents, (c) seaward return flows, such as flow along concentrated lanes known as rip currents, and (d) longshore movement of the expanding head of the rip current.

Since the rip currents have relatively high offshore velocities, the shoreward movement is restricted to wide lanes in between rip currents. As a result, the circulation pattern takes the form of an eddy or cell with a vertical axis (Fig. 2). The positions of shoreward motion and seaward return are much dependent on the submarine topography, the configuration of the shoreline, and the height and period of the waves. Periodicity or fluctuation of current velocity and direction is a characteristic of flow in the nearshore system (Shepard and Inman, 1950). This variability is primarily due to the grouping of high waves followed by low waves, a phenomenon which gives rise to surf beat (Munk, 1949b).

The direction of longshore current is primarily dependent on two factors: (1) the direction of wave propagation, and (2) the rise in water level due to the shoreward mass transport of the waves, which is greatest in the zones of highest breakers along a beach (wave convergence zones). The longshore currents commonly flow away from these zones of highest waves.

There may be processes other than rip currents by which water may be returned seaward (Munk, 1949a). Our observations indicate the importance of a net seaward drift along the bottom inside the breaker zone and a net shoreward movement at the surface. Comparisons of the offshore and onshore components of surface and bottom currents in the surf zone are given in Table I. This table shows that there is a definite tendency for bottom longshore currents to have a small offshore component. The lack of a pronounced onshore component for the surface currents is probably due to the method of observation (see footnote to Table I on following page).

To date we have not found any indication that these differential net movements between top and bottom extend any distance outside of the breaker zone, but rather that the current moves shoreward from top to bottom in one area and seaward from top to bottom in another outside of the breaker zone.

TABLE 1

Comparison of the offshore and onshore tendencies between surface and bottom currents in the surf zone. Based on all observations, irrespective of magnitude.

Location	Surface Currents[1]			Bottom Currents[2]		
	No. of Obs.	% Offshore	% Onshore	No. of Obs.	% Offshore	% Onshore
Torrey Pines Beach	139	54.7	45.3	512	81.4	18.6
Pacific Beach	162	47.5	52.5	220	81.8	18.2
Mission Beach	224	50.0	50.0	383	76.3	23.7
Total No. of Obs.	525			1115		
Average %		50.5	49.5		79.7	20.3

[1]Surface currents were measured by free floating kelp and dye and thus represent measurements of a relatively thick surface layer and not the surface itself which is known to have a greater onshore tendency (Shepard and Inman, 1950, p. 200).

[2]Bottom currents were measured with volley balls given slight negative buoyancy.

STRAIGHT BEACHES WITH PARALLEL CONTOURS

Most beaches have some curvature and the adjacent bottom topography is usually somewhat irregular. However, there are a sufficient number of beaches which are essentially straight with parallel bottom contours to warrant consideration. This type of beach is ideal for theoretical treatment and was therefore chosen for the investigations of longshore currents by Putnam, Munk and Traylor (1949). Their momentum approach has been widely used. It relates the velocity V, of the longshore currents to the wave height H, period T, angle of approach α, and slope i of the beach, according to the equation:

$$V = \frac{a}{2}\left[\sqrt{1 + \frac{4C \sin\alpha}{a}} - 1\right] \qquad (1)$$

where $a = (2.61\ i\ H \cos\alpha) / (kT)$

and, $C = \sqrt{2.28\ g\ H}$ is the wave velocity, k is the beach friction coefficient (hydraulic roughness), and g is the acceleration of gravity.

Field measurements partly reported by Shepard (1950) have indicated only partial agreement with current velocities predicted from the equation. The discrepancies appear to be due to:

1. Rip currents which normally increase the flow in the dominant direction on the up current side of the rip zone and decrease or reverse the current on the other side (Fig. 3).

2. Cell-like circulation of the nearshore system, which exists even under conditions of normal wave approach (breaker crests essentially parallel to the beach). According to the formula there should be no current under conditions of normal approach. However, with large breakers longshore currents of velocity of one or more knots have been observed for limited distances despite this normal approach.

3. The fluctuating nature of longshore currents which requires measurements over a long interval of time in order to obtain an average that is suitable for comparison with the predicted value.

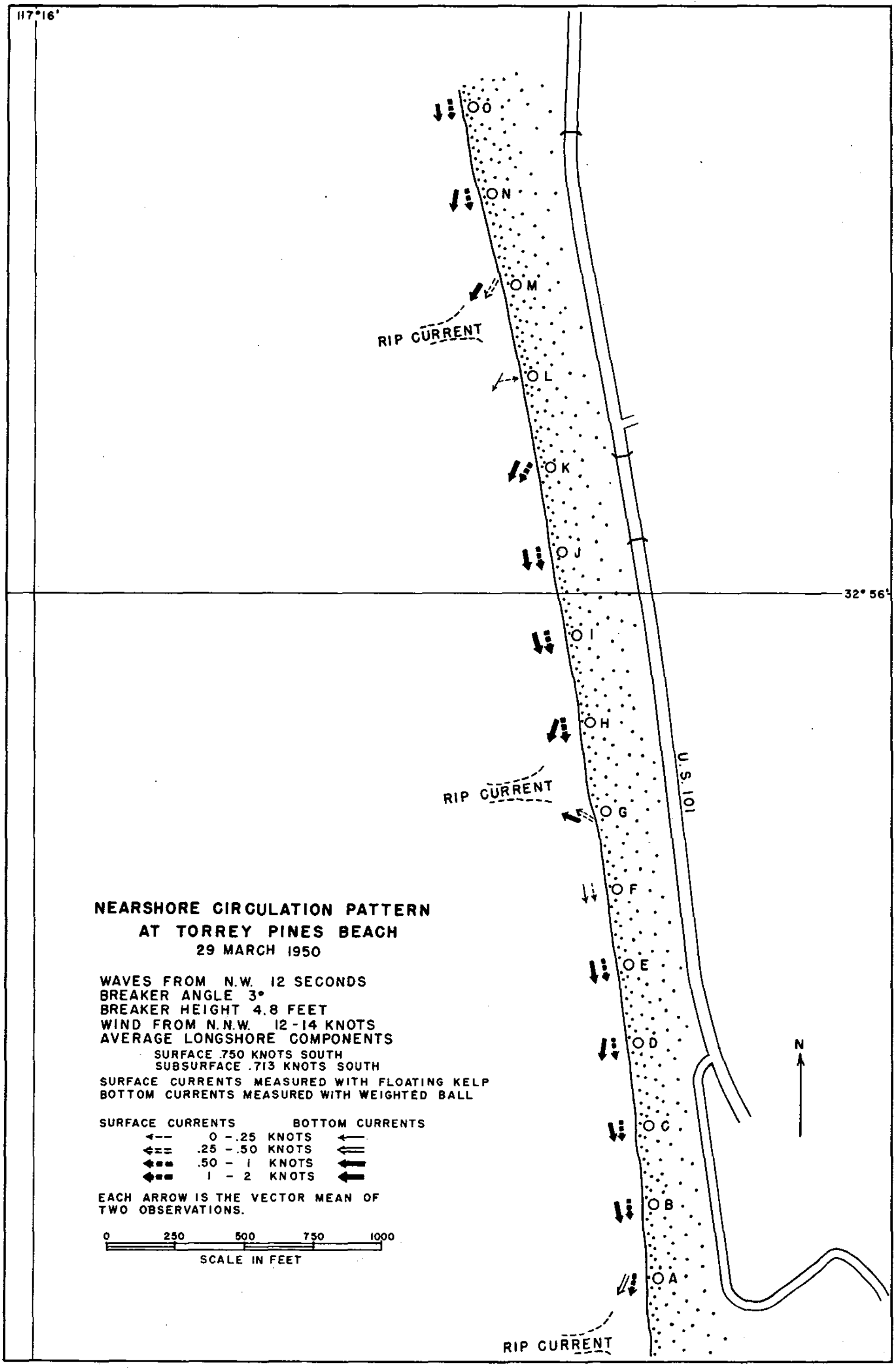

Fig. 3
Showing the influence of circulation pattern and rip currents on the direction and magnitude of longshore currents on a straight beach. Note that in the lee of rip current zones the longshore current is much reduced (station L) and sometimes reversed in direction (station G).

4. The fact that beach friction coefficient (k) was assumed to be a constant for a given beach. Further measurements indicated that it is a function of current velocity (Fig. 4).

5. Many beach approaches are terraced, making application of the formula impossible.

The nearshore circulation is frequently the result of waves approaching the coast from more than one direction, making current predictions extremely difficult.

Despite these discrepancies the longshore currents outside straight beaches with parallel submarine contours appear to be essentially the result of the factors considered by Putnam, Munk and Traylor (1949).

The average longshore component of currents observed along two relatively straight beaches with parallel bottom contours in the San Diego area were compared with the currents predicted from equation 1. Preliminary plotting of the observed versus the computed velocity showed a rather large scatter, the amount of disagreement apparently being a function of the velocity of the current. This suggested the possibility that the friction coefficient, k, is not a constant for a given beach, but varies with the velocity.

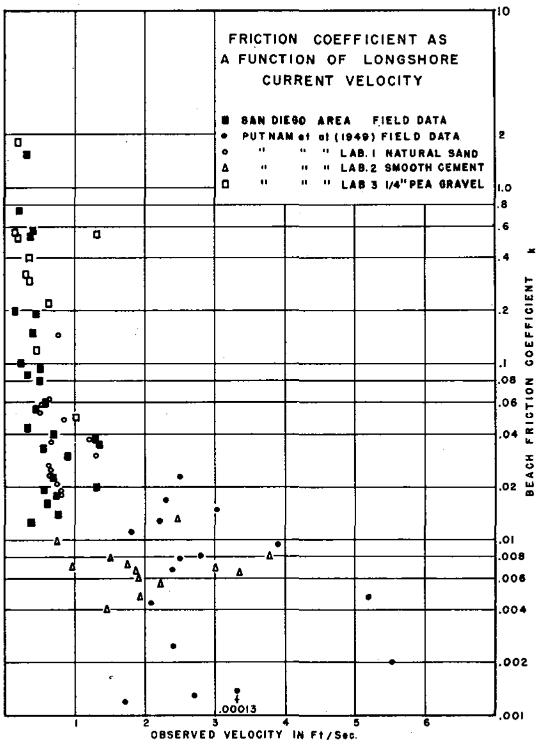

Fig. 4

Using the observed value of the longshore current, the value of k was computed for all of the field and laboratory observations listed in Putnam et al (1949) and for those recently obtained in the San Diego area. The coefficient k is plotted as a function of the observed velocity in Fig. 4. Inspection of this figure strengthens the contention that the coefficient k is a function of velocity, particularly for values of current of the order of two feet per second and less. Predicted values of longshore current based on k values from Fig. 4 have shown good agreement with field observations.

The regular spacing between rip channels cutting through the longshore bars along the straight, fine-sand beaches of northern Oregon and southern Washington indicates that the general features of the nearshore circulation pattern described in this report also apply to these northern beaches. At low tide the distance between rip channels can easily be ascertained from car speedometer readings. Observations of this type were made along the straight beaches of Clatsop Spit, Oregon, and Leadbetter Spit, Washington, on the 17th and 18th of June, 1950. The mean distance between rip channels was found to be 0.25 statute miles. The standard deviation of the distance between channels was found to be 0.09 miles for a ten mile section of beach along Clatsop Spit. These observations were made during a period of low waves.

BEACHES BORDERED BY IRREGULAR SUBMARINE TOPOGRAPHY

As yet there has been no quantitative approach to the prediction of currents or circulation along beaches bordered by irregular submarine topography. The situation differs from that existing along straight beaches with parallel contours in that the direction of longshore currents is dependent not only on the angle of wave approach but on the localized piling up of water on the beach at points of wave convergence. This local rise of sea level, the degree of which is still un-

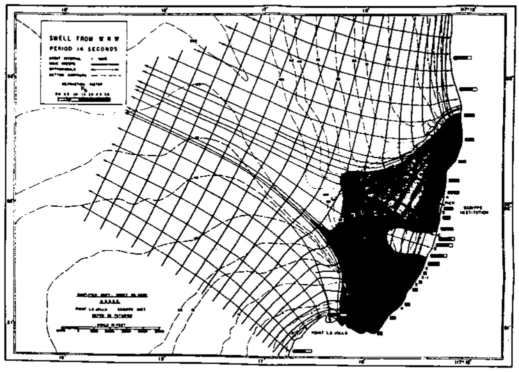

Fig. 5. Wave refraction diagram showing areas of wave convergence and divergence near La Jolla. The relative wave height is given by the length of the bar at various points along the beach. The letters A through H locate the same beach stations shown in Figs. 6 and 7. (From Munk and Traylor, 1947.)

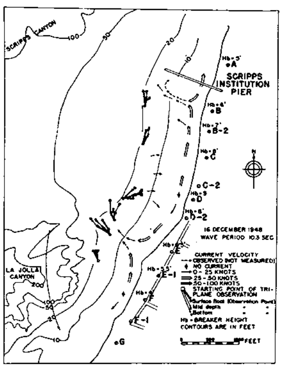

Fig. 6. The cell-like features of the nearshore circulation at Scripps Beach, La Jolla. The point of wave convergence is at station D, as shown by breaker height notation. (From Shepard and Inman, 1950.)

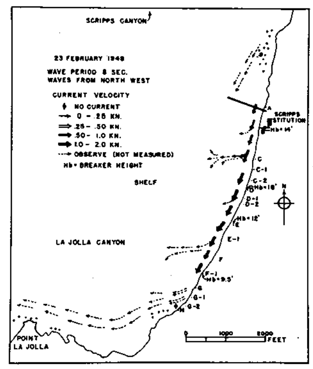

Fig. 7. For short period waves the wave convergence is not as pronounced, and the direction of longshore current is controlled by the direction of wave approach. (From Shepard and Inman, 1950.)

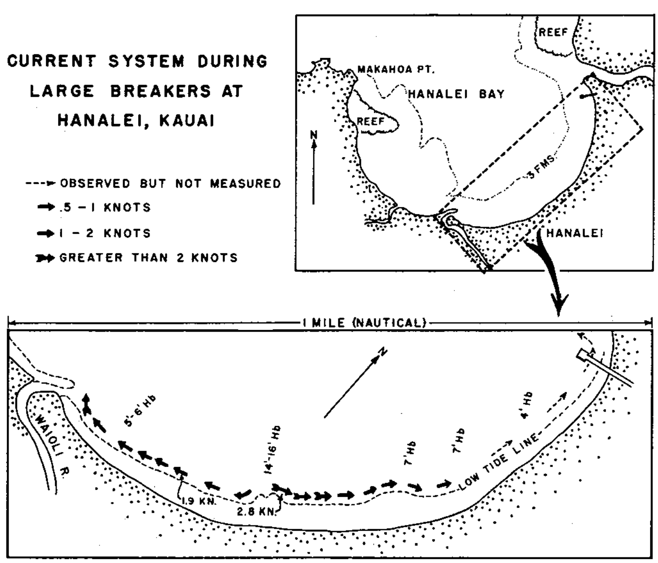

Fig. 8

known, results in a gradient along the beach and accompanying flow in both directions away from the convergence. These currents may be opposed in direction to the angle of wave approach.

Submarine canyons. Near those places where submarine canyon heads approach closely to the coast there are striking examples of wave convergence. At the canyon head the wave orthogonals diverge, decreasing the wave height, whereas they converge on either side, increasing the wave height (Fig. 5). In the La Jolla area it is not uncommon for the breaker height to be 10 times as high north of the two canyon heads as it is inside the heads. Current measurements outside the breakers at the convergence north of La Jolla Canyon indicate a movement towards the beach from the top to the bottom (Fig. 6). Inside the breakers the longshore currents flow away from the convergence in either direction. Rip currents commonly develop at points between the convergence and the divergence zone, where the waves are intermediate in height.

The current systems have been studied in two areas where canyon heads closely approach the coast: at La Jolla (Shepard and Inman, 1950) and at Mugu (Inman, 1950). In both areas the circulation pattern was found to be primarily dependent on the wave period. The longer period waves resulted in a sufficient degree of convergence so that the longshore currents flowed away from the zone of convergence. In this case the circulation cell is well developed and essentially fixed in position (Fig. 6) except as shifted slightly by changes in direction of wave approach. For shorter period waves the convergence is not as pronounced and the direction of wave approach becomes the controlling factor in determining the direction of longshore currents (Fig. 7). In this case the circulation cells are less stable and the position of the rips along the beach is variable. At La Jolla a strong blow accompanied by short period waves approaching diagonal to the coast produces constant current directions with the most pronounced rip at the bend in the coast in the down current direction (Fig. 7).

Crescentic Bay. On the north coast of Kauai in the Hawaiian Islands, Hanalei Bay forms a large crescentic bight into the main trend of the coast. During periods of high waves a large variation in wave heights was observed in a short distance along this bay (Fig. 8). In about a quarter of a mile breaker heights were observed to decrease from as much as 18 feet to 5 feet in one direction and at about half that rate in the other direction. This contrast is only partially explained by the small reefs which fringe both sides of the entrance. The highest breakers were comparable to those on the open coast so that they do not represent a convergence except relative to the adjacent portions of the bay. The longshore currents respond to this relative convergence in the same way as observed in areas adjacent to canyon heads. Currents as high as three knots were measured. At the point of greatest breaker height there is a low ridge extending seaward and the wave crests are bent around this ridge so that they come in directly against the strong outflowing currents. The flow is so strong as to produce a pronounced trough along the shore. In places these troughs are bordered by escarpments in the sand. As can be seen in Fig. 8, the flow away from the zone of high waves extends along the shore as much as three-quarters of a mile.

It is felt that these two cases including submarine canyons and crescentic bay by no means exhaust the possibilities of developing a local convergence with resulting divergent flow of longshore currents.

Small crescentic bays often show currents flowing in at one side and out on the other. During a 60 mile an hour gale at Nahant, Mass., currents were observed flowing in on one side of a small bay with velocities averaging about 4 or 5 knots. The outflow on the other side was much slower, indicating that there was a subsurface return flow. The surface inflow was continuous although considerably retarded during wave troughs.

CIRCULATION IN RELATION TO OBSTRUCTIONS

Points, breakwaters, and piers all influence the circulation pattern and alter the direction of the currents flowing along the shore. In general these obstructions determine the position of one side of the circulation cell. In places where relatively straight beaches are terminated on the down current side by points or other obstructions, a pronounced rip extends seaward. During periods of large waves having strong diagonal approach these rips can be traced seaward for one or more miles.

At the north jetty of the Mission Bay Breakwater a seaward flow was frequently observed in the current lee of the jetty. This same seaward flow north of the jetty exists under most other observed directions of wave approach (Fig. 9).

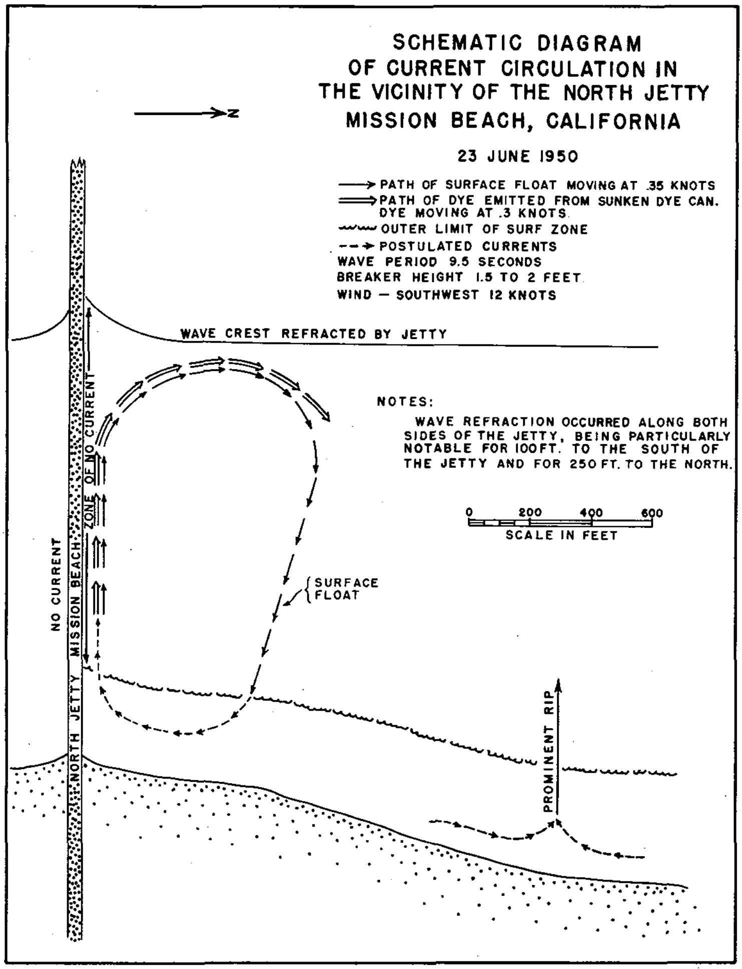

Fig. 9

Where prominent points of land interrupt the predominant longshore current flow, currents opposite in direction to that of the coast in general are likely to develop in the current lee of the point. Examples of this reverse flow have been observed in the lee of Dana Point (See Fig. 1 for location). Indirect evidence of currents flowing in this reversed direction are found where spits extend north in the current lee of obstructions such as at Morro Rock along the central California coast.

ACKNOWLEDGMENTS

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CHAPTER 6

HARBOR SURGING

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SIGNIFICANCE OF SURGING ON HARBOR USAGE

Surge is the name applied to wave motion with period intermediate between that of ordinary wind waves and that of the tides; say from one to sixty minutes. An additional characteristic of surge is that it is usually of very low height; perhaps 0.3 ft. is typical. This type of wave motion has been observed along the entire Pacific coast of the United States (Coast and Geodetic Survey, 1943), and in some places, notably Los Angeles Harbor, has been of serious concern to harbor authorities (Calif. Inst. of Tech., 1945).

Although the height of surge may be so small that, coupled with the very great wave length, the motion cannot be visually observed as a wave train, the horizontal water motion may be large, and it is this factor which accounts for the importance of surge in harbor operations. Since the height of the surge wave is very small compared to either the wave length or water depth, the classical Airy wave theory (Beach Erosion Board, 1942) may be applied to the problem with small error. Thus, the horizontal amplitude of water motion is substantially constant from surface to bottom, and is equal to $\frac{HT}{2\pi}\sqrt{\frac{g}{d}}$, where H is the wave height (vertical distance from crest to trough), T is the wave period, d the water depth, and g the acceleration of gravity. The average water particle velocity is therefore $\frac{H}{\pi}\sqrt{\frac{g}{d}}$.

This result may be easily verified in the case of a standing wave by reference to a diagram such as Fig. 1. If at some initial time the wave profile is in one extreme position as shown by the solid line, then a quarter wave period later the entire surface is horizontal, the water above the mean level in each crest flowing to the right and to the left to fill in half of each adjoining trough, and another quarter period later the surface is in the other extreme, as shown by the dashed line. Therefore, in one-half a wave period, a volume of water proportional to the area shown cross-hatched flows through each nodal section of depth d. The average flow rate, Q, is therefore:

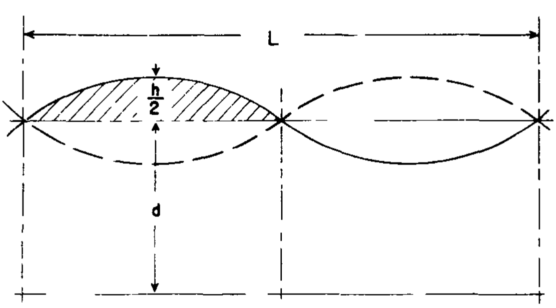


Fig. 1
Diagram for water motion in a standing wave.

$$Q = \frac{2\int_0^{\frac{L}{4}} \frac{H}{2}\cos 2\pi\frac{x}{L}\,d\,x}{\frac{T}{2}}$$

or $$Q = \frac{HL}{\pi T}$$

since $$L = CT = \sqrt{gd}\ T$$

$$Q = \frac{H\sqrt{gd}}{\pi}$$

and the average velocity through the nodal section becomes:

$$V = \frac{Q}{A} = \frac{H\sqrt{gd}}{\pi d} = \frac{H}{\pi}\sqrt{\frac{g}{d}}$$

As a numerical example, a 3-minute period surge, 0.3 ft. in height, would cause oscillatory horizontal water displacements of 9 ft. in a harbor depth of 30 ft., with an average velocity of 0.1 ft. per sec. or 0.06 knot.

Since a ship is small compared to the wave length of a surge (5600 ft. in the foregoing example) it may be expected to move in space with the water motion unless rigidly restrained. Serious damage has occurred where such surge-excited ship motion has been resisted by dock and pier structures. Fig. 2 is a record of typical surge motion and resulting damage at the Terminal Island, California, Navy Base prior to the construction of the mole.

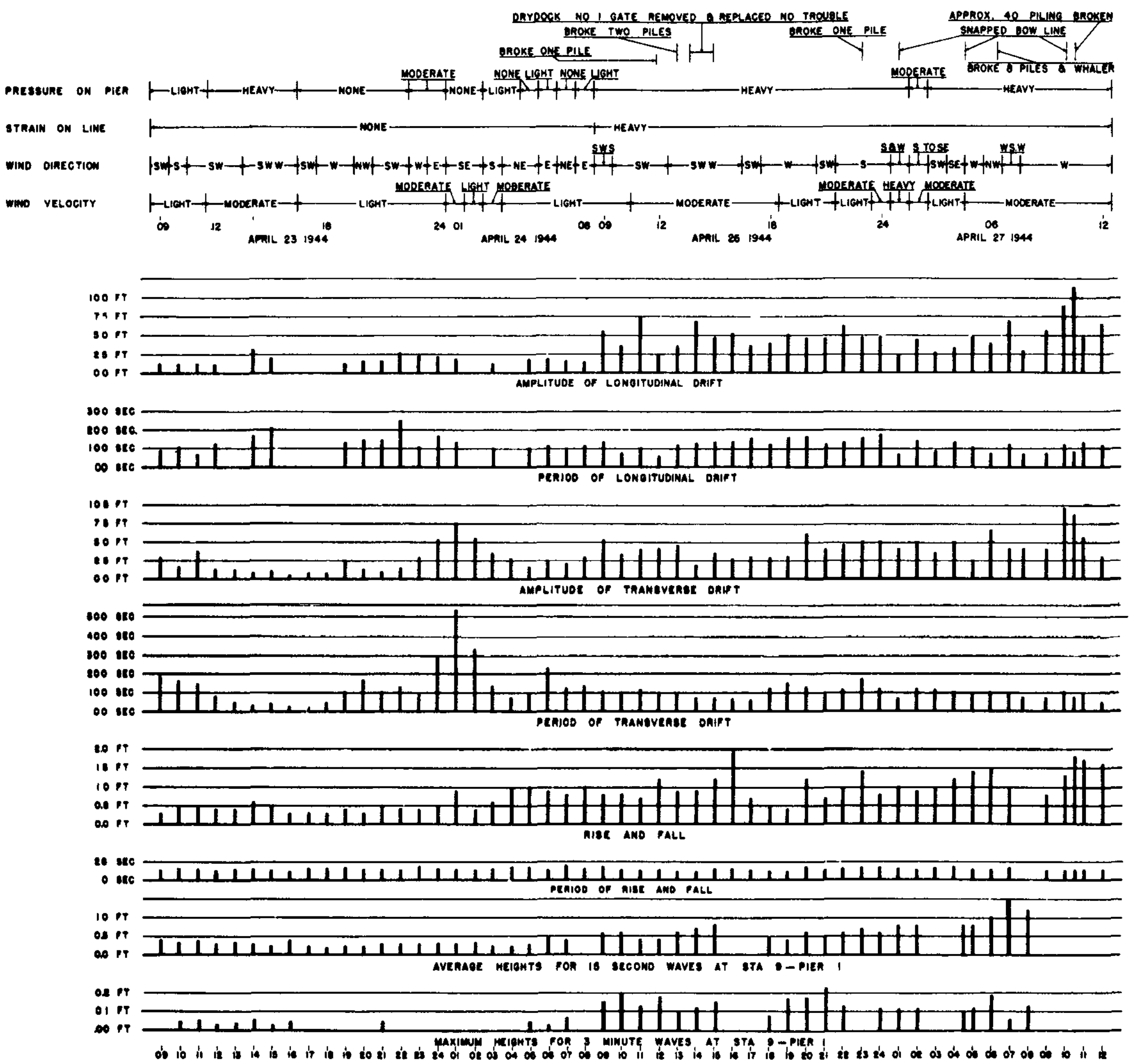


Fig. 2. Record of surge and damage, U.S. Naval Drydock, Long Beach, California.

CHARACTERISTICS OF LONG PERIOD WAVES

Wave Behavior. Like other surface waves, long period waves may be of either progressive or standing type. Because of their large wave length, standing surge waves account for special problems and effects in harbors. A standing wave results from the superposition of two identical progressive wave trains traveling in opposite directions. Where the water particle motions due to each of the wave trains coincide, they mutually reinforce and the motion is doubled; conversely, where the motions are opposed the resultant is zero. The result is that at fixed positions a half-wave length apart the vertical amplitude of the water motion is a maximum, and half way between these antinodal points the vertical motion is zero. In a progressive wave train, the horizontal velocity of partial motion is greatest at points beneath the moving troughs and crests, but in a standing wave the horizontal velocity is zero at antinodes and a maximum at the position of no vertical motion, or nodal points. Fig. 3 illustrates these characteristics of a standing wave.

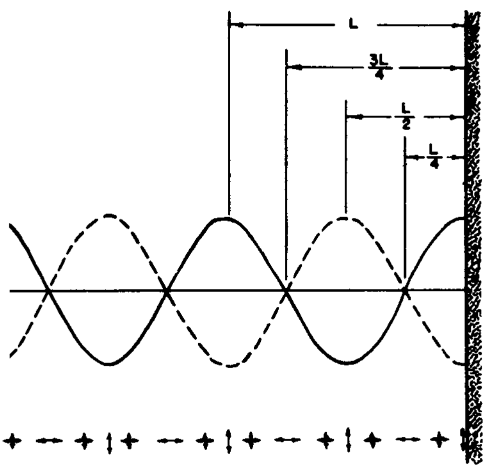


Fig. 3. Profile and water motion of a standing wave.

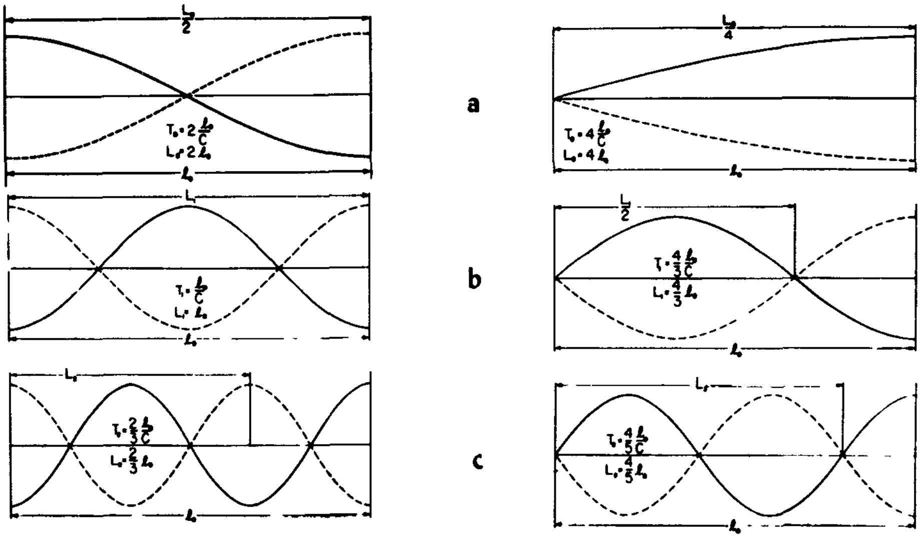
The important distinction between the two wave types as far as harbor surge is concerned is, therefore, that if the surge is due to a progressive wave train, all areas in the harbor experience the same maximum horizontal motion, whereas if the surge is of the standing wave type, there will be distinct areas of active and quiet water.

It is apparent that a standing wave always exists to seaward of a reflecting barrier, the reflected and incident waves combining to produce this result. In this connection, it should be noted that the reflective properties of a shoreline are determined by the scale of the irregularities of the shoreline with respect to the length of the waves. Thus, a given stretch of irregular shoreline will appear "straight" to long period waves, and the incident wave energy will be concentrated in a well-defined reflection, whereas the same shoreline will scatter reflected short period waves in all directions with resulting diffusion of energy.

If two parallel reflecting shorelines are oppositely disposed in a basin, a wave train will travel between them, each wave being successively reflected from one to the other, until damped out by frictional forces. If the distance between the reflecting shorelines is such that the time required for a wave to travel from one boundary to the other is an integer multiple of half the wave period, the standing wave patterns produced by reflection from each boundary will coincide, and a condition of free oscillation of the basin will result. In the case of a basin open at one end to the sea or other large body of water, and closed by a reflecting boundary at the other, it can be shown that the condition for free oscillation will be realized if the length of the basin is such that the wave travel time is an odd-integer multiple of one-quarter of the wave period (Sverdrup, Johnson, and Fleming, 1942). It may be noted that the term seiche is applied to such free basin oscillation.

Basin oscillation, or seiching, is analogous to the motion of a spring-mass system, pendulum, or other mechanical or electrical oscillating system; once started, the motion persists unless brought to rest by outside forces. Fig. 4 illustrates the fundamental and first two harmonic modes of oscillation of basins.

In any type of oscillatory system, if a periodic excitation is applied at one of the free or natural periods of the system, the motion will increase to an amplitude determined by the damping of the system; this phenomena is termed resonance. Thus in the present case, if a long period wave train whose period corresponds to the fundamental or an harmonic period of the basin enters the harbor



Closed Basin Open-Ended Basin

Fig. 4
Modes of basin oscillation
(a) Fundamental, (b) First Harmonic, (c) Second Harmonic

from the open sea, a condition of resonant basin oscillation will result. The significance of resonant basin oscillation is that all of the wave energy coming into the area is concentrated in one standing wave system, with resulting build-up of large amplitude vertical and horizontal water motion at the positions of nodes and antinodes. This amplification may result in a serious surge condition in a harbor basin even though the exciting long period wave train is nearly totally excluded from the harbor by a breakwater or mole. Thus, model studies of the Terminal Island Mole Basin (Calif. Inst. of Tech., 1945) have shown wave heights within the mole 50% greater than those in the outer harbor for critical surge periods.

Where the water depth in a basin is substantially constant, the velocity of shallow water, long period, waves may be taken as $\sqrt{gd}$, and the previously mentioned conditions for resonant periods become:

For closed basins:

$$T = \frac{2\,\ell}{(k+1)\sqrt{gd}}$$

where ℓ is the basin length
$k = 0,1,2,3,$ etc.

For open-ended basins:

$$T = \frac{4}{(2k+1)} \quad \frac{\ell}{\sqrt{gd}}$$

Where the water depth is not uniform, the critical periods may be computed by numerical integration of the equations in the following form:

For closed basins:

$$T = \frac{2}{k+1}\int_0^{\ell} \frac{d\,x}{\sqrt{gd_x}}$$

For open-ended basins:

$$T = \frac{4}{2\,k+1}\int_0^{\ell} \frac{d\,x}{\sqrt{gd_x}}$$

where d_x is the depth at distance x from the end of the basins.

These relations have been verified by model experiments for the cases of uniform water depth, and parabolic variation of water depth, at the University of California (Johnson, 1949). These experiments also investigated the damping effect of sills located at nodal points in the standing wave pattern, with the result that a sill of height approximately one-third the water depth was found to reduce the wave amplitude by approximately 50%.

Sources of Long Period Disturbances. The question of the sources of the long period wave disturbances which cause surge problems in harbors is one which can not be answered completely at the present time. Early thinking on this subject was confined to the assumption that surge was a phenomenon always related directly to the basin in which it occurred, hence was a free oscillation of the basin following some initial disturbance such as a piling up of water due to winds, atmospheric pressure anomalies, or local seismic activity. This explanation is indeed the only one admissible for the case of seiching of completely land locked basins such as lakes, but recent observations of surge activity in coastal regions have demonstrated the inadequacy of this theory as a general condition. The existence of surge conditions in harbors at intervals completely uncorrelated with local atmospheric or seismic disturbances has focused attention on the proposition that long period, low amplitude wave trains exist in the open sea, and where these wave trains enter a harbor they constitute a surge. If the wave period coincides with the natural period of the harbor basin, the waves will excite resonant oscillation, with resulting increase in the severity of the surge.

One obvious source of such long period wave trains in the sea are seismic sea waves or "tsunamis" generated by distant submarine earthquakes, Fig. 5. This source is quite limited in frequency of occurrence, but the recently developed theory of "surf beat" due to Munk (1949), offers a source of long period sea waves that is quite general in occurrence and so satisfies the observations of frequent periods of surge in coastal regions.

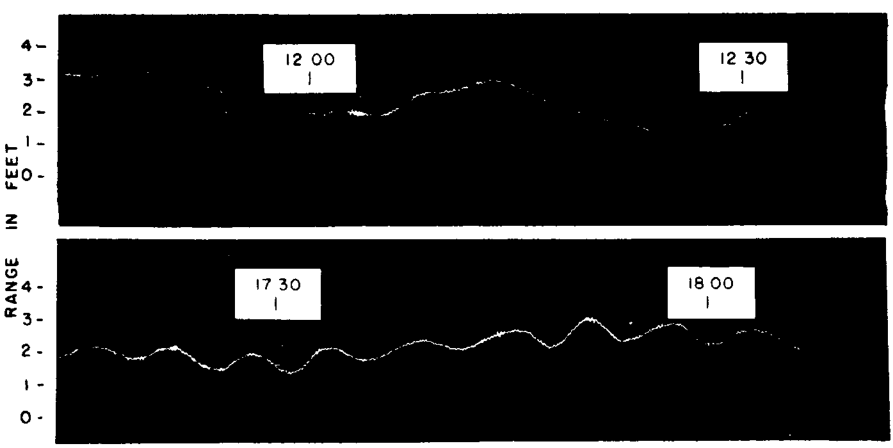


Fig. 5
Marigram record of 20- and 6-minute surges at Terminal Island due to the Alaskan Earthquake of April 1, 1946

The theory of surf beat indicates that in the process of wave breaking, a small fraction (approximately 1%) of the wave energy which enters the breaker zone is reflected back to sea as a long period wave. This behavior is due to the nonlinearity of the wave breaking process; the phenomenon of frequency demultiplication being a characteristic of non-linear systems (Den Hartog, 1940). Thus, a storm in some remote region of an ocean basin may produce a period of high surf along some distant shoreline, and the long period waves radiated from this shore may then travel across vast ocean distances in the same manner as tsunamis to produce a surge condition in harbors thousands of miles from the generating area. The theory of surf beat has been verified by experiments at the Scripps Institution of Oceanography (Munk, 1949); it remains to determine its importance as a source of harbor surge. The techniques for such an investigation are available and should be put in operation, since if it is as important as the foregoing discussion assumes, it should be possible to forecast periods of surge by a combination of present-day techniques for wave forecasting, to determine generating areas, and for tsunami warning, to determine the path of the long period waves.

REMEDIAL MEASURES

Since the principal problem associated with harbor surging is unwanted ship motion, especially where ships are moored in close proximity to fixed structures such as docks or piers, alleviation of the problem will be obtained only by the reduction of ship motion to a tolerable minimum. Two approaches to this requirement are apparent; (1) the reduction in amplitude of the surge waves in the harbor, and (2) the development of mooring systems which will limit the amplitude of ship motion.

In connection with the first named approach, the reduction of breakwater gate width, or harbor opening, to the minimum required for navigational purposes will reduce the amount of long period wave energy which can enter the harbor from the sea, with resulting reduction in surge activity in the harbor. By way of illustration, model studies of the Navy Mole Basin at Terminal Island, California, indicated that a reduction in gate opening from 2000 feet to 600 feet resulted in approximately 50% reduction in surge amplitude in the basin (Fig. 7). The location of the harbor entrance is not so important in reducing entering surge excitation, since the entrance width is usually small compared to the wave length of the long period surge waves. As a result, the diffraction phenomena at the entrance are relatively independent of the direction of wave approach, and on the harbor side the transmitted wave energy is directed nearly uniformly in all directions.

Model studies are also useful in surge problems since they enable the mapping of water motions in all parts of a harbor for any assumed surge condition, and thus permit the rational choice of "quiet" areas for dock and pier location. Since the degree of water motion may vary by a factor of 10 to 1 in various parts of a harbor due to the characteristic standing wave patterns which are produced, model studies may make significant contributions to a harbor design project. Figs. 6, 7, and 8, which are the results of model studies made for the U.S. Navy, Bureau of Yards and Docks, illustrate this behavior for the cases of Terminal Island and Apra Harbor, Guam (Calif. Inst. of Tech., 1949). In Fig. 6 the results of a disturbance survey of the entire Terminal Island mole basin are presented in the form of contours of equal vertical amplitude. It is apparent that for surge periods of 3 and 6 minutes the harbor is divided into sharply delineated areas of minimum and maximum activity, whereas the disturbance level is nearly uniform throughout the harbor for ordinary sea waves of 15-sec period. In Fig. 7, the vertical amplitude in a particular area near the Navy piers is plotted as a function of wave period. The peaks of activity correspond to the several resonant periods of the basin. Fig. 8 summarizes the results of a horizontal water motion survey of Apra Harbor for surge periods of 1.9 and 3.8 minutes, corresponding to two resonant periods of the harbor. Again, a wide variation in degree of water motion within a short distance is exhibited.

The second approach to the solution of harbor surge problems, that of improvements in ship mooring practice, has received very little attention as yet. To be successful, a system must be developed that is virtually rigid, since if the ship moves at all, it acquires kinetic energy, and can then only be brought to rest by the conversion of this energy into strain energy of deflection of the mooring system. The catenary suspensions of cable mooring systems, even extraordinary ones such as shown in Fig. 9, which were used at Terminal Island prior to construction of the mole, may permit deflections as large as six or eight feet, hence some entirely new type of mooring will have to be devised.

It is also important that the elastic constants of the mooring system be proportioned to the mass of the ship so that the natural period of the ship-and-mooring system is not close to the expected period of harbor surge, since if these

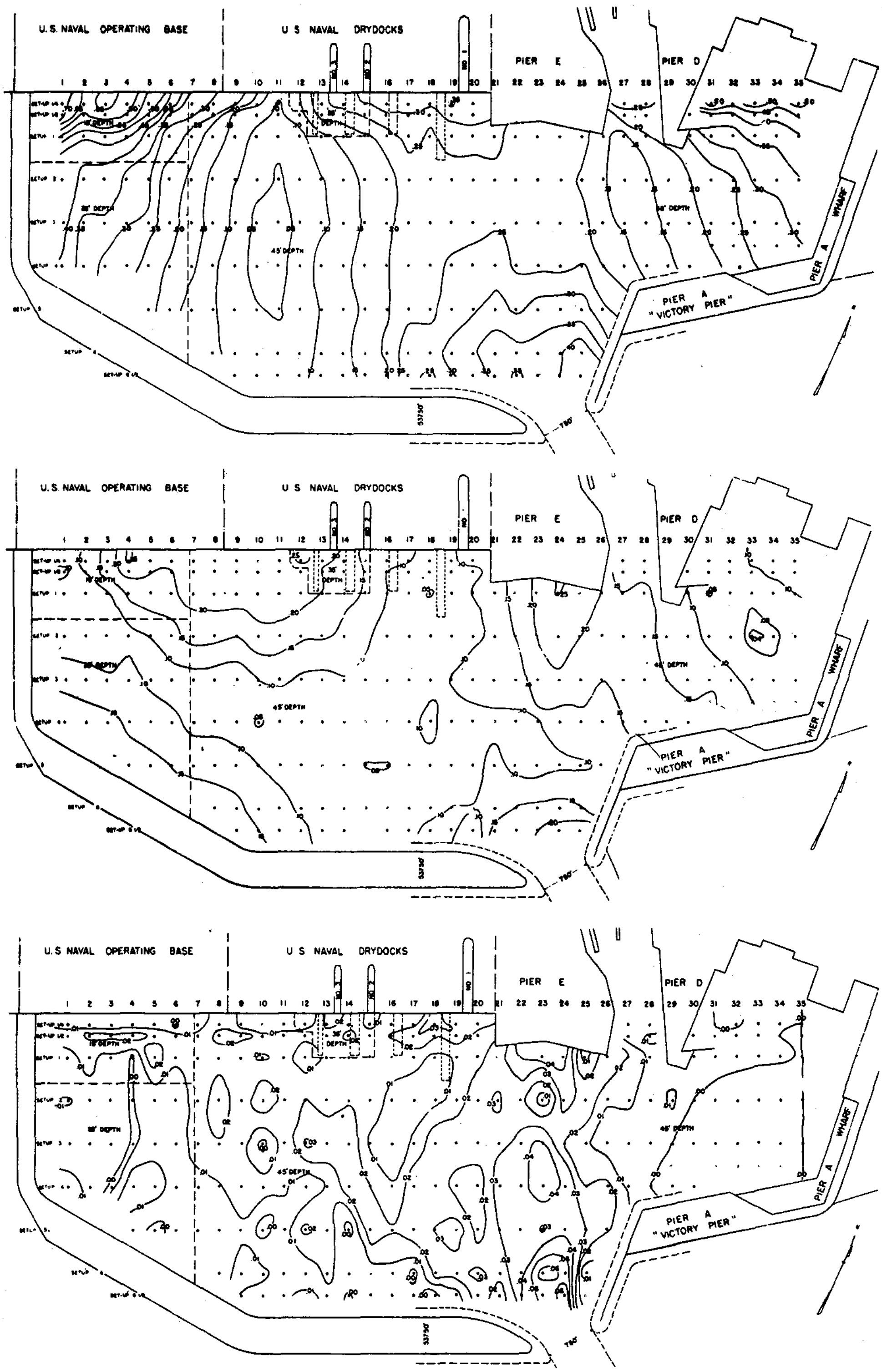


Fig. 6
Contours of vertical amplitude, Terminal Island Mole Basin for three wave and surge periods
(a) 6-minute (b) 3-minute (c) 15-second

periods are nearly equal the ship will be driven at resonance by even small surge waves with resulting large amplitude ship motion and high stress in the mooring system. The effect of the elastic properties of the mooring system on ship motion and mooring stress recently has been analyzed in great detail by Wilson (1950).

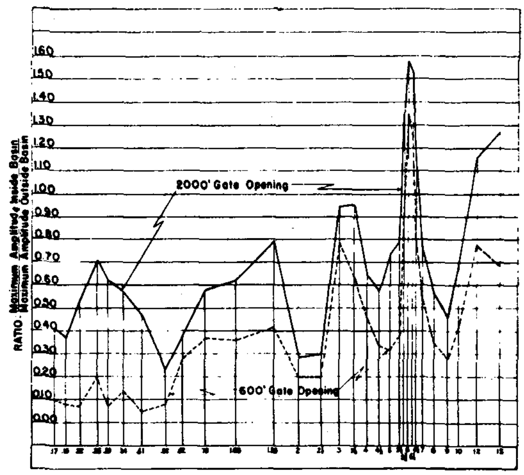
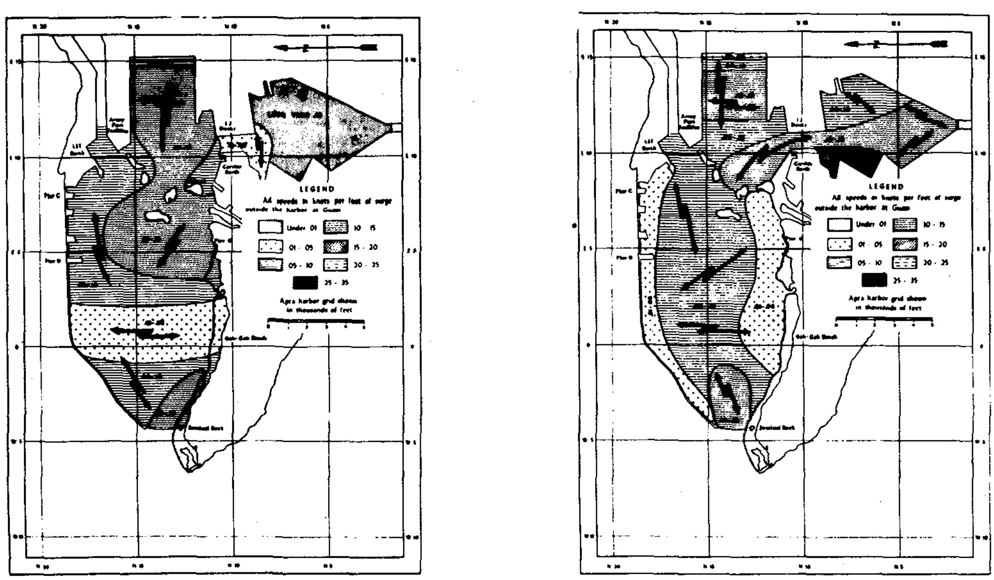


Fig. 7. Vertical amplitude as a function of wave period in vicinity of Navy Piers, Terminal Island, California



1.9-minute period

3.8-minute period

Fig. 8. Horizontal water motion due to surge, Apra Harbor, Guam, M.I.

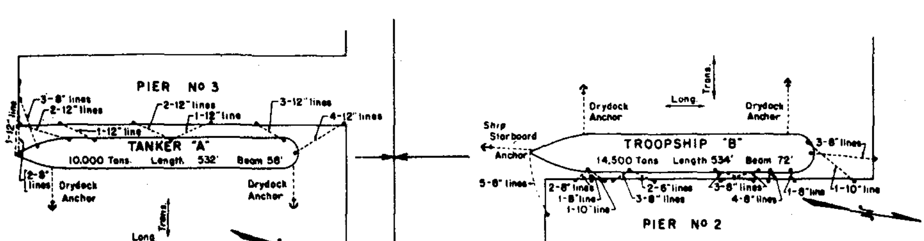


Fig. 9. Typical mooring systems

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PART 2
BASIC DESIGN DATA

CHAPTER 7

WAVE RECORDERS

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INTRODUCTION

Several satisfactory instruments are available for recording the height and period of ocean waves, and new improved gages for this purpose are being designed. The actual procurement of wave data is no longer a major problem, but the present theories interpreting these data and the methods of data analysis leave much to be desired. Definitions of characteristic wave height and wave period are vague, as no specific period of observation is designated for determining these measurements. Analysis techniques and results are inconsistent. Preliminary studies of the statistical distribution of wave heights are encouraging, but no simple method of describing the waves with regard to period has been developed. Current hydrodynamic wave theory is apparently in error, and reexamination of this basic theory in regard to the hydrodynamic attenuation factor should be made.

WAVE HEIGHT AND WAVE PERIOD RECORDERS

To obtain comparable experience in the installation, operation and servicing of the significant wave recorders developed prior to 1947, the Beach Erosion Board (1948) tested simultaneously eight different types of wave gages at Atlantic City, New Jersey during May, 1947. The instruments tested were as follows:

1. Underwater Type
    a. University of California Mark III Shore-wave recorder
    b. Woods Hole Shore-recording Wavemeter
    c. Inverted Echo Sounder
2. Surface Type
    a. Float-operated Recorder
    b. Parallel-wire Gage (large wire type)
    c. Parallel-wire Gage (small wire type)
    d. Step-resistance Gage (series type)
    e. Moving-picture Camera
3. Aerial Type
    a. Stereoscopic Cameras

A summary of these tests with a critical analysis of each gage tested appears in a publication of the Beach Erosion Board (Caldwell, 1948). This publication concludes that,

> "....none of the gages, in the form tested, were satisfactory measuring instruments adapted to a long-term measurement program of surface waves. Both underwater pressure gages were satisfactory for the production of records of bottom pressure changes, but the transfer of these records to water surface fluctuations was unsatisfactory. The echo-sounder was found to give an erroneous record. The parallel-wire gages were not found to be adapted to the study because of structural defects and the fluid nature of the calibration curves. The step-resistance gage met all requirements for a short period, but was found to become sluggish and unreliable in continued use. Stereo or movie camera records, although giving accurate pictures of the sea surface, were not considered adapted to a long-term measurement program because of processing difficulties. It was concluded that the step-resistance type gage held the most promise for development into a satisfactory gage with a minimum expenditure of time and effort."

Accordingly, the existing step-resistance gages were modified and improved by the Beach Erosion Board and two types (series type and parallel type) were developed and are in use today. The development of underwater pressure recorders and improved procedures for analyzing pressure records has been continued by the Scripps Institution of Oceanography, the University of California, Institute of Engineering Research, Berkeley, and Woods Hole Oceanographic Institution. As the Beach Erosion Board (Caldwell, 1948) thoroughly reviews the advantages and disadvantages of the significant recorders developed prior to 1947, the remainder of this discussion will be concerned only with recorders used or developed since 1947.

STEP-RESISTANCE GAGE

The step-resistance gage comprises a series of electrical contact points (modified spark plugs) installed at 0.2 ft. intervals along a sealed pipe. The spark plugs are connected to a resistance circuit housed within the pipe. The gage is attached in a vertical position to a supporting structure, such as a pier, with the bottom below the lowest expected wave trough. As the top of the gage must be above the highest wave crest, 25 ft. normally is allowed for its length.

Power is supplied to the gage through a 115 volt a.c. constant voltage transformer, the primary of which is connected through a timing switch to provide automatic programing. This alternating current, supplied to the gage to prevent polarization, is converted through a selenium bridge rectifier to a proportional d.c. current, which drives the recording unit mechanism. A Brush magnetic pen recorder, which has a high frequency response and is capable of recording the shortest period wave, generally is used.

The values of the resistors connected to the contact points of the gage are adjusted so that a straight-line variation exists between the current and the length of the gage submerged. Therefore a record obtained of the variation of the gage current is also a record of the rise and fall of the sea surface. Such a record includes tide stage as well as wave height. Wind chop and wave form are also included in the record providing the response of the recorder is sufficiently rapid.

Three step-resistance gages are in operation on the Pacific Coast at the present time. Located along the California Coast, the first of these gages was installed in May, 1948 at Huntington Beach; another was installed in July, 1948 at El Segundo, and the third was installed during December, 1949 at Mission Bay. With periodic cleaning at four to six month intervals, these recorders have been in continuous operation. Other gages of the step-resistance type have been installed along the Gulf of Mexico and the Atlantic Coast.

Series type step-resistance gage. In the series type gage, the resistors are connected to form a series circuit with the junctions between resistors tied to the contact points. As the sea rises, it shorts all resistors tied to submerged contact points, causing an increase in current proportional to the number of contacts below the surface.

Due to the relatively high value of the resistors connected between the contact points, the film which receding water leaves on the instrument makes it susceptible to current leakage. For this reason, the series type resistance gage is restricted to measurement in fresh water where current leakage caused by the water film is negligible.

Parallel type step-resistance gage. In the parallel type gage, one end of each resistor is connected to a spark plug, the other end being connected to the gage voltage supply. The sea serves as a current path between the contact points and a ground rod, which is connected to the other side of the voltage source. As the spark plugs are submerged, the resistors are added in parallel. The values of these resistors are so selected that the current flowing in the gage is proportional to the number of contact points submerged.

The accumulation of water film does not affect the operation of this type step gage as the resistance values are small in comparison with the water film resistance. However, as the resistance path between the contact points and the ground rod must be small in comparison to the resistor value, the parallel type step gage is restricted to measurement in salt water.

PRESSURE TYPE GAGES

Surface waves are often measured by recording the subsurface pressure fluctuations and translating these fluctuations to the surface by means of theoretical relationships. The relationships between surface wave characteristics and recorded pressure variations, and the methods of analyzing these pressure records are discussed below.

To obtain a pressure record, a transducer, which converts water pressure fluctuations into electrical current or voltage signals is used to record the pressure fluctuations. The various types of transducers designed for this purpose utilize inductance bridges, strain gages, thermocouples, potentiometers attached to bellows, coils in magnetic fields, and capacitive bridges. Housed in a waterproof case known as a pressure head, the transducer is placed at some point below the water surface and connected by an electrical cable, often several miles long, to a suitable recording unit located on shore. In general, these pressure type "shore wave recorders" are designed to register pressure variations on a continuous strip chart without distortion of the pressure wave form. The system should, therefore, respond linearly with pressure and have a speed of response sufficient to follow the maximum rate of change of the pressure being recorded. The chart paper is divided into uniform rectilinear, or curvilinear, divisions, and the recording mechanism is usually the direct writing type (ink or hot wire). Operation of the system is made simple as unskilled personnel often attend the shore installation. Automatic programing devices operate the recorder according to prearranged schedules.

As a basis for comparing the various pressure type shore wave recorders, the following outline is presented. (This outline of requirements was made prior to building the Mark IX instrument, designed to incorporate the most desirable features of the pressure-type recorder, which is now under development at the University of California.)

A. Record requirements -- reproduction of subsurface pressure variations without distortion.

   1. Uniform response to all wave periods to be recorded.
   2. Linear response with pressure.
   3. No response to temperature variations.

B. Chart requirements.

   1. Direct writing -- pen or hot wire recorder.
   2. Rectilinear coordinate paper.
   3. A chart paper roll sufficient for 36 hours of continuous recording.
   4. Two-speed chart drive to provide continuous recording with automatic fast speed sampling.

C. Remote recording requirements.

   1. Up to five miles of cable.
   2. Adaptable to radio and telephone telemetering.

D. Power requirements.

   1. 115v. a.c. or 115 v.d.c. power line input.
   2. Capable of emergency battery-operation (dry batteries preferred).

E. Operation requirements.

   1. A simple routine to be performed by unskilled operators.

      a. Mark date on charts at specific times.
      b. Change charts.
      c. Wind clocks regularly.

   2. Completely automatic operation.

a. Automatic program timing of fast and slow chart speeds.
b. Warning devices for replacement of charts and winding of clock.
c. Complete fuse protection.

F. Cost requirements.

1. Low cost of manufacture (use of standard parts whenever possible).
2. Inexpensive and expendable pressure head (the more expensive parts and equipment should be reserved for the shore unit).
3. Simple and inexpensive means of analyzing records and calibrating the equipment.

G. Calibration requirements.

1. Static calibration valid for all records.

a. Calibration independent of wave period (minimum wave period of three seconds).
b. Calibration independent of depth of installation (maximum depth of 100 feet).
c. Calibration independent of temperature of water.
d. Calibration independent of input voltage when power line is used (maximum variation of 95 to 135 volts).

2. Simple calibration equipment -- mercury manometer and source of air pressure.

H. Pressure head requirements.

1. Two-years operation without maintenance (fatigue life of 5 million cycles).
2. Light weight and rugged construction for easy handling.
3. Uniformity of operation regardless of position.
4. Inexpensive and expendable.

Mark III shore-wave recorder -- University of California. The transducer of the Mark III recorder (Chinn, 1949) consists of a potentiometer linked to a bellows system which is acted upon in one direction by the average pressure through a slow leak and in the other direction by dynamic pressure. This pressure head senses the pressure variations due to the waves but does not respond to tide changes. The shore installation comprises a special bridge and electronic power supply and an Esterline-Angus pen recording milliammeter. The advantage of this recorder is that its calibration is linear and independent of wave period and water depth, and that the system can be operated from 36 volt batteries with a total load current of 90 milliamperes. However, there are several disadvantages. The pressure head is heavy, difficult to handle and is easily damaged. The life of the potentiometer is limited to about six-months operation.

Mark V shore-wave recorder -- University of California. The transducer of the Mark V recorder (Isaacs and Wiegel, 1950) comprises a 32 junction thermocouple installed in a gas-filled rubber bellows. The reference junctions are in contact with the sea through the pressure head case while the active junctions are acted upon by the gas. The pressure fluctuations of the sea act upon the bellows producing temperature fluctuations in the gas and generating voltages through the thermocouple. As the average gas temperature adjusts and becomes that of the sea, average pressure and tides are not registered by the gage. The shore installation connected to this pressure head is a Leeds and Northrup recording millivoltimeter. The chief advantage of this system is its inexpensive components. The pressure head is simple in design, inexpensive, rugged and easily handled, and the shore installation consists only of a standard recorder. Chief among the disadvantages of this recording system is the difficulty and expense encountered in calibrating the equipment and analyzing the records. Calibration of the instrument is not independent of wave period, depth of installation, and short period temperature fluctuations. Also disadvantageous is the fact that gradual leakage of moisture into the rubber bellows cuts the life expectancy of the pressure head to an average period of about three months.

Woods Hole shore recording wave meter -- Woods Hole Oceanographic Institution. The transducer of the Woods Hole Recorder (Klebba, 1949) consists of a coil and magnet arranged so that the total magnetic flux linking of the coil varies as the water pressure fluctuates. This is accomplished by attaching the coil to a bellows which is deflected by the pressure fluctuations. A slow leak applies average pressure to one side of the bellows while the other side is acted upon by the dynamic pressure. Tide changes are not recorded. Recording of the flux linkage of the coil can be made with a General Electric photo-electric recorder, a direct writing servomechanism type recorder, or with the Woods Hole photographic recorder. The latter provides a record that can be used directly with the frequency analyzer developed at that institution.

RECORDERS CURRENTLY UNDER DEVELOPMENT

The Scripps Institution of Oceanography has mentioned in their progress reports the development of a recorder which utilizes strain gages in the transducer and is referred to as the Mark VIII shore wave recorder. Preliminary models have been built and now are undergoing field tests.

The University of California, Berkeley, is developing two additional wave gages; (1) the Mark VI shore wave recorder and (2) the Mark IX shore wave recorder. The Mark VI, still in the laboratory, utilizes strain gages to measure the deflection of a flat plate diaphragm which is acted upon by the water pressure. One side of the plate is sealed at atmospheric pressure so that the gage records total pressure. Electrical balancing will enable cancellation of the static pressure reading; tides will be recorded. The recorder is intended to be very rugged and have a high frequency response necessary to record pressure variations in and near the surf zone. The shore unit of the recorder will be a standard Brush magnetic pen recorder. The Mark IX shore wave recorder is an improved version of the Mark III potentiometer type gage designed to be a general purpose instrument with a long life for permanent installations. The shore recording unit is an Esterline-Angus recording milliameter. Two models of this recorder are now undergoing field tests; one at Elwood, California and the other at Point Arguello, California.

WAVE DIRECTION MEASUREMENT

No reliable instruments have been built to date which will measure wave direction. The Rayleigh disk, which was considered a possible solution to this measurement problem, has been investigated and proven unsatisfactory both from an experimental and theoretical standpoint. A study of the behavior of this disk under the action of two or more wave systems was recently completed by the Beach Erosion Board (Hall, 1950). The conclusions of the Beach Erosion Board (Hall, 1950) were:

> "It appears that the records made under natural conditions confirm as nearly as might be expected the theoretical analysis of the behavior of the disk when acted upon by two sine wave systems.
>
> It also appears from the study that the presence of two or more wave systems, the presence of strong currents other than wave currents such as rip tides, and the uncertainty in analysis of the records proves the Rayleigh Disk to be unreliable as a wave direction indicator."

Wave direction can be determined from aerial photographs providing suitable lighting conditions exist and that the water surface is not confused by local wind chop. Visual observation of the sea from any vantage point will enable an approximate determination of wave direction but again local wind may confuse the surface. Suggested as a possible solution to this problem has been the placement of three pressure recorders in a triangle and timing the arrival of the waves at the various recorders. Difficulty may arise with this method due to the short crestedness of ocean waves. Waves recorded at one corner of the triangle may not be evident, or may be of a different wave form, at another corner of the triangle, making timing of the wave difficult.

ANALYSIS OF WAVE RECORDS

Due to the irregularity of ocean waves in height and period, the definitions of terms employed in describing the waves is very important. The following basic definitions have been accepted (Folsom, 1949):

1. Wave height is the vertical distance between the crest and the preceding trough.

2. Characteristic wave height is the average height of 30 percent of the highest waves.

3. Wave period is the time interval between the appearance at a fixed point of successive wave crests.

4. Characteristic wave period is the average period for the well-defined series of highest waves observed.

5. Wave direction is the orientation of the line of travel of the largest well-defined waves.

The definitions of characteristic wave height and period are incomplete in that the length of observation is not specified. A minimum period of ten minutes has been suggested by Scripps Institution of Oceanography (Munk, 1944) while a minimum of twenty minutes has been suggested by Folson (1949). Little evidence has been published as to the effect of the observation period but, as in measuring any statistical quantity, the longer the period taken the better should be the results providing conditions remain constant. However, using the longer period would mean greater expense in regard to the amount of chart paper to be used in recording data and the amount of time required for analyzing the records.

Analysis of wave records for wave height. In analyzing the wave records made at various locations along the Pacific Coast during the last several years, the following step-by-step procedure has been used by the University of California, Berkeley:

1. Select a continuous record of about a twenty-minute duration which was made while the recorder was operating at fast speed (three inches per minute) and at approximately the time for which wave information is desired.

2. Determine the total number of waves recorded during the selected interval by:

   a. Measuring the wave period of all well-defined waves in the selected interval of the record and computing an average value.
   b. Computing the total number of waves by dividing the determined average wave period into the total seconds covered by the record interval.

3. Determine the significant wave height by:

   a. Measuring the height of the highest 30 percent of the total number of waves in the selected interval.

   b. Computing the average height of the measured values.

4. Determine the average height of the highest 10 percent of the waves as in step number three.

5. Determine the maximum wave height in the selected twenty-minute period.

Excellent correlations have been made of ratios obtained from wave height data recorded daily at several points along the Pacific Coast and analyzed according to the above procedure (Wiegel, 1949). The ratio of the average height of the highest 10 percent to the average height of the highest 30 percent has been found to be 1.29. This ratio was determined from wave data recorded at Point Arguello, California to be 1.30; at Point Sur, California to be 1.27; and at Heceta Head, Oregon to be 1.30. A substantial number of the daily values of this ratio agreed within 10 percent with the average value derived from readings taken over a fourteen-month period at Point Sur and Heceta Head, and over a three-month period at Point Arguello.

Also from the above data, the average ratio of the maximum wave height to the average wave height of the highest 10 percent was found to be 1.46; while the average ratio of the maximum height to the average height of the highest 30 percent was found to be 1.87. Again a substantial number of the daily values of these ratios compared favorably, agreeing within 20 percent, with the average value. In a tabulation of wave data taken from two widely separated stations in the Atlantic (Cuttyhunk, Massachusetts, and Bermuda) by Seiwell (1947), a constant of 1.57 was found for the average ratio of the average height of the highest 30 percent to the average height of all waves.

The agreement of daily values to the average values of various ratios discussed above and the agreement among values determined at widely separated stations indicated that a definite statistical distribution of wave heights is generated by a storm.

Evidence to further substantiate this theory is found in the results of a statistical analysis conducted by Putz (1950) at the University of California, Berkeley. Analysis was made of twenty-five wave records selected from various localities and made at various times of the year to obtain good sampling. Putz (1950) found evidence that the statistical frequency distribution of observed wave height in a twenty-minute interval is approximately constant in form and, for a first approximation, requires for its complete description only the determination of a typical height, such as the "significant wave-height." The wave-height distribution of all twenty-five pressure records matched, with reasonable accuracy, a Pearson Type III frequency function with a 0.8 positive skewness and proportionality of the mean and the standard deviations.

Utilizing this mathematical model, Putz (1950) computed values for ratios reported by Wiegel (1949) and Seiwell (1947). The value of maximum wave height determined from the model was taken as the probable maximum wave in two twenty-minute intervals as used by Wiegel (1949) in determining his daily maximum wave height. Excellent agreement was found among these three sources as shown in Table I.

TABLE I

Comparison of Wave Height Ratios
for Various Pressure Recorders
and a Frequency Function

Basis of Calculations	Computed Ratios				Remarks
	$\frac{H_{1/3}}{H_{ave}}$	$\frac{H_{1/10}}{H_{1/3}}$	$\frac{H_{max}}{H_{1/3}}$	$\frac{H_{max}}{H_{1/10}}$	
Point Arguello, California wave recorder		1.30	1.85	1.42	3 months of data
Point Sur, California wave recorder		1.27	1.85	1.46	14 months of data
Heceta Head, Oregon wave recorder		1.30	1.91	1.47	14 months of data
Cuttyhunk, Massachusetts wave recorder	1.57				10 months of data
Bermuda wave recorder	1.57				4 months of data
Average of wave record values	1.57	1.29	1.87	1.46	
Pearson Type III frequency function MODEL	1.57	1.29	1.81	1.41	Model based on 25 selected records

Analysis of wave records for wave period. Analysis of wave records for the characteristic period is accomplished by measuring the average period of the larger, well-defined waves appearing on the record. This is comparable to measuring the characteristic height of the waves by determining the average height of the highest 30 percent of the waves. The characteristic period of the waves does not describe the period-distribution, however, as the characteristic height describes wave-height-distribution. Although wave heights have been found to follow a simple mathematical distribution even though the waves may be arriving from two or more storm areas, wave periods do not follow a simple distribution if more than one generating area exists. Additional information is needed to adequately describe wave periods.

The need for more accurate methods of analyzing wave periods has led to the development of two types of electrical-mechanical analyzers, (1) a frequency analyzer and (2) an auto-correlation function analyzer.

The frequency analyzer measures the presence of the various sinusoidal frequency components in the record and produces a frequency distribution curve. Even though this analysis may give an accurate mathematical representation of the data, the validity of its physical representation has been questioned by Seiwell (1949, 1950). A study of the frequency distribution curves of pressure type wave recorders by the Admiralty Research Laboratory (1947) and later by Munk (1947a, 1947b) indicates that this type analysis is useful in tracking storms and in correlating meteorological and wave data.

The second type of analyzer, which is based on the auto-correlation function, has been investigated at the Marine Physical Laboratory, University of California (Rudnick, 1949) and at the Woods Hole Oceanographic Institution (Klebba, 1949). Although still in the process of development, this method shows promise of more accurately describing the physical characteristics of surface waves than the frequency analyzer.

Analysis of under-water pressure records. The analysis of pressure records for wave period is the same as the analysis of surface wave records. The records differ, however, in that the short period waves are not registered to the same degree as the long period waves by pressure recorders due to the hydrodynamic pressure attenuation of the water. As a result, many of the shorter period waves may not appear on the pressure record.

If the technique of measuring the periods of only the larger, well-defined waves of the record is followed (as described in the above section), the measured period will be approximately the same as would be obtained if the record were made with a surface type gage. For locations on the exposed coast, the short period waves, not recorded by pressure, generally are generated by local wind. Irregular and of small amplitude, these waves are neglected in the analysis of surface records.

In several cases, attempts have been made to utilize the hydrodynamic attenuation of short period waves by installing gages in deep water (about 600 feet) so that only the waves of long periods (the characteristic forerunners of storms) will be recorded. These long period waves are recorded by pressure heads installed in shallow water, but are "lost" in the record of shorter period waves. Installations of this type of instrument have been made, but due to instrument difficulties no satisfactory records have been obtained.

To obtain the surface wave heights from the pressure record, two factors are required; (1) the calibration of the instrument and (2) the pressure response factor relating the subsurface pressure fluctuations to the surface wave. Thus, if

H = wave height at the surface (in feet);

C_1 = calibration factor of the instrument (expressed in feet of water pressure variation per chart division);

K = pressure response factor based on the depth of the instrument, the depth of the water and the length (or period) of the wave being recorded;

R_1 = reading of the instrument;

the following equation is used to obtain the surface wave height:

$$H = C_1/K\ (R_1) \qquad (1)$$

The calibration factor for most instruments in use today is a constant independent of wave period and depth of the instrument. The instrument provides a record of the pressure variations at the instrument which is accurate in amplitude and wave form.

The relation of the subsurface pressure fluctuations to the surface wave has been determined theoretically for two dimensional, irrotational motion of an incompressible fluid in a relatively deep channel of constant depth (Folsom, 1947). The response factor K has been shown to be:

$$K = \frac{\cosh 2\pi d/L\ (1 - z/d)}{\cosh 2\pi d/L} \qquad (2a)$$

where

z = depth at which the pressure variation is being measured (in feet),

d = depth of water at the instrument (in feet),

L = length of the surface wave (in feet).

When z = d, the pressure variation is measured at the bottom and equation 2a reduces to:

$$K = \frac{1}{\cosh 2\pi d/L} \qquad (2b)$$

Pressure records do not enable the direct measurement of wave length; the wave length must be calculated from the wave period using the following equation:

$$L = \left(\frac{gT^2}{2\pi}\right) \tanh 2\pi d/L \qquad (3)$$

Where T = wave period (in seconds).

Suitable graphs and tables (Wiegel, 1948) are available for the solution of these equations. Graphs have been prepared which enable the response factor (K) to be determined if the water depth (d), instrument depth (z) and wave period (T) are known. Two errors arise when the above equations are used to determine the response factor (K) for ocean waves; (1) an average or characteristic period must be used in the equation while the actual wave period is continuously varying and individual waves are not sinusoidal in form, (2) wave heights computed from these equations have been shown by several observers to be from six to twenty-five percent too low.

Considering the first of these two errors, greater accuracy probably could be attained if the pressure response factor (K) were determined for each wave and the equivalent surface wave were individually computed. This procedure might be feasible from a practical standpoint if the statistical distribution of wave height and wave period could be established so that fewer waves need be analyzed to completely describe the state of the waves. (See the above section on "Analysis of wave records for wave height").

The second of these two errors emphasizes the need to reconsider the basic theory which does not agree with experiment. Every observer who has simultaneously measured the surface waves and the subsurface pressure fluctuations has found the theoretical response factor determined from equation 2a to be too small. Ten random measurements made at the Waterways Experiment Station (Folsom, 1947) indicated an average correction of 1.07 should be applied to equation 1. Seventeen laboratory measurements at the University of California, Berkeley, indicated an average correction of 1.10 (1949). Field data reported by the Woods Hole Oceanographic Institute (Admirality Research Laboratory, 1947; Seiwell, 1947) indicated a correction factor in excess of 1.20 while the three sets of field data obtained at the University of California (Folsom, 1946) indicated values of 1.06, 1.08, and 1.18.

EXISTING WAVE DATA

A summary of the periods of time for which records are known to exist at various localities along the Pacific Coast of the United States is presented below. This information may prove of value to engineers engaged in the design of structures at certain localities where wave data, however meager, will be of assistance.

For the past few years a number of wave recorders have been installed and operated along the Pacific Coast more or less continuously by various institutions and government agencies. The University of California, Berkeley, in cooperation with the U.S. Navy, instituted a program of wave recording at several points along the Pacific Coast in 1947. Charts from these records have been analyzed for the "significant" wave height and the wave period. Wave direction also was determined in some instances from synoptic weather charts by the wave forecasting method described in Chapter 8. These various data have been summarized in tabular form and distributed to various individuals, commercial concerns, and government agencies that were interested in this type of information (Isaacs and Schorr, 1947). In a limited number of cases summaries of wave data have been published (Wiegel, 1949; Wiegel and Kimberley, 1950).

Table II shows the periods for which wave data are available for the various recorders located as shown in Fig. 1. Inquiries regarding the data from any particular recorder should be directed to the address listed in footnotes to Table II. In addition to the records from recorders on the Pacific Coast, it is of interest to note that the University has recorded waves at Apra Harbor, Guam, M.I. for the period January 19 through July 5, 1949, and at Pokai Bay, Oahu, T.H. from October 20 to October 27, 1949.

In addition to statistical wave data obtained from recorders, a certain amount of data have been assembled by the hindcasting procedure described by R.S. Arthur in Chapter 8. The most comprehensive compilation in this field was a study by the Scripps Institution of Oceanography in cooperation with the U.S. Engineers Office, Los Angeles, on wave conditions at five open sea localities along the California Coast for the three-year period 1936 to 1938, inclusive (Scripps Institution of Oceanography, 1947). Hindcasts, for shorter periods of time, have been made for various localities along the Pacific Coast by the Department of Engineering, Berkeley. A summary of the localities and periods for which hindcast data are available is given in Table III for both the Scripps and Berkeley studies.

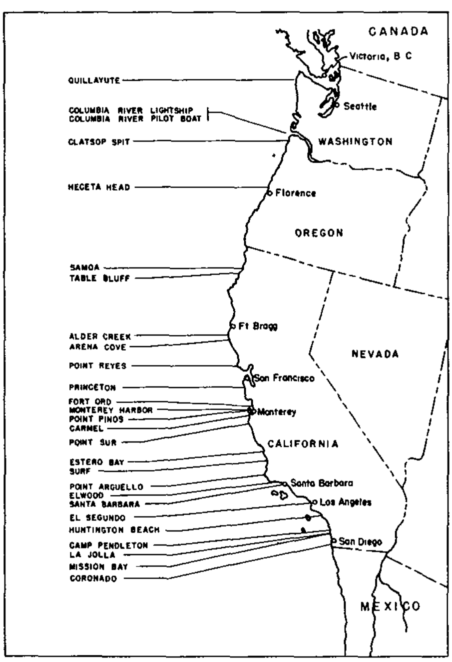

Fig. 1. Pacific Coast Wave Recording Stations.

TABLE II

Pacific Coast Wave Recorders

Location	Type	Installed	Abandoned	Remarks
University of California, Berkeley*				
Quillayute, Washington	UC Mk. V	10-27-48	11- 3-48	Cable destroyed in storm.
Columbia River Lightship	Visual	8- 1-33	8-31-36	Observations made in cooperation with the Corps of Engineers and the Coast Guard.
Columbia River Pilot Boat, Oregon	Visual	10- 9-49	11-29-49	Not observed on Nov. 24 and 25, 1949.
Clatsop Spit, Oregon	UC Mk. V	5- 8-50	6-14-50	Inoperative on several occasions for 2 or 3 days.
Heceta Head, Oregon	UC Mk. III	5-16-47	11-23-48	Not operating 4-14-48 to 11-6-48.
Samoa, California	Sighting bar	8- 5-44	2-24-45	Observations also made during Dec. 1945 and January 1946.
Table Bluff, California	Sighting bar & photos	2-26-45	1-29-46	Not observed on several occasions.
Alder Creek, California	Sighting bar	8-26-44	2-16-45	Observations approximately four miles north of Pt. Arena.
Arena Cove, California	Sighting bar	10- 3-44	12-31-44	
Point Reyes, California	Sighting bar	7- 7-44	11-26-45	
Princeton, California	UC Mk. II	10- 3-45	10-26-45	Sighting bar observations also made at Miramar Hotel during parts of July and October 1945.
Fort Ord, California	Sighting bar	11- 1-44	8-31-45	Mark V recordings were taken on several occasions in March 1950.
Monterey Harbor, California	UC Mk. III	5-15-46	8- 7-46	Recorder installed on Municipal Pier inoperative on several occasions.
Point Pinos, California	UC Mk. III	3-21-50	Active	Recorder inoperative on several occasions for 2 or 3 days.
Carmel, California	UC Mk. III	9- 1-46	10-18-46	Not operating 9-6-46 to 10-10-46.
Point Sur, California	UC Mk. III	4-25-47	Active	Inoperative 9-14-47 to 10-16-47 and 7-12-48 to 9-21-48.
Estero Bay, California	Various	12- 4-44	8-31-45	Observations made by sighting bar, recorders and from photographs.
Surf, California	Sighting bar	1-29-45	3-12-45	
Point Arguello, California	UC Mk. III	6- 6-48	Active	Inoperative 10-11-48 to 3-16-49, 4-16-49 to 8-9-49 and 3-6-50 to date.
Elwood, California	UC Mk. IX	8-10-50	Active	
Santa Barbara, California	UC Mk. V	4-20-50	Active	
Camp Pendleton, California	UC Mk. V	3- 7-49	Active	Not operating 3-21-49 to 5-5-49 and 1-16-50 to 7-1-50.
La Jolla, California	Sighting bar	6-22-44	9- 4-45	Not observed during the period 7-25-44 to 11-30-44.
Coronado, California	Sighting bar	11-30-44	9- 7-45	Not observed during the period 3-9-45 to 3-24-45.
Scripps Institution of Oceanography**				
La Jolla, California	Various	-	-	Exact periods of operation are unknown.
Beach Erosion Board***				
El Segundo, California	B.E.B.	7-26-48	Active	Not operating 3-7-49 to 3-25-49 and 6-17-49 to 6-31-49.
Huntington Beach, Calif.	B.E.B.	5-29-48	Active	Not operating 1-4-50 to 4-27-50.
Mission Bay, California	B.E.B.	12-13-49	Active	First 3 months of record are of questionable value.

For information on data from the various recorders, inquiries should be directed to:

* Director, Institute of Engineering Research, University of California, Berkeley 4, California.

** Director, Scripps Institution of Oceanography, La Jolla, California.

*** President, Beach Erosion Board, Corps of Engineers, 5201 Little Falls Road, N. W., Washington 16, D.C.

TABLE III

Statistical Wave Data Compiled by the Hindcasting Method

Location	Period of Hindcast
University of California, Berkeley	
Columbia River Pilot Boat	10- 7-49 to 12-23-49
Clatsop Spit, Oregon	2- 5-50 to 4- 1-50
	5- 1-50 to 6-16-50
Heceta Head, Oregon	4-25-47 to 1- 1-48
Humboldt Bay Entrance, California	8- 5-44 to 8-31-45
	11-19-45 to 7-31-46
Table Bluff, California	12- 3-45 to 1-31-46
	1- 4-45 to 8-31-45
Pt. Arena, California	10- 1-44 to 2-30-45
Pt. Reyes, California	7- 7-44 to 9-30-44
	10- 1-44 to 8-31-45
Fort Ord, California	4- 1-45 to 9-29-45
	1- 3-45 to 3-31-45
	3-20-46 to 7-31-46
	2- 5-50 to 4- 1-50
	5- 1-50 to 6-16-50
Point Pinos, California	2- 5-50 to 4- 1-50
	5- 1-50 to 6-16-50
Carmel, California	4-24-46 to 8- 3-46
Pt. Sur, California	4-25-47 to 1- 1-48
	2- 5-50 to 4- 1-50
	5- 1-50 to 6-16-50
Estero Bay, California	4- 1-45 to 9-29-45
	10- 5-45 to 10-22-45
	4-12-44 to 4-31-44
	5-12-44 to 3-31-45
Pt. Arguello, California	7-28-45 to 9-29-45
	5-31-50 to present
La Jolla, California	6-22-44 to 8-13-44
Coronado, California	4- 1-45 to 8-31-45
	11-30-44 to 3-31-45
Scripps Institution of Oceanography, La Jolla	
Lat. 42.5° N Long. 125.0° W	1936 - 1938 Inclusive
Lat. 40° N Long. 125.0° W	" " "
Lat. 37.5° N Long. 123° W	" " "
Lat. 35.0° N Long. 121° W	" " "
Lat. 33.0° N Long. 120.0° W	" " "

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CHAPTER 8

WAVE FORECASTING AND HINDCASTING*

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INTRODUCTION

As a result of wartime research on ocean surface waves a method has been available since 1943 for the prediction of wave characteristics of interest to engineers (O'Brien and Johnson, 1947). The initial stimulus for the development came during the planning of the invasion of North Africa, and the methods subsequently devised were later used in a number of amphibious operations (Bates, 1949). The same techniques have found useful peacetime application in problems connected with coastal engineering. Much of the application to date has consisted in applying wave prediction techniques to historical rather than current meteorological data, hence the term "wave hindcasting."

The wind-generated waves in the ocean are conveniently divided into the three categories sea, swell, and surf. The term sea refers to waves under the direct influence of generating winds, and swell to waves which have left the generating area and are subject to decay in regions of weak winds or calms. The breakers which result when waves move from deep to shallow water comprise surf. The transformation of sea and swell into surf has been described in Chapter 3. With regard to prediction it suffices to mention that the computation of the shallow water characteristics from given deep water characteristics is readily accomplished from available graphs after refraction diagrams have been drawn (Hydrographic Office, 1944; Johnson, O'Brien, and Isaacs, 1948). The present discussion is confined to a brief consideration of, (1) forecasting sea and swell, (2) the significance and application of the forecast, and (3) hindcasting and its applications.

FORECASTING SEA AND SWELL

Sverdrup and Munk (1947) have obtained relationships for the growth and decay of waves by considering the energy transfer from wind to waves during growth and the reverse transfer from waves to atmosphere during decay. Empirical data have been utilized in evaluating certain coefficients and constants of integration. The growth of waves depends upon wind velocity, the duration of the wind, and the distance over which the wind blows, called fetch. Observations have shown that as the wind blows over a fetch, the wave height and period over the up-wind part shortly reach a steady state. As time passes the steady state region expands over the whole fetch. Generally, duration rather than fetch is critical in limiting wave growth. The forecasting graph which has been constructed from Sverdrup and Munk's relationships for the transient state is reproduced (Fig. 1a).

The wave height, H_F, and period, T_F, at the down-wind end of the fetch are altered by air resistance as the waves leave the fetch and decay. The change in H_F and T_F and the travel time required depend upon T_F and the decay distance D. The relationships of Sverdrup and Munk (1947) for decay have been utilized in preparing the decay graph (Fig. 1b).

The basic data required for a sea and swell forecast are wind velocity and duration, fetch, and decay distance. These data are obtained from synoptic weather maps or from weather forecasts made from such maps. The wave forecasting theory assumes that the wind velocity is constant over the fetch for the duration of the wind. In practice, this condition is never attained, and skill and judgment are

*Contribution from the Scripps Institution of Oceanography, New Series No. 530. This work represents results of research carried out for the Office of Naval Research, Department of the Navy; and the Beach Erosion Board, Department of the Army, under contract with the University of California. The paper was largely prepared while the writer was visiting at the University of Washington, Oceanographic Laboratories.

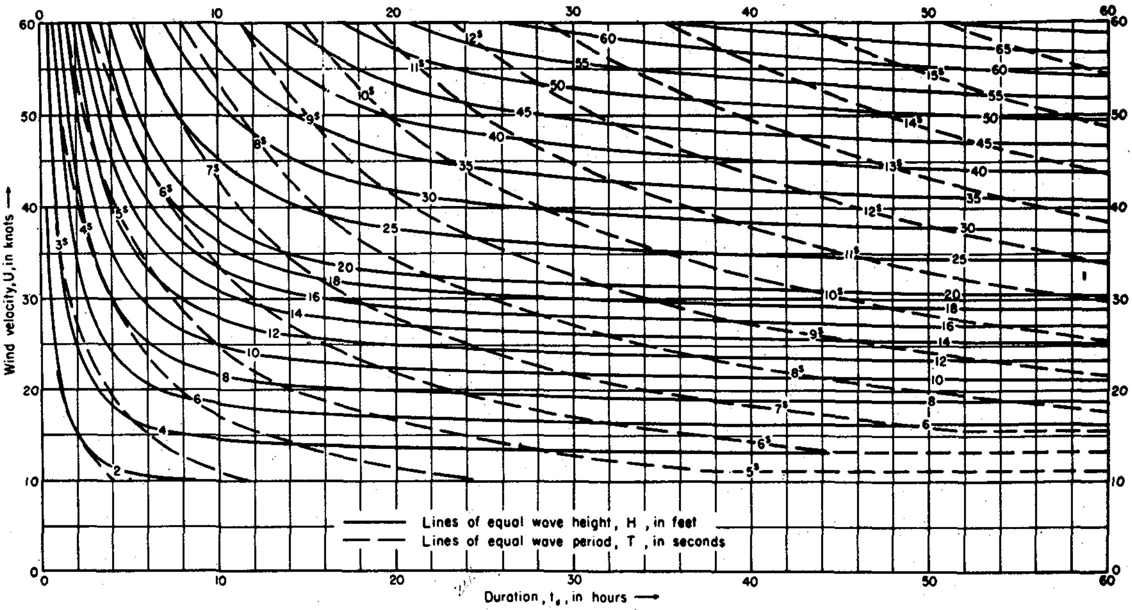


FIGURE 1a. WAVE HEIGHT AND WAVE PERIOD AS FUNCTIONS OF DURATION OF WIND AND WIND VELOCITY

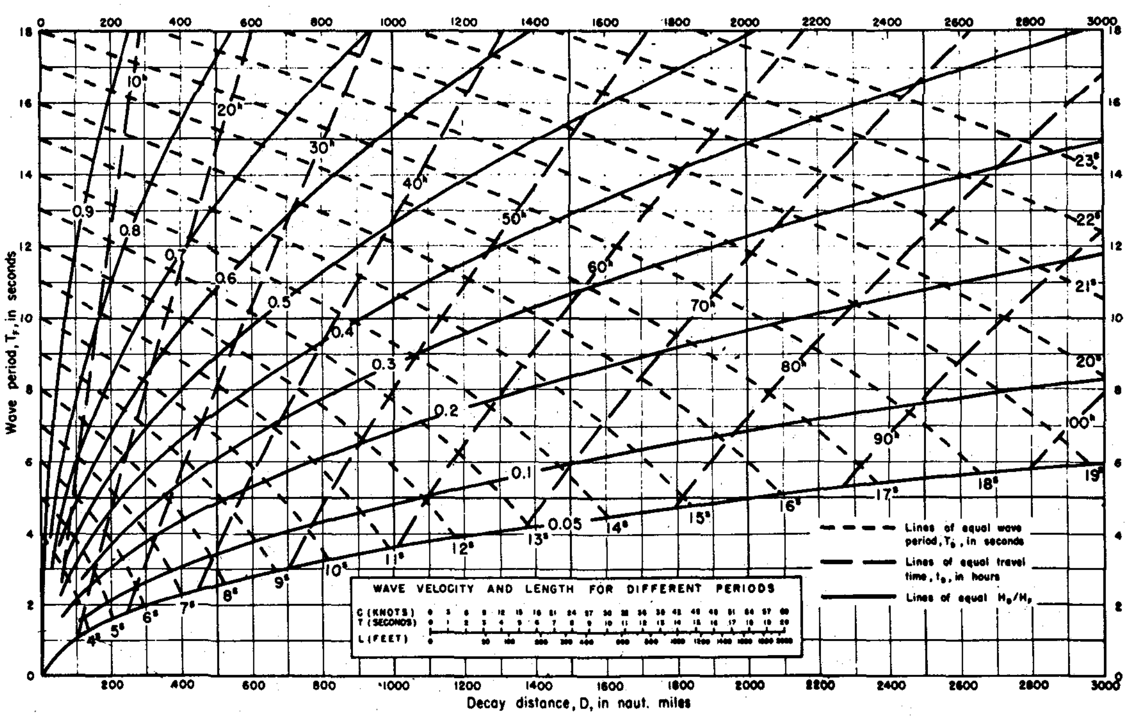


FIGURE 1b. WAVE PERIOD AT END OF DECAY DISTANCE, TRAVEL TIME, AND RATIO BETWEEN WAVE HEIGHT AT END OF DECAY DISTANCE AND AT END OF FETCH AS FUNCTIONS OF DECAY DISTANCE AND WAVE PERIOD AT END OF FETCH

required in selecting fetch, duration of the wind, and average values of wind velocity such that accurate forecasts are obtained. For this reason, persons with experience in meteorology have usually been selected for training in wave forecasting. During World War II a number of meteorologists in the armed services were trained to use the Sverdrup-Munk methods. The successful application in the field is indicated by Bates (1949), who concludes that "the techniques were basically correct and could be modified by meteorologists trained in the methods to provide reliable forecasts for amphibious operations wherever they might be held."

THE SIGNIFICANCE AND APPLICATION OF THE FORECAST

The waves in the sea are extremely complex in the sense that wave height, period, and length vary widely with respect to space and time in an apparently irregular fashion. This complexity and irregularity have made it essential to introduce statistical measures in order that wave records and observations may be characterized in a meaningful manner and the significance of the forecast be established. In many applications, only the heights and periods of the higher waves in a wave train are of practical significance. For this reason, the average height and period of the highest one-third of the waves, after ripples and waves of height less than one foot are eliminated from consideration, are useful statistical measures. These averages have been called "significant wave height" and "significant wave period," respectively (Sverdrup and Munk, 1946).

In some engineering applications, averages for the highest ten percent of the waves, or in some cases only the maximum wave height, are the important statistics. Munk (1944), Seiwell (1949), and Wiegel (1949) have determined the ratios between various statistical measures (see Table I). These ratios show variation with respect to locality and time; however, the time fluctuations at a given locality appear to be small enough so that the ratios are of great practical use.

TABLE I

Ratios of mean wave height, average of highest ten percent, and maximum height to significant wave height

	Scripps[1] (swell)	Point[2] Sur	Heceta[2] Head	Point[2] Arguello	Cuttyhunk[3]	Bermuda[3]
Interval over which averages formed	46 waves	three 20 min. intervals per day			2 minute intervals	
Ratio of mean to significant	0.67				0.64	0.64
Ratio of highest 10 percent to significant		1.27	1.30	1.30		
Ratio of maximum height to significant		1.85	1.91	1.87		

[1]Munk (1944); [2]Wiegel (1949); [3]Seiwell (1949).

The question arises as to how the wave height and period as predicted on the basis of the Sverdrup-Munk method compare with the various statistical measures. Munk (1944) has shown that the usual visual observations are best identified with significant height and period. Since visual observations were used in developing the forecasting method, it would appear that predicted wave heights and periods might be considered to be significant heights and periods. Isaacs and Saville (1949) have demonstrated that this is the case by a comparison of predicted heights and periods with significant heights and periods extracted from records made at several west coast stations. Preliminary indications show the same results for a study on the east coast sponsored by the Beach Erosion Board (James, 1951).

The investigations described in this section serve to indicate the significance of the wave forecast with respect to the actual waves occurring in the sea.

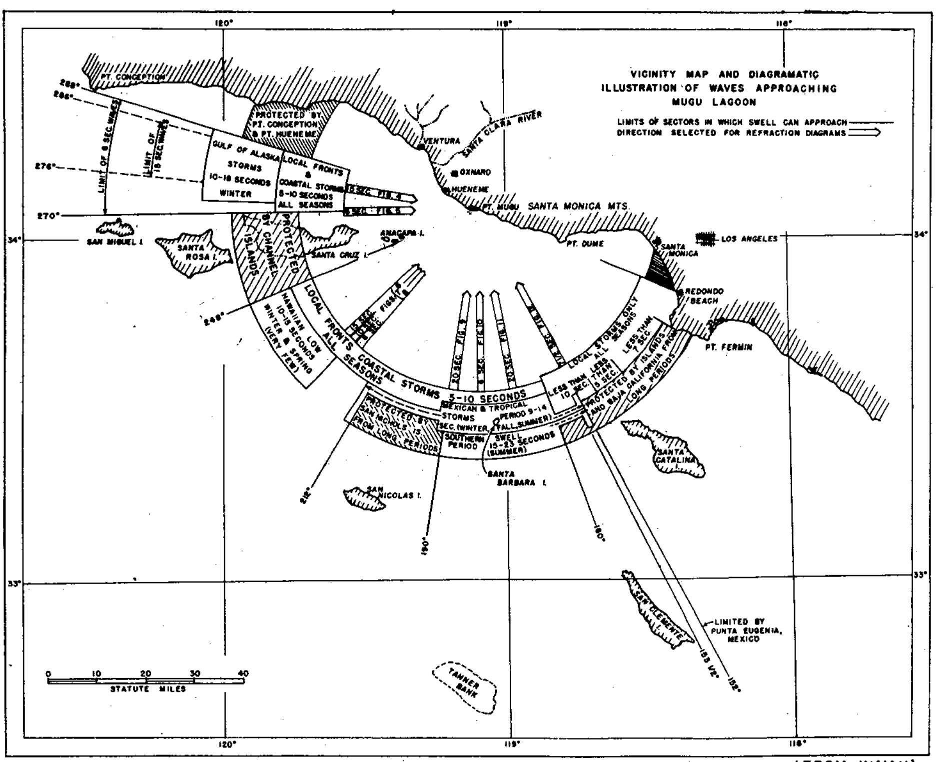
Ratios such as those given in Table I are extremely important in this regard. For application in problems of sand erosion during construction of coastal works, predictions of significant wave height and period may be desirable. On the other hand, in problems of structural strength under wave action, predictions of the maximum wave height are needed. Wave forecasting has advanced to a stage where it is possible in many localities to furnish useful predictions of the various wave characteristics to engineers engaged in the construction of coastal works.

HINDCASTING AND ITS APPLICATIONS

Because of the lack of wave records or adequate visual observations of waves in many regions, there has been a demand for wave data obtained by hindcasting (Burt and Sauer, 1948). Accumulated wind data are generally sufficient for most localities in the Northern Hemisphere to permit computation by means of the forecasting method of past wave characteristics. In particular, the Daily Synoptic Series, Historical Weather Maps, Northern Hemisphere, have been found most useful in this regard (see Chapter 10).

The hindcasting technique can provide statistical wave data over a large area in a short period of time. In one application for the Corps of Engineers, Los Angeles District, two forecasters and three clerks computed in about four months, the deep-water wave characteristics over a three-year period for the California Coast (Scripps Institution of Oceanography, 1947). The cost of such data is not large relative to the cost of installing and maintaining wave meters and analyzing the resulting records over a three-year period.

Such data have proven useful in providing a basis for the design of coastal structures such as off-shore drilling installations. Use has also been made in examining the question of the stability of beaches prior to construction of coastal works. Inman (1950) has used wave data provided by the hindcasting technique in a coastal beach study in the vicinity of Mugu Lagoon, California, and two illustrations are reproduced from his report. Fig. 2 shows the sources and period ranges for waves approaching the coast near Mugu Lagoon from various directions. Fig. 3



(FROM INMAN)

Fig. 2

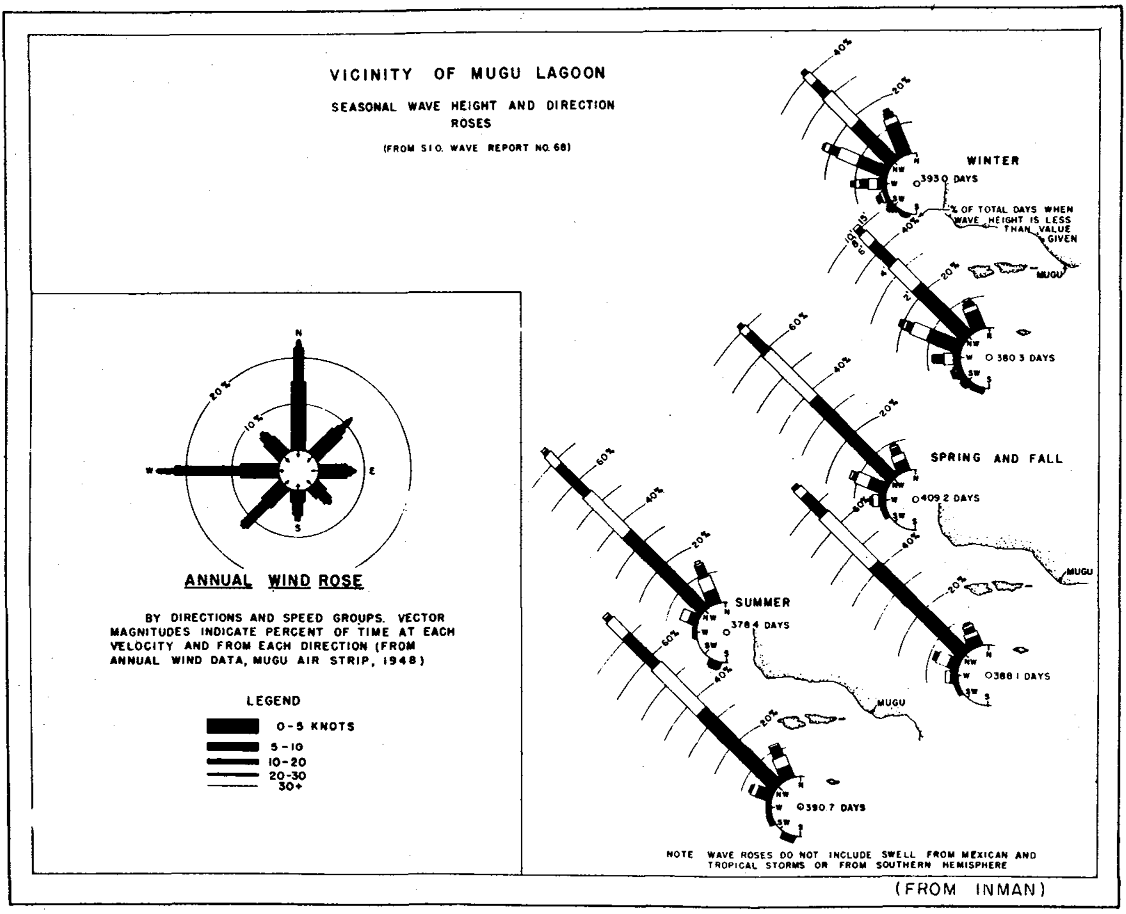


Fig. 3

shows seasonal wave height and direction roses for a three-year period for the portion of the coast near Mugu Lagoon. The source material for both figures was obtained by an examination of past meteorological situations and hindcasting from historical weather maps.

Hindcasting has also been useful in providing information on the characteristics of waves which have caused failures in coastal installations (Horrer, 1950). In the past, adequate records have often not been kept of the wave conditions associated with such failures even though the damage has been carefully noted. Hindcasting can help fill in the gap in order that future designs may be improved.

Finally, it is to be emphasized that although hindcasting is a useful technique for providing wave data it should supplement rather than supplant wave records and observations. Every individual hindcast is subject to the uncertainties contained in the forecasting method, and may, therefore, be in error compared to the actual wave conditions. In the statistical accumulation of many hindcasts these errors tend to compensate, but the security of the results must always depend ultimately on comparison with actual observations.

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CHAPTER 9

THE ENGINEERING APPLICATION OF SEA AND SWELL DATA

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INTRODUCTION

In planning for any marine construction, information on ocean waves should be considered essential. Such information must be available to select the best season of operations and to indicate the types of floating equipment that can be operated economically with a minimum of lost time. Furthermore, from such advance information it will be possible to estimate whether or not special observing and forecasting services will be required to provide for safe and efficient day-to-day operations.

Readily available sources of ocean wave information are the sea and swell atlases published by the U.S. Navy Hydrographic Office. These atlases, providing world-wide coverage on a monthly basis, have been compiled from visual observations of sea and swell recorded aboard merchant and naval vessels. These data therefore provide a means for making rapid evaluations of wave and swell conditions in the area of operations and along routes to and from this area. If the site is on an exposed coast, it is also possible to estimate surf conditions.

Although the data are not ideal for many specific purposes, they are very helpful when used to derive index values of wave conditions. Such values will indicate changes from month to month and from one locality to another. An engineer with experience in one region can, therefore, by use of such index values tell whether conditions in another area will be worse or better. If other environmental features such as winds, currents, temperatures, etc , are also taken into account, it is possible to designate analogous areas so that experience in known regions can be extrapolated into unfamiliar regions.

WORK FEASIBILITY

In order to interpret wave data, it is essential to have an understanding of work feasibility. This is a measure of efficiency and safety of one or more types of operations expressed in terms of one or more environmental factors. For example, small craft may operate unhampered while waves are small, but if the wave heights exceed some limit, the efficiency and safety decrease as the waves grow larger, until with waves above a certain size operations must be stopped entirely. Other more complicated situations can be imagined where more than one variable must be taken into account. Thus, the efficiency of dredging operations may depend upon waves and currents, while the efficiency of pile-driving may depend on rainfall as well as wave height. The importance of the concept of work feasibility is very obvious if the operation must be conducted in a region of extreme and severe conditions where effectiveness will be marginal, or must be limited to brief periods of time.

Unfortunately our knowledge of work feasibility is extremely limited. There exists a reasonable amount of information about many of the environmental factors, but the applications of such data to engineering operations will be qualitative until such a time as more is known about the relationships between these factors and various types of operations. Fortunately a firm of consulting oceanographers working with the oil companies on the Gulf Coast has, during the last several years, been able to make a start on this problem. Table I, based on these studies, shows the limits of performance in terms of wave heights for a number of different operations (Glenn, 1950). Such relationships can be employed in several different ways. If, for example, the percentage frequency of occurrence of waves of different heights is known for any region, it is possible for the planner to estimate the amount of time that any operation can be conducted efficiently, with marginal

TABLE I

GENERALIZED PERFORMANCE DATA FOR MARINE OPERATIONS
(According to Glenn)

Type of Operation	Wave Heights* (in feet) for: Safe, Efficient Operation	Marginal Operation	Dangerous and/or Inefficient Operation
Deep Sea Tug			
Handling oil and water barge	0-2	2-4	> 4
Towing oil and water barge	0-4	4-6	> 6
Handling derrick barge	0-2	2-3	> 3
Handling and towing LST-type vessel	0-3	3-5	> 5
Crew Boats, 60-90 feet in length			
Underway	0-8	8-15	> 15
Loading or unloading crews at platform	0-3	3-5	> 5
Supervisor's Boats, fast craft 30-50 feet in length			
Underway at cruising speed	0-2	2-4	> 4
Loading or unloading personnel at platform or floating equipment	0-2	2-4	> 4
LCT-type Vessel and Cargo Luggers			
Underway	0-4	4-5	> 5
Loading or unloading at platform	0-3	3-4	> 4
Loading or unloading at floating equipment	0-4	4-5	> 5
Buoy Laying (using small Derrick Barge)	0-2	2-3	> 3
Platform Building			
Using ship-mounted derrick	0-4	4-6	> 6
Using large derrick barge	0-3	3-5	> 5
Pipe-line construction	0-3	3-4	> 4
Gravity-meter exploration using surface vessel (limiting conditions caused by instrument becoming noisy)	0-4	4-6	> 6
Seismograph Exploration using craft under 100 feet in length	0-6	6-8	> 8
Large Amphibious Aircraft (PBY)			
Sea Landings and Take-offs	0-1.5	1.5-3	> 3
Boat-to-plane transfer operations in water	0-1	1-2	> 2
Small Amphibious Aircraft	0-1	1-2	> 2

*Wave heights used are those of the average maximum waves. Height limits given above are not rigid and will vary to some extent with locality, local wind conditions, experience of personnel, etc.

efficiency, or with dangerous working conditions. Furthermore, during a marine operation, day-to-day scheduling can be based upon wave forecasts expressed in operational terms. Disaster and evacuation plans also can be formulated so that they can be carried out with safety and speed before the situation becomes critical. It should be pointed out that the information in Table I is based on observations and experience gained on the Louisiana coast. It is believed that the relationships are generally usable in other areas but further checking is desirable.

EXISTING TYPES OF PUBLISHED SEA, SWELL, AND WIND DATA

Unfortunately, a direct correlation between work feasibility and wave height based on published sea and swell data is not possible. Sea and swell observations are usually described in codes based on a 10-point scale describing the sea surface in general terms, such as "smooth", "rough", "mountainous", etc., without specific reference to wave heights. Swell is reported in such terms as "short and high". However, these general terms have the approximate numerical wave height equivalents shown in Table II which illustrates the sea and swell codes currently in use by cooperating marine observers of the U.S. Navy Hydrographic Office. The coarseness of these observations has cast some doubts on their reliability. When used with judgment, however, the data are of considerable value, particularly since no other data of this sort exist for the oceans of the world except that provided by the "hindcast" method. British and Japanese sea and swell atlases have been compiled from essentially similar data based upon visual observations. These data cover several decades, the files at the U.S. Navy Hydrographic Office dating back to 1904. The data are published by these governments in the varying formats shown in Figs. 1 and 2. Of the several methods of presentation, the "rose" employed in the American presentation (Hydrographic Office, 1943 to 1950), is the most suitable for the technical user. However, it should be mentioned that the American rose is oversimplified and that data are omitted if the condition occurs less than 7% of the time; moreover, the data are obscured if the frequency of occurrence is between 7% and 14% of occasions. The detailed tabulations are on file, however, at the U.S. Navy Hydrographic Office and can be made available upon request for the cost of reproduction of the tabulation sheets.

In addition to tabulations of sea and swell conditions, the frequency of occurrence of various wind forces may also be of great use in providing enough data to provide a reliable sample, climatically speaking. Such a sample should extend over at least a 10-year period and contain more than 100 observations per month. Mariners and marine meteorologists for years have used the rule of thumb that on the high seas the wind force is one number higher than the sea state. This correlation strongly biases the wind and sea data in existence today, as found by A. E. Parr and W. T. Edmondson in an unpublished analysis carried out in 1943. The approximate correlation permits wind strength, when reported, to be converted into height of local wind waves for exposed water areas, thereby increasing the number of observations available. Suitable corrections may be made for fetch limitations if the wind blows over a water area of 150 miles or less. Climatic wind data exist for many coastal points throughout the world and also are presented in very suitable "rose" form in the British Air Ministry series of Monthly Meteorological Charts for various oceans.

CONVERSION OF PUBLISHED SEA, SWELL, AND WIND DATA INTO FREQUENCY OF WAVE HEIGHT

With the necessary climatic data at hand on sea state, swell condition, and wind force for a given area, it is possible to convert these values into terms of specific wave height and thus into terms of work feasibility for the type of marine construction to be conducted. The equation for determining the frequency of occurrence of the specified wave height is as follows:

$$P = 1/2(W + X) - k(100-Y) \qquad (1)$$

where, P = Probable frequency of suitable wave height

W = Probable frequency of suitable wind force

TABLE II

SCALES USED IN SEA AND SWELL ATLASES OF THE U. S. NAVY HYDROGRAPHIC OFFICE

SEA CONDITIONS, U. S. HYDROGRAPHIC OFFICE SCALE

ATLAS TERMINOLOGY	*Approx. Height of Sea*	Code Figure	*Seaman's Description*
CALM	0	0	CALM—Sea like mirror.
LOW	Less than 1 foot	1	SMOOTH—Small wavelets or ripples with the appearance of scales but without crests.
	1-3 feet	2	SLIGHT—The waves or small rollers are short and more pronounced, when capping the foam is not white but more of a glassy appearance.
MEDIUM	3-5 feet	3	MODERATE—The waves or large rollers become longer and begin to show whitecaps occasionally. The sea produces short rustling sounds.
	5-8 feet	4	ROUGH—Medium waves that take a more pronounced long form with extensive whitecapping and white foam crests. The noise of the sea is like a dull murmur.
HIGH	8-12 feet	5	VERY ROUGH—The medium waves become larger and begin to heap up, the whitecapping is continuous, and the seas break occasionally; the foam from the capping and breaking waves begins to be blown along in the direction of the wind. The breaking and capping seas produce a perpetual murmur.
	12-20 feet	6	HIGH—Heavy, whitecapped waves that show a visible increase in height and are breaking extensively. The foam is blown in dense streaks along in the direction of the wind. The sea begins to roll and the noise of the breaking seas is like a dull roar, audible at greater distance.
	20-40 feet	7	VERY HIGH—High, heavy waves developed with long overhanging crests that are breaking continuously, with a perpetual roaring noise The whole surface of the sea takes on a white appearance from the great amount of foam being blown along with the wind. The rolling of the sea becomes heavy and shocklike.
	40 feet and over	8	MOUNTAINOUS—The heavy waves become so high that ships within close distances drop so low in the wave troughs that for a time they are lost from view. The rolling of the sea becomes tumultuous. The wind beats the breaking edge of the seas into a froth, and the whole sea is covered with dense streaks of foam being carried along with the wind. Owing to the violence of the wind the air is so filled with foam and spray that relatively close objects are no longer visible.
		9	NOTE—Qualifying condition applicable to the previous conditions, e. g., (5-9). A very rough, confused sea.

SWELL CONDITIONS

ATLAS TERMINOLOGY	Approx. Height in Feet	Description		Approx. Length in Feet	Code Figure
CALM	0	No swell		0	0
LOW	1-6	Low swell	Short or average	0-600	1
			Long	Above 600	2
MEDIUM	6-12	Moderate	Short	0-300	3
			Average	300-600	4
			Long	Above 600	5
HIGH	Greater than 12	High	Short	0-300	6
			Average	300-600	7
			Long	Above 600	8
	....	Confused			9

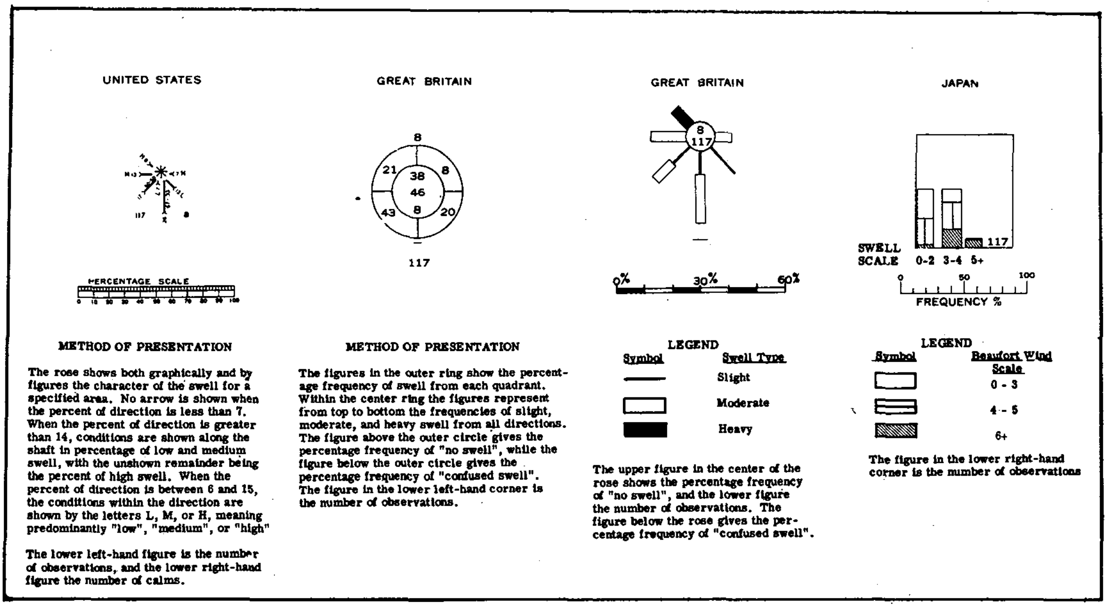

Fig. 1
Methods utilized by meteorological and hydrographic activities of Great Britain, Japan, and the United States for presenting climatic data on swell conditions.

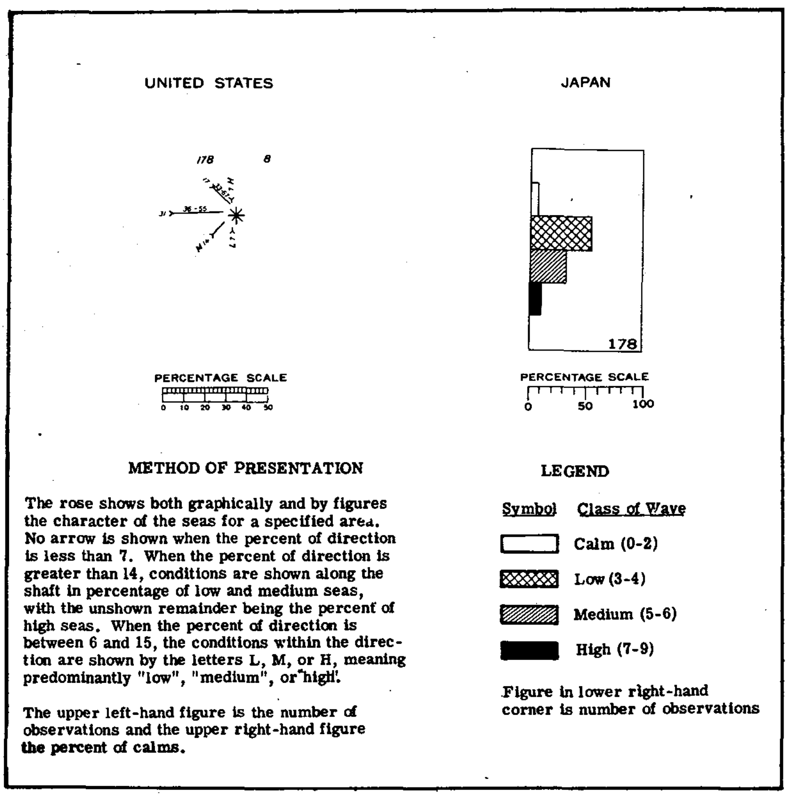

Fig. 2
Methods utilized by hydrographic activities of Japan and the United States for presenting climatic data on sea conditions.

X = Probable frequency of suitable sea condition

k = Factor defining likelihood of limiting swell condition occurring during the periods of suitable wind and sea conditions

Y = Probable frequency of unsuitable swell conditions

If there are 200 or more "state of sea" observations for a given month, wind observations do not need to be included in the computation. If the number is less, however, wind observations should be incorporated so as to give a better climatic average. The value of "k" is difficult to assess, for in some parts of the world, the sea may be calm, and yet swell occurs which is high enough to inhibit operations. In other words, the value of "k" may vary from zero to 100 percent. In practice, a very low value of "k" is used and upon occasion, a value of zero is indicated.

As an example of the method, assume that a determination was desired of the frequency with which waves 3 feet or more in height occurred at a given point totally exposed to oceanic conditions. The monthly occurrence of the defining factors is reported to be as follows:

Wind of force 4 or less: 85% frequency

Calms and low seas: 75% frequency

Moderate and high swell, plus half of the occurrence of low swell: 30% frequency

Under such circumstances, wind and wind wave conditions would be considered favorable 80% of the time. If the value of "k" were 0.2, then swell over 3 feet in height would occur when seas were below that height or about 6% of the time (0.2 x 30%). When these values are substituted in the equation just given, the equation becomes:

$$P \doteq 0.5\ (85 + 75) - 0.2(30) = 74$$

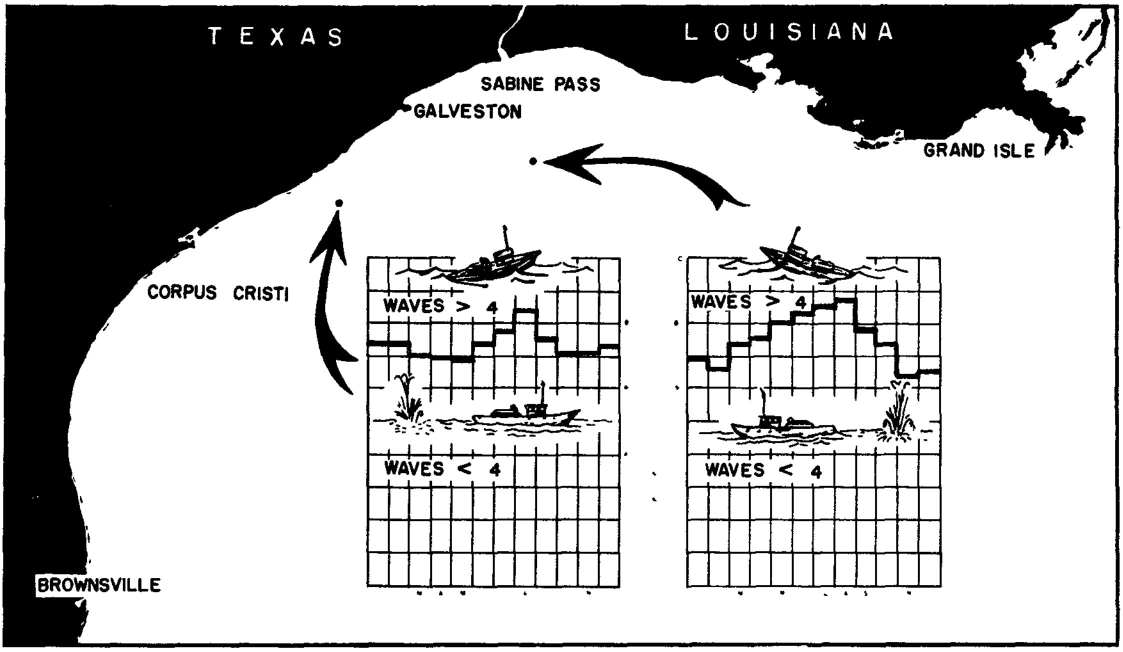

Fig. 3
Probable frequency of occurrence of waves above and below 4 feet in height at two selected locations in the Gulf of Mexico.

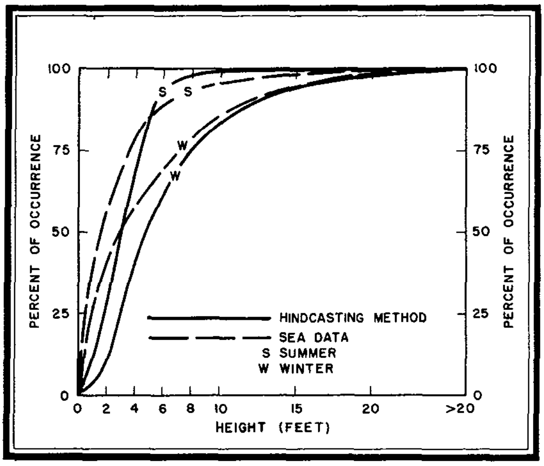

Fig. 4
Comparison of wave height data obtained by the hindcasting and climatic techniques for the location, Latitude 28°North, Longitude 177°30'East, near Midway Island.

The probable frequency of occurrence of waves with heights of 3 feet or less is thus 74%, or this condition will exist on a total of 22 overall days per month. Another example is shown in Fig. 3, where the occurrence of wave heights below 4 feet has been computed for each month of the year at two points in the northwestern Gulf of Mexico. Comparison of wave conditions at the two points indicate that seasonal improvement in working conditions extends from February to August for the eastern location, whereas there is no appreciable improvement until June at the western location.

COMPARISON OF CLIMATIC AND HINDCASTING METHODS FOR OBTAINING WAVE DATA

The accuracy of the method for computing wave heights from climatic wave data is open to some question. Fortunately, oceanographers at the Scripps Institution of Oceanography have made "hindcasts" of average wave heights in the open sea by the method outlined by R. S. Arthur in Chapter 8. A comparison of the two methods for a location in the Pacific near Midway Island is shown in Fig. 4. The number of observations and hindcasts used is approximately the same in each case -- 90 per month. The best correlation is obtained when no correction is made for the occurrence of swell independently of sea, i.e., the "k" factor is zero. The greatest discrepancy is noted between wave heights of 4 to 12 feet, the zone in which the mariner is commonly believed to overestimate wave heights. A comparison of various aspects of the two methods for obtaining probable wave conditions is given in Table III.

TABLE III

COMPARISON OF CLIMATIC AND HINDCASTING METHODS FOR PROVIDING WAVE INFORMATION

Aspect	Comparison
Accuracy	Hindcasting provides more specific information on wave height, period, and direction than does the climatic method when: a. There is adequate weather map coverage, preferably at 12-hourly intervals or less. b. Locations are outside the equatorial region bordered by latitudes 20° north and south. Inside this region, the hindcasting method fails and the climatic method is the only one available unless wind observations are used from shore sites. c. There is no swell from the Southern Hemisphere. (Both methods fail to provide adequate information on this condition). d. There are several swell trains or the height of swell is low compared to that of the local wind waves. (Climatic observations from beach sites detect such low swell, but marine observers do not.)
Cost	Climatic method is 25 or more times faster than the hindcasting method. Majority of tabulation in the climatic method can be conducted by sub-professional personnel, but most of work in hindcasting method is at the professional level.

ROUTE SELECTION

In addition to using tabulated wave data for computing conditions at an exposed oceanic site, the data may be applied to evaluating the suitability of routes to the site, since problems arise in the dispatching of small craft and towed barges to points hundreds of miles away through exposed waters. Such an evaluation is based on computing the frequency of occurrence of suitable conditions along the several routes under consideration. Since each route goes through several areas for which sea and swell conditions are given, it is necessary to arrive at a resultant value of the frequency of occurrence of acceptable wave heights in the areas making up a route. The resultant value can be obtained by correlating the frequencies of occurrence for each component area with each other. The minimum time during which suitable conditions exist simultaneously in each of these areas would be that of the areas being completely independent of each other, or the product of the percentage frequencies for each area. The maximum time during which suitable conditions would exist simultaneously is the value of the lowest component frequency of the associated areas. The exact degree of correlation falls somewhere between these maximum and minimum values. The mean of these extremes is probably a satisfactory value for the purpose of comparing one route against another.

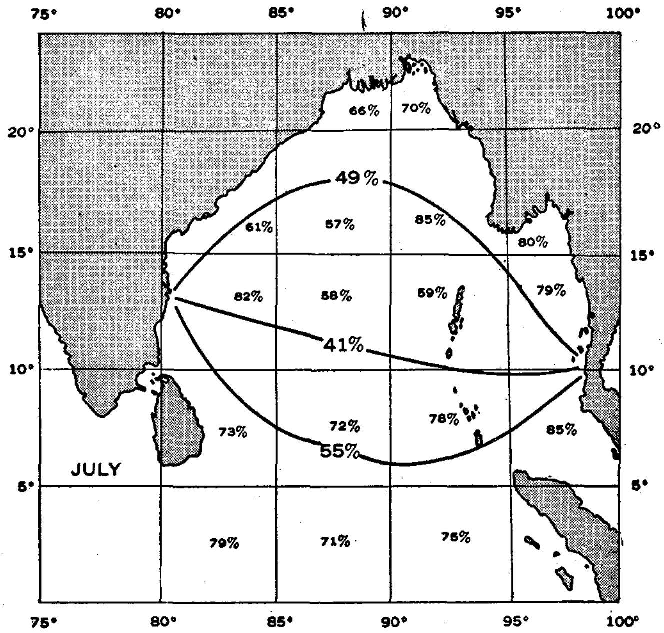

Fig. 5
Probable frequency of suitable conditions for crossing the Bay of Bengal during the southwest monsoon in July. The frequency of suitable conditions for each 5°-quadrangle of latitude and longitude is also shown for this same month.

As an example of such a route selection, a problem is shown here which was worked out for crossing the Bay of Bengal from India to Malaya during the summer monsoon. The pertinent frequencies of occurrence of suitable conditions for a given type of operation are shown in Fig. 5. The direct route is noted to pass through four quadrangles with desired conditions existing 82%, 58%, 68%, and 82% of the time respectively. Such values give the best possible frequency of occurrence of suitable passage as 58%, the worst 24%, and the probable value of occurrence about 41% of the time. Values computed for routes to the north and south of the direct route indicate that the southern route is the best of the three and is likely to have suitable conditions about a third of the time more frequently than does the direct route.

COMPUTATION OF SURF CONDITIONS

The coastal engineer is as interested, if not more so, in wave conditions in shallow water, including the surf zone, as he is in conditions outside this zone. Climatic wave data can also be used in this problem. However, the determination of probable surf heights is not as easy or as accurate as the one just described for determining wave conditons offshore, because the transformation of deep water waves into surf is difficult to determine for a short strip of beach even when full information is at hand on the deep water characteristics of the wave trains, on the beach topography, on the offshore hydrography, and on local tidal currents. A method worked out by the junior author, however, and described elsewhere (Bates, 1948) does permit the climatic data to be converted into frequency of surf heights along a given orientation of beach for the area for which the wave data are available, it being assumed that such a coast has smooth and parallel depth contours offshore. The method takes into account the three major factors required in surf forecasting, namely, deep water wave height, approximate wave period, and the angle which the deep-water wave crests forms with the coastline. By following eight successive steps too lengthy to be discussed here, the probable monthly frequency of a given surf height then can be determined.

UNPUBLISHED TYPES OF VISUALLY OBSERVED WAVE DATA

In addition to the published sea and swell atlases summarizing conditions at sea, there exists a large amount of observations of sea conditions for coastal points throughout the world. Although in manuscript form, these data are generally on file at the U.S. Navy Hydrographic Office. Weather observers have noted the sea state for periods ranging from 1 to 9 years at 157 coastal localities on the European coast, and at 3 localities in Asia Minor. Observations have been made from 9 German lightships in the North and Baltic Seas between 1936 and 1937, and by French shore observers at several points on the western coast of French Morocco since 1931. In addition, observations were made visually two or three times a day for 57 points in southern and eastern England and on the French invasion coast between March and November, 1944. Similar observations made during several months in 1945 are available for 8 points along the Bay of Bengal. On the North American coast, the U.S. Coast Guard has made measurements using wave staffs between 1944 and 1947 at 12 coastal points in the United States and Alaska, as well as one location each in Puerto Rico and Hawaii. Fifteen thousand wave observations have been collected since 1947 by Glenn and Associates from numerous drilling platforms off the Louisiana coast. In the far Pacific observations were made on Okinawa and Iwo Jima for several months during the invasions of those islands in 1945.

INSTRUMENTALLY OBSERVED WAVE DATA

Instrumental observations of waves are much rarer. With the exception of certain Russian wave-measuring equipment operated on the Black Sea coast, the oldest set of instrumentally recorded wave data is that obtained with the drum recorder at the Scripps Institution of Oceanography, which began operation in the late 1930's (Shepard and LaFond, 1940). The more recent Atlantic and Pacific coast installations of wave recorders have been described by F. E. Snodgrass in Chapter 7. In the Gulf of Mexico, the Beach Erosion Board wave recorder installed on a well platform of the Humble Oil & Refining Company five miles off Grand Isle, Louisiana, is the sole source of instrumentally observed wave data for that body of water. The Woods Hole Oceanographic Institution has collected some data from Bermuda and at Cuttyhunk, Massachusetts. In England, the British Admiralty has maintained a wave recorder on the Atlantic coast of Cornwall from 1944 to the present time. The former Deutsche Seewarte of Hamburg, Germany, operated one or more recorders upon occasion in the North Sea.

VALUE AND COST OF WAVE DATA

As can be seen from this resumé, the locations for which instrumentally recorded data are available throughout the world are still relatively sparse, and the data are, with only a few exceptions, available only in manuscript reports. In view of this situation, the climatic data, even though based on visual observations, appear to be the best currently available on a worldwide basis. The limitations of these data have already been outlined in Table III, but many of these restrictions can be overcome if the data are treated as indexes instead of rigid numerical values for the occurrence of a certain wave height. Using the index concept, the data lend themselves readily to rapid comparisons of changes of wave conditions from one month to the next for a given location or to the comparison during a given month of one locality with another. If these qualitative data are further keyed into the experience of an individual for a given locality or into the instrumentally recorded conditions for a wave recorder, a more reliable extrapolation is possible for areas in which neither experience nor instrumentally recorded data are available.

The costs of marine work are unusually high. With even small dredges chartering for $900 or more per day, pile drivers renting at $300 and up per day, and small channels silting up within 30 days of dredging because of wave action, as one concern found after spending $60,000 on the job, it is easy to understand why the oil industry has spent 234 million dollars to drill 190 wells in the Gulf of Mexico since 1938. Costs in harbor construction are certainly of the same order of magnitude, although a larger pereentage of the funds may come from public, rather than private sources. With costs such as these, increases in efficiency from adequate planning against wave action provide an enticing way to save money. The cost of a preliminary survey of wave conditions based on tabulated sea and swell data is less than $100 per location, and often considerably less than half that amount. If the preliminary study indicates that wave conditions are important factors in design and operation, a more detailed study is called for based on an elaborate analysis of wave conditions, using hindcasting and the preparation of wave refraction diagrams. The cost of such a study will probably range between $2,000 and $4,000. The initial study also may have indicated that the work must be conducted during periods when wave heights may approach or exceed safe working limits upon occasion, making proper scheduling of work and storm evacuation plans a necessity. In such a case, accurate wave and weather forecasts should be acquired through special arrangement with the government weather service if the construction job is being conducted by a government agency and through arrangement with a private oceanographic consulting service if the construction is by a private concern. In the latter case, such a service will cost about $5,000 to $10,000 per year. While this amount may sound like a large figure for a type of service not commonly employed in the construction industry, the Gulf Coast oil operators are finding such a service saves far more than it costs, particularly in reducing loss and damage to marine equipment. As a result of this, it is interesting to note that in 1947, the services of an oceanographic forecasting firm were used in drilling fewer than 25% of the wells in the Gulf of Mexico. This percentage has risen to over 85% today. In no case has an operator failed to renew his contract for the service when the information was being supplied by a well-qualified consulting organization.

SUMMARY

To summarize, the costs of marine construction are high and are generally affected by the amount of wave action to which the site is exposed. In the preliminary stages of planning and bidding on such construction projects, the mass of visually and instrumentally observed wave data which already exists can be exploited at nominal cost for guidance in final design and the planning of actual construction by utilizing the work feasibility concept. If wave action proves to be an important consideration, a detailed study of the wave factor based on modern oceanographic knowledge should be made. If wave action makes the actual construction difficult or dangerous, special wave forecasts prepared by competent agencies should be acquired for use in day-to-day operations. As in other technical fields of endeavor, the organization which takes these steps and thereby utilizes the best scientific information bearing on an operating problem will ultimately enjoy better results in the long run than a concern which does not.

ACKNOWLEDGMENT

The authors wish to thank A. H. Glenn of A. H. Glenn and Associates, New Orleans, Louisiana for permission to use Table I concerning work feasibility.

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Shepard, F.P., and LaFond, E.C. (1940). Sand movements along the Scripps Institution pier: Amer. Jour. of Science, vol. 238, pp. 272-285.

Chapter 10

RESULTS OF THE WARTIME HISTORICAL AND NORMAL MAP PROGRAM*

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The practically global extent of World War II created a need for world weather information more pronounced than in any former period. Insufficient knowledge of weather in remote places, inadequate data on diverse weather elements, and lack of information about conditions extending into the stratosphere, led to the initiation of many projects involving the plotting, analyzing, and drawing of thousands of weather maps and charts. This material proved to be valuable in providing guides to characteristic weather patterns which many meteorologists used in making difficult forecasts for military and transport operations in unfamiliar regions.

In November 1941, at the request of meteorologists of the Army Air Forces, U.S. Navy, and Weather Bureau, a project was started to analyze daily northern hemisphere sea-level maps for the 10-year period 1929-1939. The responsibility for producing the maps was assumed by the Weather Bureau. A unit of more than 100 map plotters was established in Washington, and an assembly line was set up for extracting the required data from thousands of regional weather maps and publications and plotting them on the northern hemisphere base maps. An analysis unit of 12 of the best available synoptic weather analysts was assembled at New York University, and the analyzed maps were then "polished" by a small crew of girls who soon became so expert with a contour pen that with one swift motion they could ink in a wavy isobar extending around the hemisphere. These maps when reduced photographically and printed became so useful that the series was extended to cover the 40-year period 1899-1939. The urgent demand for the maps made it necessary to finish the added 30-year series in a shorter time than it took to complete the first 10-year series. The plotting, analysis, and drafting staff became so large that at one period late in 1943 over 1,000 persons, including 60 Army officers, were employed. Since it was recognized that a series of historical maps based merely on sea-level conditions would not be sufficient to meet the demands of 3-dimensional weather analysis and forecasting, upper-air charts of many types and other specialized charts and summaries were prepared as part of the same general program.

The value of the charts goes far beyond their original military applications. By breathing life into a mass of inert data, the "frozen assets of meteorology" as Prof. Humphreys calls them, the project has provided an indispensable aid for future research and a key to the solution of peace-time operational problems. It is not intended that the so-called "normal" maps be considered as the final word. As new data are accumulated and as more extensive synoptic charts are drawn both for the surface and for upper levels in the atmosphere, it is hoped that a new set of normals will be constructed. As for the historical charts, it is earnestly desired that they be continued as an international meteorological undertaking and enlarged to portray the daily patterns of the state of the atmosphere over the world.

Most of these maps and charts are now available, and an attempt is made here to describe the type and content of each for the benefit of interested individuals and agencies. The material falls into three classes: (A) Daily Historical Weather Maps, (B) Normal Monthly Weather Maps, and (C) Miscellaneous Maps, Charts, and Tables.

A. DAILY HISTORICAL WEATHER MAPS

(For the most part, the data for these maps consist of reports from national meteorological services, ship reports, and information gathered from manuscript maps.)

*Reproduced, by permission of the authors, from Bull of the Meteorological Society, vol 28, April 1947, pp. 175-178.

1. Northern Hemisphere, Sea Level: Consists of monthly volumes of daily synoptic weather maps for 1300Z for the period January 1, 1899-June 30, 1939, inclusive. Analyzed with the assistance of the meteorological staffs of New York University, California Institute of Technology, and University of California at Los Angeles. 486 volumes, size 13" x 13", most of them published during the Winter 1943-44.

(Northern Hemisphere, Sea Level and 500 mb, for October and November 1945 were published by Headquarters, Army Air Forces Weather Service, Washington, D.C. The October issue includes a complete set of teletype weather reports, surface and upper air, received for each map, listed by country, in the code form used by those countries at the time of the reports. Six more months are ready for printing. It is planned to continue this series indefinitely if possible.)

2. Northern Hemisphere, 3,000 Dynamic Meters: Consists of monthly volumes of 0900Z maps for each day from January 1, 1935 to December 31, 1940, and one 1200Z map for each day from October 1, 1932 to December 21, 1934. Basic data plotted are pressure, temperature, mixing ratio, wind direction, and wind speed at 3,000 dynamic meters. Analyzed with the assistance of the meteorological staffs of University of California at Los Angeles and Massachusetts Institute of Technology. 99 volumes, size 13" x 13", published January 1946.

3. North America, 10, 13, 16 km: Each volume contains twice-daily (1100 and 2300 EST) maps for a single month at a single level for the period 1940-1942. Analyzed with the assistance of the meteorological staff of University of Chicago. 108 volumes, 7-1/2" x 10". Published October 1945.

4. Southwest Pacific, Sea Level: Consists of monthly volumes of daily synoptic maps (0700, 120th East meridian time) for the period of 1932-1934, covering the area bounded by 70°E and 145°W longitude, and 30°N and 50°S latitude. Analyzed with the assistance of the meteorological staff of N.Y.U. 24 volumes, 8" x 13". Published August 1944. Unanalyzed maps available for 1935-37.

5. East Asia, Sea Level: Daily sea-level maps, plotted and analyzed for May, October, November, and December 1937, and March, April, and May 1938. Analyzed with the assistance of the meteorological staff of New York University. In addition, daily sea-level maps were plotted but not analyzed for March, June 1937; June 1935; April 1939. Maps extend approximately from 90°-155°E longitude and from 5°S to 55°N latitude. Unpublished. (Some of these maps were utilized in preparation of many special studies, such as the book "Weather and Climate of China," AAF Weather Div. Rept. 850, 2 vols., 1945, for sale by Supt. of Documents.)

B. NORMAL MONTHLY WEATHER MAPS

1. Northern Hemisphere, Sea Level: Contains normal monthly and annual sea-level distribution of pressure as determined for the 40-1/2-year period (1899-1939) covered by the "Daily Historical Weather Maps, Northern Hemisphere, Sea Level" (A, 1 above). One volume size 13" x 13". Published April 1946. Tables are also available for constructing biweekly normal sea-level charts for the pressure distribution over the northern hemisphere.

2. Northern Hemisphere, Upper Levels: Contains pressure and temperature normals for each month at 10,000 feet, 20,000 feet, 10 km, 13 km, 16 km, and 19 km. List of extensive source material in preface. Analyzed with the assistance of the meteorological staff of N.Y.U. One volume, 13" x 13". Published in 1944.

3. Northern Hemisphere Isograms of Wind Speed and Stream Lines, Upper Levels: Contains isograms of wind speed and stream lines for Northern Hemisphere, one map for each month at each of the following levels: 10,000 feet, 20,000 feet, 10 km, 13 km, 16 km, and 19 km. Based on "Normal Monthly Weather Maps, Northern Hemisphere, Upper Levels" (B, 2 above). Analyzed with the assistance of the meteorological staff of N.Y.U. Unpublished.

4. Normal Contour Maps, Northern Hemisphere, 700 mb: Contains normal height contours and isotherms of the 700-mb surface for each month. Based on "Daily Historical Weather Maps, Northern Hemisphere, 3,000 Dynamic Meters" (A, 2 above). Unpublished.

5. Normal Contour Maps, Northern Hemisphere, 500 mb: Contains normal height contours of 500-mb surface. Consists of three maps for each month, each map extending over a period of ten days. Based on "Normal Monthly Weather Maps, Northern Hemisphere, Upper Levels" (B, 2 above). Unpublished.

6. Normal Thickness between 700 and 1,000 mb, Northern Hemisphere: Consists of monthly maps of height differences between the 700-mb surface (taken from B, 3 above) and the 1,000-mb surface. Latter surface contours computed from Napier Shaw's normal sea-level pressure values. Unpublished.

7. Normal Vertical Cross Sections, Northern Hemisphere: Contains average vertical cross sections from the surface to a height of 19 km, and from 10°N to 70°N latitude, constructed along meridians spaced 20°, starting from Greenwich, by months. Displays (a) temperature and departure of density of dry air from standard atmosphere in grams per cubic meter; (b) N-S and E-W wind components; (c) altimeter correction in feet and lines of zero temperature anomaly. In addition, each map shows the terrain profile covered by the cross section. Based primarily on "Normal Monthly Weather Maps, Northern Hemisphere, Upper Levels" (B, 2 above). Analyzed with the assistance of the meteorological staff of N.Y.U. 648 charts, size 17" x 27". Unpublished.

C. MISCELLANEOUS

1. Extreme Temperature Maps, Northern Hemisphere, Upper Levels: Contains isotherms of absolute maximum and absolute minimum temperature for each month and at each of the following levels: 10,000 feet, 20,000 feet, 10 km, 13 km, 16 km, and 19 km. In the back of the volume there are six maps showing the annual range of normal monthly temperature, one map for each level. Analyzed with the assistance of the meteorological staff of N.Y.U. One volume, 13" x 13". Published July 1945.

2. Five-Day Mean Pressure, Sea Level: Contains 5-day mean maps (2 maps per week) for period covering winter seasons of October 1932-June 1939, inclusive. Based on "Daily Historical Weather Maps, Northern Hemisphere, Sea Level" (A, 1 above). Project still in progress. Unpublished.

3. Five-Day Mean Pressure, 10,000 Feet: Contains 5-day mean maps (2 maps per week) for period covering winter seasons October 1932-June 1939. Based on "Daily Historical Weather Maps, Northern Hemisphere, 3,000 Dynamic Meters" (A, 2 above). Project still in progress. Unpublished.

4. Mean Monthly Pressure, Northern Hemisphere, Sea Level: Contains mean sea level pressure for each month for the period January 1899-June 1939, inclusive. In addition, each map carries the zonal index value for the particular map and a graph of the mean pressure at each latitude (pressure profile) as well as the monthly normal pressure at the same latitude. Based on "Daily Historical Weather Maps, Northern Hemisphere, Sea Level" (A, 1 above). Unpublished.

5. Mean Monthly Pressure, Northern Hemisphere, 10,000 Feet: Contains mean 10,000-ft pressure for each month for period October 1932-December 1940, inclusive. Based on "Daily Historical Weather Maps, Northern Hemisphere, 3,000 Dynamic Meters" (A, 2 above). Unpublished.

6. Normal Pressure and Tendencies for the United States: Data tabulated and summarized by Works Projects Administration, Fort Worth, Texas. Contains (a) four pressure tendency charts for each month for the hours 0100, 0700, 1300, and 1900 EST, each chart containing 3 maps (size 4-1/2" x 7") showing 3-, 6-, and 12-hourly tendencies; and (b) normal hourly station pressure charts (size 10" x 15") for each month for approximately 100 airport stations in the United States. Based on the 1931-1940 barograph trace charts for same stations. One volume. Published in 1943.

7. Normal Flying Weather for the United States: Cooperative project of Army and Weather Bureau; basic data tabulated and summarized by Works Projects Administration at New Orleans (1938-1939) and by Federal Works Agency, Work Projects Administration at New Orleans, New York, and Birmingham (1939-1943). Contains charts (size 4" x 7-1/2") of monthly, seasonal, and annual percentages of frequency of occurrence of various visibilities, ceiling heights, and of the general weather phenomena (fog, haze, smoke, precipitation, thunderstorms, etc.) at more than 300

Weather Bureau Airport Stations. Based on hourly observations for the stations for periods ranging from three to fifteen years. One volume. Published in 1945.

8. Wind Frequency Distribution, North Atlantic Ocean, 6 and 10 km: Contains Baillie wind roses representing frequencies of wind direction and wind speed at 6 km and 10 km, one wind rose map for each month and at each of the two elevations. The second half of the publication contains a percentage frequency distribution of integrated wind components along and at right angles to the principal air routes across the North Atlantic. Based on 0400Z pressure maps for the 3-year period 1942-1944, plotted and analyzed especially for this project. One volume, 6-1/2" x 9". Published September 1945.

9. Wind Frequency Distribution, Northwest Pacific Ocean, 3, 6, and 10 km: Contains Baillie wind roses representing frequencies of wind direction and wind speed at each of the 3-, 6-, and 10-km levels for two-month periods. Based on "Daily Historical Weather Maps, Northern Hemisphere, Sea Level" (A, 1 above) for 1939-1940, "Wind Frequency Distribution, North Atlantic Ocean, 6 and 10 km." (C, 8 above), and pilot-balloon data for several Pacific Ocean stations. One volume, 8-1/2" x 12". Published July 1945.

10. Northern Hemisphere, Tracks of Pressure Centers, Jan. 1899-June 1939: Contains tracks of high and low pressure centers, giving successive positions at 24-hour intervals on a chart of the Northern Hemisphere. There are three charts per month for highs, and three charts per month for lows; each chart covers approximately a ten-day period. Based on "Daily Historical Weather Maps, Northern Hemisphere, Sea Level" (A, 1 above) for the period Jan. 1, 1899 to June 30, 1939. Analyzed with the assistance of the meteorological staff of N.Y.U. Unpublished. (A study of frequencies of highs and lows by 5-degree squares for the period 1929-38 was made by L. W. Sheridan, U.S. Weather Bureau, in 1945. Results of this study may be found in Transactions of the American Geophysical Union, vol. 26, Part I, August 1945.)

The unpublished material listed is still in manuscript form. However, in some instances photostat, photoprint, or microfilm copies are available. Requests for published or unpublished charts should be sent to Chief, U.S. Weather Bureau, Washington 25, D.C.

CHAPTER 11

COAST AND GEODETIC SURVEY DATA — AN AID TO THE COASTAL ENGINEER

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Washington, D. C.

INTRODUCTION

I am grateful for the opportunity to participate in this timely Institute and to present to this distinguished group of engineers and scientists something of the work of the Bureau, which I am privileged to head, insofar as that work relates to the problems of coastal engineering.

In its long career of surveying and chartering the coastal waters of the United States and possessions, a career which dates back to the early part of the nineteenth century, the work of the Coast and Geodetic Survey has been associated with the problems of the coastal engineer. Its successive hydrographic and topographic surveys of the coastal regions furnish basic data for the study of changes in the coastline and adjacent underwater topography and the means to arrest these changes; its tide and current surveys provide the fundamental data necessary in the design of waterfront structures and in harbor improvement; and its geodetic control surveys provide an accurate base for the preliminary study and final construction plans for large-scale improvement projects. To a lesser extent the geomagnetic and seismologic data of the Bureau have also been used by the coastal engineer (Fig. 1).

These and other data comprise a vast reservoir of precise facts concerning the coastal regions of our country. We are constantly adding to this storehouse of knowledge, both qualitatively and quantitatively, through improvements in instruments and techniques. In the field of hydrographic surveying, for example, the development of electronic methods for depth measurement and position fixing has made it feasible to survey large sections of our coastal waters with a completeness of detail and position accuracy comparable to that usually obtainable on a ground topographic survey. This was not possible with the older methods of wire sounding and celestial observation.

In this discussion, I should like to describe briefly the nature of our activities, the kind of data we accumulate, and the criteria by which such data should be evaluated.

BUREAU ACTIVITIES RELATED TO COASTAL ENGINEERING

TOPOGRAPHIC SURVEYS

Of primary importance in problems involving coastal engineering is a knowledge of the location, nature, and form of our sea coasts. Topographic surveys provide this information. Since land features are an essential part of marine charts, the Bureau has been making topographic surveys almost from the very beginning of its work. Scales of topographic surveys are usually 1-20,000 or larger (1-10,000 or 1-5,000), depending upon the importance of the area and the extent of detail to be shown.

Previous to 1922 all topographic surveys were made from the ground using the planetable method. Among ground surveying methods this was probably the most superior for mapping purposes. The terrain was mapped as the surveyor saw it, and no notes were required to be kept for after-plotting in the office. This method made the survey available, as it progressed, for immediate use by hydrographic parties. Some 5,000 surveys were made by this method. Since 1922 an increasing use has been made of aerial photographs. Ground topographic methods are rapidly giving way to the more economical and more expeditious method of aerial photogrammetry. Field inspection of photographs and manuscript maps provides identification of doubtful detail on the photographs, such as the location of the high-water line along a sandy beach, and for the special needs of the hydrographic parties.

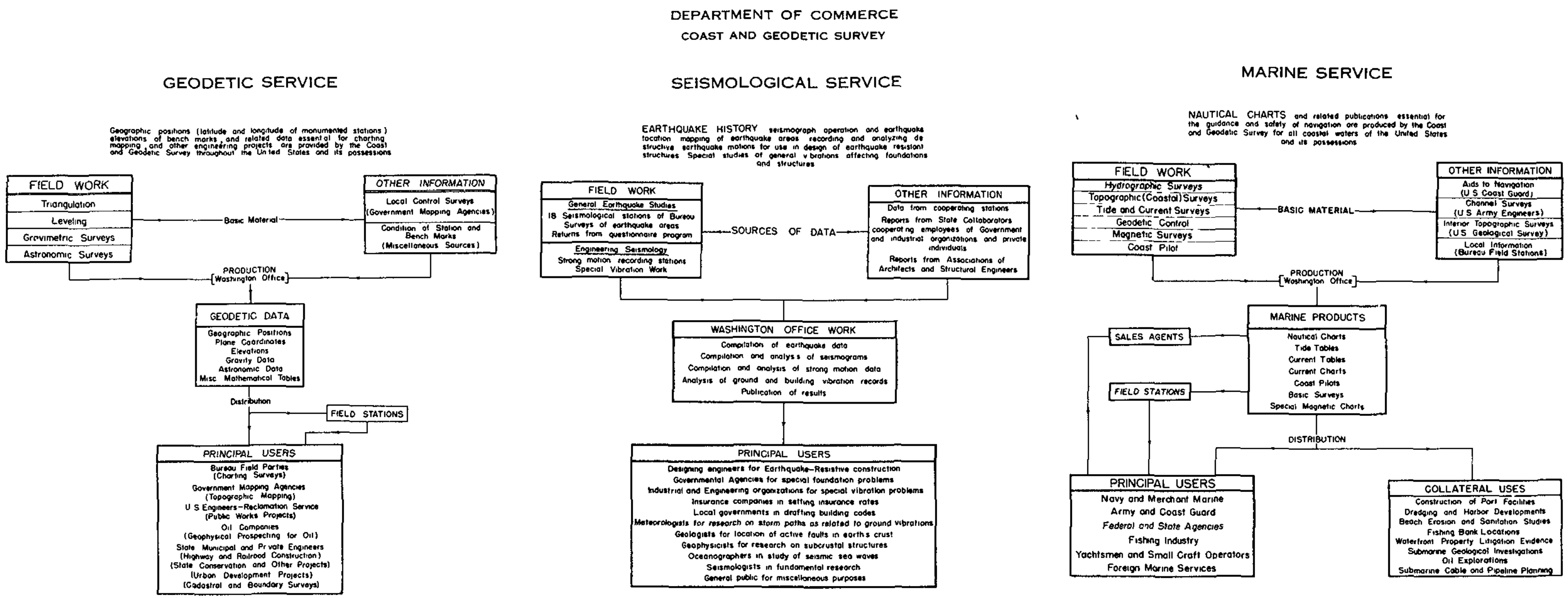


Fig. 1. Types of services available by the Coast and Geodetic Survey.

For its photogrammetric work the Coast Survey has designed a special nine-lens camera, together with transforming, rectifying, and stereoplotting equipment. At an elevation of 14,000 feet, approximately 125 square miles are photographed in a single exposure. To cover the same area with single-lens photographs at the same scale would require twenty exposures.

The high water line. The most significant feature on a topographic survey of the coast is the shoreline It is the dividing line between land and water. On Coast and Geodetic Survey charts it closely approximates the mean high-water line. The topographer attempts to locate this line on his survey as accurately as is consistent with the methods used and the purpose for which the survey is intended.

The true mean high-water line is the intersection of the plane of mean high water with the shore. The identity of this line on the ground for purposes of fixing property boundaries can be determined accurately by means of levels run from established bench marks. The line can also be identified on aerial photographs taken at the time of mean high water. For charting, such precise methods have not been found necessary, and the topographer usually identifies the mean high-water line from distinctive lines of drift, compactness of sand, and other physical appearances of the beach. His judgment is the basis for the location of the line on our topographic surveys.

(Accuracy of determination.) The closeness with which the line can be identified depends upon the character of the shore. On a rocky or steep-to beach, the identification would be accurate because a short horizontal distance is covered or exposed during the rise or fall of the tide. On gently sloping beaches, the estimation of the high-water line is necessarily approximate, and the line is seldom located with greater accuracy than within 10 feet of its true position. The many predetermined triangulation points within the area of a topographic survey provide a constant check on the work and no large errors can accumulate.

High-water line in tidal marshes, etc. In areas of marsh grass, mangrove, or other similar marine vegetation, where the shoreline is generally obscured, the surveyor makes no attempt to locate the actual mean high-water line, but rather a line that marks the outer or seaward edge of the marsh or marine vegetation, because to the navigator this is the visible shoreline. The true mean high-water line in such areas is generally a meandering line located somewhere between the outer edge of the vegetation and the fast land inshore. Under certain stages of marsh development, the whole of the marsh area may be precisely at mean high water, but from the topographic survey alone, no inference can be drawn regarding the position of the mean high-water line. Other collateral information is usually necessary, such as the contemporary hydrographic survey of the area.

The low water line. Another feature on coastal topographic surveys that is important to the coastal engineer is the low-water line -- symbolized by a series of dots. A word of caution is necessary regarding the use of this line from topographic surveys. From the survey alone, there is no evidence that the line is the true low-water line. During a large part of the time when the topographer is at work the low-water line is covered and it is impossible for him to locate it by measurement. The low-water line on our topographic surveys is a sketched line and does not represent a definite plane of reference. For charting purposes, the line is developed from the hydrographic survey, supplemented wherever necessary with information from the topographic survey.

HYDROGRAPHIC SURVEYS

Hydrographic surveying, as applied to the work of the Coast Survey, consists essentially of measuring depths -- this is, taking soundings -- and determining the definite locations of the depths even though their positions may be out of sight of land. Besides their value for charting, hydrographic surveys provide the coastal engineer with information on the underwater slope immediately adjacent to the shore. Successive surveys reveal whether there has been a steepening or flattening of the shore and throw light on probable future changes.

In the offshore areas the measurement of great depths can now be obtained in a matter of seconds, making it possible to take thousands of soundings in areas where formerly only a few scattered ones were economically feasible. This is greatly augmenting our knowledge of the ocean floor which not only contributes to the safety of navigation but provides important data for use in several of the earth sciences.

Modern methods. During the century and a quarter that we have been engaged in charting work, many changes have taken place in the methods of hydrographic surveying, each change resulting in an accumulation of more accurate and more detailed information. The most significant of these changes occurred during the periods following the two World Wars. The measurement of ocean depths by handlead and wire gave way to echo sounding, which is a measurement of the time a sound echo takes to return from the bottom beneath the vessel. The uncertain methods of position determination, such as dead reckoning and celestial observation, have been supplanted by electronic methods for measuring distances to known points previously determined.

(Echo sounding.) Modern echo-sounding equipment, designed for use in small launches, furnishes detailed information on depths of water close to the beach. The continuous profiles traced by graphic recorders show characteristic patterns of the underwater topography and details which were frequently missed by the older handlead-and-line method of sounding. An interesting example of underwater topography was disclosed recently in a hydrographic survey in San Francisco Bay off one of the beaches. A distinctive saw-tooth pattern closely resembling sand wave forms was disclosed by the fathogram. Another fathogram survey of an inshore area in the Gulf of Mexico revealed a narrow along-shore ridge with a depth of 6 feet of water over it. This hitherto unknown ridge occurred between inshore and offshore depths of 12 feet.

The fathograms of echo-sounding equipment also show quite clearly, under certain conditions, layers of silt or other loosely distributed sediment overlaying the substrata, such as are commonly found in bays, lakes, or estuaries.

Experience has shown that the most effective method of surveying an inshore area, where the bottom slopes gradually, is by a system of sounding lines parallel to the beach, with an occasional cross check line. The first line of soundings is run as close to the high-water line as practicable. The next lines are closely spaced and do not exceed 50 meters. This interval is increased as the survey progresses offshore. Our operations are planned to take advantage of periods of high tide and calm weather which afford the best conditions for inshore sounding.

(Shoran and E.P.I.) For position fixing in hydrographic surveying we have developed two electronic systems. One is Shoran, or Short Range Navigation, which is an adaptation of a method used by the Air Forces during World War II for strategic bombing. Shoran constitutes no new principle in hydrographic surveying, but it does apply a new and effective method for measuring distances from control points. Shoran does the job quickly, accurately, and under adverse weather conditions. Shoran is based on the fact that radio waves travel through the atmosphere at a very nearly constant velocity of approximately 186,000 miles per sec. Success of the method is due to the accomplishment of electronic engineers in devising means for accurate measurement of the remarkably small time intervals involved in the travel of electromagnetic waves to a target and back. In the familiar radar, dependence is placed upon the reflection of radio waves from natural objects encountered. Shoran strengthens and specializes this principle by use of responding radio stations set up at known points, which return intensified signals. Distances can be read to hundredths or even thousandths of a mile. Two such distances are measured simultaneously and thereby determine the position of the survey ship. Shoran will measure distances with a probable error of about 8 meters in a single measurement.

The radio frequencies used in Shoran are in the ultra-high-frequency bands and are of the order of 250 megacycles per sec. The range of the system is limited to line-of-sight distances. With a normal installation, control is possible over an area extending 50 to 75 miles from the ground stations. We have used Shoran successfully off the Atlantic coast and in Alaskan waters off the western Aleutians during the past 5 years.

Because of the line-of-sight limitation of Shoran, the Coast and Geodetic Survey developed an Electronic Position Indicator (E.P.I.) for use beyond the limits of Shoran. E.P.I. utilizes very low frequencies of the order of 2 megacycles per sec. which can be detected for great distances because they follow the curvature of the earth. The principles of position fixing with E.P.I. are essentially the same as with Shoran. The range of E.P.I. does not depend on elevation and therefore the system can be used with control stations of relatively low height. Distances close to 300 miles have been measured in the Gulf of Mexico. The over-all accuracy of the system is about 75 meters and is independent of distance from the control station.

(Accuracy of echo soundings.) Questions often arise as to the accuracy of the new methods of hydrographic surveying. With respect to depth measurement, it can be stated that, insofar as the equipment is concerned, the visual type of echo sounder, known as the "Dorsey No. 3," which was developed by the Coast and Geodetic Survey, has a probable reading accuracy of 1/10 ft. for soundings of 100 fathoms and less. The graphic recorder type of echo sounder using supersonic frequencies (up to 25 kilocycles per second) gives good results for depths of from 3 ft. under the keel to greater than 4,000 fathoms. The accuracy of these instruments is about 1:1,000 on the expanded shoal scales and about 1:100 on the compressed deep-water scales.

Echo-sounding instruments are usually calibrated for a velocity of 4,800 ft. per sec., whereas the velocity of sound in the different oceans varies from 4,500 to 5,100 ft. per sec. Depths read on the instrument must therefore be corrected for this difference, which constitutes perhaps the greatest single factor affecting the accuracy of an echo sounder. Inasmuch as velocity is dependent upon the temperature, salinity, and depth of the water and as these factors vary both seasonally and regionally, our specifications call for a determination of these physical characteristics with such frequency that the resulting depths will not be in error by more than 1 percent of the true depth. Thus, a sounding of 100 ft. should be correct within 1 ft., and a 1,000-ft. sounding within 10 ft.

It is of interest to note that, in the early period of echo sounding, the standard of comparison was the leadline or wire, and echo soundings were frequently corrected from comparisons made with these standards. With improvements in the time-measuring device and in operational techniques, greater accuracy was gradually achieved. Today echo soundings are considered more accurate than the old standard of comparison, since they are not subject to the inherent and uncontrollable errors and difficulties associated with the leadline and wire.

(Low-water line on hydrographic surveys.) On hydrographic surveys the line of zero soundings, generally called the low-water line, represents the line where the plane or datum adopted for the soundings intersects the shore. On the Atlantic coast the reference plane is mean low water, while on the Pacific coast it is mean lower low-water. Soundings obtained inshore of the zero line would be shown as minus depths on the survey, indicating that the area is exposed at low water.

In general an attempt is made to fully develop the low-water line on our hydrographic surveys wherever tidal conditions permit. This is not always possible, particularly along an exposed coast, such as large portions of the California shoreline. Many of our surveys therefore do not show a continuous low water line. For charting purposes low-water in such cases is represented by an interpolated line based on the high-water line on the topographic survey and the inshore soundings on the hydrographic survey. In using such information for purposes other than charting, the engineer should keep in mind the accuracy limitations imposed by the survey methods used. The low-water line is located by methods adequate for purposes of navigation. To accurately delimit the line for fixing property boundaries would require, as in the case of the high-water line on topographic surveys, running lines of levels from established bench marks or using an aerial photographic method.

(Character of sea bottom from hydrographic surveys.) The determination of the character of the sea bottom -- that is, its consistency, color, and classification--is an essential part of every hydrographic survey. Although only the immediate top layers of the bottom are sampled, the value of such information to the mariner for choosing an anchorage, to the fisherman for avoiding types of bottom likely to damage his equipment, to the engineer engaged in dredging operations or in underwater construction, and in a limited sense to the student of the earth sciences, is recognized and our field parties are given detailed instruction for gathering such data.

The frequency with which data on bottom characteristics are obtained depends upon the nature of the area and the method of survey. In harbors, anchorages, and channels and on shoals and banks the coverage is generally more complete. Along the open coasts and in large bays, where tests have indicated that a sameness of bottom material is to be expected, fewer samples are taken.

The character of the bottom is determined either by "feel" or by bringing up a sample for examination with the leadline or with a special snapper device. No samples are retained as a permanent record, except by pre-arrangement with a scientific institution or a commercial establishment. The basis for the classification of sediments is the size of the particles composing them. No mechanical analysis is used for typing a sediment. An estimation of its dimensions is made by eye, and classification is based on an available table covering the range from "ooze" to "boulder." Standard abbreviations have been adopted on our survey sheets and charts for indicating the character of the sea bottom; for example "hrd SShP" denotes "hard sand, shells, pebbles." The adjective part of the characteristic is shown with lower-case letters and the noun part with capitals. An appropriate legend covering abbreviations is included on our nautical charts. Because of their larger scale, many more bottom characteristics appear on our survey sheets than are shown on the published charts.

HORIZONTAL CONTROL SURVEYS

It was recognized at an early period in the history of the Coast Survey that an undertaking of such vast magnitude could not be attacked as a problem in ordinary surveying. To be of lasting value, the shape and size of the earth must be taken into account, and accurate latitudes, longitudes, and azimuths determined from astronomic observations and from triangulation. This was the method adopted at the beginning of our work. The great network of horizontal control which has since been established along our coasts and in the interior of the country forms a rigid geodetic framework for all our surveys and charts (Fig. 2).

A network of monumented points. To span the continent from coast to coast and from north to south, widely spaced arcs of triangulation were first established. This gave a preliminary framework. Intermediate arcs, spaced 40 to 60 miles apart, provided control for boundary determinations and general purposes of the various States. The next logical step has been followed in the last decade and work has begun on filling in the areas between these arcs. There are now 115,000 miles of arcs of first- and second-order triangulation in the United States, and monumented stations have been established at an average of about 10 miles along the various arcs. Monumented stations are also placed at 4-mile intervals along the main highways in agricultural areas with closer spacing in metropolitan areas and along the coasts. In addition, a large number of prominent objects, such as water tanks, church spires, cupolas, chimneys, etc., have been located in connection with the triangulation, making a total of about 150,000 stations for which geographic positions (that is, latitude and longitude) have been determined. To facilitate the use of the triangulation stations by local surveyors and engineers, a nearby azimuth mark is established which gives directional control and avoids the need of establishing a true meridian line.

Although great strides have been made by the Bureau in establishing this basic network of control for the country, our job is far from completed. Our concept of adequacy is continually changing, as it necessarily must, with the ever-widening needs of commerce and industry. Our present policy is to provide for at least one triangulation station in each 7-1/2-minute quadrangle map. To coordinate local urban surveys, at least one Federal base line is measured in metropolitan areas of 100,000 population or over.

Development of the North American 1927 datum. All the horizontal control work of the Coast and Geodetic Survey, with few exceptions, is now referenced to a single geodetic datum, the North American 1927 Datum. This is a fact of great practical significance to the engineer who uses the horizontal control data of the

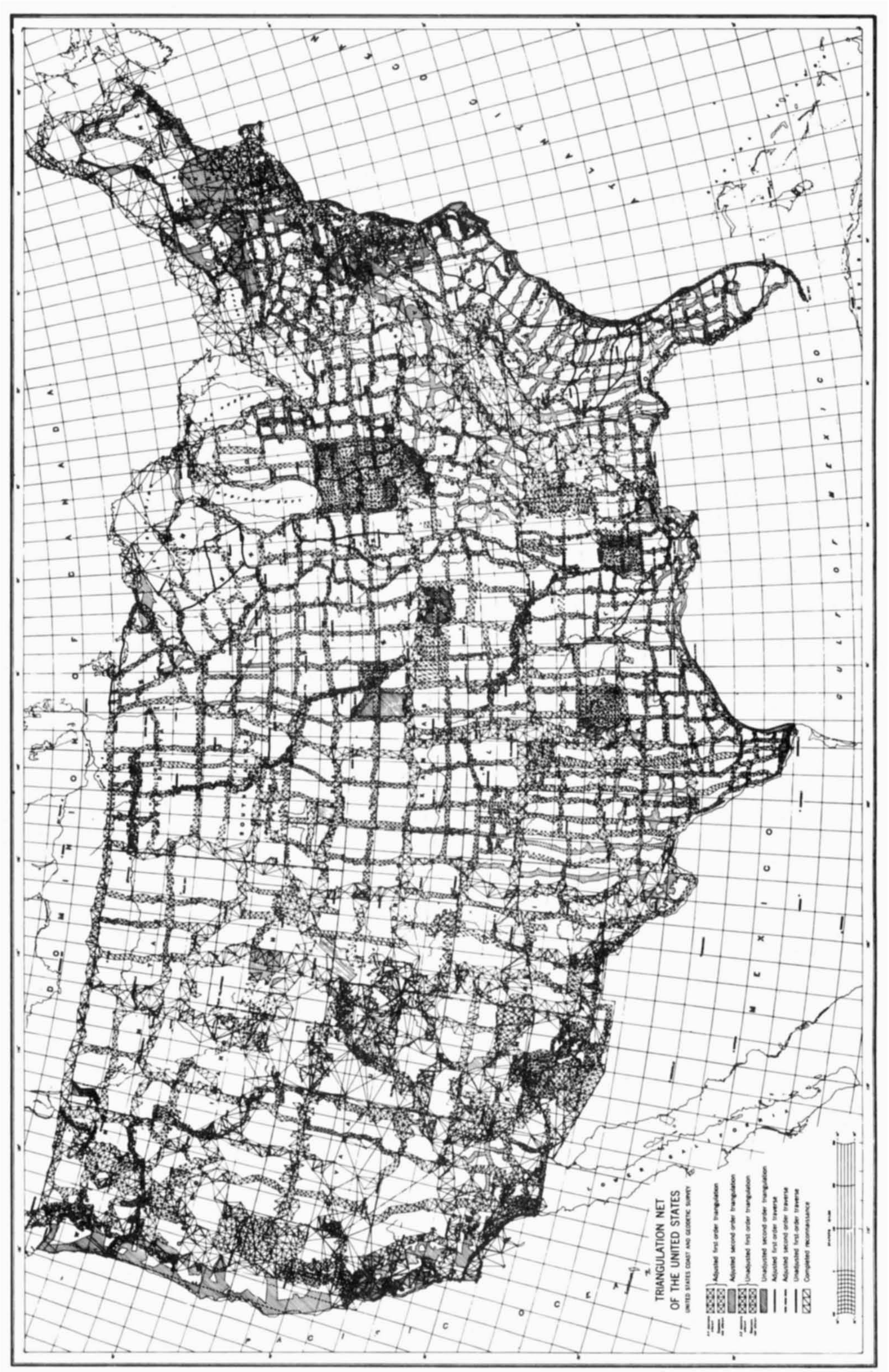


Fig. 2. Triangulation network of the United States

Bureau or the surveys based on such control. But it is important to remember that this was not always the case.

The horizontal control surveys of the United States were begun during the early part of the nineteenth century and existed at first as separate surveys, each based on one or more astronomical determinations of latitude, longitude, and azimuth. Examples of such detached surveys were the early triangulation in New England and along the Atlantic coast; a detached portion of the transcontinental arc centered on St. Louis, and another portion of the same arc in the Rocky Mountain region; and three separate surveys in California, in the vicinities of San Francisco, Santa Barbara Channel, and San Diego. With the lapse of time these separate pieces of triangulation were expanded until they joined and difficulties immediately arose. The positions of common points computed from different triangulation schemes were found to differ by varying amounts, because each scheme was based on its own astronomic determinations, and such determinations are affected by the irregular distribution of masses in the earth's crust.

In any engineering or scientific undertaking involving a large area, such as the United States, it is essential that full coordination and correlation be obtained of the surveys, maps, and charts of the area. A depth along the coast or a point on shore can have but one latitude and longitude, which should be the same on every map or chart on which such feature appears. This can be accomplished only by establishing a single geodetic datum for the area, that is, by having the position of a single point in the country as the initial or datum to which all other stations are referred. This became possible about 1900, when the transcontinental arc was completed which joined all the detached portions into one continuous triangulation.

On March 13, 1901, a single datum was adopted by the Bureau and was named the United States Standard Datum. This datum corresponded to the one in use in the New England States, and hence did not change the latitudes and longitudes of triangulation stations in that area. In 1913, Canada and Mexico adopted this datum and its designation was changed to North American Datum to reflect its international character. The two datums are, however, identical.

As the triangulation of the country expanded and the principal arcs were completed it became necessary to make a unified adjustment of the whole network, first the western half -- begun in 1927 -- and later the eastern half. This new adjustment assigned new values to all points except station Meades Ranch, which was held fixed. The new datum was called the North American 1927 Datum.

The engineer who uses surveys or charts made prior to the adoption of the North American 1927 Datum must keep in mind this development of a single geodetic datum. From the standpoint of datums the greatest differences in the geographic positions of triangulation stations occurred with the adoption of the United States Standard Datum.

Accuracy of horizontal control. Control surveys are classified as nearly as possible according to the accuracy of the resulting lengths and azimuths of the lines. Since the absolute errors of these quantities cannot be ascertained, indirect gages must be used. For triangulation, the principal criterion is whether the discrepancy between a measured length of a base line and its computed length, as carried through the network from the next preceding base, is less than a certain fraction of the measured length. For first-order work, the computed length must agree on an average within 1 part in 75,000 or about 1 ft. in 14 miles. Base lines are measured in both directions and are generally from 4 to 8 miles long with a probable error for the mean of the two measurements averaging about 1 in 2,000,000.

Another important indirect gage of the accuracy of the final results of triangulation is the average closure of the triangles. To insure adequate agreement among the component parts of the triangulation, basic criteria have been adopted for the observations. For first-order work, the requirements are an average triangle closure not in excess of 1 sec.

State plane coordinates. The results of the triangulation of the Coast and Geodetic Survey are expressed in terms of geographic coordinates -- that is, latitude and longitude. Such coordinates are most convenient where extensive areas

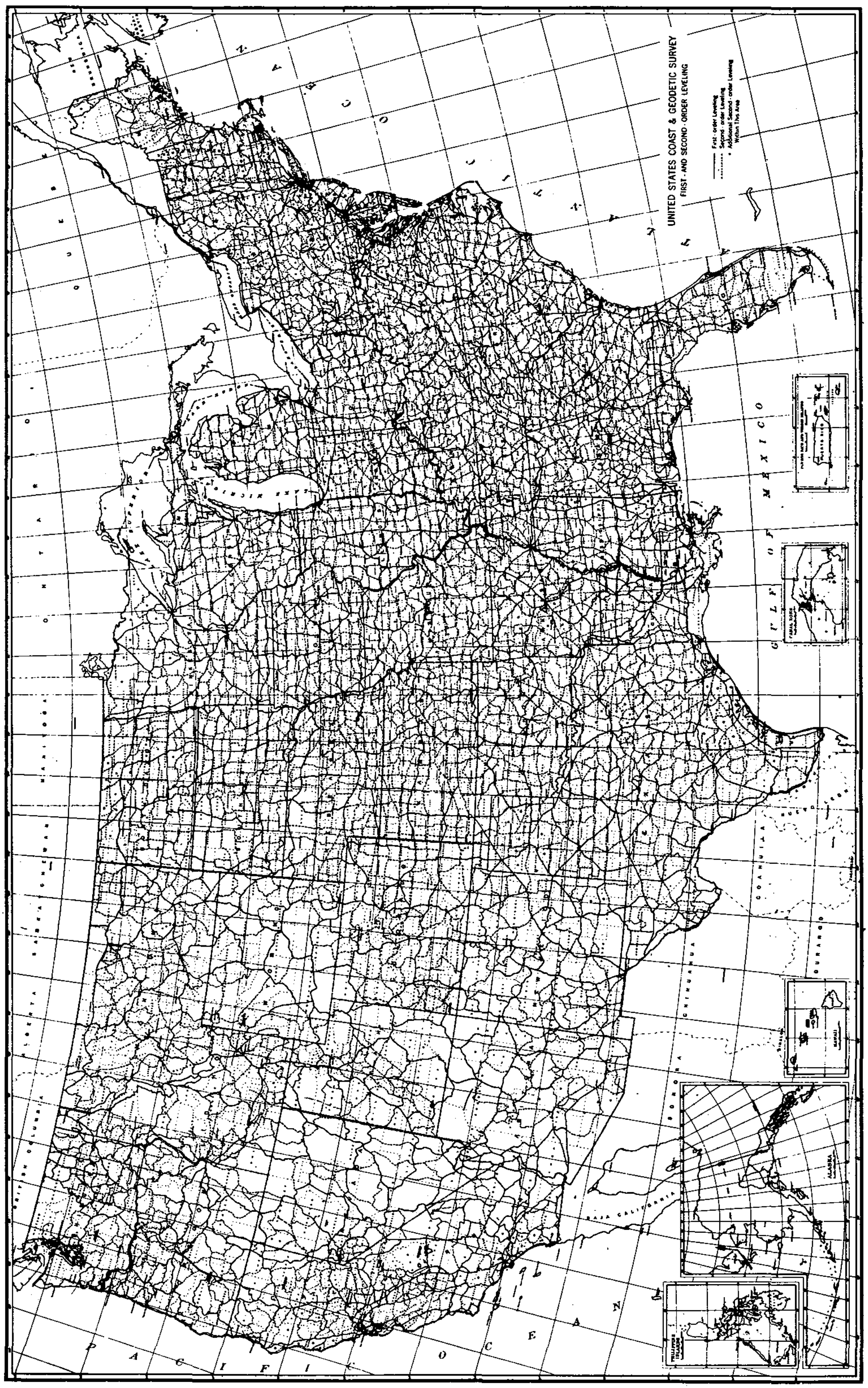


Fig. 3. Precise leveling lines of the United States

are involved. They constitute a universal system, all points of which are directly related. We have recognized, however, that engineers and surveyors, unfamiliar with the computational methods involved, hesitate to use geographic coordinates for surveys of smaller areas, such as those performed within a state, county, or city. Considering the desirability of having all surveys tied into the Federal network of geodetic control, the Coast Survey in 1933 devised systems of plane coordinates for each state and developed the formulas for the transformation of geographic coordinates to their corresponding X and Y values. By these systems, points on the earth's surface are mathematically projected upon a surface which can be developed into a plane. Surveys between such points can then be treated as though the work were accomplished on a plane instead of a spheroidal surface.

In the practical field use of State Coordinates, the engineer needs only the plane coordinates of the triangulation stations and the plane azimuth to an azimuth mark within the area covered by his survey. These data are sufficient for him to run and adjust his traverse, using the ordinary methods of plane surveying, and obtain an accuracy of not less than 1 part in 10,000, without any corrections for scale error. The local engineer may bring his computations to geodetic accuracy, if such refinement should be desired, because the State Coordinate Systems are based on definite systems of projections with definite scale corrections.

VERTICAL CONTROL SURVEYS

In providing for the vertical control of the country, the Coast and Geodetic Survey followed a program starting with widely spaced lines 100 miles apart, later supplemented by lines spaced at 50-mile intervals (Fig. 3). At the present time, we are providing area leveling with lines spaced 6 miles apart connected to the wider spaced lines in a consistent network of levels. Bench marks are set at 1-mile intervals along each line. For convenience in running and in the subsequent use of bench marks, the lines of leveling usually follow the routes of highways and railroads. There are now more than 370,000 miles of first- and second-order leveling in the United States and upward of 275,000 established bench marks.

First-order leveling represents the most exact method of determining elevations. Lines are run in both directions and the two runnings must be such that in a 100-mile circuit the error of closure will on an average be only slightly over an inch.

Sea level datum of 1929. The reference datum for elevations in the vertical control network is MEAN SEA LEVEL. Originally, the leveling was extended from a tide station at Sandy Hook, N.J., to furnish accurate vertical control for the transcontinental arc of triangulation following approximately the 39th parallel. As the net expanded and new circuits developed with additional sea level connections, it was found desirable to make adjustments in the elevations. The first adjustment was made in 1899, with partial readjustments in 1903, 1907, and 1912. A complete readjustment of the network was made in 1929, in which sea level was held fixed as observed at 26 tide stations, 5 in Canada and 21 in the United States. Elevations in this adjustment are referred to as being based on the "Sea Level Datum of 1929."

TIDAL SURVEYS

The tidal work of the Coast and Geodetic Survey had its origin in the need for reducing to a common level, or datum plane, soundings taken at different stages of the tide during hydrographic surveys, so that nautical charts would show all depths referred to a uniform datum. The further needs of the mariner were met with the publication of tide tables giving the predicted times and heights of the tide annually in advance. Within recent years the needs of the engineer have been given consideration, and in 1922 systematic tide and current surveys were begun of all the important harbors.

Engineering aspects of tides. Tidal observations are of particular interest to the coastal engineer. The rise and fall of the tide is a continuing phenomenon and varies from day to day and from place to place. Thus, at New York the mean range is about 4.5 ft. while the maximum range may be 7 ft. At the Atlantic entrance to the Panama Canal the range is less than a foot while at the Pacific entrance it averages 12.5 ft. On the other hand, at Anchorage, Alaska, a rise and fall of approximately 35 ft. may be encountered on certain days.

Tides also differ in the character of the rise and fall. At New York, for example, there are two tides a day of approximately equal range; at San Francisco there are two tides a day of unequal range; and at Pensacola, there is but one tide a day.

These and other aspects of the tidal phenomenon are matters which the coastal engineer must take into account in planning or designing waterfront structures.

The accumulated tide records of the Coast and Geodetic Survey furnish the fundamental data required in the establishment or datum planes based on tidal definition; in the prediction of tides; in the determination of mean and extreme ranges of tide; and in the study of changes in the plane of mean sea level and its correlative coastal stability.

Primary tide stations. Although tidal constants and tidal datum planes may be established (within certain limits of accuracy) from observations extending over a month or a year, for geodetic and scientific purposes continuous observations for a period of 19 years are required. This takes into account all changes due to astronomic causes and tends to balance out the disturbing effects of wind and weather.

For the collection of tidal data, the Bureau has in operation some 80 primary stations at coastal ports where automatic tide-gage installations provide continuous graphic records of the rise and fall of the tide over a long period of years. Most of the basic data are obtained from this source. In addition, short-period observations are obtained in connection with our hydrographic operations or for some other purpose. By means of simultaneous observations with primary stations, short-period observations may be converted to long-period means with an accuracy closely approximating the 19-year cycle.

(The 1924-1942 epoch.) Long-period observations indicate a slow secular change in sea level. Therefore, in defining tidal datums, it is necessary to identify them with a particular 19-year group. To make datums comparable at all localities the same group of years must be used. In the Coast and Geodetic Survey the 1924-1942 epoch has been adopted for datum plane reference.

(Changes in sea level.) Long series of tidal measurements furnish the only quantitative data for the study of changes in sea level and the important geophysical problem of coastal stability, that is, whether any given coastal region is rising or sinking relative to the sea. For example, along the Atlantic coast of the United States, investigations indicate that in the last 20 years mean sea level has risen, or the land has subsided, about 0.3 ft.; while on the Pacific coast sea level has risen about 0.1 ft. On the other hand, at Skagway, Alaska, observations over the last 40 years show a fall in sea level of over 2 ft.

Determination of datum planes. Besides the datum of mean sea level, which is used for the vertical control surveys of the country, other datums are established from tide observations for use in our hydrographic surveys and nautical charts, and for other purposes.

(Chart datums.) Chart datums are selected primarily for their practical utility to the navigator and depend upon the characteristics of the tide in a given area. The aim is to provide the mariner with as wide a margin of safety in navigating his vessel as will be consistent with prevailing conditions. From the standpoint of navigation, the critical part of the tidal cycle is at the time of low water. At this time depths in a channel or over a shoal area are at a minimum. If a datum higher than low water were to be used as a reference plane, depths shown on the nautical charts would be greater than actually exist at the time of low water. This might result in giving the mariner a false sense of security, particularly in areas where the controlling depth approaches the draft of his vessel. Another practical advantage of a low-water datum is that tidal corrections given in the Tide Tables, which the mariner uses in conjunction with the chart to find the depth at a specified time and place, will be mostly additive values. It is for these reasons that low-water datums have been adopted for the nautical charts of the Coast and Geodetic Survey.

But even low-water datums differ on the different coasts of the United States, depending upon the prevailing type of tide. On the Atlantic coast, for example, the tide is of the semidaily type, with two tides a day of approximately equal range. Successive low waters differ but slightly and the adopted chart datum is MEAN LOW WATER, which is the mean of all the low waters in a given area. On the Pacific coast, the tide is of the mixed type, with two tides each day of unequal range. Successive low waters exhibit a marked inequality and the datum of MEAN LOWER LOW WATER is used, which is the mean of all the lower of the two low waters each day.

The advantages of using a mean lower-low-water datum over a mean low-water datum for Pacific coast charts are similar to those described above for low-water datums. The important thing to keep in mind is that the selection was dictated by the practical needs of navigation. Its use should be so appraised.

In order that datum planes once derived may be preserved for future use, they are referenced as so many feet below bench marks established in the vicinities of tide stations. This makes the recovery of datum planes a simple matter, so long as the bench marks are maintained.

Tidal surveys of important harbors. In some areas a need has developed for more detailed tidal information than is provided by the nearest primary stations and those stations established in connection with hydrographic surveying. To supplement these sources special tide surveys have been made in selected areas. Numerous tide-gage installations, at carefully selected sites, are required in most coastal harbors, or in any system of tidal waterways, to determine the varying times and heights of high and low waters at critical points. The first of such systematic surveys was begun in New York Harbor in 1922.

Prediction of tides. Mention has been made of the use of the Coast and Geodetic Survey Tide Tables for navigational purposes. These tables, which give the daily predictions of the times and heights of the tide at the important ports of the world, can also serve the coastal engineer who may wish to know in advance the height of the tide that may be expected at a given time and place.

To predict the tides for any given port, tidal observations must first be obtained to determine the characteristics of the tide at that port. From these data, predictions can then be made for any date in the future. Tide prediction is a complicated mathematical process; however, the work has been greatly simplified through the design and use in the Bureau of a tide predicting machine which can reproduce the tide in nature by solving equations involving as many as 37 variables. Tide Tables are published by the Bureau approximately 6 months in advance.

(Accuracy of tide tables.) To test the accuracy of the Tide Tables, comparisons have been made for different ports between predicted tides and observed tides. These tests indicate that, under normal weather conditions, the predicted tides closely approximate the actual tides. At Los Angeles, for example, where the tide is of the mixed type with a mean range of 3.8 feet, a full year of comparisons showed that 90 percent of the predicted times of high and low waters agreed within 5 minutes of the observed times; 98 percent of the predicted heights agreed within half a foot of the observed heights; and 59 percent agreed within one-tenth foot.

TIDAL CURRENT SURVEYS

Observations of the strength and direction of tidal currents along our coasts and in tidal waterways have been made by the Coast and Geodetic Survey in connection with hydrographic operations and as special surveys, as an aid to navigation. Currents must also be considered by the engineer engaged in the maintenance and improvement of channels and harbors, in marine construction and improvement of beaches, and in the problem of sewage disposal.

Comprehensive tidal current surveys have been made of our more important harbors and waterways. In recent years current observations have been greatly expedited by the Bureau's development of the Roberts Radio Current Meter, which not only measures the velocity and direction of the current but transmits the data by radio to a central receiving station. As many as eight meters can be operated simultaneously in an area.

Tidal current data are used in the Current Tables published by the Bureau annually in advance. The tables give daily predictions of the times of slack water and the times and velocities of strength of flood and ebb currents for numerous places along our coasts and in our waterways.

AVAILABILITY OF COAST AND GEODETIC SURVEY DATA

An important part of the work of the Coast and Geodetic Survey is the dissemination of its technical information, which is meticulously collected, analyzed, and compiled, and made available to the public in the form of charts, maps, and printed publications. Although our nautical and aeronautical charts and related publications are well known to mariners and aviators, we frequently find that much of the fundamental data on which these products are based is unknown to those dealing with engineering and scientific problems.

Some of these data, of special interest to the coastal engineer, and the forms in which they are available are described in the balance of this paper. Illustrative samples of indexes and other forms of available data are included at the end.

TOPOGRAPHY AND HYDROGRAPHY

Copies of field surveys. Photographic copies of field topographic and hydrographic surveys are available at the nominal cost of reproduction. Detailed topographic and hydrographic surveys have been made since 1834 of most of the coastline of the United States and Alaska and of our island possessions. Along some portions of our coast, surveys have been repeated at periodic intervals.

Most of the topographic surveys compiled from aerial photographs are published as lithographic prints. Planimetric maps show nearly all interior topographic features except contours and elevations. More than 1,000 planimetric maps are now available which cover the greater part of the Atlantic coast and extensive areas of the Gulf and Pacific coasts.

Notwithstanding their accuracy limitations, as already explained, these surveys nevertheless represent an authentic historical record of the evolution of our coastline and the underwater features, useful in the study of beach erosion and protection and for other engineering purposes. The scales are large enough (usually 1:10,000 and 1:20,000) to provide the engineer with a background of information from which a quantitative appraisal can be made of changes.

An important by-product value of our topographic and hydrographic surveys is their use in waterfront boundary disputes where the seaward limit of property is defined by the high- or low-water line. Our early surveys may be very informative in cases where it is important to determine, as of a specific date, whether a coastal strip was exposed, or covered with water, at high tide.

A cautionary note must be sounded, however, regarding the use of original field surveys made at different periods. Because different geodetic datums were used at various periods in the history of the Bureau, surveys must be brought to the same datum before an accurate comparison can be made of shorelines or other data. Copies of old surveys furnished from the Washington Office usually show at least one projection intersection based on the North American 1927 Datum.

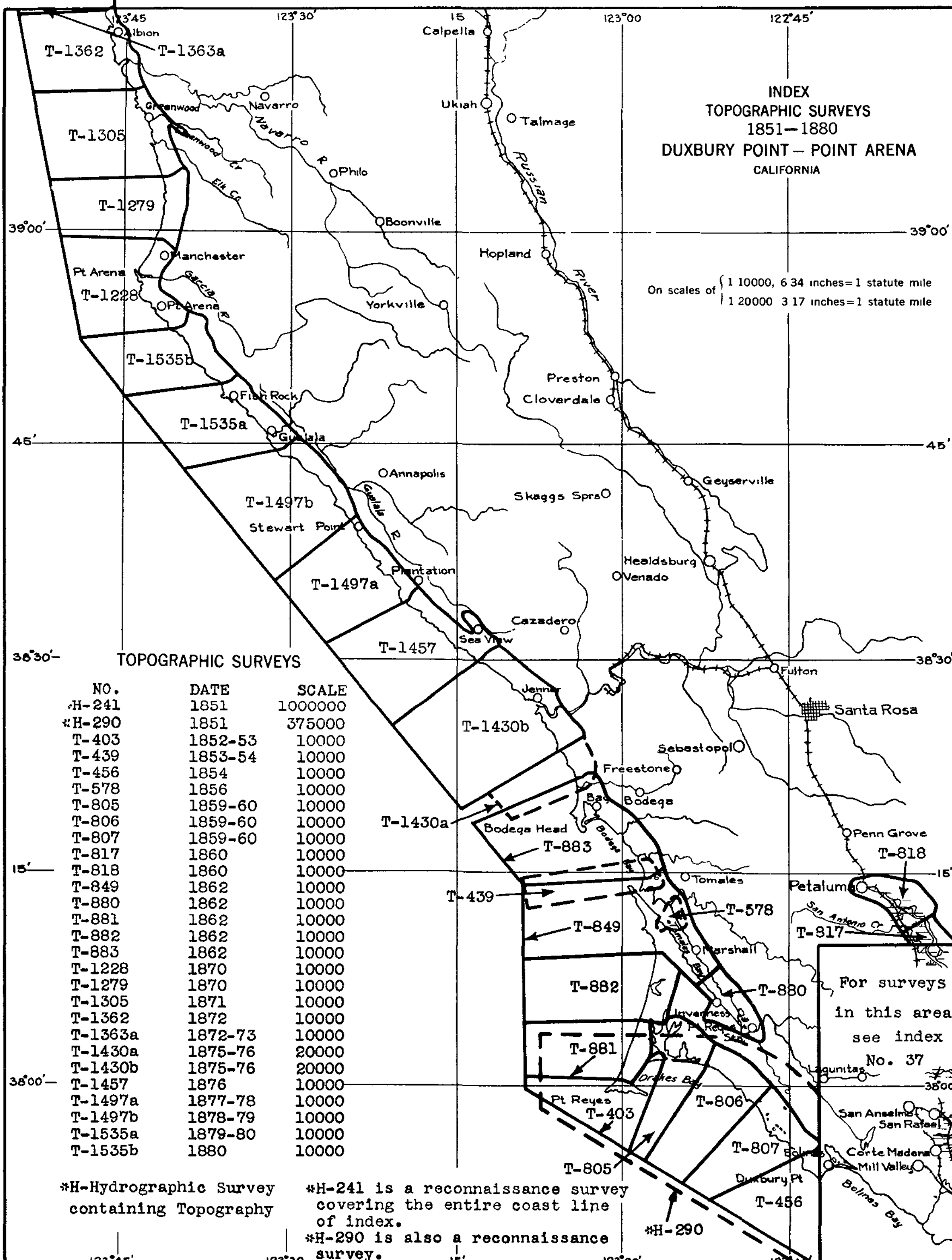
Survey indexes. Index maps -- showing the date, area covered, and scale of each survey -- are available for the planetable and planimetric surveys along the Atlantic, Gulf, and Pacific coasts, and for the hydrographic surveys along the Atlantic and Gulf coasts (Figs. 4 - 9, inclusive). They are furnished on request without cost.

CONTROL SURVEY DATA

State index maps. Index maps are available showing the triangulation and leveling accomplished in each state. The scales of the maps are large enough (1:650,000) to show graphically the locations of the individual stations and bench marks. On the leveling map the lines are numbered and the engineer desiring information on any particular level line can refer to the line by its number.

DEPARTMENT OF COMMERCE
U S Coast and Geodetic Survey
Washington, D C

Topographic Index No 39 A



TOPOGRAPHIC SURVEYS

NO.	DATE	SCALE
*H-241	1851	1000000
*H-290	1851	375000
T-403	1852-53	10000
T-439	1853-54	10000
T-456	1854	10000
T-578	1856	10000
T-805	1859-60	10000
T-806	1859-60	10000
T-807	1859-60	10000
T-817	1860	10000
T-818	1860	10000
T-849	1862	10000
T-880	1862	10000
T-881	1862	10000
T-882	1862	10000
T-883	1862	10000
T-1228	1870	10000
T-1279	1870	10000
T-1305	1871	10000
T-1362	1872	10000
T-1363a	1872-73	10000
T-1430a	1875-76	20000
T-1430b	1875-76	20000
T-1457	1876	10000
T-1497a	1877-78	10000
T-1497b	1878-79	10000
T-1535a	1879-80	10000
T-1535b	1880	10000

*H-Hydrographic Survey containing Topography

*H-241 is a reconnaissance survey covering the entire coast line of index.
*H-290 is also a reconnaissance survey.

Fig. 4

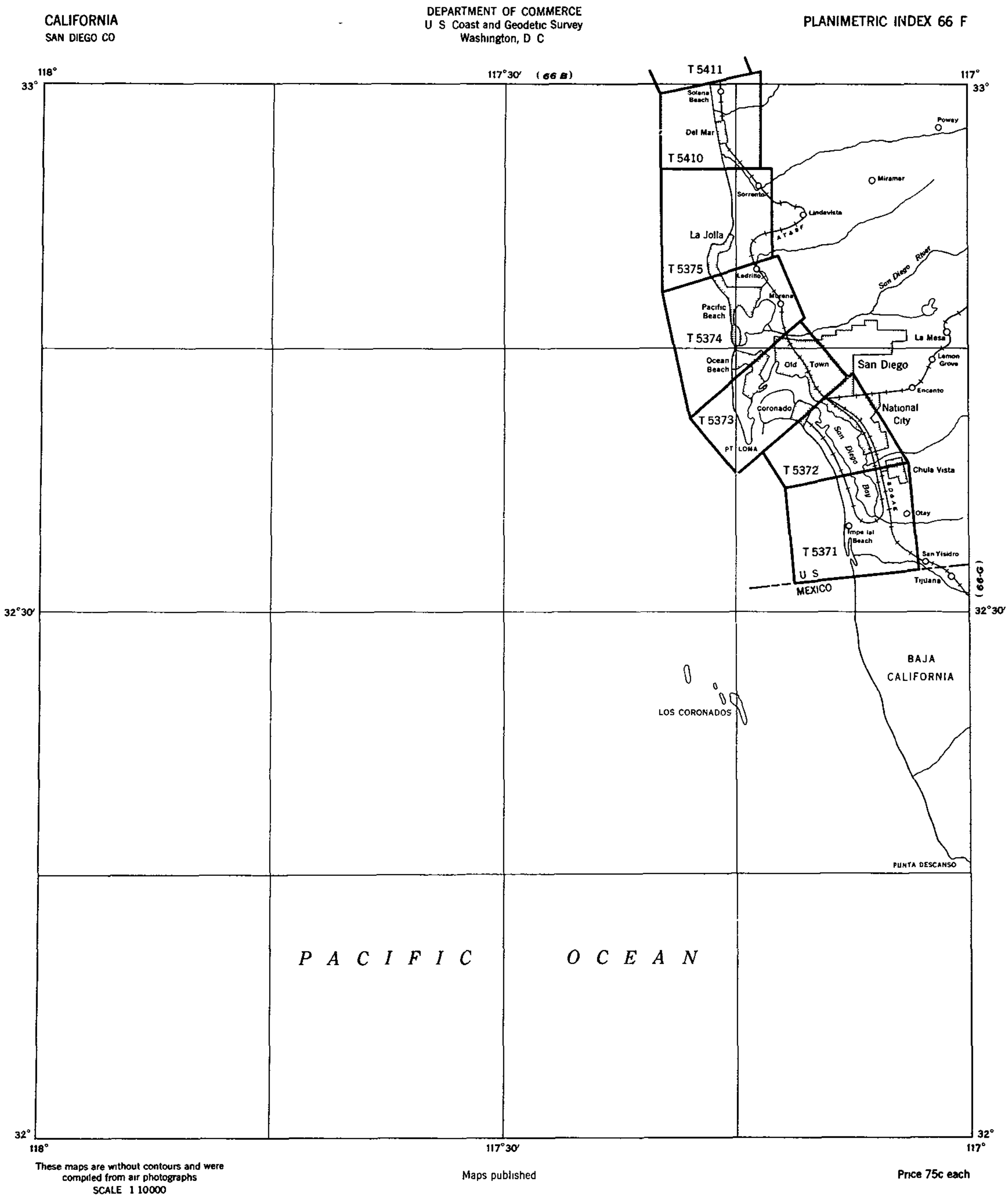

CALIFORNIA
SAN DIEGO CO
DEPARTMENT OF COMMERCE
U S Coast and Geodetic Survey
Washington, D C
PLANIMETRIC INDEX 66 F
118°
33°
117°30′ (66 B)
117°
32°30′
32°
T 5411
Solana Beach
Del Mar
T 5410
Poway
Miramar
Sorrento
Linda Vista
La Jolla
T 5375
San Diego River
Pacific Beach
T 5374
La Mesa
Lemon Grove
Ocean Beach
Old Town
San Diego
Encanto
Coronado
National City
T 5373
PT LOMA
San Diego Bay
T 5372
Chula Vista
Otay
Imperial Beach
T 5371
San Ysidro
U S
MEXICO
Tijuana
(66-G)
BAJA CALIFORNIA
LOS CORONADOS
PUNTA DESCANSO
PACIFIC OCEAN
These maps are without contours and were compiled from air photographs
SCALE 1 10000
Maps published
Price 75c each

Fig. 5

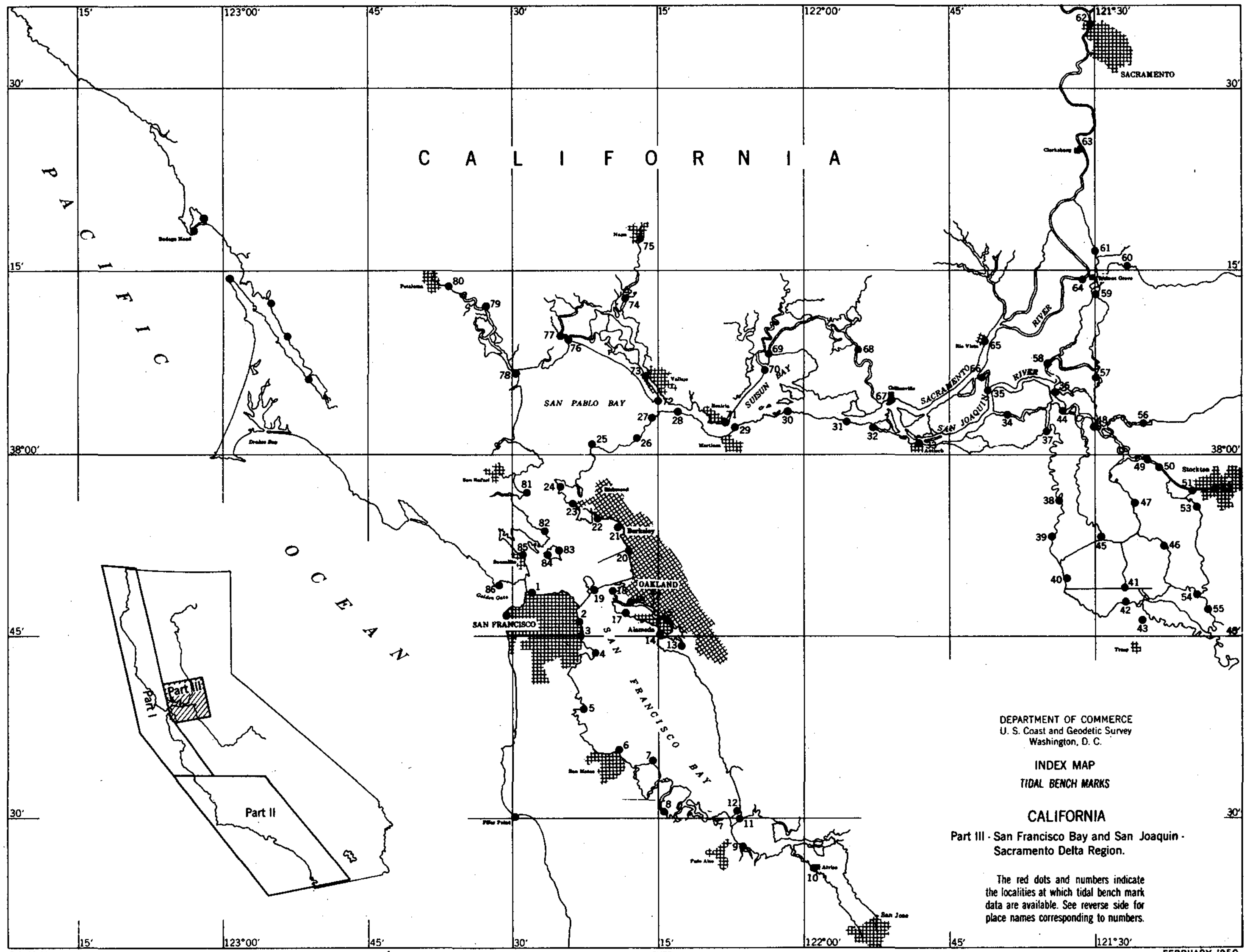

CALIFORNIA
PACIFIC
OCEAN
SACRAMENTO
SAN PABLO BAY
SUISUN BAY
SACRAMENTO RIVER
SAN JOAQUIN RIVER
SAN FRANCISCO
OAKLAND
SAN FRANCISCO BAY
Part I
Part II
Part III
DEPARTMENT OF COMMERCE
U. S. Coast and Geodetic Survey
Washington, D. C.
INDEX MAP
TIDAL BENCH MARKS
CALIFORNIA
Part III - San Francisco Bay and San Joaquin - Sacramento Delta Region.
The red dots and numbers indicate the localities at which tidal bench mark data are available. See reverse side for place names corresponding to numbers.
B-3458-11 ++
OVER
FEBRUARY 1950

Fig. 6

INDEX MAP NUMBER (see reverse side)	NAME	INDEX MAP NUMBER (see reverse side)	NAME
1	Presidio, San Francisco	44	Prisoners Point, Venice Island
2	Rincon Point, San Francisco	45	Borden Highway Bridge, Middle R
3	Potrero Point, San Francisco	46	Union I Highway Br , Middle R
4	Hunters Point (Point Avisadero)	47	Holt, Whiskey Slough
5	Point San Bruno	48	Ward Cut, Ward Island
6	San Mateo (U S M S School)	49	Blackslough Landing
7	San Mateo Bridge	50	Eldorado Pump, San Joaquin R
8	Smith Slough, San Francisco Bay	51	Jacobs Road, San Joaquin River
9	Palo Alto Yacht Harbor	52	Stockton, San Joaquin River
10	Alviso, Alviso Slough	53	Borden Highway Bridge, San Joaquin River
11	So Pac RR Bridge, Dumbarton Point	54	Junction, Old & San Joaquin Rivers
12	Dumbarton Highway Bridge	55	Mossdale Bridge, San Joaquin R
13	Oakland Municipal Airport	56	Bishop Cut, Disappointment Slough
14	Alameda (Electric Light Plant)	57	Terminous, Mokelumne River
15	Oakland (Park Street Bridge)	58	Georgiana Slough Entrance
16	Oakland (Inner Harbor)	59	New Hope Bridge, Mokelumne R
17	Alameda (Naval Air Station)	60	Benson Ferry Bridge, Mokelumne R
18	Oakland Mole (7th St), Oakland	61	Snodgrass Slough (Highway Bridge)
19	Yerba Buena & Treasure Islands	62	Sacramento, Sacramento River
20	Berkeley	63	Clarksburg, Sacramento River
21	Point Isabel	64	Walnut Grove, Sacramento River
22	Richmond (Inner Harbor)	65	Rio Vista, Sacramento River
23	Point Richmond	66	Three Mile Slough Entr , Sacramento River
24	Point Orient & Point San Pablo	67	Collinsville, Sacramento River
25	Pinole Point, San Pablo Bay	68	Meins Landing, Montezuma Slough
26	Hercules, Refugio Landing	69	Montezuma Slough Entrance
27	Selby, Carquinez Strait	70	Suisun Echo Board, Suisun Bay
28	Crockett, Carquinez Strait	71	Benicia, Carquinez Strait
29	Suisun Point and Vicinity	72	Carquinez Strait Lighthouse and Vicinity
30	Port Chicago, Suisun Bay	73	Mare I Naval Shipyard & Vallejo
31	Mallard Ferry Wharf Suisun Bay	74	Brazos Drawbridge, Napa River
32	Pittsburg, New York Slough	75	Napa, Napa River
33	Antioch, San Joaquin River	76	Sonoma Creek Entrance
34	Webb Ferry, Webb Tract, False R	77	Tubbs I Wharf, Sonoma Creek
35	Three Mile Slough Entr , San Joaquin R	78	Petaluma Creek Entrance
36	Bouldin Island, Potato Slough	79	Lakeville, Petaluma Creek
37	Franks Tract Pump, Sand Mound Slough	80	Upper or Quarry Drawbridge, Petaluma Creek
38	Orwood, Old River	81	Point San Quentin and Vicinity
39	Borden Highway Bridge, Old River	82	California City
40	Clifton Court Ferry, Old River	83	Angel Island (East Garrison)
41	Grant Line Bridge, Union Island	84	Angel Island (West Garrison)
42	Naglee Bridge Old River	85	Sausalito and Vicinity
43	Holly Sugar Refinery, Sugar Cut	86	Point Bonita, Golden Gate

NOTE Unnumbered red dots on the index map on the reverse side indicate nearest tidal bench mark locations along the Northern California Coast

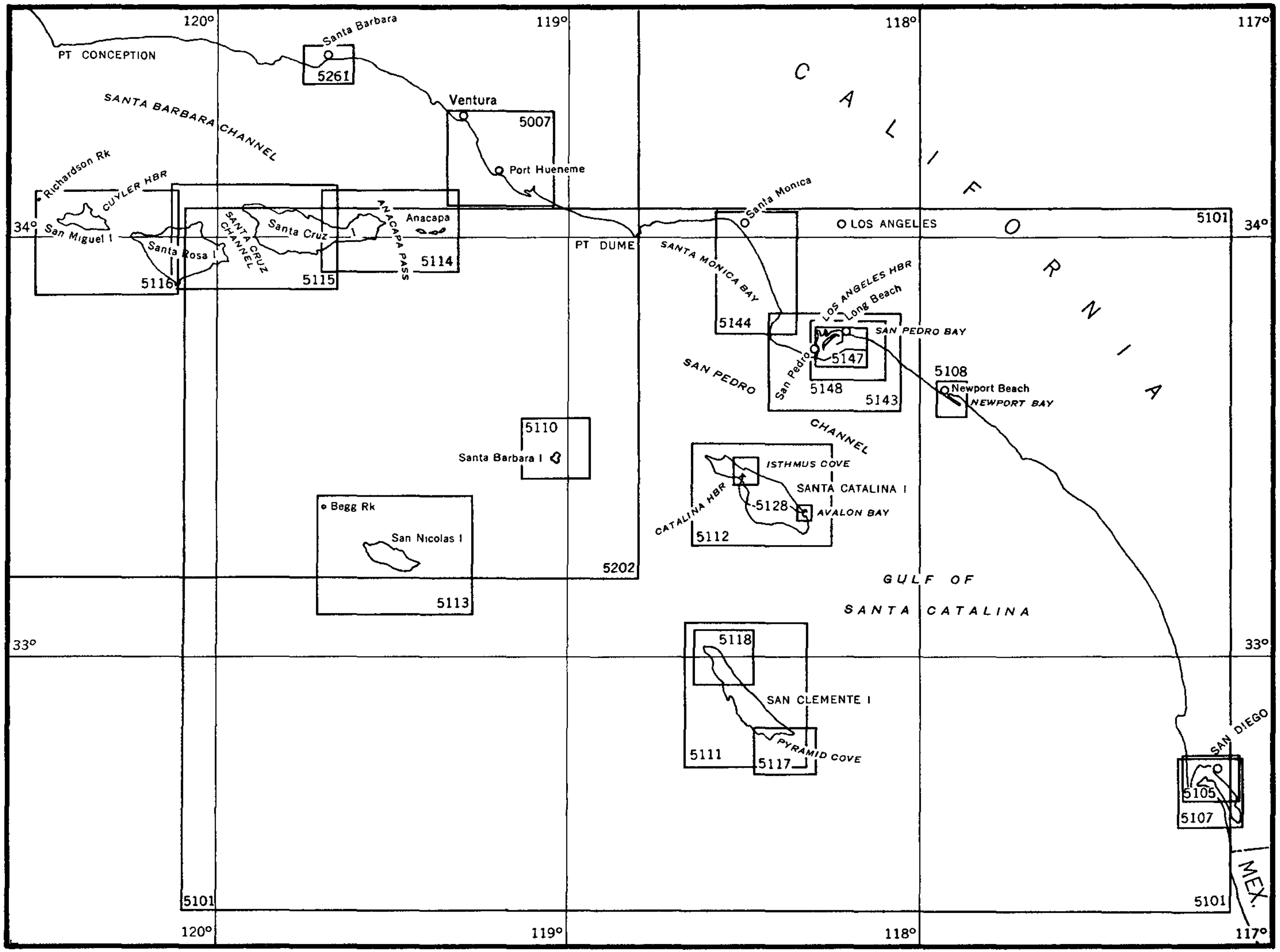
Tidal bench mark locations in the State of California are shown on three index maps, as follows

Part I - Northern California
Part II - Southern California
Part III - San Francisco Bay and San Joaquin - Sacramento Delta Region

Tidal bench mark data are available for the above locations and may be obtained by writing to the Director, U S Coast and Geodetic Survey, Washington 25, D C **In requesting these data, please refer to both the index map numbers and the names of the particular localities in which you are interested**

GENERAL, COAST AND HARBOR CHARTS - PACIFIC COAST

SAN DIEGO TO POINT CONCEPTION, CALIFORNIA



COAST AND GEODETIC SURVEY DATA — AN AID TO THE COASTAL ENGINEER

U. S. Department of Commerce
Coast and Geodetic Survey
Washington, D C

Diagram No 17
SEPT 1949

Fig. 7

GENERAL, COAST AND HARBOR CHARTS—PACIFIC COAST
SAN DIEGO TO POINT CONCEPTION, CALIFORNIA

No.	Price	Title	State	Scale	Size of border (inches)	Edition Date
		GENERAL CHARTS				
5101	$0 75	San Diego to Santa Rosa Island	California	1 234,270	32×48	Jan 1947
5202	75	Point Dume to Purisima Point	"	1:232,188	31×46	Dec. 1940
		HARBOR CHARTS				
5007	25	Pt. Mugu to Ventura	California	1 40,000	25×28	May 1941
		Ventura		1 20,000		
		Port Hueneme		1 10,000		
5105	.75	North San Diego Bay	"	1 12,000	33×44	Oct. 1947
5107	75	San Diego Bay	"	1 20,000	33×37	Jan. 1948
5108	75	Newport Bay	"	1 10,000	28×37	Aug 1940
5110	75	Santa Barbara Island	"	1 20,000	32×37	Mar 1935
5111	75	San Clemente Island	"	1.40,000	33×39	Feb 1946
5112	75	Santa Catalina Island	"	1 40,000	27×38	Feb 1945
5113	75	San Nicolas Island	"	1 40,000	31×41	Oct. 1944
5114	50	Anacapa Passage	"	1 40,000	22×37	July 1945
		Prisoners Harbor		1·20,000		
5115	75	Santa Cruz Channel	"	1 40,000	28×42	Aug. 1945
5116	75	San Miguel Passage	"	1 40,000	28×39	Nov 1945
		Cuyler Harbor		1 20,000		
5117	75	Pyramid Cove and approaches, San Clemente Island	"	1 15,000	33×44	Apr 1938
5118	50	San Clemente Island—Northern Part	"	1 20,000	30×32	May 1940
		Wilson Cove		1 5,000		
5128	25	Catalina Harbor, Isthmus Cove and Avalon Bay, Santa Catalina Island	"	1 10,000	22×27	Oct. 1935
5143	50	Los Angeles Harbor and vicinity	"	1.40,000	25×37	June 1947
5144	50	Santa Monica Bay	"	1 40,000	22×34	Feb 1948
5147	75	Los Angeles and Long Beach Harbors	"	1 12,000	33×45	Oct. 1943
5148	75	San Pedro Bay	"	1.18,000	29×45	July 1945
5261	25	Santa Barbara	"	1.20,000	20×27	July 1939

U. S. Department of Commerce
Coast and Geodetic Survey
Washington, D C

Diagram No 14
APR 1950

CHARTS OF THE ATLANTIC INTRACOASTAL WATERWAY

NORFOLK TO KEY WEST

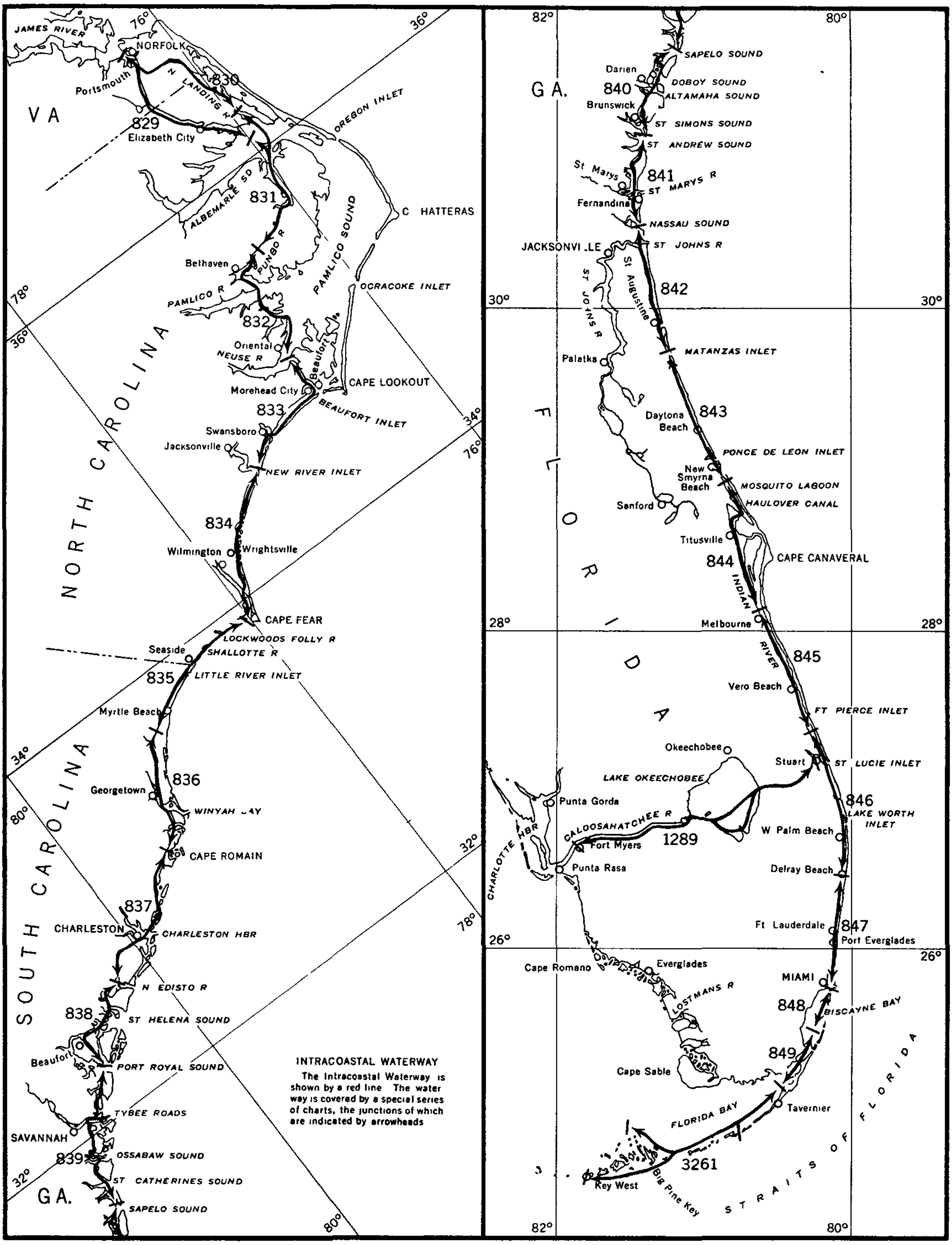


Fig. 8

CHARTS COVERING THE ATLANTIC INTRACOASTAL WATERWAY (INSIDE ROUTE)—NEW YORK TO KEY WEST

Arranged in order of progression southward

NEW YORK TO NORFOLK

No.	Price	Title	Scale	Size of border (inches)	Edition Date
		The following coast and harbor charts, listed with others on pp 4 to 7 inclusive are relisted here as those best suited for this section of the route			
369	$0 75	New York Harbor, N Y and N J	1 40,000	34×43	Feb 1947
1215	75	Approaches to New York—Fire Island Light to Sea Girt Light, N Y and N J	1 80,000	33×41	Feb 1947
825	25	Manasquan Inlet to Little Egg Harbor, N J	1 40,000	22×34	July 1946
826	25	Little Egg Harbor to Longport, N J	1 40,000	22×34	June 1949
827	25	Longport to Cape May, N J	1 40,000	22×34	Oct 1943
1218	75	Delaware Bay, N J and Del	1 80,000	31×37	Jan 1942
294	75	Delaware River—Smyrna River to Wilmington, N J and Del	1 40,000	29×45	Sept 1943
570	50	Chesapeake and Delaware Canal, Md and Del	1 20,000	22×34	Feb 1947
1226	75	Chesapeake Bay—Sandy Point to Head of Bay, Md	1 80,000	34×38	Aug 1942
1225	75	Chesapeake Bay—Cove Point to Sandy Point, Md	1 80,000	35×38	May 1943
1224	75	Chesapeake Bay—Smith Point to Cove Point, Md and Va	1 80,000	30×41	Feb 1947
1223	75	Chesapeake Bay—Wolf Trap to Smith Point, Va	1 80,000	31×39	July 1943
1222	75	Chesapeake Bay Entrance, Va	1 80,000	33×43	Dec 1946

NORFOLK TO KEY WEST

No	Price	Title	Approximate Scale	Size of border (inches)	Date
		The following series of Atlantic Intracoastal Waterway (Inside Route) charts shows the route in strips of convenient widths, with 3 strips to each sheet			
829	$0 25	Dismal Swamp Canal—Norfolk to Albemarle Sound, Va and N C	1 40,000	22×34	May 1938
830	25	Norfolk to North River, Va and N C	1 40,000	22×34	Feb 1943
831	25	North River to Alligator River-Pungo River Canal, N C	1 40,000	22×34	Feb 1943
832	25	Alligator River-Pungo River Canal to Neuse River, N C	1 40,000	22×34	Jan 1938
833	25	Neuse River to New River Inlet, N C	1 40,000	22×34	June 1946
834	25	New River Inlet to Southport, N C	1 40,000	22×34	Sept 1942
835	25	Southport to Socastee Cr , N C and S C	1 40,000	22×34	Jan 1943
836	25	Socastee Cr to McClellanville, S C	1 40,000	22×34	Aug 1942
837	25	McClellanville to Wadmalaw River, S C	1 40,000	22×34	Oct 1942
838	25	Wadmalaw River to Port Royal Sound, S C	1 40,000	22×34	Oct 1942
839	25	Port Royal Sound to Johnson Creek, S C and Ga	1 40,000	22×34	Apr 1943
840	25	Johnson Creek to Brunswick River, Ga	1 40,000	22×34	July 1942
841	25	Brunswick River to Nassau Sound, Ga and Fla	1 40,000	22×34	Mar 1943
842	25	Nassau Sound to Matanzas Inlet, Fla	1 40,000	22×34	July 1941
843	25	Matanzas Inlet to Mosquito Lagoon, Fla	1 40,000	22×34	Nov 1938
844	25	Mosquito Lagoon to Eau Gallie, Fla	1 40,000	22×34	May 1942
845	25	Eau Gallie to Walton, Fla	1 40,000	22×34	June 1938
846	25	Walton to Delray Beach, Fla	1 40,000	22×34	Jan 1943
847	25	Delray Beach to Miami, Fla	1 40,000	22×34	Mar 1943
848	25	Miami to Elliott Key, Fla	1 40,000	23×34	Oct 1939
849	25	Elliott Key to Florida Bay, Fla	1 40,000	22×34	Oct 1939
3261	25	Barnes Sound to Key West, Fla	1 80,000	22×34	July 1941
1289	75	Okeechobee Waterway including Lake Okeechobee	1 80,000	33×46	Mar 1949

The following charts are recommended to supplement the special Atlantic Intracoastal Waterway series For diagrams see pages 6 to 10 inclusive

291 Lake Worth Inlet and Palm Beach, Fla
419 Ocracoke Inlet and part of Core Sound, N C
420 Beaufort Inlet and part of Core Sound, N C
440 Savannah River and Wassaw Sound, S C and Ga
447 St Simon Sound, Brunswick Harbor and Turtle River, Ga
448 St Andrew Sound, Ga
452 Norfolk Harbor and Elizabeth River, Va
453 Fernandina Harbor, Ga and Fla
491 Charleston Harbor Entrance, S C
538 Neuse River and Upper Part of Bay River, N C
546 Port Everglades, Fla
547 Miami Harbor, Fla
571 Port Royal Sound and inland passages, S C
573 Ossabaw Sound and St Catherines Sound, Ga
574 Sapelo and Doboy Sounds, Ga
575 Altamaha Sound, Ga
577 Fernandina to Jacksonville, Fla
582 Fort Pierce Harbor, Fla
584 Key West Harbor and approaches, Fla
777 New River, N C
787 Winyah Bay, S C
792 Stono and North Edisto Rivers, S C
793 St Helena Sound, S C
794 Parts of Coosaw and Broad Rivers, S C
795 Shark River, Manasquan River and Bay Head Harbor, N J

DEPARTMENT OF COMMERCE
U. S. Coast and Geodetic Survey
Washington, D. C.

Hydrographic Index No. 66 J

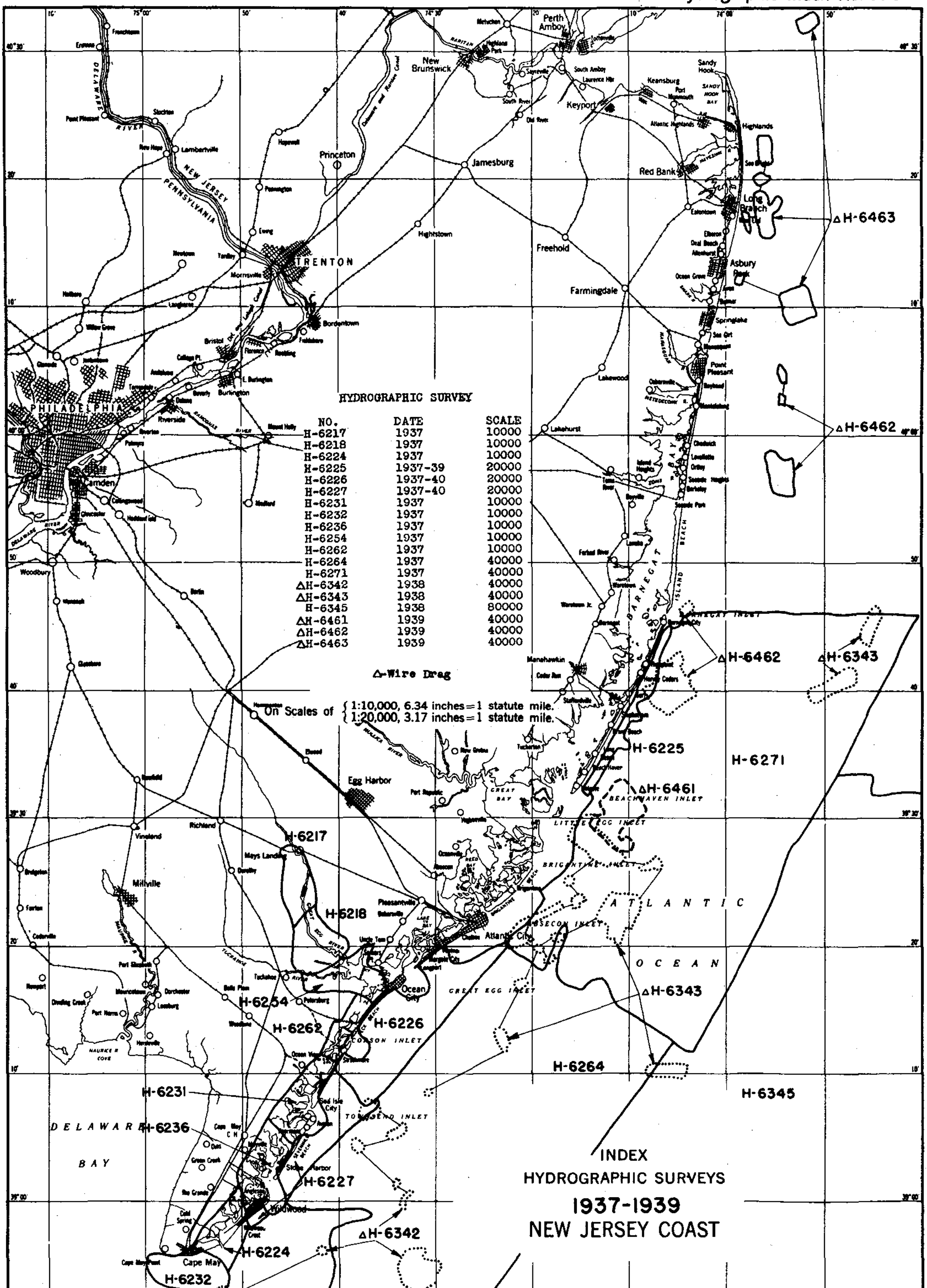


Fig. 9

Triangulation and leveling data. Triangulation and leveling data are issued in several series in loose-leaf form. One series gives the latitudes and longitudes of the established triangulation stations and the lengths and azimuths of the lines to contiguous stations; another the descriptions of the stations; and a third the plane coordinates of the stations, together with the grid azimuths to adjacent stations. Descriptions and elevations of bench marks are given in a fourth series.

TIDES AND CURRENTS

Tidal bench mark data. Tidal bench mark data are available for each tide station. These include the elevations of the bench marks above the basic hydrographic datum; the date and length of the tidal series; and a table showing the relations between the basic datum and other tidal planes in general use, for conversion of elevations to any of these planes. Heights of observed or estimated highest and lowest tides are also given. In addition, special index maps are prepared for each coastal state showing, by place name and number, the localities for which tidal bench mark data are available. Where spirit level connections have been made between the tidal bench marks and the geodetic bench marks of the vertical control survey net, information can be furnished on the relationship between the hydrographic datum and sea level datum.

Tidal current charts. For many areas, where comprehensive current surveys have been made, TIDAL CURRENT CHARTS are published showing graphically, by a set of 12 charts, the direction and velocity of the tidal current for each hour of the tidal cycle.

COMPILED CHARTS

The nautical charts of the Coast and Geodetic Survey are compiled from the basic topographic and hydrographic surveys of the Bureau, supplemented by data from the Corps of Engineers, Coast Guard, Harbor Boards, and other agencies.

Charts are published on different scales to meet the various needs of navigation. They are revised frequently to reflect the many natural and man-made changes that are constantly taking place along our coasts.

Surveys and charts. In using charts for purposes other than navigation, the distinction between a survey and a chart must be kept in mind. Perhaps the principal distinction is that the former, whether hydrographic or topographic, shows the condition as of a specific date and is the result of a field examination. A chart, on the other hand, is the result of an office study and compilation, is usually on a much smaller scale than the field survey, and may show information obtained over a long period of time.

(Dates on charts.) Many misconceptions have grown up regarding the import of our published charts, particularly with reference to the publication dates shown. The significance of the several dates on Coast and Geodetic Survey nautical charts should be understood by all who have occasion to use them.

When a new nautical chart is printed, the date (month and year) and edition number are given in the publication note, which is placed in a central position in the lower margin of the chart. This date is known as the PUBLICATION DATE, and remains unchanged until a new edition is printed, when the date and edition number are changed. A new edition is printed when it becomes necessary to chart important corrections too numerous to be applied by hand.

An additional printing of a chart which includes any change in any portion of the chart is designated as a new print and the year, month, and day are noted in the lower left margin of the chart. This date is known as the NEW PRINT DATE. For each new print an additional date is added. New prints include corrections which generally are not of sufficient importance to require a new edition.

One other date appears on nautical charts which it is well to keep in mind. This is the CORRECTION DATE. It is a stamped date placed in the lower right margin of the chart and represents the date to which all essential changes for lights, buoys, beacons, recently reported dangers, and other critical information have been corrected.

The user of a chart should therefore not be misled by the edition or printing dates on a chart. These dates bear no relationship necessarily to the date when the survey was made or when certain material was obtained. If the precise date pertaining to any section of the chart is required, recourse must be had to the original material from which the chart was compiled.

HOW TO OBTAIN DATA

Nautical charts and related publications (Coast Pilots, Tide Tables, Current Tables, Tidal Current Charts) of the Coast and Geodetic Survey can be purchased at its Washington Office and field offices (Fig. 10) and from authorized sales

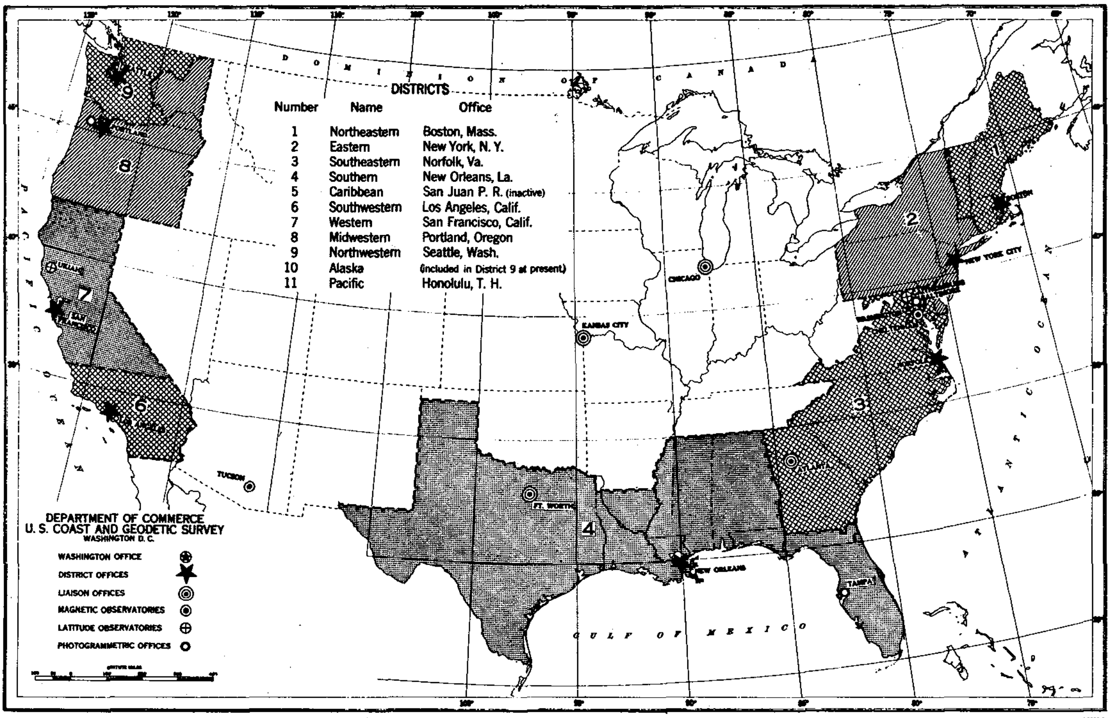


Fig. 10

agencies at the principal seaports of the United States and possessions. These offices and agencies are listed in the Catalog of Nautical Charts published by the Bureau. Copies of field surveys, planimetric maps, triangulation and leveling data, bench mark data, and information on temperature and density of sea water are also obtainable from the Washington Office. Other printed publications, such as manuals, are obtainable on a sales basis from the Superintendent of Documents, Government Printing Office at Washington, D.C.

CONCLUSION

In conclusion, I would like to say that it is my hope that out of this brief presentation will come a better understanding and a wider use on the part of engineers of the results of our activities. The Coast and Geodetic Survey has an important interest in the problems of the coastal engineer. Our surveys are uncovering new and interesting facts about the coastal regions. We are pleased to make these available to all who have occasion to use them.

CHAPTER 12

THE BEACH EROSION BOARD*

Dabney O. Elliott
President, Beach Erosion Board
Department of the Army
Washington, D C.

INTRODUCTION

The purpose of this paper is to describe the methods by which, and the extent to which the Federal Government participates with local agencies in the control of beach erosion. The Beach Erosion Board of the Corps of Engineers is the instrumentality through which this participation is effected. However, before describing this Board, it is necessary to sketch very briefly the background of the beach erosion problem as viewed from the national standpoint.

The necessity for the control of beach erosion by one means or another has no doubt been recognized from the beginning of the practice of coastal engineering in the United States. The early technical records of the Corps of Engineers contain numerous references to the mutual effects which navigation structures and the adjacent shorelines exert upon each other. As an example, chosen at random, I may mention the construction in 1874 of twelve stone groins along the shore of the State of Connecticut between Welshs Point and Indian River, and of a stone jetty at the mouth of that river in the following year, to stabilize the shoreline and to prevent the movement of sand into the navigation channel of that river.

Interest in the related phenomena of wave action, tidal currents, and beach erosion increased materially after the turn of the century, as witness the contemporary professional writings of well known officers of the Corps of Engineers. Noteworthy among these was the report of Captain D. D. Gaillard on "Wave Action in Relation to Engineering Structures" published in 1904. Individual beach erosion problems were studied in the Engineer Districts and boards were occasionally convened by the Chief of Engineers to consider important or critical cases. However, during the first quarter of the present century the approach to the general problem of beach erosion was primarily through individual case studies. The attack was therefore piecemeal in character. The organization by the Chief of Engineers of the "Board on Sand Movement and Beach Erosion" in January 1929 appears to have been the first attempt by the Federal Government to approach the problem on a comprehensive scale. This Board had a short life, being superseded in 1930 by the Beach Erosión Board.

Meanwhile, the State of New Jersey had been seriously concerned with the damaging erosion which was then occurring along the major portion of its ocean front shoreline. In the early twenties the State Board of Commerce and Navigation succeeded in obtaining from the Legislature the necessary authorization to undertake studies with a view to developing means of protection. A State Engineering Advisory Board thereupon was created for the purpose. This Board followed the pattern established by the British Royal Commission (1906-1911), i.e., to use the coast itself as a laboratory and, by the analyses of observed phenomena, to deduce methods of erosion control. Thus, New Jersey was the first state in the Union to enter the field of shore protection and beach erosion control. The close similarity between the New Jersey State Engineering Advisory Board and the Beach Erosion Board, leads to the deduction that the former board served as a model for the organization of the latter one.

THE BEACH EROSION BOARD

The fundamental law establishing the Beach Erosion Board is dated July 3, 1930. This law accomplished two principal objectives. First, it created the Board and defined its duties; and second, it established the procedure by which cooperative

*The opinions expressed in this paper are those of the writer and do not necessarily represent the views of the Chief of Engineers or of the Department of the Army.

beach erosion studies and reports are made. These reports are made by the Corps of Engineers, as the agent of the Federal Government, at the behest of local political subdivisions (state, county, or municipality), the cost being borne equally by the two cooperating agencies. They will be discussed in some detail later.

The scope of the Beach Erosion Board's activities was at first restricted largely to the preparation and review of cooperative reports, and so continued until the end of World War II. Post-war legislation has, however broadened very materially the Board's powers and responsibilities. In 1945 legislation was enacted authorizing the Board to undertake a program of general investigation and research, and to publish technical information relating to the problem of beach erosion and its control. In the following year, the principle of Federal cooperation in beach erosion control was established by a law authorizing the Federal Government to participate in the cost of construction of works designed to protect the shores of publicly owned (non-Federal) property. The protection of Federal shore property is of course exclusively a Federal responsibility.

In this connection, it is of interest to mention that, in 1935, legislation was enacted which recognizes the obvious relationship which exists between seacoast navigation structures and adjacent shoreline processes. This legislation requires that reports on proposed navigation improvements at the mouths of rivers, or at entrances to inlets consider the probable effects which these improvements will have upon the adjacent shores.

The time element in this summary of our national legislation is significant. The Board was created in 1930 and at the same time the principle of Federal participation in beach erosion control studies was established. It was, however, fifteen years later that the Board was authorized to undertake the essential function of research and another year elapsed before the principle of Federal assistance in the cost of protective construction was established.

ORGANIZATION AND DUTIES OF THE BOARD

The Beach Erosion Board consists of seven members appointed by the Chief of Engineers. Four are officers of the Corps of Engineers, the senior one of whom is the President and Resident Member of the Board. The remaining three members are civilians selected with regard to their special fitness from among the engineers of state agencies cooperating with the Department of the Army in beach erosion control. These civilians serve without remuneration by the United States although they are reimbursed for travel and personal expenses incurred while on Board duty. The office of the Board is located in Washington, D.C. A small board staff operates under the direct supervision of the President. Although not specifically required by law, an additional Engineer Officer serves as Executive Officer of the Office.

The duties of the Board, as fixed by law, are as follows:

a. To furnish technical assistance in the conduct of beach erosion control studies;

b. To review the reports of these studies;

c. To make inspections and examinations as necessary of localities under study;

d. To conduct general investigations and research;

e. To publish from time to time useful data and information concerning beach erosion and its control.

In reviewing a beach erosion control report, the Board is required to state its opinion as to:

a. The advisability of adopting the proposed beach erosion control project;

b. The public interest (if any) involved in the proposed improvement;

c. What share (if any) of the cost should be borne by the United States.

Since the Board does not remain continuously in Washington, D.C., its routine duties and responsibilities are discharged very largely through the instrumentality of the President and Board staff. The review of cooperative reports and the determination of the advisability of recommended beach erosion control projects (including the share of the cost recommended to be borne by the United States) are, however, duties which the Board habitually performs in board session. These responsibilities are never delegated to any individual.

Board meetings average five or six a year. When assembled for these meetings the Board normally inspects as a body some critical or important reach of shoreline of current interest. As a result Board meetings are more frequently held in coastal cities than in Washington, D.C. In addition to these inspections, individual members make inspections as opportunity and time permit. By this practice the Board is able to keep currently posted as to major beach erosion problems.

COOPERATIVE BEACH EROSION CONTROL REPORTS

As stated above, cooperative reports are made by the Corps of Engineers at the request of local political subdivisions (States, Counties, or Municipalities). The costs are borne equally by the United States and the local cooperating agencies. The objectives of a cooperative report are normally to ascertain the causes and extent of beach erosion along a specific reach of shoreline, and to determine the most feasible plan for improving and protecting it against further attack. It is important to differentiate between this type of report and the so-called preliminary examinations and survey reports of the Corps of Engineers. These latter reports are made in response to Congressional directives. Cooperative reports on the other hand are made at the instance of local governmental agencies. Specific Congressional authority is not required. When completed they are the joint property of the United States and of the cooperating State, County, or Municipality. The desires of the local agency must obviously be kept in mind while the reports are being prepared.

The procedure followed in the preparation of a cooperative report is a fairly simple one. The local governmental agency desiring the report first contacts the local District Engineer (of the Corps of Engineers). The latter prepares a preliminary analysis of the problem, a proposed program of field work, and an estimate of cost. These are forwarded to the Office of the Beach Erosion Board for comment. Representatives of this office then visit the field, inspect the reach of shoreline to be studied, and reach agreements with the District Engineer and the local cooperating agency as to the program to be followed and the estimated cost. When these details have been settled, the local governmental agency submits a formal letter of application for the cooperative study. On approval by the Chief of Engineers this application becomes the contract covering the work. The local District Engineer then proceeds with the field work. The contribution by the local cooperating agency may be either in cash or in the form of services performed in connection with the study.

The completed report contains an analysis of the causes of erosion, and presents a recommended plan of improvement and protection. The report also contains conclusions as to the economic feasibility of the project as a whole and as to the amount of public interest and the amount (if any) of the recommended Federal contribution toward the cost of the project. This report is forwarded through channels to the Chief of Engineers who in turn refers it to the Beach Erosion Board for review. After review by the Board, the report is referred to the Bureau of the Budget for a finding as to whether the proposed work should be included in the current national construction program. Thereafter, it is forwarded by the Secretary of the Army to Congress. If the report is approved by the Congress, the project is included as an item in an authorization act. Thereafter funds must be appropriated before Federal assistance can become a fact.

Several points are noteworthy in connection with this procedure. A public notice is issued by the Division Engineer as soon as practicable after the report has been completed by the District Engineer. This notice briefly describes the proposed plan of protection including the cost and the recommended amount of the Federal contribution thereto. Interested parties are thus given an opportunity to forward their comments to the Beach Erosion Board for consideration prior to the completion of Board action on the report. If the comments are sufficiently important, the Board may hold a public hearing prior to completing action. In practice, however, comments in response to public notices are not numerous and public hearings have almost never been necessary up to the present time. The reason lies in the fact that the District Engineer and the cooperating agency keep in close contact during the preparation of the report. Comments are therefore usually made while the plan is in its formative stage and can therefore be disposed of before the report reaches the Board.

Federal participation in the cost of protection is by law restricted to public property. Nevertheless cooperative reports always cover the protection of such privately owned shores as may be within the limits of the study areas, and normally include proposed plans for the protection of these shores even though Federal funds cannot be applied toward the construction of the proposed structures. In fact, instances frequently occur in which the aggregate length of privately owned shoreline under study considerably exceeds that of the publicly owned shoreline. This is typical of the states of Ohio and Connecticut where progressive cooperative studies are now under way which will ultimately cover the entire shorelines of these two states.

It is to be noted here that the law does not permit private individuals to apply for cooperative reports even though they are willing and able to meet the local share of the cost. Applications must originate with political subdivisions (normally the State, the County, or Municipality).

FEDERAL PARTICIPATION IN COST

The legislation which authorized Federal participation in the cost of shore protection (Public Law 727, Seventy-Ninth Congress) was passed in 1946. This law states that "with the purpose of preventing damage to public property and promoting and encouraging the healthful recreation of the people," the United States will "assist in the construction, but not the maintenance, of works for the improvement and protection against erosion" of the shores of the United States which are owned by States, Municipalities, or other political subdivisions, provided the Federal contribution to the cost of construction of protective works does not exceed one-third of the total cost. This law, therefore, provides for Federal assistance but limits it to publicly owned property. It recognizes the element of improvement, as well as protection, and lastly, it establishes public recreation as a legitimate objective. The maximum amount of Federal participation is fixed, but within that maximum the actual Federal contribution in any individual case is discretionary.

Since the passage of this Act, five projects involving Federal assistance have been authorized by the Congress but as yet no funds for this assistance have been appropriated. Fifteen additional cooperative reports, including five now before the Board, have been completed and will probably result in additional Congressional authorizations during the coming year. However, the number of projects fully processed to date is too small to indicate with certainty the working limits of policy which will in the future govern the amount of Federal assistance in individual cases. The following considerations in this connection represent the personal views of the writer. They are not to be interpreted as representative of Board policy.

It is seemingly axiomatic that Federal assistance in any beach erosion control project must rest on two fundamental conditions, i.e., substantial public interest and economic soundness. The term "interest" is used here to connote a real public advantage and not merely a favorable opinion. Economic soundness is obviously a prerequisite to Federal aid. Projects which do not meet these two fundamental requirements should not be considered as eligible for Federal assistance.

In many cases, the substantial public interest mentioned above will consist predominantly of recreational features although not necessarily confined thereto. A shore protection project may well be intended solely to prevent erosion damage to valuable public shore front property and may contain no element either of improvement or of public recreation. Such a project should not be denied Federal aid merely because recreational benefits do not exist. Moreover, when these recreational benefits do exist, they should be substantial, reasonably certain to develop, and should accrue to the general public. Recreational benefits which can be enjoyed only by small groups of fortunately situated property owners do not, in the opinion of the writer, warrant Federal assistance. Local governmental agencies should give reasonable assurance that the recreational areas resulting from proposed improvements will be adequately regulated to insure the continuance of the expected benefits throughout the life of the projects. When pollution of adjacent water areas menaces the recreational value of a project, the abatement of this pollution by local agencies should be made a condition of Federal assistance.

As stated above, the law recognizes the element of improvement as well as protection. Nevertheless this law apparently limits Federal assistance to a maximum of one-third of the cost of protective construction. Thus, in the opinion of the writer, improvements which amount substantially to the creation of new beaches, or to the widening of existing beaches beyond the requirements of adequate protective construction, should be undertaken wholly at local expense. Federal participation should be limited to a maximum of one-third of the construction cost of the protective measures needed to stabilize these improved beaches.

One provision of the law authorizing Federal participation in the cost of shore protective works has not as yet been mentioned. This is the so-called "Highway Clause," which authorizes Federal participation in the cost of repair or replacement of seawalls or similar structures previously built to protect public highways against erosion by waves and ocean currents. This clause is of rather narrow application and is hedged about with restrictions. It will not be discussed in detail here.

The foregoing legislation applies to the continental shore of the United States, including those of the Great Lakes, but does not apply either to Alaska or to the insular possessions. Consequently, a seeming paradox exists which allows Federal cooperation (including sharing costs) with territorial governments in the preparation of cooperative reports but does not authorize Federal assistance in the actual control of beach erosion. This inconsistency is, however, not important at the present time. Beach erosion control reports originating in our territories will no doubt be considered on their individual merits and Congressional authorization of these projects will include approval of whatever Federal assistance may be recommended in the reports. The present law does not authorize Federal assistance to the territories but on the other hand, it does not bar such assistance.

It will be recalled that, prior to the end of World War II, the Board restricted its activities largely to the preparation and review of cooperative reports. During that period the field work of these reports was done by local District Engineers. The reports themselves were, however, written by the Board staff under the supervision of the Board itself. In 1946, the increasing prospective volume of this cooperative work made it appear wise to decentralize to the Districts the preparation of the entire reports. This practice has been followed since that year and is gradually demonstrating its advantages. As was to be expected, practical knowledge of beach erosion control engineering is now being more widely disseminated in the field than was previously the case. In addition, the field agencies are more keenly aware of the intimate relations which exist between navigation improvements and the adjacent shorelines. The function of the Board staff in this connection has gradually changed from that of an operating agency to a supervisory and consultant body.

INVESTIGATION AND RESEARCH

In retrospect it seems unfortunate that legislative authority to undertake a program of research and general investigation was not received by the Beach Erosion Board before 1945. This delay was probably due to the fact that the original concept of the Board was that of an operating agency rather than an investigative one. This was perhaps inevitable. Nevertheless, the result has been a delay of about one decade in the research program. The war years were not lost by any means. Indeed during those years, marked advances were achieved in our knowledge of beach erosion processes by reason of the military research programs which were undertaken as incident to naval and amphibious warfare. Institutions of learning, such as the University of California and Scripps Institution of Oceanography, were very active in carrying out these war programs.

There is nevertheless still a wide field for further investigation. The following random comments are the personal opinions of the writer and are not to be considered as representing Board opinion. They are, however, presented here as they illustrate the existing need for further research from the standpoint of practical coastal engineering.

a. The existing method of estimating the volume of littoral drift by measuring the quantities of sand impounded by jetties and similar structures occasionally produces results completely inconsistent with other observed phenomena. Improved methods for determining the direction and quantity of littoral drift are badly needed.

b. The character and amount of onshore and offshore sand movement, as distinct from littoral drift, are not yet thoroughly understood. In the study of extended reaches of shoreline critical areas have been found to exist where the observed directions and estimated volumes of littoral drift indicate that pronounced erosion (or accretion) should occur. Study of these critical areas, however, fails to reveal the existence of this erosion (or accretion). This leads to the conclusion that unobserved sand movements are acting to maintain an equilibrium.

c. The effect of submarine canyons on the littoral movement of beach material is a subject which merits a considerable amount of investigation.

These comments by no means exhaust this field of speculation. They merely illustrate the present need for further study. The program which the Beach Erosion Board is currently carrying out is now about three years old. It is being carried on by the Board staff in Washington, D.C., and also by contracts with New York University, the University of California, and Scripps Institution of Oceanography. The Board staff maintains a field party in California (based at present at San Diego) which is investigating the nature and extent of sand movements along certain beaches. The results of a somewhat similar study at Long Branch, New Jersey, have been published recently.

It is expected that about two additional years must elapse before results of interest to the practical beach erosion control engineer will begin to emerge from this research program. By that time, the Bulletins and Technical Memoranda published by the Board will be of great value to the individual engineer seeking a practical solution to a field problem. For the present, however, the research program is devoted to the development of fundamental concepts of wave motion, sand movement, etc. This is necessary before the practical solution to individual engineering problems becomes feasible.

CONCLUSIONS

As a result of twenty years of work, a wide coverage of our shorelines by individual beach erosion studies has been obtained. This coverage is increasing. As of the date of this writing, sixty eight cooperative reports have been completed, including five now under Board consideration. Eleven additional reports are expected to reach the Board during the current fiscal year and the load during the following fiscal year will probably be equally as heavy. An appreciable volume of additional coverage is obtainable from the discussions of shoreline changes which are contained in current reports on proposed navigation improvements. The current research program is producing a fund of useful data and basic information which will be of ever increasing value as the program develops. From this mass of information, certain concepts as to the characteristics and processes of our major reaches of coastline are gradually developing. The reasons for these concepts and the differences between different coastal regions are slowly becoming apparent. Much remains to be done but there is room for optimism that our knowledge of beach erosion control engineering will make more rapid advances in the near future than at any time in the past.

PART 3

COASTAL SEDIMENT PROBLEMS

CHAPTER 13

THE GEOLOGICAL ASPECTS OF COASTAL ENGINEERING

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INTRODUCTION

A natural beach system is in equilibrium when there is a balance between sand supply and erosion such that the volumes of material entering and leaving the system are just equal. If the erosion rate exceeds the supply rate, a beach retrogrades; if the erosion rate is less than the supply rate, a beach progrades. Unfortunately, coastal engineering works, which are meant to improve the shore for commerce or recreation, often upset this delicate balance with very deleterious results: great accretions of sand and high dredging costs, accelerated beach erosion and much property damage. It is the task of the geologist to determine the secular equilibrium conditions of a beach system and to supply the coastal engineer with the information he needs to control the natural forces acting on the shore in such a way that this equilibrium is maintained. In order to accomplish this task, the geologist needs to make a thorough study of the source, transportation, and deposition of beach sediment. He must determine the stable position of the shore line and the profile of equilibrium of the beaches through detailed physiographic investigations. The geological report can and should close with the prediction of just what will happen to a natural beach system if man introduces a disturbing element.

CONTINENTAL GEOLOGY OF TRIBUTARY WATERSHEDS

If an investigation of sediment sources reveals that the principal sources of beach-building material are the drainage basins tributary to the shore under consideration, as is the case in southern California for example, then it is necessary to make a detailed study of the continental geology of these basins. Such a study includes hydrology, physiography, stratigraphy, and sediment supply (Handin, 1951).

In order to make an estimate of the quantities of sediment supplied to the beaches, the geologist first measures or estimates the rate of terrestrial sedimentation. Quantitative estimates are impossible without direct measurements of silting rates in reservoirs or catch basins or empirical equations based upon such measurements. The Forest Service makes use of such equations in determining sedimentation rates in mountainous watersheds (Anderson, 1949). The equations used are of course applicable to 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. In the absence of direct measurements or applicable equations only qualitative estimates can be made. In southern California for example, the hydrologic data indicate that rainfall is appreciable during a limited time only, but that during this time intermittent streams can become torrential and can then transport enormous quantities of sediment. Such knowledge at least permits the geologist to say that the sediment supply is discontinuous and that significant amounts of material are moved to the shore only once in several years, whenever there is a major flood. On the other hand, there may be evidence that there is never much runoff in, say, a region underlain by very pervious rocks.

Consideration must be given to the types of rocks being eroded in the source areas. Only sand-sized material remains on the beaches; silt and clay are lost in deep water offshore. A careful stratigraphic study is necessary to determine the relative abundances of sand and fine sediment in the source rocks. In the case of

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crystalline rocks particularly, the geologist must know what sort of weathering processes are active in the region so that he can determine how much of the weathered rock remains in the form of sand grains and how much is lost in solution or decomposed into clay and silt. Weathering depends largely upon climatic conditions which are then another factor to be considered.

Even if there are excellent data on terrestrial sedimentation rates, it may be very difficult to estimate how much material reaches the shore. How much sediment is lost along the way between the mountains and the stream mouths? It is now extremely difficult to measure directly the total load of a stream. The measurements of losses in transit must be indirect. If the streams are degrading or appear to be at grade, one can assume that all the source material ultimately reaches the shore. But if the streams are aggrading, deposition rates along the channel must be measured and the losses subtracted from the estimate of sediment supply to the beaches.

It is sometimes possible to use coastal engineering data on accretion rates to determine the contributions of sediment by tributary streams. The rate of accretion of sand at the Santa Monica breakwater has been used to estimate sedimentation rates in the Santa Monica Mountains (Handin and Ludwick, 1950a). The quantity of material deposited by the Santa Clara River during the great flood of 1938 was determined from the volume of sediment in the delta.

In summary, a study of tributary watersheds should reveal terrestrial sedimentation rates, percentages of sand-sized material, and percentages of source sediment ultimately reaching the shore. Both direct and indirect methods can be applied to the measurements.

BEACHES AND COASTAL PHYSIOGRAPHY

A detailed study of beaches and the physiography of the adjacent coastal areas can solve a great many local problems. Some of the following features might be included in such a study: foreshore slope, berms, cusps, sediment type, nature of the backshore, general shape of the beach, presence of seacliffs, seawalls, or dunes, general shape of the beaches, terraces and wave-cut benches, inlets, offshore bars, spits, headlands, and many more. An example of the application of such a study to the solution of a particular problem is the establishment of the fact that in southern California a negligible quantity of sediment has its source in the sea cliffs bordering the coast (Shepard and Grant, 1947). In general the cliffs are protected from direct wave attack by narrow "buffer" sand beaches even along rugged, mountainous beaches of the coast. In regions where coast land is being eroded by wave action, another purpose of the physiographic investigation is to determine the rate of supply of sediment from that source. This is usually accomplished by measuring the rate of retreat of sea cliffs.

But by far the most important purpose of this study is to establish the natural equilibrium of the beach system. The facts gathered in the investigation are used to interpret the recent geologic history of the coast. The shore line changes which have taken place during historic time are determined from a study of old maps and charts. It is then possible to establish whether or not the present shore line is stable, and, if not, what the trend of further changes is likely to be.

PETROGRAPHIC STUDIES

A petrographic study, including mechanical and mineral grain analyses, can be a powerful tool to use in the solution of the problem of the source, transportation, and deposition of beach sediment. Sediment samples to be analyzed are collected from selected sites in the source areas, along the channels of source streams, on the beaches, on dunes, and on the shelf offshore. If the source rocks contain distinctive diagnostic minerals, it is possible to trace sediments from the source areas to the shore. One can show, for example, that much of the beach sand of Santa Monica Bay is derived from the Santa Monica Mountains; whereas the beach sands of Ventura County to the northwest are not (Handin, 1951). By comparing the mineral content of sand samples taken along the shore line, it is possible to determine the direction of the prevailing littoral drift. Finally one can show where the coastal sands are ultimately deposited by investigating the mineral content of dune sands and submarine sediments.

Much can be learned about the nature of the sediment transporting agents by using the concept of the "variation series." Several samples taken along a shore line generally have different median grain sizes and different percentages of mineral constituents. The changes from sampling point to sampling point along the shore line may show a progressive trend. Along the Ventura County coast, for instance, there is an increase in median grain size of the beach sands with distance away from the source streams, which is ascribed to a progressive loss of fine material from the littoral zone. On the other hand, there is a decrease in grain size along the shore of Santa Monica Bay, which is probably due to a progressive decrease in the competency of longshore currents. Heavier minerals seem to settle out of suspension to remain on the beaches, while the lighter ones are carried out beyond the plunge zone to be lost in deep water. Therefore there is an increase in the heavy mineral content of many beach sands as the sediment is transported away from the source areas (Handin, 1951).

Still another use of the petrographic study is to determine the characteristics of the beach sediment which is in equilibrium in its natural environment. Such information is valuable to the engineer who must bring in material with which to construct an artificial beach. If his material is finer than the sand which would be found on a natural beach in the same environment, it is quickly removed by the waves and lost in deep water. If his material is too coarse, the artificial foreshore is likely to be so steep that the beach is useless as a recreational facility.

SUBMARINE GEOLOGY

An investigation of the offshore submarine topography is important since one must have detailed knowledge of bottom contours for wave refraction studies. The history of changes in the bottom topography are of course as significant as changes in the position of the shore line in determining the natural profile of equilibrium and the stable configuration of the shore line. Successiv soundings may indicate that sediment is being deposited or that material is being removed.

In solving the problem of the ultimate deposition of beach sediment, a study of bottom deposits is indispensable. The petrography of submarine samples may reveal the source of the sediment, and in so doing establish whether the bottom sediments are residual or are being deposited now.

In southern California there are unique features which require special consideration: the submarine canyons which head very close to shore. In the first place, these canyons exert a powerful influence on incoming waves. Erosion at Redondo Beach is greatly intensified by wave convergence resulting from refraction over the canyon just offshore. In the second place the canyons very probably trap much sediment moving in the littoral zone. At Port Hueneme the jetties are built out almost to the head of a canyon, and it is very unlikely that littoral drift can bypass the harbor entrance. The inevitable result is serious depletion of down coast beaches.

GEOLOGICAL ASPECTS OF WAVE STUDIES

The geologist is most concerned with average wave conditions prevailing for a long time. The prevailing waves have much to do with shaping the coast line. The source of the majority of swells reaching the California coast is in the North Pacific Ocean, so that waves approach the shore from the northwest much of the time (Scripps Institution of Oceanography, 1947). The result is that headlands are asymmetric with long western tangents and short northeast trending re-entrants.

The direction of wave approach determines the direction of littoral drift, which is of course of prime interest to the coastal geologist, as well as the coastal engineer. Unfortunately, the direction of the littoral drift can be determined analytically with the aid of wave refraction diagrams only when very complete meteorologic data are available. In the absence of such data, the geologist must look to physiographic and petrographic evidence.

Wave heights are probably the most important factor determining the grain size of beach sands. Foreshore slopes in turn depend upon grain size among other things, so that it is well for the geologist to make a study of prevailing wave heights. He should also observe the period and angle of approach of waves, since together with height, these factors largely control longshore current velocities. Seasonal changes in wave heights and angles of approach are responsible for seasonal variations in longshore current capacity and competence (Handin and Ludwick, 1950b).

CONCLUSIONS

There has been time to touch upon only a few of the more generally important functions of the geologist in the coastal engineering program. Indeed, it is difficult to differentiate between problems typically handled by a geologist or by an engineer. For the study of sediment sources, the engineer furnishes the hydrologic data; the geologist provides the stratigraphic evidence. For the beach studies, the geologist makes the petrographic analyses; the engineer measures the profiles. But perhaps the greatest contribution the geologist can make is a philosophical one. He always asks himself: what will be the effect of an engineering structure upon a natural equilibrium that was established long before man came along, an equilibrium man can upset so easily.

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CHAPTER 14

ESTIMATING QUANTITIES OF SEDIMENT SUPPLIED BY STREAMS TO A COAST

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Most beaches which are in equilibrium depend for their maintenance upon a continuous supply of sediment. This supply rate has been estimated for instance for the beach at Santa Barbara to be of the order of magnitude of 350,000 cu. yards of sand per year (Johnson, 1948). As long as this supply rate prevails no accumulation of sand occurs on the beach. The sand must, therefore, be used up at the beach either by wear or by transport along the beach and, eventually, by deposition into some deeper water. A continual supply of that order of magnitude very rarely becomes available at the shore itself by wave erosion, instead the bulk of supply of sand to beaches usually is derived by rivers. The appraisal of the different river channels for their sediment supply to the beaches near their mouth becomes thus a major factor in the maintenance work of many beaches.

Usually, the sediment load of a river is given as an average annual rate or if somewhat more care is used in the determination of the load, it is divided into a bed-load rate and a suspended-load rate. Even this unfortunately is insufficient for an estimate of the contribution to the beach. This is because most beach sediments are much better sorted than the river sediments. In order to predict the rate of the rather narrow range of grain sizes that contribute to the beach load, the rates of transport in the river must be known individually for all grain sizes. This is a rather ambitious problem, but it may be solved today, at least in part, for most rivers. Before the procedure of such a load determination is described, it is necessary, however, to describe the process of the transportation of sediment in such rivers.

It is most helpful to begin this discussion with the trivial sounding remark that all sediment particles which pass from the river to the beach must have been eroded somewhere in the watershed and must then be transported from there to the beach by the stream system. The significance of this statement lies in the fact that both conditions must be fulfilled and that either one of the two may limit the rate. It becomes actually advantageous to introduce special terms for the two parts of the load: it is customary at present to call the part of the load which is limited in its supply by the availability through erosion the "wash-load", while that part which is limited by the ability of the stream to transport it, is called "bed-material load", or for short, "bed load" (Einstein, Anderson, and Johnson, 1940). This definition of the two parts suggests already that the two parts must be determined in an entirely different manner.

The rate of transport of the wash-load of a river is governed by the erosion on the watershed. It is known today that the rate of erosion in an entire watershed depends on a large number of parameters such as the geology, the steepness of the terrain, the plant cover (as divided into perennial cover -- such as forests and pasture, temporary cover as some broadcast crops and row crops), the climate (the seasonal distribution of the precipitation, duration of frost periods), the size of the watershed, and many other influences. Some attempts have been made to predict the sediment production of watersheds on a statistical basis from available reservoir surveys (Brown, 1945). In most of the cases considered in this latter analysis the bulk of the load was wash-load, and the contribution of the bed material load may be neglected in view of the low degree of accuracy of such an estimate. The prediction of a breakdown of these total rates into different grain sizes has not been attempted as yet.

The most promising present day approach in the determination of the wash-load of a river is the direct measurement by suspended-load sampling (Iowa University, 1948). This method is expensive, however, and time consuming. It may take from one to ten years of continuous observation to predict the wash-load of a river, depending on the regularity of its flows. The expense is high, not only because of the cost of sampling in the field but to a large part also due to the analysis of the samples in the laboratory.

Most beaches contain only particles which are measurable by sieving, making the analysis of finer sediment fractions by other means unnecessary. It is possible, therefore, to reduce the laboratory cost of such a load study for beach supply to a minimum, if sieving of composite samples is used exclusively.

An entirely different approach may be used for the prediction of the rate at which bed material is moved. The presence of these particle sizes in the stream bed already indicates that sometimes, at least, larger rates of these particles become available in the watershed than the stream is able to move away. This surplus of material, therefore, was accumulated in the bed and represents a reservoir of such particles. Over such a bed a rate of movement will be maintained which is a strict function of the flow condition. Not only will a surplus of supply above the carrying capacity be deposited in the bed, but also any deficiency of supply is eliminated by scour from the bed. Thus, <u>bed material always moves according to capacity.</u>

The capacity rates for bed material may be calculated by so-called bed-load equations, which are more or less general relationships between the flow-variables, the particle size and weight, and its rate of movement. Theoretically, at least, these equations permit the prediction of the transport rates of bed particles over a bed of known composition as caused by any given flow. In reality, however, most such equations are derived for a bed of uniform particle sizes, and if applied to a mixture of different sizes of bed materials, the assumption is made that the mixture will not undergo any segregation, and that the size analysis of the transport is the same as that of the material forming the bed. Very often, especially where the range of grain sizes is small and where the rates are large, this assumption leads to the correct result; in many other cases, however, it is entirely misleading. This fact led to the recent development of newer formulas which permit the prediction of the individual bed-load rates of the different bed components in terms of discharge (Einstein, 1950).

While it is advisable to determine, or at least to check the otherwise derived wash-load rates by direct measurement in the stream, such a measurement often is impractical for the bed-material load. Here, the analytically determined capacity load is usually more reliable than the measurement unless there is for instance a fully effective basin available in which the load can be measured volumetrically and analyzed by sampling. But if such a basin is available in a stream, it will prevent the sediment from moving all the time, and what we measure is not the rate of sediment supplied to a beach, but the part which is diverted from the beach. With the measurement of the bed-material load often impossible and almost always impractical, it is most important to base the load calculations on an appropriate description of the channel and of the flow conditions in the channel. This description must include the following items:

1. The geometry of the channel, as described for instance by a longitudinal profile and a set of cross sections.
2. The bed composition, as determined by sampling in the surface layers and by size analysis.
3. All causes of hydraulic friction, in addition to that on the bed, must be known, such as banks, vegetation, islands, or severe meandering.
4. A dependable record of the expected flow rates, preferably in the form of a flow duration curve or in any other form which may be reduced to a flow duration curve. If past flow records are used, they must cover a significantly long duration and apply to the reach in question.

Practical work in this field has revealed that, particularly here in the West, the lack of flow records (especially for smaller watersheds) presents today the most stringent restriction on the application of bed-load formulas for the purpose of total-load determination.

The limiting size between bed-material load and wash load may be derived according to definition as the lower limit of sizes found in the bed in significant amounts (the size of which 5 or 10 percent are finer has been proposed) (Einstein, Anderson, and Johnson, 1940.) Present investigations at the University of California, Berkeley, however, point to the fact that this simple division into the two modes of transport by a limiting grain size is oversimplified and, thus, not logical. A more general and basically more satisfactory description is found if the transition between wash load and bed-material load is made gradually. The capacity load on a movable bed has been defined as the rate at which the different sizes of bed material may be moved by the flow without any change of location or composition of the bed. Experience indicates that there often exists for the finer components of the bed not only one rate at which this condition is fulfilled but an entire range of rates, the lower limit of which has been studied in the past while the upper limit is under investigation today. These two limits diverge more and more from one another as the particle size decreases. The known lower limit may be interpreted as the minimum rate necessary to maintain the bed under the given flow conditions, while any additional rate up to the maximum as given by the upper limit must be classed as wash load. The two limits seem to diverge very fast with decreasing grain diameter, such that the assumption of a single limiting size is unsatisfactory only in a small transition range of diameters. But, if just this size range is an important component of the beach material, the rates for these sizes must be estimated from the watershed erosion like the wash load proper.

In conclusion it may be summarized that the rates at which an alluvial stream transports the sediment sizes of its bed can be calculated by means of bed-load equations, while the finer components in many instances must be measured or estimated from measurements in similar watersheds.

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CHAPTER 15

LITTORAL PROCESSES ON SANDY COASTS

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THE GEOLOGICAL ASPECT OF SHORE PROCESSES

Seacoast littoral transport may be defined as the movement of sediment along the coastal region by currents primarily induced by waves and tides. It is a phase of the geomorphic process by which sediments forming the earth's surface seek an environment conducive to permanent deposition.

In his classic work on the subject of shore processes and shoreline development, Johnson (1919) reviewed with great thoroughness the work of earlier and contemporary students of that subject and introduced his concept of littoral transport. Johnson (1919) reached the conclusion that marine forces attacking a shore would produce over a limited period of time a "profile of equilibrium," at which stage the degree of slope at every point on the littoral berm would correspond exactly with the ability of wave energy developed at that point to dispose of the debris there in transit. The equilibrium profile would vary in detail within limits fixed by the variability of wave energy and the resistance of the sediments to transport. On the basis of ultimate development, he visualized diminution of the rate of delivery of sediments to the littoral zone by reason of flattening of the land mass, and further reduction of littoral material by marine abrasion and consequent removal from the littoral zone, resulting in dominance of the marine forces to the extent that they are capable of reducing broad land areas to a plane of marine denudation.

At the conclusion of his discussion of the forces responsible for littoral transport, Johnson stated that it is much easier to describe the complexities of these forces and the mistakes which are frequently made in interpreting them than it is to present a solution in a given case which is not open to criticism. To this expression of the problem the author heartily subscribes. Johnson (1919) further stated his opinion that "the time will come when our present limited knowledge of both wave and current action will be enormously extended by means of improved mechanical appliances, permitting actual observation of sediment movement at considerable depths and exhaustive studies of limited coastal areas under varying conditions." His predictions have been borne out to a limited extent but much work remains to be done before the mechanics of littoral transport can be stated conclusively. The work of Johnson (1919) remains the outstanding contribution of this century to the fundamental principles of shore processes.

The comprehensive work of Twenhofel (1939) on sedimentation treats only briefly the subject of littoral transport, but he discusses a concept of importance to the study of littoral processes which can be summarized briefly in his terms "base level of erosion", "base level of deposition" and "profile of equilibrium." He defines the marine base level of erosion as the lowest level to which marine agencies can cut a bottom. The base level of deposition is the highest level to which a sedimentary deposit can be built. His concept provides that the base level of deposition due to marine agencies coincides with the base level of erosion, resulting in a single surface that would be the base level of deposition over places that are filled and the base level of erosion over eroded surfaces. When, during an intermediate stage of development, erosion and deposition become so nearly the same that the surface is being neither raised nor lowered, it is defined as a profile of equilibrium. A profile of equilibrium is thus transitory, and may exist temporarily far above the base levels of erosion and deposition while rates of supply and loss remain equal but in the stage of final development it would attain those levels. He does not attempt to evaluate the forces or

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factors governing these phenomena. Following his concept, a sandy coast which is being acted upon by littoral forces (waves and tides) is in a position above the base level of erosion and deposition and is therefore temporary in a geologic sense.

A third comprehensive treatment of littoral processes has been presented by Shepard (1948). In addition to presenting extensive data on his examinations of submarine canyons and the materials which form the continental shelves, Shepard (1948) takes issue with the concept that the continental shelves are formed as wave cut or wave built terraces. He contends that the presence of rock and coarse sediments in abundance on the outer shelves, and lack of evidence that the surface sediments do not progressively decrease in size with increase in the distance from shore, are not consistent with the earlier theories as to origin of the shelves. He suggests that they may be of complex origin, and proposes as contributing factors sea level changes during the glacial period, the influence of glacial deposits in glaciated areas, and tectonic changes due to extensive faulting in many coastal regions.

It does not appear to the author that the evidence of Shepard (1948) is intended to discredit completely the earlier theories that wave action is the primary force in shaping the shelves, but rather to explain the factors which also influence them, even though in terms of geological development these latter factors may serve only to interrupt temporarily, by what Johnson (1919) terms an accident in the cycle, the development of the so-called base level of erosion and deposition, or ultimate profile of equilibrium.

The preceding resumé of the geological aspects of littoral processes has been presented in very abbreviated manner as an introduction to the problems of the engineer who is called upon to analyze and design works for the improvement or protection of the shore, or to predict the effect of proposed coastal works upon the natural shore processes. It has often been demonstrated that failure to analyze properly the littoral characteristics of a site may reduce the effectiveness of the improvement for its intended purpose or require maintenance expenditure far in excess of that anticipated. It may also result in unforeseen costly damage to adjacent shores.

Johnson (1919) cites the remarkable disagreement which has existed among investigators in different localities concerning the precise mechanics of littoral transport. He ascribes this condition as due at least in part to the difference in dominant factors prevailing at different localities which makes doubtful the existence of a single hypothesis which would fit all cases precisely. The engineer nevertheless must establish a basis for analyzing specific problems, and where facts are not reasonably obtainable he must substitute opinion. The author has been for many years a student of coastal processes, and has progressively developed a concept of the fundamental mechanics of seacoast littoral transport which, although lacking in quantitative application, has been a useful guide in the analysis of specific problems. This concept does not purport to be either a final or a complete summary of the subject. It has been changed many times over the years and is therefore quite likely to undergo some change in the future. The features involving considerable doubt will be readily apparent. In the following discussion the author will attempt to stress the gaps in present knowledge, with a view to emphasizing the necessity for further investigation.

THE MECHANICS OF LITTORAL TRANSPORT

The transport of sediment by flowing water has long been a subject of broad interest to the engineering profession. The accelerated program in recent years for the control of floods, reclamation, and soil conservation has brought about extensive research in the United States with a view to solving the problems of sediment transport in our streams, rivers and watersheds. While these studies have added to our knowledge of the subject there remains much to be learned about the true mechanics of sediment transport on the surface of the land.

Undersea sediment transport is similar in fundamental process to that which occurs on the upland. The sediments which form the ocean bed have the same variable characteristics of density, size, shape and position as the remainder of the earth's surface. The fluid characteristics of the sea vary only slightly from those of our fresh water streams and rivers. The flow characteristics are the same in the sea as in water flowing over the land with respect to sediment transport capability. The principal difference in the problem of the two regions lies in the extremely complex flow pattern in the littoral zone as compared with that in a confined waterway. Einstein (1948) has concisely and effectively defined the relationship of the problem in the two regions.

Along the sea coasts of the continents, measurements have been made of rates of erosion or accretion at specific localities, usually where barriers, which intercept material in the process of littoral transport, have been erected. The evidence developed from such measurements, supplemented by consideration of the littoral forces involved, forms the basis for the author's concept of littoral transport. An attempt will be made to state this briefly. The views expressed are not original, but an effort has been made to express them in logical sequence to aid the coastal engineer in his analysis of specific problems.

Littoral transport. The movement of water over the bed of the sea exerts a tractive force upon the surface particles on the bottom. When the force exerted exceeds the resistance of the particle to movement, transport takes place. The characteristics of transport are thus dependent principally upon the velocity and direction of water movement; upon the size, density, shape, and position of the surface particle; and upon the slope of the bed. The author chooses to identify that portion of the coastal slope over which littoral material is transported by currents primarily induced by waves and tides as the littoral berm.

In a confined channel with unidirectional flow, it has been possible, in a limited degree, to establish relationships of the variables stated above. Because of the range of variability of turbulent flow and material characteristics, a rigorous solution applicable to all conditions encountered in nature has not been practicable. The variability of flow characteristics in the littoral zone of a seacoast is probably the most complex of all waterways in which sediment transport is of interest. The waters in this region are in motion by reason of currents caused by wind, tide, atmospheric pressure, density, temperature, and waves. To reduce all the components to a resultant seems at present to be an insurmountable task.

Wave induced currents. The currents considered to be most important to material transport on an open seacoast are those induced by waves. Ordinarily, only wave currents cause bottom velocity sufficient to set bed material in motion. For practical purposes, the limiting depth for a measurable wave-induced current is half the wave length. Movement of bed material observed in depths exceeding 400 ft. has been attributed to wave action, but excluding extraordinary occurrences, the limiting depth of such movement is probably of the order of 200 ft. The wave current is oscillatory, moving in the direction of wave travel during passage of the crest and in the opposite direction during passage of the trough. In deep water, the current path is orbital in a vertical plane. In shallow water, it is elliptical at the surface and approximately horizontal at the bottom. The orbital movement is irrotational, resulting in mass transport of relatively small magnitude in the direction of wave travel. In deep water, the theoretical wave form is such that a horizontal component of the orbital velocity is the same in each direction. To a degree governed by the height-length ratio of the waves and the coastal slope, the crest steepens as the wave moves up the slope, and if measured at still water level, becomes shorter than the trough. This has the important effect of unbalancing the oscillatory current, the greater velocity being in the direction of wave travel. The current, in a space of 1/2 the wave period, is obliged to accelerate from the rest to maximum velocity and decelerate to rest. Maximum velocity attained is about 1.6 times the average velocity. Because deformation of the wave is coincident with its ability to exert force upon the bottom, the velocity differential favoring onshore transport is believed to prevail over the surface of the littoral berm.

Bed material movement. The combined effect of mass transport and velocity differential give the wave currents greater competence to move material in the direction of wave travel. Offsetting this onshore component are gravity (to move onshore, the material must move up the slope), return flow due to mass transport, and the currents produced by wave reflection. If the bottom is regular, the shore straight for appreciable length, and aligned normal to the direction of wave travel, the return flow and wave reflection may produce a current which flows seaward along the bottom in the surf zone, commonly observed as "undertow." If irregularities in the bed develop, the return flow may be localized in swift streams flowing offshore, commonly called "rip tides" but more accurately named rip currents. If the wave direction is not normal to the shore, the reflected currents may be visualized as following an orbital or elliptical path in a horizontal plane, with an alongshore component favoring the direction of wave travel in the shoreward half of the orbit or ellipse, and opposing it in the seaward half. When the waves approach from two or more directions coincidentally, the return flow may again occur in the form of rip currents, probably more numerous, less conspicuous, and with less individual power and duration than in the case cited above. Thus throughout the waters overlying the littoral berm, there exists a pattern of forces vectoral in character and infinite in number, with equilibrium at any point very unlikely.

Material sorting and slope. Considering only the effect of the velocity differential in the oscillating (unbroken) wave current upon material sorting we find that greater competence of the shoreward component should cause persistent shoreward movement of the more resistant particles. Theoretically, for a given depth, slope and wave, there is a characteristic bed material which would be shifted back and forth but would not depart from its oscillitory orbit, which could be classified as equilibrium material. For convenience in discussion let us assume that density, position and shape characteristics of the bed material remain constant and that resistance to movement is governed by size of the particles. Assuming a fixed slope and wave, the equilibrium particle size would increase as the depth decreases, because the current competence in a shoreward direction likewise increases as the depth decreases. For a fixed depth and wave, the equilibrium particle size would increase as the slope steepens, because gravity increases in influence as the slope steepens, therefore the same particle would advance to a higher position on a flat slope than on a steeper one. For a fixed depth and slope, the equilibrium particle size would increase as the bottom velocities decrease, that is, as the period or height of the wave decreases, for weaker currents cannot push the larger particles as far uphill. The significance of this hypothesis lies in the reasoning that all particles larger or smaller than the equilibrium size are in transit shoreward or seaward, respectively. Further reasoning follows that if the limits of variability of slope, depth and wave characteristics are known, they can be translated to terms of bands paralleling the shore, with limits fixed by depth, within which material of specified characteristics remains. Because of the nature of the variables, these bands must necessarily overlap. For example, if we consider the 10-ft. depth curve on a typical California shore, at which depth the slope is about 1:40, we find that the depth curve moves seaward and shoreward over a range of more than 300 ft. due solely to the effect of tide. At greater depths, the slope is ordinarily flatter, and the band widens. The variability of bottom current characteristics produced by waves introduces a much broader band. Slope variability is obviously a combination of cause and effect, since the slope is constantly in a state of adjustment while the transporting forces and the material characteristics are seeking equilibrium. A storm wave episode concurrently with a falling spring tide may shift a large volume of beach sand offshore to considerable depth. Continuance of the same storm through the interval of a rising spring tide may bring the same material to shallower depths, where subsequent weaker waves may return it to the shore. A complete correlation of beach and offshore surveys with wave and tidal data, over a suitable range to confirm or adjust this hypothesis, still remains to be accomplished.

At the seaward limit of the littoral berm, wave directions are only slightly influenced by the bottom, and may be at a very sharp angle with the bottom contours. As the waves progress up the slope, their directions are changed by refraction, the trend of change always being toward an alignment normal to the contours. Thus those forces affecting material transport which result from mass transport and current velocity differential although weaker as the depth increases, tend to have a greater alongshore component in the deeper area of the littoral berm than in the shallow area. In the concept of orbital or elliptical currents developed by return flow, an opposing current tends to counteract the alongshore wave current component in the deeper region. Little or no knowledge exists of the nature of resultant transport in the deeper portion of the littoral berm.

Surf zone. In the shoreward portion of the littoral berm, the concept of orbital or elliptical return flow tends to augment the alongshore component produced by the wave angle. The resultant flow pattern has the appearance of a current paralleling the shore, commonly called the littoral current. In this portion of the berm, known as the surf zone, the wave characteristics previously discussed change abruptly. Upon reaching a depth 1 to 1-1/2 times the wave height, depending upon bottom slope and reflected currents, the waves reach a state of critical steepness (crest orbital velocity greater than velocity of wave advance) and break. The manner of breaking varies widely between ranges characterized as the "spilling" type and the "plunging" type. A high degree of turbulence exists in the vicinity of the breaking point. A surface current moving shoreward develops, passing over the remnant of the reflected current from preceding waves flowing seaward along the bottom. The wave may re-form and break again a number of times before reaching the shore, depending upon slope and wave period. Within the surf zone, bed material is brought into suspension, in quantity depending upon the degree of turbulence, and both bed load and suspended load transport are in progress. A littoral current, if present, importantly affects net material transport in this region. The material brought into suspension by the breaking wave is carried landward by the surface current until it settles into the reflected bottom currents, whence it is carried seaward again. Net transport landward or seaward is again dependent upon material characteristics, depth, wave characteristics, and beach slope, the latter being constantly in adjustment toward equilibrium. The angles of incidence and reflection govern the path of material particles, although the path is not necessarily angular, thus there is an alongshore component of material transport within the surf zone favorable to the direction of wave travel even though the so-called littoral current in unidirectional pattern should be non-existent.

Offshore bars. The concept of greater competence of the onshore phase of the oscillatory wave current admits that under favorable conditions of wave, depth, and material characteristics, net transport shoreward may occur in the area seaward of the surf zone. At some point on the slope of the berm, onshore transport will be checked by the effect of gravity and reflected currents. At this point, conditions are favorable to an accumulation of material, known as an offshore bar. The position at which the bar will form is dependent upon depth, slope, material and wave characteristics. When depth change is rapid, as during spring tides, or when wave characteristics are non-uniform, the position favorable to bar building shifts rapidly shoreward or seaward, and a measurable bar is unlikely to develop. Once started, a bar may govern the breaking point of waves for the range of depth change for the lower range of tides. At certain localities on the Pacific coast, it is not uncommon for a substantial bar to form during the period of neap tides, and to merge with the shore during ensuing spring tides, advancing the high water shore line as much as 100 ft. in the space of a few days. Subsequent slope adjustment, over a longer period, restores the shore to normalcy. Less conspicuous bar development and disintegration is considered to be normal to littoral processes. At the more mature stages of its development, the larger type offshore bar may have a pronounced effect upon drift phenomena, principally by creating a semi-confined channel paralleling the shore within which swiftly moving currents flow, altering the normal path of the return flow currents previously discussed.

Summary of the problem. The preceding hypothesis conceives radically different processes of littoral transport in those portions of the littoral berm which lie respectively seaward and shoreward of the breaking point of waves. A feature distinctly common to both regions is that the dominant forces involved are those produced by wave action. Another common feature is the importance of the character of bed material upon the manner of transport. In the surf zone, the transport takes place both in suspension and along the bed. In the offshore zone, the forces involved appear capable of producing only bed load transport. Relative values of net alongshore transport in the two zones is unknown. It has been generally believed that the surf zone is by far the most important in this respect. Mounting evidence of major volumetric changes in offshore areas has led the author to suspect that the littoral processes in the deeper zone may be of equal or greater magnitude than those occurring in the surf zone. Research now in progress under guidance of the Beach Erosion Board is expected to develop much additional data on this subject. The importance of littoral transport in the offshore zone will be discussed below.

ANALYSIS OF LITTORAL CHARACTERISTICS

In the opinion of the author, knowledge of the littoral characteristics of the area in which engineering works are being considered is of vital importance to the coastal engineer. Any coastal structure which extends into the sea will both affect, and be affected by, the littoral processes. Failure to understand and evaluate these effects properly is likely to materially alter the economic value of the completed work. Likewise, projects which would change the natural depth, such as dredging to deepen an inlet, will be governed to a considerable extent with respect to maintenance cost by the rate and mode of littoral drift at that point.

There is no clear-cut path leading to a solution of all such problems. Positive quantitative analysis is still not possible at any location, but deductions from pertinent factual data can produce approximations which will increase in accuracy as experience is gained. The basic factors involved are the magnitude and direction of the littoral forces and the source and disposition of littoral material. If these are properly understood, an approximation of the direction and rate of littoral drift can be made.

The direction and magnitude of littoral forces. Pursuant to the general concept of littoral processes recited herein, the currents induced by waves are considered to be the dominant transporting force. Thus it would appear that the initial step in the problem is a statistical determination of wave characteristics. The data required are direction, height and period of all waves reaching the seaward limit of the littoral berm for a sufficient length of time to encompass a reasonable meteorological cycle for the tributary wave generating zone. Such a compilation, covering a 3-year period, was derived for the Los Angeles District, Corps of Engineers, from synoptic weather maps by Scripps Institution of Oceanography (1947) for five points in deep water off the coast of California. Limits of accuracy of the methods employed are not yet established, but when applied to localities where the rate and direction of drift has been established by measurement, reasonable agreement is apparent. Recording wave gages have been operated at several localities in the United States in recent years, but statistical data from that source covering a suitable period is still lacking.

Wave measurements derived by observation (recording wave gages) would be expected to produce the most accurate obtainable statistics of the height and period, which can be reduced theoretically to terms of energy, work, or power as described by R. L. Wiegel and J. W. Johnson in Chapter 2. Unfortunately no satisfactory means has yet been devised for measuring and recording wave direction, therefore the directional component of the transport capacity cannot now be determined by means of wave gages. This method of deriving statistical wave data has additional disadvantages in that observations at a single site are applicable to a very limited area, and the length of time required to establish an adequate cycle for statistical purposes is often prohibitive for the problem at hand. Mechanical advancement and the passage of time will doubtless overcome these disadvantages and statistics based upon direct observation will ultimately become available.

Sea and swell charts for the open oceans published by the U.S. Navy, Hydrographic Office, (see Chapter 9) provide statistical wave data based on observations by ships at sea and will often be the only immediate source of such data. Accuracy of the charts for any specific area is dependent upon the number of observations reported and the care exercised in making the observations. Wave statistics from this source will normally be available, and should be determined and evaluated regardless of data obtained from other sources. Wave statistics may also be obtained by "hindcasting" wave characteristics from synoptic weather charts, mentioned earlier as a method employed for the coast of California (Scripps Institution of Oceanography, 1947). This method is advantageous in that a substantial period of time may be covered with reasonable expenditure of effort, and the results can be assembled readily in any form desired. Its accuracy is dependent upon the limitations of theory and empirical values, upon the skill and experience of those making the "hindcasts," and upon the accuracy of synoptic weather charts employed (see Chapter 8).

Wave statistics determined by either of the latter two methods establish the wave characteristics in deep water. To determine changes in wave direction, length and height over any point on the littoral berm, wave refraction studies, and diffraction studies where applicable, are made. The transformation of waves in shallow water has been explained in detail by M. A. Mason in Chapter 3 and methods of determining the effects of refraction and diffraction are described by J. W. Dunham in Chapter 4. By means of wave statistics and refraction and/or diffraction studies, it is thus possible to determine an approximation of the vectors and the resultant of wave energy, power, or work, at any point on the littoral berm.

This method of analysis has been employed at several localities on the California coast where predominant direction of littoral drift has been previously established by observing the effects of barriers, and excellent agreement is apparent as regards direction. A satisfactory basis for application of the resultant wave factor in terms of sediment transport capacity has not been established. In the Scripps Institution of Oceanography report (1947) it was suggested that wave work might be a useful parameter for the transporting force, and work factors were computed for that purpose by wave periods and directions. Experimenting with this parameter during wave studies in Santa Monica and San Pedro Bays engineers of the Los Angeles District (1950) computed values of what has been termed the "littoral drift factor" from the following formula:

$$Q = k\, w\, e \sin\alpha_b \cos\alpha_b$$

Q = littoral drift factor = total amount of material moved in littoral drift past a given point on the shore by waves of a given period and direction

w = total work performed by all waves of a given period and direction in deep water just offshore during a typical year

e = wave energy coefficient at the breaker line for waves of a given period or direction, defined as the ratio of the unit width energy at the shore to the unit width energy in deep water

α_b = angle between the wave at the breaker line and the shore

k = a constant determined by observational data and units of measure, probably varying with beach slopes and grain sizes.

Taking a summation of the values thus derived for all wave directions and periods reaching the selected point, and assuming k = unity, the Los Angeles District found some correlation in trend between computed values of the resultant Q and the measured rates of littoral drift at the various points selected. The results did not justify adoption of any empirical values for the constant in the experimental formula. It is the opinion of the author that variability in sediment characteristics governed largely by the variability in rates of supply to the littoral zone, will in most cases prohibit a mathematical relationship involving littoral forces and material characteristics which in itself can be relied upon for determining the rate of littoral drift. Also, the depth at which α_b should be measured remains in doubt. The resultant of the wave work vectors in deep water is believed to afford a means of determining the predominant direction of littoral transport, and together with a consideration of material supply, may indicate whether the rate of littoral drift is large or small.

Sources and disposition of littoral material. The littoral berm adjoining a shore segment of specific length may receive material from several sources. Sediments eroded from the upland may be delivered directly to it by tributary streams, the shore itself may be eroded by waves and the eroded material transferred to the littoral berm, or material from adjoining littoral berms may be transported to it by littoral forces. The littoral berm cannot be fed by material moving onshore from greater depths, since by definition its outer limit is the greatest depth at which sediments can be moved by littoral forces. The deeper region of the littoral berm may in certain cases be an important source of beach material, particularly in regions formerly glaciated.

Assume the case of a shore segment in which the shore and adjoining littoral berm is composed of rock strong enough to resist temporarily erosion by littoral forces. If material is fed to this shore segment from the updrift littoral berm at a rate less than the transporting capacity of the littoral forces, it will move across the rock segment leaving no residue. Individual particles will seek a path in a depth environment compatible with their characteristic size, shape and density.

If the rate of material supply is increased to exceed the transport capacity, or if the littoral forces are sufficiently reduced, sediments will accumulate over the rock surface. As those deposits reduce the depth, the littoral berm will assume a profile governed by the littoral forces. Assuming that the material characteristics remain constant in gradation the profile of equilibrium would be reached when all of the rock is covered between maximum and minimum depth limits governed by environmental characteristics of the specific material supplied.

Continued excess supply after the profile of equilibrium is reached would advance the littoral berm seaward without appreciable change in profile, causing deposit of sediments in depths greater than the littoral forces were competent to accomplish at an earlier stage.

Broadening of the littoral berm would cause dispersion of the littoral forces. If the initial equilibrium profile did not tolerate sediments above sea level to form a visible beach, such a beach would ultimately form when littoral forces in the shore region became sufficiently weakened by dispersion. Once above the sea, the material would be exposed to landward transport by winds. When the rate of landward transport reached the rate of excess littoral supply the littoral berm would become stable.

In reverse order, if the supply of littoral material should be discontinued or reduced below the transport capacity of littoral forces, the littoral berm would recede and the rock ultimately would be laid bare. Material which was forced seaward to depths beyond the initial limit of the littoral berm would remain undisturbed, and in the absence of a lowering of the sea or an increase in the depth capacity of littoral forces, would not resume littoral transit.

The preceding analogy is presented to illustrate the author's concept of the manner in which sediments permanently leave the littoral zone and to illustrate the importance of material balance in maintaining stability of a shoreline.

The littoral berm is fed in part by sediments eroded from the uplands and delivered to the shore by streams. Sediments which are transported in suspension across and to depths beyond the littoral berm before deposition are disregarded because they are of no importance to littoral processes. The littoral berm is depleted by windborne landward transport of the coarser (but not the coarsest) materials and perhaps by permanent seaward deposit of the finer materials. Comparative rates of gains and losses to the littoral berm control the position of the shoreline.

Another source of supply to the littoral berm, more active in earlier geologic time than at present for shores of principal interest, is the glacial outwash. Course materials were delivered by this means to position of considerable depth in the coastal region, from whence, by littoral processes, they may be transported shoreward to the upper regions of the littoral berm. There is considerable evidence that beaches in some localities, particularly on the north and middle Atlantic coast of the United States, may receive nourishment from this source.

LITTORAL BARRIERS

The previously stated concept of littoral processes assumes that the angle between the shore (or littoral berm) and the resultant of the littoral forces is a factor affecting the rate of littoral transport. Thus any shore segment not in equilibrium with respect to rates of supply and loss will tend toward realignment in a direction normal to the resultant of littoral forces, receding if the material supply is deficient and advancing if the supply is excessive. An abrupt change in shore alignment, such as a prominent headland, may act as a littoral barrier, causing material to accumulate on the updrift side. Prominent examples of such natural littoral barriers on the California coast are Monterey Peninsula and the Palos Verdes headland. These mark the southern extremities of Monterey Bay and Santa Monica Bay, respectively. Conclusive evidence of predominant north to south littoral drift exists at both localities. The curving shore alignment in the bight of each bay marks the realignment trend concomitant with reduction in the southerly component of the littoral forces. Extensive dune deposits in the southerly region of each bay are repositories of excess littoral material.

Inlets to tidal bays or estuaries may act as limited littoral barriers. The typical migrating tidal inlet through a barrier beach is manifested by accretion on the updrift side encroaching upon the tidal channel, causing the currents to erode the downdrift shore. The eroded material enters the littoral stream and provides nourishment for the downdrift shore. Thus there is an exchange of the source of littoral material but with generally localized effect upon the littoral regimen.

Tidal inlets on the Atlantic coast are subject to ebb and flood tidal flow of about equal intensity, at estuaries where there is no appreciable fresh water discharge. In the process of inlet migration, a portion of the littoral material is carried into the bay by flood tide currents and deposited out of the range of ebb currents or littoral forces. Each natural migrating inlet thus tends to store littoral material and deficiency in material balance, if any, must be made up at the expense of the downdrift beach.

Diurnal inequality of tides in the Pacific Ocean cause substantially different flow characteristics at tidal inlets from those described above. The sequence of tides during the spring, or highest ranges, is lower low, lower high, higher low, higher high and lower low. Maximum velocities are reached in ebb flow. At an unimproved inlet, littoral material entering the inlet during flood tide is swept seaward by the ebb and deposited offshore in the form of a bar. The depth, profile and alignment of the bar are dependent upon the littoral forces, the character of the littoral material, and tidal characteristics of the inlet. Variability of these functions has prevented the establishment of inlet bar criteria of general applicability. Inlet bars are by no means restricted to Pacific coast waters, but their pattern and behavior appears more uniform in these waters than elsewhere.

For the channels at the throats of Pacific Coast estuaries, it has been determined that the area of cross section in square feet below mean sea level is approximately equal to the tidal prism in acre feet, measured between mean higher high water and mean lower low water (Robbins, 1933). If an inlet is stable rather than migratory, littoral material is being transported across it by natural forces, and the throat dimensions above are fixed by the scouring capacity of the ebb current.

It is generally believed that the bar is the principal path followed by littoral material crossing an inlet. The uniformly fine sands found on the crests of the larger bars (Robbins, 1933) leads the author to believe that the coarser material is transported across in the region of the throat of the inlet. Evidence to a firm conclusion to that effect is presently lacking. It can be stated with reasonable assurance, however, that a stable tidal inlet with a mature bar is not necessarily a littoral barrier.

MAN-MADE LITTORAL BARRIERS

There are three basic types of coastal works which function as littoral barriers. The most common is the jetty or groin which extends from the shore across a portion of the littoral berm, acting as a dam to entrap the littoral drift. The impounding capacity of such a structure is dependent upon three factors: its length, the slope of the littoral berm upon which it is built, and the equilibrium alignment of the shore in that region (normal to the resultant of littoral forces).

If such a structure is built on a shore which is stable with respect to material balance, accretion will occur in the form of a fillet on the updrift side with alignment tending toward equilibrium. Deficiency in material balance on the downdrift side will result in erosion, the alignment likewise tending toward equilibrium. The transport of sediments whose natural habitat is a depth greater than that at the seaward end of the structure would not be affected. Accretion of the coarser sediments on the updrift side results in a slope steeper than normal, whereby these coarser sediments may be transported at greater than normal depths. Depending upon limitations of the impounding capacity of the barrier, littoral material will be transported around its seaward end at an increasing rate until the impounding capacity is reached, whereupon material balance is re-established on each side of the barrier. Stability shore alignment may be permanently altered, resulting in an abrupt offset at the barrier.

If the barrier be sufficiently long or if it extends to great depth, it may alter the character of littoral forces on a localized segment of the downdrift shore. If the region be one subject to reversals in drift direction, accretion may occur on the downdrift as well as the updrift side but to a lesser degree. The erosion zone in this case will be situated beyond the shadow of the barrier on the downdrift shore. Many examples of this phenomenon exist, notably on the California coast, the Humboldt Bay jetties, Newport Harbor jetties, and Camp Pendleton Harbor jetties.

If the equilibrium shore alignment varies only slightly from the stability alignment, the impounding area of a littoral barrier will be elongated along the updrift shore and will be proportionately large with respect to the length or limiting depth of the barrier. Examples of such characteristics are the various littoral barriers in the bight of Santa Monica Bay and on the New Jersey coast, particularly the Cold Spring Inlet jetties at Cape May. Examples of the opposite case, (the equilibrium alignment varying substantially from the stability alignment), are evident along the south shore of Long Island and at Santa Barbara.

If the littoral berm is narrow by reason of precipitous slopes at its seaward margin, a relatively short barrier may have an excessively large impounding capacity. Advance of the littoral berm to re-establish stability characteristics may require an excessive volume of material on such abnormally steep slopes. This feature has not been fully explored, but barriers suspected of possessing this characteristic are Newport Harbor west jetty, Hueneme Harbor west jetty, and Moss Landing north jetty, all in California.

A second type of man-made littoral barrier is the offshore breakwater. This structure is designed to intercept waves and to create a protected area of calm water, usually to meet navigation requirements. Its effect upon littoral processes is the reduction or elimination of the principal component of the littoral forces (waves) within that portion of the littoral berm in its lee, with the result that littoral material accumulates in the protected area. The resulting shore advance acts similarly to a jetty or groin in causing updrift shore accretion beyond the region of direct effect of the structure itself, and corresponding erosion on the downdrift shore. An outstanding example of this type of littoral barrier is the Santa Monica Breakwater (Handin and Ludwick, 1950).

A third type of littoral barrier is the dredged channel across the littoral berm. Such a channel creates greater than normal depths, with the result that littoral material accumulates therein. Maintenance of the channel by dredging removes the littoral accumulations, thus preventing restoration by natural processes of normal littoral transport. If the littoral deposits be removed and redeposited on the downdrift littoral berm in such manner as to resume normal littoral travel, material balance may be maintained and detrimental shore effect would be unlikely. If the material is removed and deposited elsewhere, deficiency of supply to the downdrift shore, with consequent erosion, is probable. The factors affecting a solution of this problem, as well as that of other littoral barriers, are discussed below.

RESTORING NORMAL LITTORAL PROCESSES

Whenever a littoral barrier produces an undesired effect, as is usually the case, it becomes necessary to examine the means by which normal littoral processes may be restored. If the impounding capacity of the barrier is small and if preservation of a navigable channel is not involved, it is probable that the restoration of normal processes solely by the forces of nature will suffice. In other cases, consideration must be given to artificial means of by-passing littoral material across the barrier. Alternative means of preserving the downdrift shore by defensive works or by restoring material balance from a source other than littoral accumulation on the updrift side of the barrier must also be considered. Economic analysis of the benefits from each method in comparison with its cost will determine which should be employed.

Perhaps the most important feature of the problem is a proper understanding of the littoral regimen of the specific problem area. Variable conditions exist in different continental regions depending upon the stage of geologic development. Along the Pacific coast of the United States, it is the author's belief that the littoral zone is fed intermittently along the shore by material eroded from the upland. Principal replenishment occurs at the time of major floods, when large delta deposits are formed and serve to feed the littoral stream during the intervening periods between floods. The natural littoral barriers establish the limits of the littoral regimen for each independent shore segment. If a man-made barrier is created within such a shore segment with impounding capacity sufficiently large, its effect may ultimately extend along the entire downdrift shore to the next natural barrier. Defection in material balance will accelerate the depletion of each delta deposit in succession along the shore. The frequency and size of the deltas, as well as the frequency of floods, will govern the rate of progress along the shore of what may be termed the erosion wave resulting from defection in material balance.

Proper understanding of the limits of the littoral regime between natural barriers is thus of paramount importance in determining the ultimate as well as the immediate effect of intervening man-made littoral barriers, the justification for remedial measures, and the best method of accomplishing such measures. Defensive works have been employed extensively in the past, often without advance knowledge of the ultimate length of shore likely to require protection, and without consideration of the comparative cost of restoring natural littoral processes in lieu of defensive works.

By-passing littoral drift has not been successfully accomplished in enough cases to establish criteria which can be applied to any locality for analysis and determination of cost. Fundamental requirements for successful by-passing are that:

1. The limiting effective depth of the littoral barrier be known.
2. Material intercepted by the barrier be removed at the rate of accumulation (resulting in no net accumulation of material on the updrift side of the barrier).
3. The material be deposited on the downdrift littoral berm, in a location fully exposed to littoral forces, and in depths not exceeding the limiting effective depth of the barrier.

At Santa Barbara, California, (Los Angeles District, 1948a) a breakwater connected with the shore served as a littoral barrier for approximately 5 years. Thereafter the updrift shore became stable and all littoral material in transit shoreward of the limiting effective depth of the structure moved around it and deposited in the harbor. Because of the protection afforded by the breakwater, it has been a simple matter to dredge all of the accumulated material biennially with conventional pipe line dredging plant and deposit it on the downdrift shore. The method employed has been successful in maintaining the downdrift shore, although the rate of depletion of the downdrift disposal area (feeder beach) between replenishments has been somewhat less than the rate of accumulation in the harbor. This indicates that the feeder beach may be shorter than optimum length, with result that some material is placed in depths greater than its natural environment from whence it is moved less rapidly than normal. Further study of the Santa Barbara problem is in progress by the University of California, Berkeley, as part of a program of research sponsored by the Beach Erosion Board.

At South Lake Worth Inlet on the east coast of Florida, by-passing of littoral material, by means of land-based plant near the jetty end, has been in progress intermittently for several years (see Chapter 34). It is the opinion of the author that the impounding capacity of the jetties at South Lake Worth Inlet has been virtually exhausted, and that littoral material is passing the barrier by natural means in substantial quantity. The by-passing operation, by removing the accumulations of coarser material moving in the shallower zone, prevents extreme shoaling of the adjacent navigation channel and maintains the downdrift beach. The quantity by-passed, reported to be 50,000 to 70,000 cubic yards per year, is about one-third the rate of accumulation in early stages of the barrier's history.

Effectiveness of this installation indicates that in certain cases, natural processes may be restored and required depth of a navigation channel maintained by by-passing only that material which has a native depth environment less than the required depth of the navigation channel. Present knowledge of the depth environment characteristics of littoral material is inadequate to enable quantitative prediction of an operation of this character. Material balance requirements must for the present be determined experimentally.

Because of the uncertainty of the effectiveness and cost of by-passing material across jettied inlets by means of land-based plant, consideration has been given to constructing littoral traps on the updrift side of inlets by means of an offshore breakwater (Los Angeles District, 1948b). This method permits use of a conventional floating pipe line dredge for the by-passing operation and is considered to be positive in results attained. The method has not previously been employed in a case where it was designed for the specific purpose of by-passing littoral drift, but is similar in operation to Santa Monica Breakwater (Handin and Ludwick, 1950) where the first fully effective by-passing operation was accomplished in 1949, 15 years after the structure started to function as a littoral barrier. Observations to determine the rate of filling of the area dredged in lee of the breakwater are in progress.

ARTIFICIAL NOURISHMENT OF THE LITTORAL ZONE

It is often necessary to restore eroded shores, or widen existing beaches, by making beach fills. The concept of material environment characteristics in the varying depths of the littoral berm has an important bearing upon determining suitability of specific material for use in a beach fill, and upon the manner in which it should be placed.

The material sorting, or selective transport characteristic of the littoral processes requires that regardless of the composition of underlying material, the surface material will adjust itself to its depth environment. If uniformly fine material is placed on the foreshore of a beach normally composed of uniformly coarse material, the fill will rapidly be shifted offshore to its proper depth. If the gradation of the fill material covers a broad range, with a substantial proportion in the size range of the native material, the surface will readily acquire the characteristics of the native beach. Seasonal changes and recessions due to storms will expose the underlying materials periodically to littoral forces, and progressive loss of the finer particles to deeper regions will occur.

In somewhat the same manner, coarse materials deposited offshore may be prevented from moving shoreward by a selective surface coating of finer material which has attained its proper environment. By occasional exposure to littoral forces, the underlying coarse material may work its way shoreward. Experiments by the Beach Erosion Board at Long Branch, New Jersey (Hall and Herron, 1950), when analyzed on the basis of the reasoning above, provide some support for the hypothesis. Non-conformity of surface samples taken from the littoral berm may also be explained by this reasoning.

Selection of material for a beach fill must therefore be based upon an analysis of the probable ultimate distribution of the material along the littoral berm and the effect that such distribution will have upon the intended purpose of the fill. If one purpose is to create a flatter offshore slope, availability of a substantial portion of fine material would be desirable. If the purpose is to armor the foreshore slope, fine material would have no value. If the purpose is to widen a beach, material of any size will serve for the major portion provided the seaward face is built of sand corresponding generally to the native beach for a thickness adequate to accommodate expected fluctuations due to storm and seasonal changes.

Beach fills amounting to more than 50,000,000 cubic yards of material, creating more than 1,000 acres of additional sandy beach, have been made at various locations on the coast of Southern California over the past 15 years. The purposes have been varied, and for the larger part of the total quantity the object was simply to dispose of excess dredged material for which no other suitable disposal area was available. Little attention was paid to the character of the material, which was for the most part sand but included in several instances clay, gravel and boulders. The author has followed the progress of littoral forces acting upon these beach fills and in all cases they have rapidly assumed the characteristic appearance of the native beach except that slope characteristics have apparently been permanently altered in some instances. It can be stated in general that with one exception in the easterly portion of Newport Beach, all of the fills have been eminently satisfactory with no detrimental effect. In the exception cited the foreshore slope has remained much steeper than normal though there is evidence that it may be flattening gradually. The material deposited in that reach of shore contained a large proportion of very coarse sand.

Sampling and mechanical analysis of materials deposited to make beach fills mentioned above were unfortunately either omitted or were too limited to provide reliable data. Subsequent sampling of the beaches has been accomplished without regard to seasonal or storm wave effect and are of limited value for analytical purposes. The author hopes to obtain suitable data for a future paper on this subject.

SUMMARY AND CONCLUSIONS WITH RESPECT TO ECONOMIC ASPECTS OF LITTORAL PROCESSES

From the standpoint of geological processes, a sandy beach is an interim phase in the ultimate development of a coastline, principally because a sandy beach can survive only if it receives nourishment at the rate of depletion. Depending upon its stage of maturity, a particular coastal segment may be advancing, retreating or in a state of approximate equilibrium. Its precise status in any phase is dependent upon material balance, that is, the rate at which sediments are delivered to the littoral zone compared with the rate at which they are removed therefrom. The ultimate stage of development may be described as a state in which the littoral forces are incapable of transporting materials which form the littoral berm, and erosive forces acting upon the land mass are incapable of delivering sediments to or from the coastal shores.

The rate of changes wrought solely by natural geologic processes is ordinarily so slow as to be of minor importance to the engineer whose mission is to protect or improve a particular shore segment. Because the engineer's problem is fundamentally dependent upon the state of material balance, he must investigate that feature sufficiently to determine the extent to which material balance is influenced by natural processes and by works of man. In this investigation he may be aided by the geologist competent to analyze the geomorphology of the region, and by measurements of the effects of man-made works in accelerating or retarding the transport of sediments within the area tributary to the shore under consideration.

It is of the utmost importance that the engineer determine the limits of the littoral compartment for a shore segment under investigation. These limits will be fixed by the natural barriers to littoral transport on each side of the shore segment, and will often be at great distances from the particular problem area. Knowledge of the limits of the littoral compartment will enable determination of the natural sources of littoral nourishment, the region within which the magnitude and direction of littoral forces must be considered, and the zone of influence upon littoral processes of structures or other works existing or being considered.

The littoral forces consist primarily of the currents induced by wave action. The rise and fall of the sea surface in the form of tides, and the configuration of the surface of the littoral berm, greatly influence the distribution of littoral forces over the littoral berm. The sediments are selectively transported according to characteristics of size, density and shape. Wave induced currents possess greater competence in the direction of wave travel, thus the more resistant sediments tend to move shoreward. A specific sediment particle is shifted shoreward and seaward while seeking environment compatible with its character but will remain within depth limits fixed by the variability of tides and wave characteristics peculiar to the region.

More or less constant onshore and offshore movement of material is accompanied by lateral movement along the shore in a direction governed by the resultant of the littoral forces. Relative rates of alongshore movement of sediments at different depths is not known at this time, although it is generally believed that the coarse sediments native to the surf zone have a greater velocity of travel along the shore than the finer materials in the offshore portion of the littoral berm. Evidence is mounting that a substantial proportion of the net rate of littoral drift over the entire littoral berm may occur in depths seaward of the surf zone.

The effect of selective transport and the transport rate at varying depths are important to the engineer in his analysis of the effect of existing or proposed littoral barriers. He may determine the predominant direction of littoral drift with reasonable accuracy by analysis of the littoral forces and the shore characteristics within the littoral compartment. In the absence of measurements of rates of accumulation at existing littoral barriers, present knowledge does not provide a means of quantitative determination of the rate of littoral drift. Even though the rate of drift be determined by measurement at a specific barrier, evidence is lacking as to the transport in progress in depths greater than the limiting effective depth of the barrier. Until knowledge of the mechanics of littoral processes becomes available, the engineer is obliged to substitute opinion based upon all the pertinent facts he can obtain.

Examples of man-made littoral barriers are groins, jetties, breakwaters and dredged channels in the littoral berm. Each has an impounding capacity depending upon its limiting effective depth (and size in the case of a dredged channel). When the impounding capacity of such a barrier is reached, normal littoral drift past the structure will be resumed. Disregarding the effect of seasonal or cyclical changes in the natural rate of littoral drift, the rate of accumulation at a barrier will be highest in the earliest stages and will decelorate as the impounding capacity is approached. This fact must not be neglected in estimating the drift rate by measuring accumulations at existing barriers.

The effect of a littoral barrier is to alter the material balance on the downdrift littoral berm. Except for a limited local area depending upon the extent and nature of the barrier, the littoral forces will not be altered. The downdrift shore must therefore supply such deficiency in material balance as the barrier may create. The ordinary consequence is erosion of the downdrift shore. The engineer planning a littoral barrier must include in his analysis the effect upon the downdrift shore in terms of magnitude, rate, duration, and economic value. If the effect thus determined is in the nature of a consequential damage, an evaluation of the damage must be considered, in conjunction with the cost of constructing and maintaining the barrier, in determining economic justification of the project.

Remedial measures to offset or prevent consequential damage, resulting or expected to result from a littoral barrier, are feasible in many cases and should be considered and analyzed economically. Such measures may be divided basically as to type into two classes, defensive works and restoration of normal littoral processes.

If the barrier is located in the updrift region of a littoral compartment, and there is no nearby source of supply such as a river delta on the downdrift shore, defensive works will rarely be an economic solution. Such works protect only the immediate shore upon which they are constructed. Defensive works in the form of groins, designed to trap littoral drift, may protect a portion of the shore while causing accelerated erosion elsewhere. Defensive works employed to offset a deficiency in material balance in the case stated above retard but do not prevent erosion, and are not likely to be permanently effective. This is a case in which restoration of normal littoral processes by mechanically transporting material across the barrier is likely to be the most practicable solution.

If a relatively short length of shore is exposed to hazard by reason of a littoral barrier, as at the downdrift end of a littoral compartment, defensive works are more likely to be warranted. Maintenance by nourishment from sources other than littoral accumulations at the barrier may in some cases be employed economically with or without defensive works.

In the opinion of the author restoration of normal littoral processes at littoral barriers would solve most of the erosion problems now existing on the coastal shores of the United States. It is believed that this method of shore protection should be analyzed and evaluated economically wherever barriers have caused a deficiency in material balance.

Much knowledge must be gained before the mechanics of littoral processes can be stated with assurance. This situation presents a challenge to the coastal engineer who must constantly deal with littoral processes. It is believed that the present state of knowledge is adequate to avoid repetition of many errors made in the past. Investigations now in progress by the Beach Erosion Board promise to add to our knowledge of the subject. Littoral barriers have been designed which incorporate means for by-passing littoral material. It will be through the construction and operation of such works that our greatest advances in knowledge will be made.

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CHAPTER 16

LITTORAL PROCESSES IN LAKES

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INTRODUCTION

Lakes differ from the oceans in being smaller, shorter lived, and tideless. Lakes may have fresh or salt water, in some instances with salinity far exceeding the normal ocean. In the most general case, a lake is a body of standing water occupying a depression in the earth's surface. The depression may be produced by a variety of geological processes, giving rise to several classes of lakes.

Lakes are classified according to origin into the following groups:

1. Glacial lakes
2. Flood plain lakes
3. Coastal or deltaic lakes
4. Deflationary lakes
5. Lakes of volcanic origin
6. Lakes of diastrophic origin
7. Artificial lakes

Each of these classes has certain attributes, of which some are important in application of engineering principles to construction problems. The characteristics are briefly reviewed as a preliminary to discussion of littoral processes.

CHARACTERISTICS OF LAKES

Glacial lakes. Glacial lakes occupy depressions developed by glaciation. Cirque lakes in rock basins are characteristic of upper parts of glaciated mountain areas. "Paternoster lakes" are chains of lakes along glaciated valleys, occupying rock basins or depressions in glacial deposits. The High Sierra and the Rocky Mountains provide numerous examples.

Continental glaciation, represented by ice sheets of the Pleistocene, produced several kinds of lakes. In areas of ice erosion (eastern Canada) many of the lakes occupy scoured rock basins. In the north central United States the glacial lakes are associated with depositional features.

The lake regions of Wisconsin and Minnesota are almost entirely of glacial origin. The lakes occupy irregularities of the glacial drift surface, produced by damming of drainage lines, by moraines, and by associated meltwater processes. These glacial lakes range from small ponds to large bodies of water with sufficient fetch to develop significant wave action.

Glacial lakes as a class are shallow. Shore features, other than rims of boulders or pebbles, are uncommon in mountain lakes. Glacial lakes in drift may have well-developed beaches, but they are commonly narrow and may be localized. The bottom deposits in glacial lakes in Wisconsin and Minnesota include sand or silt, soft dark mud, and marl (a calcareous clay), lying on the original foundation of stony clay (glacial till) or bedded sand and silt.

Flood plain lakes. Flood plain lakes are formed in valleys by normal stream processes. Ox-bow lakes are most common, produced by meander cut-off. Other lakes may occur along the flood plains of wide valleys, especially where the river level is higher than the flood plain and confined behind a levee. Less common flood plain lakes may be produced by land slides or similar phenomena.

Flood plain lakes are mainly small and ephemeral. They seldom show shore features, and tend rapidly to be occupied by vegetation to become swamps and marshes. The deposits are mainly organic-rich mud, lying on the foundation of normal flood plain deposits below.

Coastal or deltaic lakes. Lakes may form along coasts by development of barrier beaches, bars, spits, or migration of dunes which isolate bodies of water from the sea. The lakes may start as salt or brackish lagoons, becoming fresh by contributions from streams. Deltaic lakes are similar, formed by changes in river channels or by combinations of stream and coastal processes.

Coastal lakes are commonly shallow, and in some the waters may become partially restricted in circulation, giving rise to stagnant conditions. Bottom deposits range from clean sand to dark organic-rich mud. Coastal lakes are completely cut off from the sea and are tideless. They receive most of their sediment from associated streams.

Deflationary lakes. Lakes of deflation occur in arid regions, where wind work may scour shallow basins. The lakes are commonly shallow and subject to evaporation. The water may be saline, forming evaporitic deposits associated with sand, silt, and mud carried in by temporary streams. Deflationary lakes seldom have a true littoral zone, owing to the wide range in their expanding and shrinking areas.

Lakes of volcanic origin. Volcanic processes may produce lakes by damming of drainage lines by lava flows, or by development of calderas from volcanic explosions. Lake Tahoe is an example of the former, and Crater Lake, Oregon, is a type example of the latter. Volcanic lakes are small to moderate in size, and they may have steep sides, rugged outlines, and considerable depth. Associated deposits are re-worked volcanic debris.

Lakes of diastrophic origin. Earth movements are responsible for the formation of some lakes. Abrupt dislocation of rock strata (faults) during earthquakes may produce depressions or alter natural drainage. The lakes along the San Andreas fault south of San Francisco furnish examples. Slower downwarping of the earth's surface due to long-time isostatic adjustment may also develop lakes. The Great Lakes, although formed initially by combination of glacial scour and depositional disturbances of normal drainage, show evidence of basin tilting due to post-glacial elastic adjustments of the earth's surface. Basins of interior drainage, with deflation (playa) lakes, are commonly of diastrophic origin.

Lakes of diastrophic origin may range from large to small, with depths from shallow to great. Except for fault-produced lakes, many of them are among the larger lakes of the earth. The deposits in large lakes may not differ markedly from similar deposits in the oceans in terms of their physical attributes (texture, composition, and structure).

Artificial lakes. Here are included all man-made lakes, which range from small ponds to major bodies of water. Artificially dammed lakes are commonly located along valleys and are linear or branched in form, often with steep sides and moderate depth. Deposits in artificial lakes include sand and silt brought down by the original stream, which may form a delta at its mouth, and spread a blanket of finer material over the lake bottom. Deposits laid by density currents appear to be more common in these linear lakes with stream-bottom gradients than they are in more ovate, gentler-bottomed natural lakes of equivalent size.

LAKE PROCESSES

A number of lacustrine processes are sufficiently different from their oceanic counterparts to warrant mention.

Overturn. Seasonal changes in temperature in temperate regions causes an overturn of lake water. Cooling of the surface in late fall causes a sinking of the heavier water, with displacement of warmer water from below. This continues until the lake temperature is 4° C. throughout, whereupon the surface water remains at the surface until and after freezing.

In the spring the surface temperature is again raised to the point of minimum density, with another overturn. During the ensuing summer the surface water is further warmed and remains at the surface, whereas the lower cooler water may become oxygen-depleted. The main effects of overturn are a complete replenishing of oxygenated waters through the lake.

Overturn does not occur in tropical or polar regions. In the former the cooler water at the bottom is commonly stagnant. In polar regions the warmer stagnant water is at the bottom, with a layer of colder water at the top.

Ice shove. In temperate regions the work of ice may be very important in its effect on the shore. Small lakes display the effects of ice above, which may push and distort the shore deposits into irregular ridges.

Ice shove occurs where the lake surface is small enough to permit the surface ice to act as an essentially rigid mass. During extreme cold spells the ice contracts and develops cracks which fill with water and are refrozen. During a later temperature rise the ice expands and thrusts against the shore.

In larger lakes ice damage is mainly the result of ice floes moving against installations. In Lake Michigan broken ice floes frequently form extensive icefields which may be driven against shore by onshore winds. Such icefields serve as protection against severe wave action, but the ice exerts heavy pressures and abrasive forces against structures. Timber piles are particularly subject to such abrasive forces.

Life history of lakes. A characteristic of natural lakes is that they are relatively short-lived in a geological sense. Various processes tend toward the extinction of lakes, but the time required depends largely upon the size of the lake and the dominant process.

Three general processes which destroy lakes are recognized. The first is the lowering of the outlet by streams flowing from the lake. The lake level drops as the outlet lowers, until the entire lake basin may be drained. Diastrophic processes which tend to warp the lake basin may entirely eliminate a lake through slow tilting.

The third process of lake destruction involves filling the lake basin with vegetation or sediment. Sediments may be introduced by streams or by wind. Vegetation gains a foothold on the shallow borders of lakes, and gradually grows farther into the lake. Filling by vegetation is particularly common in glacial lakes, which become converted into peat bogs, muskegs, or marshes.

Artificial lakes are particularly subject to filling with sediment, as is evidenced by the numerous dammed lakes in which reservoir silting has become a serious problem.

FEATURES OF LARGE LAKES

Large lakes display the same processes as small lakes, but because of greater wave fetch, shore processes are much more pronounced, and locally or temporarily may attain the order of magnitude of average shore conditions on the oceans. Large lakes are also more important than small lakes in terms of harbor development, protective structures, beach maintenance, and other engineering aspects.

LAKE MICHIGAN

The writer is most familiar with Lake Michigan, which will be taken as an example of a large lake. The lake is about 320 miles long, and 80 miles maximum width. The surface area is 22,500 square miles. Its greatest depth is 870 ft., and its surface lies about 580 ft. above mean sea level at New York.

The western shore of the lake has been studied by the writer in more detail than the eastern. The stretch of greatest interest to the writer extends from Milwaukee to the southern end of the lake. Specifically, many of the observations on waves and currents mentioned below were made at Northwestern University, just north of Chicago, under research grants from the Graduate School.

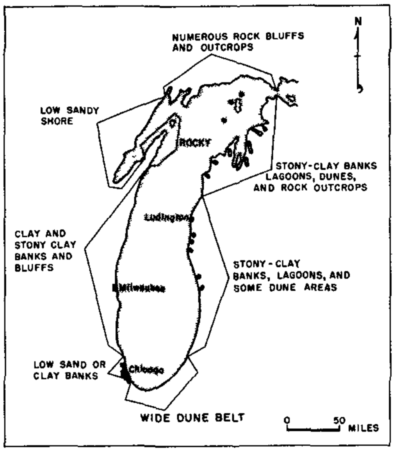

Fig. 1. Shore features of Lake Michigan

Shore features of Lake Michigan. Fig. 1 is a sketch map of Lake Michigan, showing a classification of its shores. The northern shores, including the peninsula along Green Bay, are typically rocky, with limestone bluffs and pocket beaches. The western shore is characterized by clay or stony-clay bluffs with relatively narrow beaches along most of its extent. The eastern shore is an alternation of stony-clay bluffs and sand spits with lagoons, and wider beaches on the average than the western shore. Some rocky areas occur along the northeast. The southern shore has a wide sand beach backed by an extensive dune area.

Beaches along the western shore are seldom more than 100 ft. wide. The slopes vary from 1 on 5 to 1 on 60, with an average slope of about 1 on 30. The beach composition ranges from pebbles to sand, with sand beaches more common.

The banks or bluffs along the western shore range from 10 or 15 ft. to more than 100 ft. high. The southern part of the western shore was low in its natural state, and is now almost wholly fronted by sea walls or bulkheads from Chicago to the Indiana state line.

Milwaukee, Waukegan, and Chicago have the most important harbors along the southern part of the western shore. Many parts of this stretch have also been improved with bulkheads and groins. This is especially true of the residential districts between Chicago and Waukegan, and adjacent to other cities along the lake.

Winds, waves, and currents. Along the southwestern part of Lake Michigan the winds blow from the north 17 percent of the time, from the northeast 7.5 percent, from the east 6.9 percent, and from the southeast 8.9 percent of the time. These winds control the waves which strike the western shore. From a four-year daily record at Evanston, it was found that the lake was calm or essentially so (waves less than 6 in. high) about 7 percent of the time. Waves from the northeast occurred about 47 percent of the time, from the east about 15 percent, and from the southeast 31 percent.

For the four-year period from summer, 1946 to summer, 1950, the average period of all waves, regardless of direction, was 3.6 sec. at Northwestern University. For the northeast and east waves the average was 4.4 sec., and from the southeast, 3.0 sec. Waves from the southeast were 5 sec. or less for 96 percent of the days. For northeast waves the periods were 5 sec. or less for 67 percent of the days, 7 sec. or less for 94 percent of the days; and the longest average period observed from the northeast was 10.5 sec.

For the same period, the average wave heights at the breaking point for waves from all directions was about 1.0 ft. For northeast and east waves the average was slightly larger, and from the southeast slightly smaller. The "significant wave", as an overall average, was 1.6 ft. from the southeast, and 2.3 ft. from the east and northeast. The highest average waves observed were from the east, with height 6.5 ft. The largest individual waves observed at the breaking point did not exceed 10 ft., and were in fact probably not larger than 8 ft. in height.

Other wave data, collected offshore at Milwaukee, during 1931 and 1932, showed that waves exceeded 3 ft. 39 percent of the time (in contrast with 10 percent at Northwestern). Waves more than 10 ft. in height occurred 1.1 percent of the time, in contrast with no average values that large at Northwestern.

In general, it appears that the southern half of the western shore of Lake Michigan has an expected average wave height at the breaking point of the order of 1.5 to 2.5 ft. during any given year. The maximum average wave height at the breaking point may be of the order of 8 ft., with individual waves at the breaking point not exceeding 10 ft.

Refraction plays an important part in wave heights at the breaking point. It is noticeable at Northwestern that storm waves from the east, which approach the shore head on, are larger than northeast waves, despite the much longer fetch of the latter.

Shore currents along the western shore of the lake flow southward about 50 percent of the days, and flow northward about 30 percent of the days. During calms or when the waves are from the east, no strong unidirectional currents occur. Data are sparse on currents, but it is observable that the southward-flowing currents are stronger than those flowing north, owing to the greater energy content of the northeast waves. The strongest current observed was 3.5 ft. per sec. to the south.

Changes in lake levels. Lake Michigan, in common with the other Great Lakes, displays seasonal and long-time fluctuations in level. The datum of the lake is 578.5 ft. Since 1900, the highest monthly mean level was 582 ft. in July, 1929. The lowest monthly mean in the same period was 577 ft. in January, 1926.

The major cycles of lake level have a period of 10 to 11 years, and an amplitude of the order 3 to 5 ft. Since 1900, peaks were attained in monthly average elevations during 1907-08, 1913, 1917-18, 1929, and 1943. The cycles are irregular, with secondary peaks and troughs. The cause of the cycles is not fully understood. Presumably they depend upon ground water and evaporation conditions, inasmuch as no major streams enter the lake.

Seasonal changes in lake level are shown by higher levels in the summer and lower in winter. The annual range in elevation is 0.5 to 2.5 ft. In 1949 the annual range was 1.7 ft.

The principal effect of changing lake levels is its control of shore erosion. During high stages of the major cycle, storm waves are able to undercut the bluffs, and the severity of the erosion is a function of combinations of high levels and larger-than-average storms.

Lake Michigan displays no visible tidal effects, and it is estimated that the tide is about 2 in. Irregular shorter changes in level, called seiches, occur due to variations in barometric pressure over the lake.

Sand movement. The net movement of sand along the western shore of Lake Michigan is southward. In the lake's natural state, sand derived from the bluffs in the stretch from the foot of the peninsula at Green Bay (see Fig. 1) to Chicago was slowly carried southward, and accumulated in the natural trap at the south end of the lake to form the large belt of Indiana dunes.

Seasonally there is a slight reverse in sand movement, owing to the prevalence of southeast waves during summers. This reverse movement is much less significant than the southward movement associated with fall, winter, and early spring currents moving southward.

Estimates of sand drift along the shores were made at several localities by the Beach Erosion Board in cooperation with local or state agencies. In general, it was found that the drift is lean compared with usual ocean standards. Maximum amounts are of the order of 40,000 cubic yards per year, with averages of the order of 5,000 to 10,000 cubic yards. Detailed studies show that the shore drift varies considerably along the western shore of the lake, depending upon configuration of the shore, nature of bluff material, presence of bulkheads or groins, and harbor structures.

An important feature affecting the amount of shore drift is the availability of unconsolidated materials for erosion. Surveys show that the average natural rate of erosion for the stony-clay bluffs near Milwaukee was about 2 ft. per year. It is estimated that this would feed about 35,000 cubic yards of debris into the lake per mile per year, of which 20 percent is sand and pebbles, and the remainder silt and clay. Along the bluffs farther south, the natural rate of erosion may have been 3 ft. per year, with local rates as high as 10 ft. per year along certain low sand banks.

Present-day protective structures along the lake have probably reduced the effective erosion to one-fourth or less of the natural rate. To a large degree the intervals of erosion are now confined to storm seasons during times of high lake level. The period 1943-47 was characterized by high lake levels, and erosion was markedly greater than during the preceding decade.

ENGINEERING STRUCTURES

Numerous typical shore structures, including breakwaters, sea walls, bulkheads, revetments, jetties, and groins have been constructed along the lake shore. Of these, the writer is most familiar with groins as structures for creating or maintaining beaches.

The permeable groin, developed by S. M. Wood, had some of its earliest installations in the general stretch from Chicago to Milwaukee, and numerous examples may be seen. The groins are constructed of pre-cast concrete, and have an increasing permeability lakeward.

The most common type of groin has been the short impermeable wooden type, made of piles or sheeting. Wooden cribs, filled with broken stone, are also commonly used as large groins or jetties. More recently, steel sheet piling groins have come into extensive use, some with openings to make them partially permeable.

In areas where the drift is very lean, or where the shore is protected by sea walls or bulkheads, as along the lake front in Chicago, beaches have been constructed with imported sand. The structures include a large impermeable hooked concrete jetty at the south end of the system, with shorter wooden or steel groins spaced along the beach north of the main jetty. The sand is placed in the system, and slowly migrates southward until trapped by the hooked jetty. Periodically sand may be carried or pumped to the north end of the system to renew the cycle.

In a recent study of the Illinois shore line of Lake Michigan, the Beach Erosion Board recommended similar systems for other localities with lean drift. For more normal shore conditions the Board recommended impermeable groins with a horizontal portion at about berm level, and a 1 on 25 slope under water to the low water datum. The width of the horizontal portion depends upon the width of the beach, and the slope of the lakeward portion may be adjusted to stable beach slopes for the exposure and sand size.

CONCLUDING REMARKS

The purpose of this paper is to present the general topic of lake processes to an engineering audience from a geological point of view. It is apparent that the shores of larger lakes present many problems in common with the sea coast, but the absence of tides, and of first-rank storms, means that geological processes are less marked, and structures may in general be smaller.

In contrast to many coastal areas, the Great Lakes probably have less consolidated materials in their banks and bluffs, inasmuch as the basins lie mainly in glacial deposits. Hence rates of erosion may be much greater than on harder coastal rocks. The effect of long-period changes in level also introduce problems of selecting distances above and below lake datum in structures, to allow for the more shoreward wave action during times of high levels.

The much leaner shore drift along lakes as compared to the oceans also means that problems of beach development and maintenance may be more difficult to solve. Greater reliance on imported sand in closed systems seems to be the trend in some larger communities where the demand for recreational beaches is great.

It is interesting to note that in some respects lakes may be considered as models of the ocean. For the southern part of the western shore of Lake Michigan, it is the writer's impression that the scale factor (based on wave period and height) is of the order of 1/9 for the length dimension, and 1/3 for time. Much more data are needed from systematic wave and current measurements, to obtain a better basis for comparison. Further data on the relative behavior of standard shore engineering structures under lake and ocean conditions would also be very useful.

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CHAPTER 17

CONTRIBUTION OF HYDRAULIC MODELS TO COASTAL SEDIMENTATION STUDIES

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INTRODUCTION

The use of hydraulic models in coastal sedimentation studies dates back more than 60 years. To the best of the writer's knowledge, the first such model was of the tidal portion of the Mersey River in England, and was constructed and operated by Professor Osborne Reynolds in the year 1887. Data obtained from the model tests by Professor Reynolds were used in the location and design of navigation channels from the sea to the port of Liverpool.

Since the time of Professor Reynolds, numerous models have been used in this and other countries for studies of coastal sedimentation problems. The first such study made by the Waterways Experiment Station was of Winyah Bay, South Carolina in 1933, and was followed by studies of Ballona Creek outlet near Venice, California; Galveston Harbor, Texas; Absecon Inlet at Atlantic City, New Jersey; the Umpqua River entrance in Oregon; Lynnhaven Inlet near Norfolk, Virginia; and many others. This paper covers in a general way the types of coastal sedimentation problems subject to model analysis, the types of models used, model instruments and appurtenances, field data requirements, adjustment and verification of models, testing of proposed improvement plans, the analysis of test results, and the limitations of models.

GENERAL PROBLEMS SUBJECT TO MODEL ANALYSIS

Coastal sedimentation problems of primary importance to the Corps of Engineers are those pertaining to maintenance of navigation channels and to erosion or accretion of ocean beaches. Materials transported along the coasts by the combined action of waves, littoral currents, and tidal currents, and materials discharged into the oceans by rivers and estuaries shoal the entrances and access channels to the harbors of our nation. Such shoaling is very detrimental to navigation because of decreased channel depths; also, large amounts of public and private funds are expended each year for maintenance dredging of these channels. Model studies have contributed to the design of channels and appurtenant jetties, dikes, etc., for the reduction of shoaling in numerous cases. Beach erosion problems may be caused directly or made more serious by the construction of works designed to reduce shoaling of navigation channels. Such works are usually designed to halt or diminish the movement of material to a given area; therefore, shore areas downbeach from the improvement works may be eroded due to interruption or elimination of their normal supply of material. It is therefore necessary that studies be made of all proposed jetties, breakwaters, and other control works designed to reduce channel shoaling in an effort to determine their probable effects on beach erosion and accretion. Models for testing such improvement works usually reproduce a considerable length of the shore line on each side of the problem area so that the effects of the works on adjacent areas can be noted.

TYPES OF MODELS USED

Coastal sedimentation problems are usually studied by means of a movable-bed model, in which the bed of the model in the problem area is molded of sand, crushed coal, haydite, or other erodible materials. Fig. 1, which shows the layout of the Absecon Inlet model, will illustrate the general features of the movable-bed model. It will be noted that the inlet proper and the adjacent beaches for distances of four miles to the north and south of the inlet, and offshore to about the 30-ft. contour of depth in the Atlantic Ocean, were molded of sand. The remainder of the wetted portion of the model was molded of concrete to a common elevation. This area provided space for the pipe system used to repro-

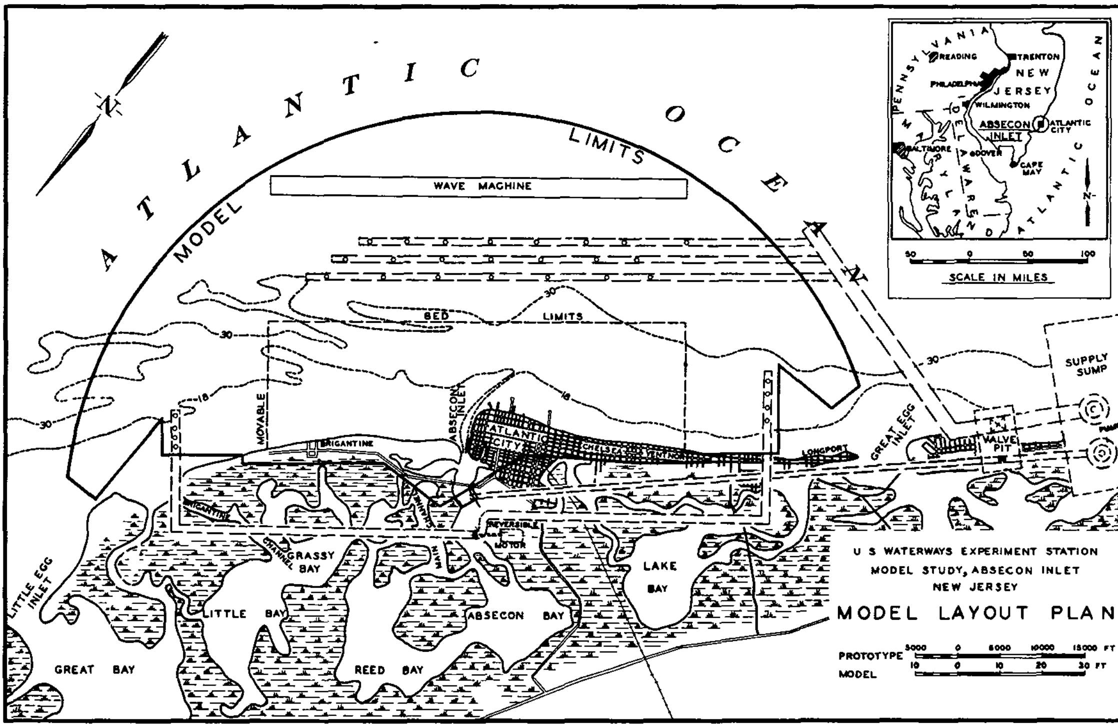
ATLANTIC OCEAN
MODEL LIMITS
WAVE MACHINE
BED LIMITS
MOVABLE
ABSECON INLET
ATLANTIC CITY
BRIGANTINE
CHELSEA
VENTNOR
LONGPORT
GREAT EGG INLET
VALVE PIT
SUPPLY SUMP
PUMP
REVERSIBLE MOTOR
GRASSY BAY
CHANNEL
MAIN
LITTLE EGG INLET
LITTLE BAY
ABSECON BAY
LAKE BAY
GREAT BAY
REED BAY
PENNSYLVANIA
READING
TRENTON
NEW JERSEY
PHILADELPHIA
WILMINGTON
ABSECON INLET
ATLANTIC CITY
BALTIMORE
DOVER
CAPE MAY
MARYLAND
DELAWARE
ATLANTIC OCEAN
SCALE IN MILES
U S WATERWAYS EXPERIMENT STATION
MODEL STUDY, ABSECON INLET
NEW JERSEY
MODEL LAYOUT PLAN
PROTOTYPE 5000 0 5000 10000 15000 FT
MODEL 10 0 10 20 30 FT

Fig. 1

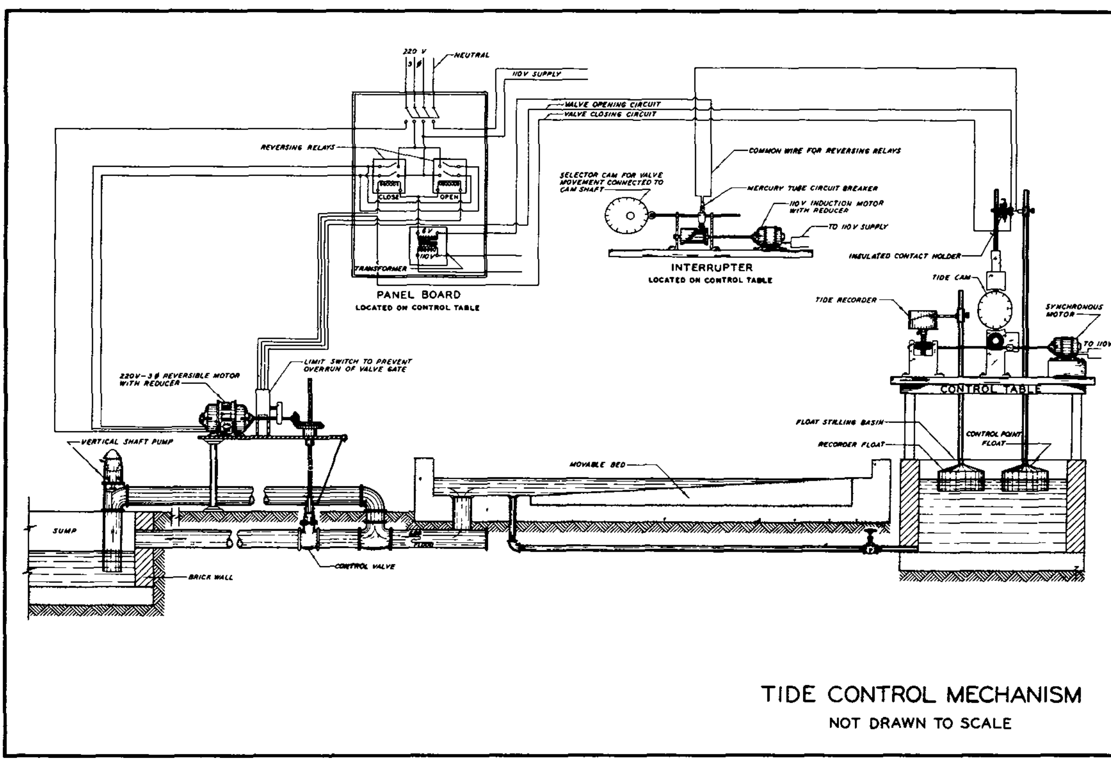
220 V
NEUTRAL
110 V SUPPLY
VALVE OPENING CIRCUIT
VALVE CLOSING CIRCUIT
REVERSING RELAYS
CLOSE
OPEN
TRANSFORMER
PANEL BOARD
LOCATED ON CONTROL TABLE
COMMON WIRE FOR REVERSING RELAYS
SELECTOR CAM FOR VALVE MOVEMENT CONNECTED TO CAM SHAFT
MERCURY TUBE CIRCUIT BREAKER
110 V INDUCTION MOTOR WITH REDUCER
TO 110 V SUPPLY
INTERRUPTER
LOCATED ON CONTROL TABLE
INSULATED CONTACT HOLDER
TIDE CAM
TIDE RECORDER
SYNCHRONOUS MOTOR
TO 110V
CONTROL TABLE
FLOAT STILLING BASIN
RECORDER FLOAT
CONTROL POINT FLOAT
MOVABLE BED
220V-3∅ REVERSIBLE MOTOR WITH REDUCER
LIMIT SWITCH TO PREVENT OVERRUN OF VALVE GATE
VERTICAL SHAFT PUMP
SUMP
BRICK WALL
CONTROL VALVE
TIDE CONTROL MECHANISM
NOT DRAWN TO SCALE

Fig. 2

duce the ocean tide, and served as a floor for the wave machine which was operated from a number of positions from south to east. The movable-bed portion of the model was molded by means of male templets suspended from permanent sounding rails, the templets being spaced not more than two feet apart so that accurate molding could be obtained.

INSTRUMENTS AND APPURTENANCES

Appurtenances usually required on movable-bed, coastal sedimentation models are tide-control mechanisms, wave machines, littoral-current reproducers, bed-load feeders, and an assortment of gages, current meters, sounding rods, and other minor equipment. One of the tide-control mechanisms in present use at the Waterways Experiment Station is illustrated schematically in Fig. 2. This mechanism consists of the following component parts: (a) a large header sloping from the model to a nearby water-supply sump; (b) a pump supplying through a separate line a constant flow from the sump into the header at a point near the model; (c) a motorized, rising-stem valve installed in the header a few feet toward the sump from the entrance of the pumpline into the header; and (d) an automatic control apparatus located within the model for regulating the operation of the valve by means of a system of floats and electric contacts. It is obvious that the complete closing of the valve will divert the full pump output into the model and produce a rapidly "flooding" tide; with the valve wide open, the full pump output plus gravity flow from the model will return to the sump and produce a rapidly "ebbing" tide. Thus, by means of intermediate valve openings, any desired rate of ebb or flood can be reproduced in the model. The automatic control apparatus for regulating the valve operations is equipped with a cam, cut to a polar plot of the tide to be reproduced, rotated by a synchronous motor at a speed determined by the computed model time scale. Riding vertically on this cam is a rod carrying a pair of electric contacts, one above the other, which rise and fall in accordance with the plotted tide curve. A third contact, placed between this pair of contacts with very slight clearances above and below, is attached to a rod supported by a float on the model water surface. Thus whenever the model water surface rises above or falls below its proper elevation at any time during the tidal cycle, the float forces the middle contact to close the circuit through the upper or lower contact, respectively. Closing of the upper circuit actuates the valve in an opening direction, which directs more of the pump output back to the sump and causes the model water surface to fall to its proper elevation; closing of the lower circuit reverses this operation, causing the model water surface to rise to its proper elevation. The tide thus reproduced can be controlled to an accuracy of approximately 0.001 ft. in the model. The control apparatus is equipped with a recording device which inks on a roll of paper a continuous record of the model tide curve, superimposed upon the prototype curve being reproduced. The prototype curve is inked by a pen riding on the plotted tide cam, while the model curve is superimposed by a pen riding on a float on the model water surface. This feature is of utmost importance during adjustment of the apparatus, and permits a visual check on the accuracy of the model tide reproduction at all times.

Various modifications of the above described apparatus are made in special cases. For example, if computations show that the discharge through the outflow valve would be so great as to require a very large valve, then separate automatic controls are installed on the inflow and outflow valves and operated in such manner that, during an ebbing tide in the model, the control on the inflow line progressively decreases the pumped supply as the control on the outflow line progressively increases the gravity flow to the supply sump. This procedure is reversed during the flooding tide, so that the two controls operate in balance at all times. The control of the inflow valve is usually fixed; that is, the valve operates through a predetermined cycle of movement at all times, while operation of the outflow valve is controlled by the contact point and float system previously described. Therefore, the outflow valve compensates for small variations in pump efficiency and other factors which are sure to occur over a period of months or even years of model operation.

Wave machines of the eccentric roller type, flap type, and plunger type have been used in coastal sedimentation models at the Waterways Experiment Station.

Since it is usually necessary to reproduce waves from several directions during the course of such a study, the plunger type machine has been found most suitable. Fig. 3 is a schematic diagram of one of the plunger type machines in present use.

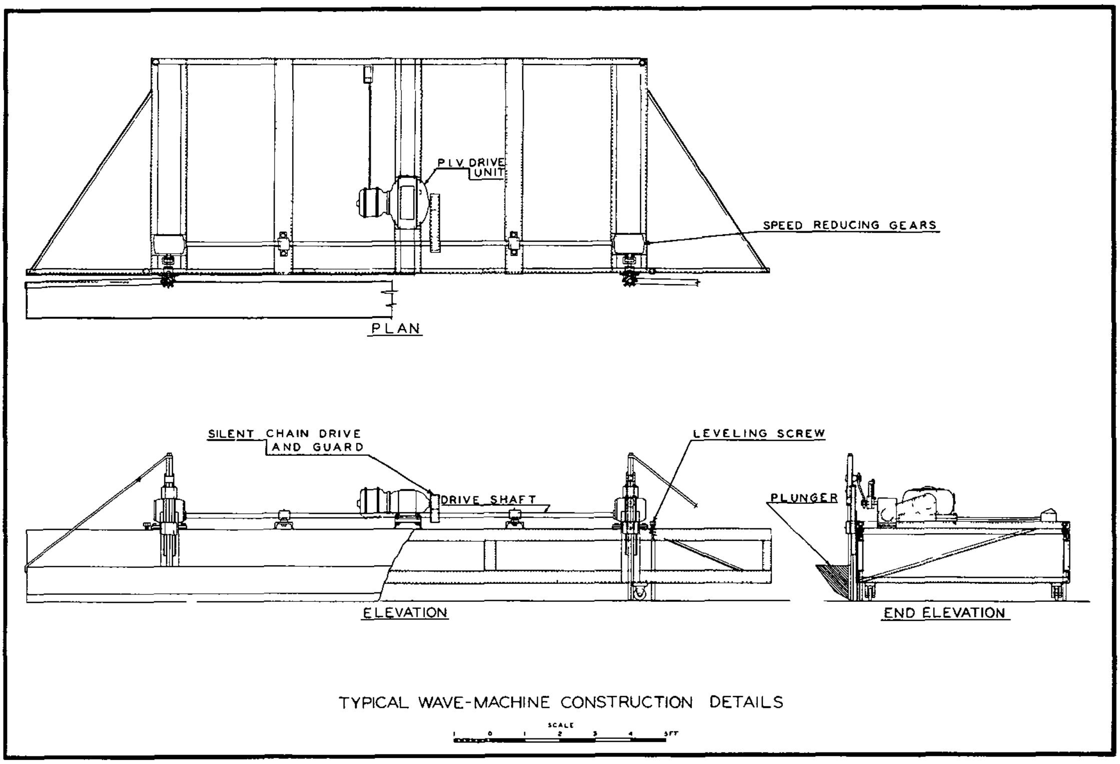

Fig. 3

This machine consists of a heavy angle-iron frame mounted on castors so that it can be moved readily. If the machine is more than about 50 ft. long, a small motor-driven winch is usually mounted on the frame and the machine is moved from place to place by means of a cable wound on a drum. An electric motor and link belt P.I.V. gear arrangement drives a number of special speed reducers, the number depending on the length of the machine, to which are connected slotted cams which are bolted to the risers from the plunger. The connection between the cams and plunger risers are adjustable so that the submergence of the plunger can be set as desired. With the above arrangement, the speed, length of stroke, and submergence of the plunger can be easily adjusted so that the plunger will reproduce the characteristics of the desired waves.

Coastal sedimentation problems are usually affected to some extent by littoral or alongshore currents. Regardless of the origin or cause of such currents, it is always necessary to reproduce them in coastal sedimentation models since they have a direct effect on the transportation of materials. Since such currents usually change direction in nature, it is necessary to provide for flow in either direction in the model. Fig. 1 illustrates the usual method for reproducing littoral currents in a coastal sedimentation model. Bays are constructed at both ends of the model, beyond the limits of the movable-bed section, and are connected by a pipe in which is installed a reversible pump. With the pump operating in one direction, water is removed from one end of the model and introduced into the opposite end, thus setting up a current from one end of the model to the other. A valve is installed in the pipeline at a convenient location so that the pump discharge, and therefore the velocity of the littoral current, can be controlled. Vanes are usually installed at the two bays so that the direction of the current can be set properly as it leaves the bays. In some coastal sedimentation problems the alongshore currents are tidal and therefore of a regular pattern, reversing in direction about every six hours and passing through a regular cycle from slack to maximum and back to slack. In such instances it is necessary to install an automatic control on the valve which will vary the discharge in such manner that the cycle of current velocities will be reproduced accurately. A pair of cam-actuated electrical switches appurtenant to the automatic control reverses the direction of the pump at the proper times.

Inasmuch as the movable-bed section of a coastal sedimentation model represents only a relatively short section of a shore, it is obvious that material must be fed artificially at the end of the movable bed from which the drift of material is moving. The amount of material to be introduced per unit of time is determined by oversvations and estimates of the amount passing the comparable point in the prototype. The material is usually added to the model by an endless belt driven by an electric motor; the material is spread evenly on the belt and is dropped into the model by rotation of the belt in accordance with a predetermined rate of introduction. Material moving out of the movable-bed section at the opposite end is carefully picked up and measured so that a constant check is kept on the rate of littoral drift of material through the model.

FIELD DATA REQUIREMENTS

The degree of accuracy attained in a model study of coastal sedimentation problems depends to a large extent on the availability of accurate and adequate field data for its design, construction, and operation. It goes without saying that a movable-bed model, adjusted on the basis of inaccurate field data, will unquestionably furnish inaccurate results during subsequent tests of proposed improvement works. It is also imperative that adequate field data be available for a proper analysis of the problem before the selection of model scales and design of the model and appurtenances are undertaken.

As will be explained more fully later in this discussion, the proper adjustment of bed movement is of principal importance in the coastal sedimentation model. It is necessary, therefore, that a sufficient number of periodic hydrographic surveys of the prototype be available to determine progressive changes in hydrography over a considerable period of time. Otherwise, the adjustment of bed movement in the model may be based on a period during which the trend of hydrographic changes is not in accordance with the long-time trend.

Once the trends and magnitudes of hydrographic changes have been established from studies of periodic surveys, it is necessary to determine what hydraulic forces are responsible for such changes and the relative importance of each of the forces involved. This information can usually be obtained from a comprehensive study of the tides, tidal currents, littoral currents, and waves in the problem area. Such studies should cover a considerable period of record so that inaccurate conclusions will not be reached because of inadequate records. It is believed that such investigations should cover at least one year of record.

One very important factor, and probably the most difficult to obtain, is the determination of the volume of material moving through the area under investigation as littoral drift. It is entirely possible that a certain section of the coast will remain stable for a period of years while hundreds of cubic yards of material per day are moving through the area from an unstable reach on one side to an unstable reach on the other. Periodic hydrograph surveys of the area would indicate that no changes were taking place; however, if an entrance channel were dredged through such an area it would shoal very rapidly. While it is practically impossible to determine the rate of littoral drift accurately except in isolated cases, some indications can be obtained from examinations of erosion or accretion characteristics of the shore line and offshore bars for considerable distances in each direction, dredging records of nearby channels, and erosion and accretion adjacent to jetties, groins, and other structures in the vicinity.

VERIFICATION OF MODELS

It is obvious that the worth of any coastal sedimentation model study is wholly dependent upon the ability of the model to produce with a reasonable degree of accuracy the results which can be expected to occur in the prototype under given conditions. It is essential, therefore, before any model tests are undertaken of proposed plans of improvement with a view to predetermining their effects in the prototype, that the required similitude first be established between the model and prototype and that all scale relationships between the two be determined.

In the case of fixed-bed model studies involving the investigation of purely hydraulic problems, the existence of hydraulic similitude with the prototype can be shown and all scale relationships determined by mathematical means. However, in the case of a problem requiring the use of a movable-bed model to study the movement of bed material under the influence of hydraulic forces, what is commonly known as hydraulic similitude with the prototype sometimes is not compatible with the all-important similitude with respect to the phenomenon of bed movement. This is brought about by the fact that the bed material used in the model fails to conform to the linear scale ratios which govern all other model dimensions. It therefore becomes necessary in such a model to depart somewhat from hydraulic similitude by adjusting the various hydraulic forces so that their effects upon the model bed material will be similar to the effects of the corresponding prototype forces upon the prototype bed material.

It is evident, therefore, that the attainment of complete hydraulic similitude between a movable-bed model and its prototype, although desirable, is not necessary. Instead, a proper analysis of such a problem requires that scale relationships be carefully adjusted to provide a high order of similitude with respect to the effects of the various hydraulic forces upon the scour, travel, and deposition of bed and beach material, which is the essence of such a study. However, the obtaining of this required similitude and the establishing of the model-to-prototype scale relationships involved are altogether impossible by mathematical means, and can be accomplished only through the empirical process of verification.

The verification of a movable-bed model is an intricate cut-and-try process of progressively adjusting the various hydraulic forces and the model operating technique until the model will accurately reproduce hydrographic changes which are known to have occurred in the prototype between certain dates. In this manner the accuracy of the functioning of the model is established and certain of the scale relationships with the prototype are determined. The verification process usually consists of the following steps: (a) two prototype surveys of past dates are selected -- the time between these dates being known as the "verification period" -- and the movable bed of the model is molded to conform to the earlier survey; (b) the hydraulic phenomena which occurred in the prototype during the verification period are simulated in the model to the proper time scale, all regulative measures undertaken in nature during that period being reproduced in the model at their proper times; and (c) the movable bed of the model is surveyed at the end of this period, and the model is considered to be satisfactorily verified only when this survey is an accurate reproduction of the later prototype survey.

However, in operating a model subject to such a variety of forces as confronts the experimenter in coastal sedimentation studies, it is absolutely necessary to abide by the principle that departure from scale simulations of the several forces should not be regarded lightly. The over-all effect of the forces acting is a resultant of the various component forces. And, as previously stated, the verification of the model is dependent upon its similitude with respect to the effects of the various hydraulic forces. It can be readily understood that the results of the introduction of some improvement plan, such as a jetty, which would alter the hydraulic regimen of an area, may be erroneous if the component forces acting in the model are definitely dissimilar to those acting in nature. Therefore, every effort should be made during the verification process to keep the model forces as undistorted as possible.

It was mentioned previously that the verification process of a movable-bed model established one of the model-to-prototype scale relationships, and this is the all-important scale relationship with respect to bed movement. Tides, currents, and waves are reproduced in the model in accordance with the time scale computed from the linear scale relationship; however, this time scale bears no relationship whatsoever to the time required for the model to reproduce a change in bed configuration which occurred in a known period of time in the prototype. Let us assume that the verification period selected for adjustment of a movable-bed model covers a three-year period in the prototype, and that the model is adjusted so that all changes in bed configurations known to have occurred during the period are reproduced accurately in the model in an actual operating time of 36 hours. It is therefore evident that the time scale for bed movement, model to prototype, is 12 hours to one year or one hour to one month.

TESTING OF IMPROVEMENT PLANS

After the verification procedure of the coastal sedimentation model has demonstrated the ability of the model to reproduce with a reasonable degree of accuracy the bed configurations which are known to have occurred in the prototype under corresponding conditions, the model is ready for tests of existing conditions and proposed improvement plans. One fixed testing procedure -- that developed during the verification phase of the study -- is followed during all such tests; the only variations between the subsequent tests being in the lengths of the tests and in the features of the proposed plans of improvement.

The first test made following a satisfactory verification of the model is a base test or test of existing conditions. The purpose of the base test is twofold: (a) to provide all possible data as to the accuracy and dependability of the functioning of the model over a relatively long period of time (the base test usually covers a period of time equivalent to five to ten years in the prototype, as dictated by the time scale for bed movement); and (b) to provide data which will serve as a basis for the comparison of results of subsequent tests in which proposed regulative works are simulated (a comparison of base test results with results of a test of regulative works makes it possible to isolate the effects of the regulative works in the model). Tests of proposed regulative works are made in exactly the same manner and for exactly the same length of time as the base test, the only difference being that the regulative measures of each plan are installed in the model at the beginning of the test of that particular plan of improvement. The movable-bed portion of the model is surveyed in detail at the end of each test of a proposed plan of improvement, and the survey is compared to that made at the end of the base test to determine the effects of the plan on the scour and deposition of bed material throughout the model. In addition, hydraulic measurements of tidal heights, current velocities, and current directions as affected by the plan, and volumetric measurements of the effects of the plan on movement of material as alongshore or littoral drift are made.

ANALYSIS OF TEST RESULTS

Direct comparisons are made between hydrographic surveys obtained at the end of each test of a proposed improvement plan and that made at the end of the base test, and comparative maps showing plus and minus changes in bed configurations as effected by each plan are made. These maps illustrate very clearly the effects of the various plans on the movable-bed section of the model, and they may be used for computations of scour and fill in critical areas if considered necessary. Comparisons of the volume of material moving out of the movable-bed portion of the model in the direction of the littoral drift during the base test with comparable volumes during tests of proposed improvement works will indicate the effects of each plan on the passage of material through the area under investigation. These data are significant in the case of tests of jetties or other coastal improvement works which might cause deterioration of downcoast beaches by decreasing their normal supply of material.

Measurements of tidal heights, current directions, and current velocities for each plan tested are compared directly to base test measurements. Such comparisons will indicate the effects of each plan on the hydraulic characteristics of the area under investigation, and are of particular significance if navigation is of importance to the problem under study. For example, a proposed jetty at a tidal inlet might indicate the desired effects on bed configurations but might also create such undesirable hydraulic conditions that it will be dropped from further consideration.

MODEL LIMITATIONS AND CONCLUSIONS

It has been emphasized during this discussion that the adjustment and verification of the coastal sedimentation model, and hence the accuracy of results to be obtained therefrom, are based upon data obtained from comprehensive prototype investigations. The completeness and accuracy of such prototype studies are most essential, since the model study would unquestionably produce erroneous results if its adjustment and verification were based upon inaccurate or incomplete field data.

In the above connection, model-prototype confirmation studies are of inestimable value to the engineer who works with coastal sedimentation models. Following the installation of an improvement plan in the prototype as a result of such a study, the question immediately arises as to how closely the functioning of the plan in nature corresponds to model predictions. Where inconsistencies are revealed through such a confirmation study, model operating techniques may be improved to the end of eliminating such inconsistencies in the future. Model-prototype confirmation studies are believed to be of such importance to the further development of coastal sedimentation model technique -- and thus to the solution of the problems involved -- that plans for a confirmation study should be included as a part of each comprehensive improvement plan that has involved study on a model.

It is apparent that the coastal sedimentation model has certain limitations, imposed largely by the characteristics of the available model bed material, the adequacy of prototype data, the distortions inherent in small-scale models used for such purposes, and the uncertainties that still exist as to the mechanics of sediment movements in both model and prototype systems. These limitations are considered to be far out-weighed, however, by the many advantages gained through the relatively inexpensive and positive expedient of model analysis as contrasted to the prohibitive and tremendous cost, effort and hazard that would otherwise be involved in achieving similar solutions by trial and error in the field.

It is to be understood that the hydraulic model is not proposed as a substitute for analytical design; nor can its use eliminate the need for extensive field investigations. The three are mutually supporting. The need for experimentation is basically fostered by the fact that present-day knowledge of coastal sedimentation phenomena has not advanced to the point where such problems can be resolved by theory alone. Until that point is reached the hydraulic model will continue to be a most useful expedient in providing information not obtainable by other means. Thus, in its usefulness to the engineer responsible for the solution of coastal sedimentation problems, the hydraulic model occupies a position of great importance lying somewhere between the provinces of abstract theory and rule-of-thumb field engineering.

CHAPTER 18

DREDGING AT INLETS ON SANDY COASTS

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Dredging in navigation channels across off-shore bars and in tidal inlets or estuaries presents special problems not encountered in dredging in more protected inland waters. The greater exposure to wave action and often-swift and complicated reversing tidal currents impose an increased hazard of loss of or damage to dredging plant, more lost working time due to adverse conditons, increased cost of insurance, and other factors that tend to add to the difficulty and cost of such work. These special hazards require special countermeasures in the choice of a dredge, the conduct of the work, and the disposal of the dredged material. A knowledge of these factors is important to those engaged in coastal engineering.

In the United States, the choice of a dredge usually lies between the hydraulic pipe-line type, some variant of the bucket or dipper type, and the seagoing hopper type. The choice is determined by the physical characteristics of the inlet, the tidal range, the nature of the material to be removed, the exposure, the weather conditions to be expected, and other characteristics of the site.

The hydraulic pipe-line dredge consists basically of a square-end, flat-bottom, scow-type hull, in which are mounted a powerful centrifugal pump with its driving and auxiliary machinery. At one end of the hull is the "ladder," hinged at the hull end, and carrying the intake pipe and the cutter shaft. The forward end of the ladder is raised or lowered through an A-frame at the forward end of the hull. At the aft end of the hull, the discharge line from the pump emerges and is carried to the disposal area or to shore on a series of pontoons. Two vertical timber or steel spuds are also supported on a frame at that end; these can be raised or lowered independently. In operation, one spud is lowered to engage the bottom and hold the dredge in place, the ladder is lowered to the bottom, the revolving cutter stirs the material into suspension, the pump draws the water and suspended material up through the intake pipe and forces it out through the discharge line to the dumping area. The ladder end of the dredge is swung back and forth while pumping by a "swinging cable" passing around a drum in the operating machinery and extending thence laterally on each side of the dredge to a heavy swinging anchor. The dredge progresses forward by swinging alternately around first one and then the other spud. The dredge usually has no propelling machinery, and must be moved by towboat.

The bucket- or dipper-type dredge is similar to the hydraulic pipe-line except that it has no pump and no intake or discharge lines. Instead of the ladder it has a hinged boom, on which is mounted a controlled bucket or dipper of one of several different types. The material dug must be deposited either within the reach of the boom or, more commonly in open water, in scows for removal to a more remote dumping area.

The seagoing hopper dredge has a ship's hull, with propelling as well as pumping machinery. The material may be picked up, while the dredge moves slowly along the dredging range, by one suction pipe in a well amidships or by two suction pipes, one on each side of the ship. The pumps discharge into hoppers or bins amidships; the heavy material settles out in the hoppers while the water passes overboard through scuppers at the tops of the hoppers. When a load has been pumped, the pumps are cut off, the section lines are raised, and the dredge proceeds to the dump. Gates in the bottoms of the hoppers are then opened and the load slides through them into the dumping area. The gates are then closed and the operation is repeated.

The hydraulic pipe-line dredge has the following advantages:

a. It draws less water than the hopper dredge.

b. It operates continuously while at work.

c. It disposes of its material nearby -- to replenish beaches, create bases for jetties, provide shore fill to create real estate, eliminate mosquitoes, or for other useful purposes.

d. It is, under favorable conditions, the least costly to operate.

It has the following disadvantages:

a. Its low freeboard.

b. Its floating pipe line, swinging cables, ladder, and spuds are easily damaged by waves or currents.

c. Its comparative inability to remove rock unless well blasted.

d. Its relative immobility and lack of maneuverability.

The dipper-type dredge has the advantages of the hydraulic pipe-line dredge as to shallow draft and continuous operation while at work, and the additional advantage that it can remove rock with comparatively less blasting or, in soft or loose rock, with no blasting. It has the disadvantages of the hydraulic pipe-line dredge as to low freeboard, easily damaged boom and spuds, and relative immobility; it must, in addition, normally load its material into scows for movement to the dump. In soft material its output is lower than that of a comparable hydraulic pipe-line dredge, but in hard material it may be able to remove material more cheaply than could a hydraulic pipe-line dredge with complete blasting.

The seagoing hopper dredge has the sole advantage that it is self-propelled and seaworthy in any reasonable weather, and, therefore, more self-reliant. It can work in waves and currents that would compel either of the other types to withdraw, and it can reach shelter more quickly in case of sudden squalls or storms. It has the disadvantage of greater draft, so that it can only work in a minimum water depth greater than that needed by the other two types; it requires a turning basin of adequate depth and area at the inner end of its work, and it loses dredging time while en route to and from the dump. A theoretical disadvantage is that the only hopper dredges in this country are owned by the United States, but this is counteracted by the fact that the United States is about the only agency now interested in dredging inlets. In any event, if local authorities (such as a municipality, port authority, etc.) require dredging in an inlet or harbor entrance that can only be done practically by a hopper dredge, it is possible to arrange that the work be done by a United States dredge, the money to cover the cost being provided by the local agency. This was recently done in Pascagoula Harbor, Mississippi, when the Federal project harbor 25 ft. deep on the bar and 22 ft. deep across Mississippi Sound was deepened by the United States to 35 ft. on the bar and 30 ft. across Mississippi Sound at the request of local interests and with $750,000 advanced by the Port Authority and the State to cover the cost. The 35 ft. depth in the entrance bar channel was so provided with the United States 3,000-cubic-yard hopper dredge Langfitt, which removed 790,168 cubic yards of soft material at a unit cost of $0.1167 a cubic yard.

All things considered, the Corps of Engineers prefers to do the dredging in exposed offshore-bar and entrance channels with seagoing hopper dredges whenever feasible. Two principal conditions limit this practice; (1) the material to be removed must be soft and (2) there must be adequate initial depth in the channel and the inside turning area to accommodate the dredge. When these conditions exist, it is often advantageous to assign some part, at least, of the inner-harbor channel also to be dredged by the hopper dredge to keep it busy whenever the outside channel is too rough for work.

The hopper dredges now in use by the Corps of Engineers range upward from the "Lyman" class, drawing 13 ft. fully loaded with 700 cubic yards, to the "Essayons," an 8,000-cubic-yard giant drawing 27-1/2 ft. loaded. The least depth in which a hopper dredge can work at full efficiency is, therefore, about 14 ft. However, dredges of the "Lyman" class draw 11 ft.-3 in. when carrying 300 cubic yards; it is, therefore, possible with these dredges to start a cut in a depth of 12 ft. by dredging less than a half load at a time. Furthermore, if the channel to be dredged is subject to a considerable tidal range, a pilot cut can be started on a depth of about 12 ft. at high water only, increasing the load and the duration of the operation as the pilot cut is deepened until the full depth of 14 ft. or more at low water is secured, when the channel is widened to its project width on full-time operation. In view of this limitation on the use of hopper dredges, the Corps of Engineers now usually proposes that the entrance channel to a small-boat harbor, even when the harbor depth is less than 14 ft., be planned for a least depth of 14 ft. when it is so exposed that use of a small hopper dredge for maintenance dredging is indicated.

When a hopper dredge is used, the dredged material is usually disposed of by dumping in deep water offshore. In many cases, when the distance to a naturally deep dumping area is so great as to make that method uneconomical, artificial dumping basins in protected water closer to the work are dredged with a hydraulic pipe-line dredge, which then removes the material dumped into the basin by the hopper dredges and pumps it ashore or into other areas of the waterway. This method has been used in the Delaware River and Tampa Harbor.

In at least two cases, hopper dredges have been successfully used to remove rock. In 1927-28, the entrance channel to Miami Harbor, Florida, was deepened to then-project depth of 25 ft. by removing some 900,000 cubic yards of material, about 30 percent of which was limestone. The rock was first surface or "doby" blasted; bundles of 5 or 6 pounds of 60 percent dynamite were tied to a heavy cord about 5 ft. apart; the cord was then laid back and forth across the area to be blasted; one detonator was exploded, and the concussion in the water exploded the rest of the string. The rock so shattered was then removed by the U.S. hopper dredges W. L. Marshall and Dan C. Kingman. A total of 217,650 pounds of dynamite was used at a cost of $62,325.37. The average cost of blasting per cubic yard of total material removed was 6.9 cents. The total unit cost of the work was 23.6 cents a cubic yard. The same method has been used in new-work dredging in Tampa Harbor. Successful use of this method requires a considerable depth of water to hold the shattering effect of the explosive down against the rock. In any work involving blasting, complaints of damage to shore structures are to be expected; whenever blasting is involved, it should be cautiously done, to the end that such complaints may be proved to be unfounded.

Under favorable circumstances, hopper dredges have also been used to remove material from entrance channels by agitation dredging. The material to be removed by this method must be soft silt or mud which will settle out of suspension only slowly; a second prerequisite is that the ebb current must be strong enough and so directed that it will carry most of the suspended material out of the channel to settle out in another area where it will do no harm. In the lower reaches of Savannah Harbor, Georgia, and in the outer-bar section of the entrance channel to Wilmington Harbor, North Carolina, hopper dredges have been used to pump such materials during the ebb flow through the bins and overboard through the scuppers, allowing the current to carry most of it in suspension out of the channel alinement before it settles to the bottom. During flood flows the dredges removed their loads to offshore dumping areas in the usual way.

When, due to lack of sufficient operating depth or other compelling considerations, inlet or bar dredging cannot be done by a hopper dredge, recourse must be had to the hydraulic pipe-line or the dipper type, despite their disadvantages. In such case, every possible precaution should be taken to minimize the hazards attending such operation. Such precautions commonly include the following:

a. The work should be scheduled for a season in which the maximum of favorable weather conditions is to be expected.

b. The dredge should be large, with as high a freeboard as possible, so that it will be affected as little as may be by ordinary wave action.

c. It should have a high dredging capacity, to minimize the time during which it will be exposed to the hazards of such work.

d. It should, whenever practicable, be used only to open up the minimum cut as to depth and width required to give access to a hopper dredge to complete the work to full project dimensions.

e. Before engaging in the inlet channel dredging, a safe haven should be provided into which the dredge and attendant plant can be quickly moved in case of danger.

f. The dredge should work from the shoreward waterway seaward, care being taken to keep the channel behind it open for a quick retreat to safety if bad weather threatens. Rapid shoaling of the channel behind the dredge has on occasion been allowed to threaten to close that avenue of escape.

g. Ample towboat service should be available at all times to move the dredge and its pipe line and attendant plant to safety on short notice.

h. Special care should be taken to emphasize and enforce all safety regulations on the dredge, pipe line, and attendant plant, and to maintain all facilities in readiness at all times for an emergency move.

i. If the dredging involves the creation of a new inlet channel to be protected by jetties, the jetting should if practicable be provided before the dredging is done, in order that the dredge may avail itself of their protection during its work.

When the dredging involves making a new cut through a barrier beach from a coastal waterway into the ocean, certain precautions should be observed in the final opening of the seaward end of the cut, particularly when the tidal range is considerable. It is good practice to carry the cut as close as possible to the high-water line on the beach before cutting through, and to provide some excess depth in the section of cut immediately back of the beach. The break-through is best made when the tide on the ocean side has turned to flood, and just as the water levels on the beach and in the cut become about equal. In this way the current through the newly made opening is at a minimum when the cut is opened; whatever current develops as the tide rises will set in rather than out, tending to carry any items of plant that may break loose inward instead of seaward. If, as is often the case, high velocities develop in the opening, with rapid erosion of the barrier plug, the material carried inward will settle out in the excess depth provided before cutting through, with a minimum of danger that it will shoal the channel sufficiently to obstruct withdrawal of the dredge and attendant plant in case of need. Such shoaling should be carefully watched, and the dredge moved to safety if its escape seems to be threatened.

In many inlets, active movement of beach and bottom material is to be expected, with rapid erosion at some points and shoaling in others. Such inlets are usually characterized by a crescent-shaped offshore bar, over which the natural shallow channel tends to shift from one alinement to another. When a dredged channel traverses such an inlet and bar, it must be expected that, unless jetties be provided, rapid shoaling will take place in some parts of the channel. It is usual to forestall frequent maintenance dredging of such a channel by providing, at the critical points, overdepth and overwidth in which shoaling material may accumulate for some time before encroaching on the project depth and width. A careful study of the given case, and an intelligent distribution of such "advance maintenance" dredging, may materially reduce the frequency and cost of maintaining the navigable depth and alinement of the channel.

In dredging in inlets, special consideration should be given to the disposal of the dredged material. Prior to undertaking the dredging, a careful study of the conditions should be made; the disposal of the spoil should be so planned in advance as to result in the maximum of advantage and the minimum of disadvantage. When the work is to be done with a hopper dredge, the spoil will be deposited either in deep water offshore or in deep holes in the inside waterway; the temptation to use the latter method to save a longer run to an ocean dump should usually be resisted, since the very existence of such a deep hole is usually prima facie evidence of current activity that will probably remove the dumped material and redistribute it elsewhere in the waterway -- probably largely in the channel from which it was originally removed. If the run to an offshore dump is uneconomically long, it will usually be preferable to dredge a dumping area in shallow inside waters with a hydraulic pipe-line dredge, rather than to use a naturally deep hole.

If a pipe-line dredge is used, the safest place for the dredged material is ashore, to fill in marsh areas, replenish an eroding beach, build out land for useful purposes, or as otherwise indicated by the conditions in the individual case. Care should be exercised to place the material far enough from the edges of the cut so that subsequent widening of the cut by wave and current action will not reach the deposit. If a predominant littoral drift exists along the shore, dredged material can often profitably be placed on or near the beach on the downdrift side of the inlet, so that it will be carried away from the inlet along the beach; this is especially true when the inlet entrance is jettied and when active erosion of the downdrift beach is in progress or may be expected. If the material must be deposited in inside waters, it should be placed in such amounts and on such alinements as will at least not adversely affect the direction and intensity of the tidal currents, and, if possible, will favorably affect them. The carrying capacity of the tidal waterway should not be so reduced as to diminish the tidal range and prism of the waterway landward of the work. The complexity of the hydraulics of most tidal inlets is such that, if the material is not to be simply and safely deposited ashore, a major study of the place and manner of its disposal is essential if unexpected damage to the waterway is not to be caused.

Many of the special aspects of dredging in tidal inlets are illustrated by some work done by the Corps of Engineers for the Navy at Ponce de Leon Inlet, Florida, during the recent war. In its natural state, Ponce de Leon Inlet is a fairly stable opening through a barrier beach near New Smyrna, on the east coast of Florida, through which Halifax River from the north and Hillsborough River from the south empty into the ocean. At its seaward entrance is the usual crescent-shaped offshore bar, over which mean-low-water depths of 1 to 5 feet prevail. Prior to the dredging, the least width of the inlet between low-water lines was about 1,800 ft.; in the north one-third of that width, a natural channel up to 37 ft. deep and 450 ft. wide between 5 ft. depth contours extended seaward; the south two-third was generally less than 3 ft. deep. The natural channel crossed the offshore bar generally perpendicularly to the shore, widening gradually to a maximum of 1,000 ft., and shoaling progressively to depths of 3.5 to 5.0 ft. over the crest of the bar about 1/2 mile offshore. At that point the channel angled sharply to the southeast, evidencing the fact that at that point the heavy preponderant southward littoral drift crossed the channel from north to south.

Along the alinement of that natural channel, a channel 20 ft. deep (14 ft. plus 6 ft. of overdepth), 200 ft. wide, and about 6,000 ft. long was to be dredged to the 20-ft. depth contour on the seaward face of the bar. A hopper dredge could not be used because of inadequate depth over the shoal; the 22-in. hydraulic pipeline dredge "General," owned by the Arundel Corporation, was leased for the work. The dredge began work at the shore end of the inlet channel on May 15, 1943; with generally favorable weather the dredge reached the crest of the bar on June 11. The weather then changed, and it was necessary to pull the dredge back into the inside waterway, where it worked until July 1. It was then moved back to the outer bar; on July 2 the swinging cable parted, and it was too rough to make repairs. On July 3 the dredge was again pulled in to safety; the company then refused to do any further work on the bar, and the contract was terminated on July 7, leaving the outer end of the work unfinished, with a least depth of 8 to 9 ft. over the crest of the bar. The total amount of material removed was 522,000 cubic yards, all soft.

The dredged material was placed in two areas. Part of it was placed off the north land point of the inlet to form a dredged mole parallel to the channel and extending about 3,800 ft. seaward from the original high-water shore line. The rest was placed off the south land point so as to extend it about 1,100 ft. from the high-water line into the inlet, reducing the width of the inlet by nearly one-half.

The material placed in the mole north of and parallel to the channel was straightway carried back southward by the littoral drift; within 6 months the mole had practically disappeared. Most of the material was carried back into the dredged channel, which shoaled rapidly from its north edge, especially in the section crossing the crest of the offshore bar. By May 1944, the crest of the bar had been restored to its original predredging elevation. The shoreward half of the channel shoaled generally less rapidly; after nearly 1 year it still retained depths of from 16 to 20 ft. instead of predredging depths of 8 to 10 ft. As the dredged channel shoaled along its north edge, the south edge was scoured, with the result that the deepest available channel at the end of the first year extended seaward from the gorge on an alinement about 6° to the southeast of the original dredged cut. About 330,000 cubic yards of material were deposited in the dredged channel cut within 1 year after the original dredging.

In summary, the following points are recommended for consideration by those responsible for dredging in inlets on sandy coasts:

a. If adequate initial depth is available and the material to be removed is soft, the work can most safely be done with a seagoing hopper dredge.

b. If a hydraulic pipe-line or a bucket dredge must be used, it should be carefully selected for its suitability to the work, every possible precaution should be taken to ensure its safety, and it should be used only to open up a pilot cut of sufficient depth and width to permit a seagoing hopper dredge to complete the dredging to full planned dimensions.

c. Dredged material should be so disposed as to produce a maximum of benefits and a minimum of adverse results. This will frequently involve a careful and intelligent study of all pertinent factors.

d. A carefully considered use of overwidth and overdepth advance-maintenance dredging can often considerably reduce the annual cost of maintaining an inlet channel.

CHAPTER 19

THE SEAGOING HOPPER DREDGE

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The scope of this paper is generally limited to a brief history of dredging, and to a description of the seagoing hopper dredge and its cycle of operations.

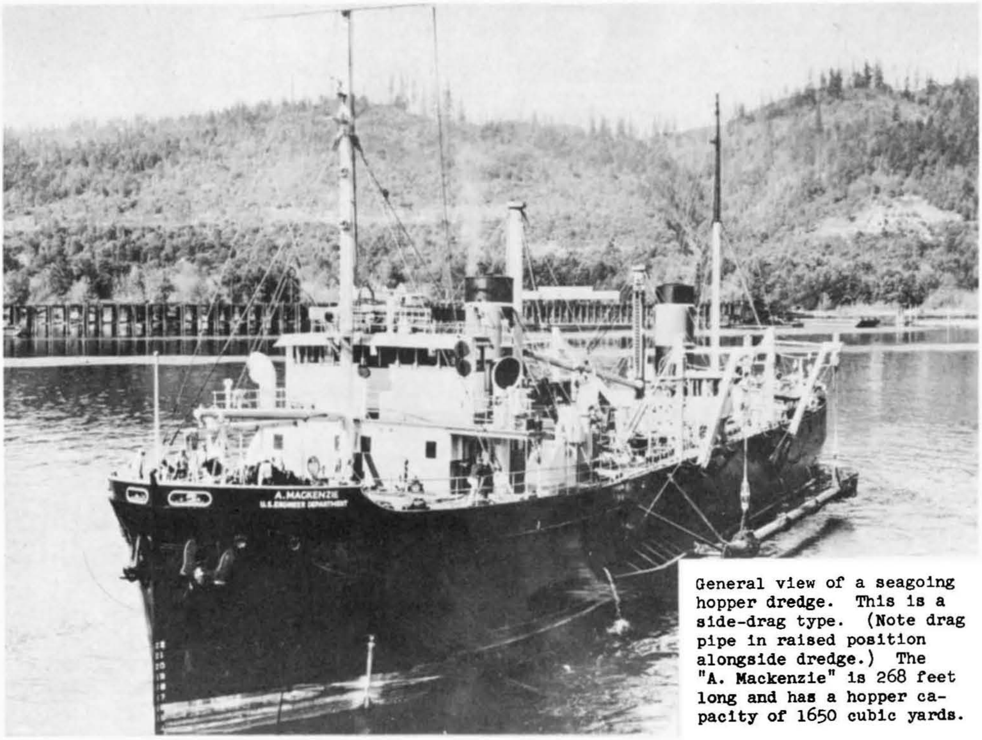
One reason for this limitation in scope is to avoid repetition of material presented by Mr. Berkeley Blackman in Chapter 18. In his paper, Mr. Blackman discusses the special problems of dredging at inlets; the types of plant used, together with their advantages and disadvantages; the methods necessary in making a cut through a sandy beach; and the precautions to be taken in disposing of materia from inlet dredging operation. He covers other particular phases of such dredging and concisely summarizes the subject with an illustrative example of the special aspects of dredging at tidal inlets and of the points which should be considered in the performance of the work. All but a few of the hopper dredges in the United States are operated by the Corps of Engineers. Therefore, this paper may be of interest to those not familiar with dredging operations of that agency.

Dredging is a special type of excavation involving the removal of material from under water. Several thousand years ago the Chinese and the Assyrians employed the primitive spoon-and-bag dredger to clean and maintain their canals. This apparatus consisted of a bag manipulated from a boat by means of a pole attached to a yoke or hoop at the mouth of the bag which was dragged along the canal bottom. When full, the bag was lifted and dumped into the boat. The method of disposal of the material is not clear, but it must be assumed that it was lifted ashore or dumped overboard, all probably by hand. Little improvement over this spoon-and-bag method of dredging was made until the advent of the industrial era in the eighteenth century. The method was then developed into a chain of buckets moving on a ladder.

Only during the past one hundred years has dredging been lifted to one of the most important methods of excavation known to engineers. During this period, hydraulic dredging was developed. Hydraulic dredging is the term applied to the method wherein material is conveyed by water through pipes as distinguished from the self-explanatory term of bucket dredging. The centrifugal pump is usually the prime mover of the mixture of material and water.

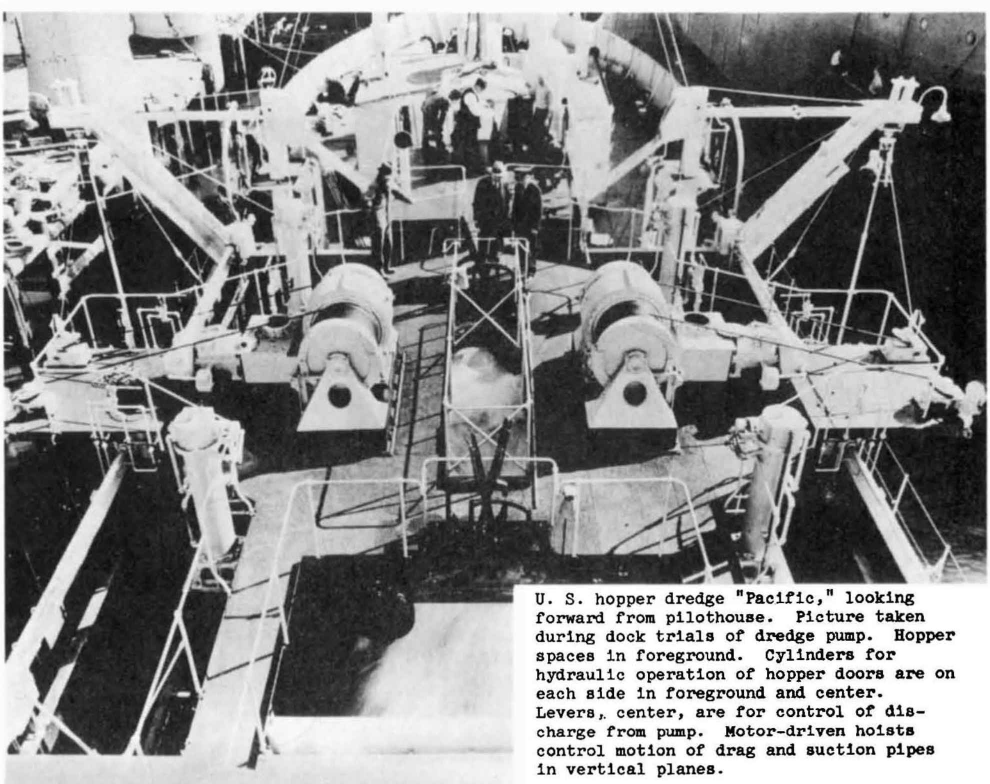
The two principal types of hydraulic dredging machines are the pipe-line dredge and the hopper dredge. In pipe-line dredges the material is discharged through floating and/or shore pipes to the disposal area, or, in rare instances, to barges which transport it to the disposal area. Hopper dredges, on the other hand carry the excavated material in bins, or hoppers, within the hold of the ship Another principal difference between the two types is that the hopper dredge is self-propelled while the pipe-line dredge is not and must be towed from job to job.

The modern seagoing hopper dredge is basically a self-propelled vessel with the molded hull of an ocean freighter or tanker, but fitted with hoppers to retain and transport material instead of holds for cargo or tanks for oil. Photographs showing some of essential features of this type of dredge are shown in Figs. 1 - 4. The seagoing hopper dredge has been developed primarily for the dredging of bars and entrance channels at coastal inlets, in other exposed locations, or where the excavated material must be carried a considerable distance for disposal. It excavates, transports, and disposes of material without anchoring and without the assistance of any auxiliary plant or equipment. Its operations do not interfere with, or obstruct, navigation of other craft as it moves along a waterway under propulsion and steerage control similar to any other vessel. Therefore, its use is also favored in busy harbors.



General view of a seagoing hopper dredge. This is a side-drag type. (Note drag pipe in raised position alongside dredge.) The "A. Mackenzie" is 268 feet long and has a hopper capacity of 1650 cubic yards.

Fig. 1



U. S. hopper dredge "Pacific," looking forward from pilothouse. Picture taken during dock trials of dredge pump. Hopper spaces in foreground. Cylinders for hydraulic operation of hopper doors are on each side in foreground and center. Levers, center, are for control of discharge from pump. Motor-driven hoists control motion of drag and suction pipes in vertical planes.

Fig. 2

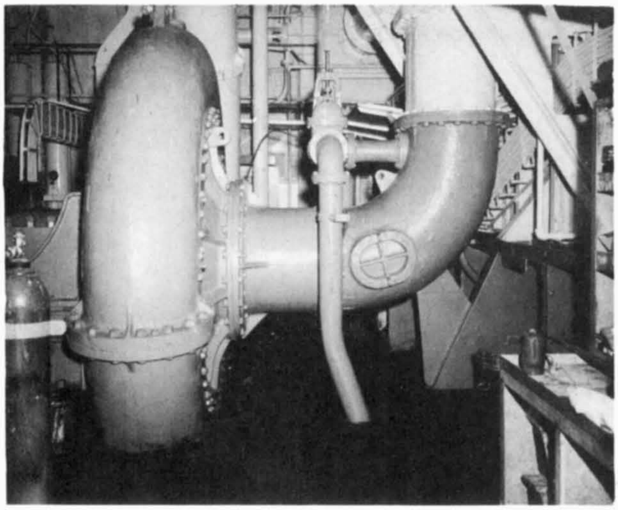


Fig. 3
Dredge pump of seagoing hopper dredge ("A. Mackenzie"). Impeller is 6 feet 9 inches in diameter. Discharge pipe (left in photo) has inside diameter of 26 inches. Pump driven by 900 H.P. electric motor at 150 R.P.M. Pump capacity 52,000 G.P.M.

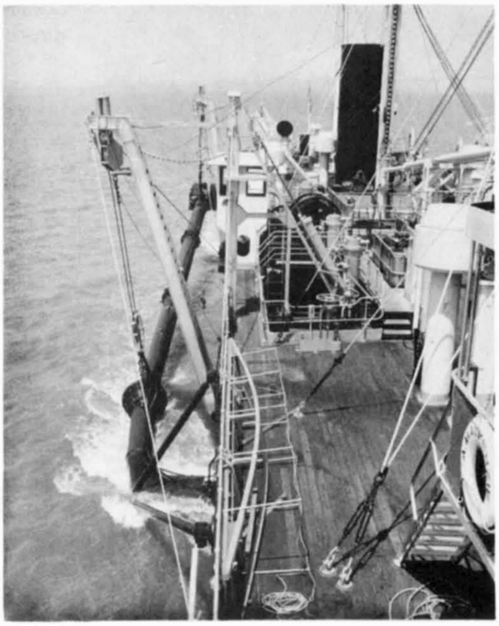


Fig. 4
General view from pilothouse of starboard side of seagoing hopper dredge ("A. Mackenzie"). Drag pipe in raised position.

Seagoing hopper dredges vary in size and capacity in accordance with the type of projects on which they are designed to be used. Small shallow-draft dredges of 500-cubic-yard capacity are about 180 ft. long; those of 1200- to 1600-cubic-yard capacity are about 280 ft. long; 3000-cubic-yard dredges are about 350 ft. long; and the largest dredge, of 8000-cubic-yard capacity, is 525 ft. long. All of the dredges have quarters and mess accommodations for crews for three-shift 24-hour-per-day operation.

Because of its principal use in dredging in exposed localities where other types of dredging equipment cannot be used, and in order that it may travel between such localities, the seagoing hopper dredge is designed to be seaworthy in all respects. The dredges are built to meet the U.S. Coast Guard requirements for "Ocean" classification and carry all prescribed navigation equipment including radio communication, lifeboats, and firefighting apparatus. Personnel are licensed or certified in accordance with U.S. Coast Guard regulations. Some of the dredging equipment is unique to the type of dredge, and warrants special discussion. The principal elements of dredging equipment are the drag-pipe assembly, the drag hoist, the pump, and the hoppers. The drag-pipe assembly is that portion of the pump suction line carried outside of the hull. One end of the assembly is hinged to the hull by a swivel-ell connection. The other carries the draghead, which is a large steel casting with a grate on the lower face, so designed and hung that it rides on the channel bottom when the drag is lowered. When operating, the hopper dredge moves along the channel at slow speed (1 to 3 knots) with its dragheads lowered in contact with the channel bottom. The mixture of bottom material is lifted hydraulically through the suction line by the pump, thence through discharge pipes to the hoppers.

The drag-pipe assembly may be hung from the vessel in any one of three ways. When in the center amidships so as to be raised and lowered in a well through the vessel, it is known as the "center-well" type. When suspended in a well at the stern, it is a "stern-well" type. When suspended from the side, it is a "side-drag" type. The latter has been adopted as a standard for the modern dredges which carry, and usually operate with, a drag on each side.

Dragheads are designed basically to secure the maximum percentage of material with minimum hydraulic losses and without admitting obstructions or choking the pump. Two general types have been evolved. The "New York," or fixed, type operates best in soft or free-flowing material while the "Pacific," or self-adjusting type, is most adaptable to dredging packed sand or other firm material. Because the drag bottom, or grate, is subject to wear through abrasive action, it is usually hard-surfaced and replaceable. Unlike most pipe-line dredges, which have cutters to loosen material, the hopper dredge depends upon "suction" and the motion of the draghead for loosening material. However, teeth have been attached to, or cast integral with, draghead grates in attempts to scarify or loosen firm material. These attempts have met with varying success.

The forward end of the drag-pipe assembly includes an elbow and is usually supported by a sponson with bearings which carry the pipe trunnions and act as pivots for angular movement of the drag in the vertical plane. A ball joint in the pipe between the elbow and drag permits minor angular movement in the transverse direction. The drag assembly is supported from davits by cables passing through sheaves to the drag-hoisting winches. The latter provide for the lowering and raising of the drag pipe. During dredging, each drag is continuously controlled by an operator called a dragtender. He must control the position of the drag in relation to the bottom in accordance with the type of material encountered and the depth of cut necessary, so as to obtain maximum dredging efficiency.

The dredging pump is usually located in the forward part of the ship and is especially designed for the type of operation involved. Like most dredging pumps, it must be constructed to resist abrasion and must be of high capacity at relatively small discharge head. The larger dredges have one pump for each drag, but several of the smaller dredges (500- to 1600-cubic-yard capacity) have one dredge pump with suction pipes from each drag uniting in a wye on the suction side of the pump.

The pump discharge pipes lead to the hoppers where suitable distribution boxes, or troughs, are equipped with gates, or valves, to regulate flow into the several hopper bins. The solid material settles in the hoppers while the water (carrying a small percentage of suspended material) flows over the top of the hoppers into troughs which carry it to overflow chutes which discharge it overboard. When the hoppers are filled with solids (or when, in the case of dredging material which tends to remain in suspension, the "economic pumping" limit has been reached) the pump is stopped, the drags are lifted, and the dredge runs at high speed to the dumping ground.

The hoppers are usually located amidships and extend from the main deck nearly to the bottom of the dredge. Their sloping bottoms are fitted with semi-watertight gates or doors for dumping material. On the modern dredges, these doors are operated by direct-acting hydraulic cylinders, the piston rods of which are connected to the door-operating rods. In keeping with the general safety precautions taken in the design and operation of the seagoing hopper dredges, the hoppers are usually located in watertight bulkheaded compartments. Necessary doors through such bulkheads are watertight, and those to engine rooms are equipped with remote controls for emergency purposes.

As with other modern vessels, the type of power plant for a seagoing hopper dredge is selected to give the most efficient performance under the design criteria of the vessel. The general types in use are turbo-electric, diesel-electric, and direct (steam or diesel engine) drive.

There are numerous other features of the seagoing hopper dredge which are unique to its type as a vessel and as a dredge. However, it is believed that the principal of these features have been mentioned herein. About 25 seagoing hopper dredges are operated by the Corps of Engineers, and this fleet moves approximately 70 million cubic yards of material annually to play an important role in maintaining harbor entrances and channels which are necessary for waterborne commerce.

PART 4

SITE CRITERIA AND THE DESIGN AND CONSTRUCTION OF COASTAL WORKS

CHAPTER 20

LOCATION OF HARBORS

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INTRODUCTION

The location of harbors is dependent on many factors, which vary widely in different parts of the country. For example, along the Atlantic coast are many natural waterways. The improvement of these waterways, especially for small craft, is comparatively inexpensive and harbors are constructed at frequent intervals. However, along the coast of southern California, natural facilities are almost nonexistent. Consequently, improvements are farther apart and correspondingly larger. The cost per boat accommodated remains about the same. In general, this discussion is based on conditions along the coast of California, particularly southern California.

The purpose of this paper is to show how the costs, functions, types, sizes, and physical features of harbors and the effect of harbors on adjacent shore lines and underground basins are related to the selection of a proposed harbor site. Typical examples, with brief discussions of the principal factors involved in the selection of harbor sites, are given later in this paper.

FUNCTIONS OF HARBORS

In general, public harbors may be divided into four classifications according to the function the harbor is designed to fulfill. These are: commercial, recreational, and fishing harbors, and harbors of refuge. Harbors used exclusively for military or naval use are not included in this discussion. Also, it must be recognized that many harbors have multiple uses that include elements of all four classifications in varying degrees. The selection of any harbor location is directly affected by the function that the harbor is designed to fulfill. This, of course, presupposes that a need exists for the particular kind of harbor considered.

For example, the commercial harbor should be so located as to best expedite the movement of goods, and should be so designed as to be readily adaptable for future expansion of both land and water facilities. The history of all our major ports is one of continued expansion and improvement. However, a harbor of refuge is a haven where vessels may seek shelter in heavy weather or may obtain emergency supplies and repairs. This type of harbor is generally in more remote sections of the coast, and emphasis is placed on so locating the harbor as to provide a safe entrance during severe storms. The selection of a location for a fishing harbor is governed by proximity to fishing grounds, adequate housing for fishermen, service facilities for fishing boats, and markets for fish caught.

TYPES OF HARBORS

Harbors are either natural or artificial. If a harbor is natural, its location is determined and only its most efficient use or improvement must be ascertained. In early days, communities grew up around these natural harbors and harbor improvements came about as natural processes.

Artificial harbors are generally developed to fulfill specific needs. These harbors may be either offshore facilities or interior basins, depending on location and natural terrain. Where conditions permit, a harbor with entrance jetties and interior basins is the most satisfactory. However, because of prohibitive rights-of-way costs or of rugged terrain, dredging an interior harbor is often impracticable, and an offshore harbor protected by a breakwater system or a combination of natural headlands and breakwaters must be constructed. Each type of harbor has its own inherent advantages and disadvantages, which must be weighed before selection of the location for a proposed harbor. If the location of the harbor has been otherwise determined, this location may govern the selection of the type of harbor.

DETERMINING THE BEST LOCATION FOR HARBORS

Careful consideration is given to many factors in determining the best location for a harbor. Some of these factors are general and aid in determining whether a harbor should be provided in a general locality. Others are specific and aid in pin-pointing the location of a harbor or even the location of components of the harbor. These factors are all interrelated and must be compared with one another to determine the relative weight each factor may have in a specific case. Major factors are briefly discussed in the following paragraphs.

POPULATION CENTERS

Except harbors for refuge only, harbors generally should be near large centers of population. This is particularly important for commercial harbors where a nearby outlet for consumer goods results in lower transshipment costs. The costs of both imports and exports are affected by the overland shipping distances involved. This was one argument advanced by the proponents of a harbor at Santa Monica against a harbor at San Pedro. Large population centers also provide manufactured products for export and a labor market of dock and ship workers.

Recreational harbors also should be near large centers of population or should be combined with commercial fishing harbors and harbors of refuge to be justified. The basic protection for an artificial harbor is so expensive that it is warranted only if a large number of boats use the harbor. These boats can only be provided by a large tributary population.

For commercial fishing harbors, proximity to large population centers is a factor but not so important as it once was. The present practice of quick freezing permits a longer period for distribution, and quick sales to the consumer are not mandatory. However, a large nearby population reduces handling costs and provides labor for canneries and fisheries.

REQUIREMENTS OF FISHING

Requirements vary for different types of fishing. Tuna clippers, which are large boats equipped with refrigeration, travel long distances for tuna. Because their catch must be sold to canneries, they generally operate from the nearest commercial harbor. Smaller boats, which are not equipped with refrigeration, fish for sardines and pilchard. Because these fish spoil rapidly, the boats must be unloaded not later than the day after the fish are caught. Accordingly, the fishing radius for this type of fishing is more limited and harbors are required at more frequent intervals. The hook-and-line fishermen who supply the market with fresh fish operate small one- or two-man boats that have a very limited safe cruising radius. Unless harbors are provided at distances of not over 15 to 20 miles from the fishing grounds, those grounds cannot be exploited. Along the coast between Morro Bay and Monterey Bay are many such unexploited fishing grounds. The State of California Division of Fish and Game has requested that consideration be given to the development of an adequate number of harbors in this reach of coast line.

DISTANCE BETWEEN HARBORS

Studies are now being made to determine the need for a chain of small-craft harbors along the coast of California and also to determine whether these harbors can be economically justified. They would be spaced at intervals that would permit a boat at no time to be more than 2 or 3 hours from a haven of refuge. As a result, harbors would be about 30 miles apart. A harbor proposed between two existing harbors 50 to 70 miles apart would be about halfway between the two existing harbors, providing a satisfactory location was available. If the existing harbors were 80 to 100 miles apart, two harbors probably would be developed, or, if only one could be justified at the time, it probably would be about one-third of the distance from one of the existing harbors.

TRIBUTARY AREAS

The tributary area to any harbor is primarily determined by the access routes to that harbor as compared with those to other adjacent harbors. Other factors, such as labor or through rates, may sometimes be of more importance. For example, cotton has been shipped by rail from the Sacramento and San Joaquin Valleys to Galveston or other Texas ports and then shipped to the Orient via the Panama Canal. This cotton should have been shipped through San Francisco Bay. Westcoast ports are making every effort to eradicate the conditions that lead to such an abnormal distortion of normal tributary areas.

Owners of small craft live not only in population centers immediately adjacent to harbors for small craft but also in distant communities. Many residents of Bakersfield now keep their boats at Los Angeles and Long Beach Harbors. Because Bakersfield is considerably closer to Ventura than to Los Angeles, residents of Bakersfield probably would keep their boats at Ventura if adequate facilities were available. The needs of an entire back tributary area must be considered in determining the location of a new harbor or the necessity for improvements at existing harbors.

ACCESS ROADS

Access roads, both rail and highway, are important considerations in determining the extent of a tributary area and the most favorable location for a harbor. Included under this heading is the necessary relocation of highways and highway bridges. This presents a serious problem in the consideration of the proposed small-craft harbor at Alamitos Bay, where a secondary highway over a trestle bridge crosses the entrance channel and a railroad crosses the site of the central part of the proposed harbor. A considerable readjustment of access roads would be required to properly serve the harbor area.

Access roads are important because owners of boats and suppliers of service to boats must be able to reach them from the land side. This is a major problem in determining the best location for harbors of refuge near Point Conception and Point Arguello, where the local sponsoring agencies probably would have to provide several miles of access highways. Even where adequate access roads exist, they affect the location of a harbor. Every effort is made to select that location where the greatest benefit would result to the greatest number of people and where the use of existing facilities would not be adversely affected.

SIZE REQUIREMENTS

In southern California, basic protection at a very high cost generally must be provided for any proposed harbor. Also, harbors must be larger and farther apart than east-coast harbors to be economically justified. Size is sometimes a decisive factor in determining the location of a harbor. For example, Playa del Rey is the only suitable location in Los Angeles County for a small-craft harbor of the size proposed. At one time local interests at Laguna Beach requested consideration of a harbor in a small canyon between Laguna and South Laguna. However, the request was withdrawn when they realized that the available space was entirely inadequate.

Important considerations in the work of planning and designing a harbor include selecting the location of the entrance and determining its size. A harbor entrance should be of sufficient width to accommodate anticipated traffic with ease and safety. It should not be so wide as to induce shoaling between the jetties and the attendant development of a meandering channel. In southern California, the tidal prism of a harbor is never sufficient to maintain the entrance naturally. Accordingly, the wider the entrance, the longer must be the entrance jetties to prevent shoaling.

In planning an entrance channel to upper Newport Bay, local interests were faced with a choice between the Balboa Island channel and the Upper Bay channel. The Balboa Island channel between the upper bay and the ocean would be a shorter channel and would reduce traffic congestion in the lower bay. However, because of ledge rock and high rocky bluffs, local interests have expressed the opinion that a channel entrance of adequate width through the Balboa Island channel would be neither practicable nor economically feasible.

SURGE

Surge is generally a major factor in determining the location of the integral parts of a harbor rather than in determining the location of the harbor itself (see Chapter 6). The causes of surge are quite complex and individually may have some bearing in determining the location of a harbor. Short-period waves seldom cause surge. They could only do so if the basin were of such shape and size as to develop a period of resonance that would combine a damping and amplifying effect on waves of certain short periods. Generally, the surge results from a convergence of long-period waves. In determining the location of a harbor entrance, areas of concentrated long-period waves are carefully avoided if possible.

Model studies are often made to determine the best locations for component parts of a harbor so that surge will be reduced the greatest practicable extent. A model study was made by the United States Waterways Experiment Station at Vicksburg, Miss., for the Long Beach Harbor Department to assist Long Beach in the planning and layout of its East Basin (see Chapter 23). As a result of this study, major changes were made in the location of the basin entrance and in the layout of the interior basin. Data obtained from the model study indicated that these modifications would materially reduce surge in the new basin. Similar studies were made for the Los Angeles District to determine the proper length and location of a proposed detached breakwater extension to reduce surge and wave action in Anaheim Bay Harbor.

In Los Angeles Harbor, a surge condition exists in East and West channels and in slip 230 on Terminal Island. If a model study had been made when the harbor was being planned, this surge condition might have been reduced or eliminated.

PHYSICAL FEATURES

The importance of physical features in determining the best location for a harbor is readily apparent. These features, which include natural protection, availability of land for service areas, natural sloughs for harbor development, streams, and drainage, outweigh all other considerations. This is reasonable because the physical features of a coast line affect the many other factors that must be considered before a harbor is constructed.

The planner and designer must fully utilize all natural protection that would help to provide a less expensive and a safer harbor. Use of natural sloughs would reduce costs of rights-of-way and of dredging. Southern California streams, which are dry most of the year, do not augment the tidal prism in maintaining an entrance channel. During flood periods these streams often carry large quantities of debris that shoal interior basins and channels. Diversion of streams may be difficult and expensive. Drainage sometimes is a problem, especially where a harbor is constructed in low marshland. Filling sufficient ground around the harbor for its service and operation may result in drainage problems in the rest of the low-lying areas.

EFFECT ON THE ADJACENT SHORE LINE

Along the coast of California, sand movement is generally from up coast to down coast or north to south. Minor exceptions occur at major headlands, where the direction of the coast line produces localized areas of up-coast drift, and at many small pocket beaches. Also, a general reversal of the normal trend of sand movement occurs during periods of southern swell. The construction of offshore breakwater or harbor entrance jetties results in interception of this down-coast-trending littoral drift and in accretion on the up-coast side and erosion on the down-coast side. This process continues until the harbor is filled or until sand passes around the jetties, shoaling the entrance channel as it passes. If nothing is done, the process eventually ends but the harbor is no longer navigable. In the meantime, millions of dollars in damage may occur to down-coast areas.

This danger has been recognized for some time. The Corps of Engineers is required to consider the effects that any proposed harbor would have on adjacent beaches for a distance of 10 miles on both sides of the harbor. The damage is small in isolated or rugged areas but is of primary importance in improved areas with population centers and popular beaches. The necessity for bypassing of sand past the harbor entrance to maintain the harbor's navigability and to prevent damage to down-coast beaches by maintaining the normal flow of littoral drift is always given careful consideration. The cost of bypassing of sand is included in the costs of the project. A choice in location may occasionally occur to minimize the danger of erosion damage. For example, a survey was recommended for a harbor at Malibu instead of at Point Dume because Malibu was down coast from the highly developed beach colony and any improvements probably would not cause erosion of this valuable area.

WAVE EXPOSURE

Wave exposure is always considered in determining the location of a harbor and of the integral parts of a harbor, especially the entrance channel. By the use of refraction diagrams, zones of convergence and divergence may be readily determined. With these zones established, the harbor entrance can be so located as to provide optimum safe navigating conditions. A slight shift in location often will result in lowering the design wave several feet. Thus, the size of protective structures may be safely reduced with resultant savings in cost.

An example of a poorly located structure subject to wave exposure is the Standard Oil Co. pier at El Segundo. The pier is opposite a submerged ridge at about the 12-fathom depth. Long-period waves (18 sec.) from the northwest are affected by this ridge, which causes them to converge near the pier. Because of this convergence, waves more than 20 ft. in height have been observed at the pier when wave heights a short distance on both sides of the pier were not more than 6 ft. in height. Company officials state that if as much had been known about wave refraction when the pier was built as is now known, the pier would have been built elsewhere. At Redondo Beach a study of refraction diagrams resulted in a slight change in the location of the breakwaters with a marked reduction in wave exposure at the entrance. The masking effect of islands in reducing wave heights is considered in determining the location of a harbor and the height of the design wave.

SALINE CONTAMINATION

The problem of saline contamination is especially important in southern California when considering an interior harbor such as Playa del Rey or a ship channel such as proposed at Laguna Dominguez. At Playa del Rey, a geological study of underground aquifers indicates that the proposed dredging would not increase the saline contamination of ground water. However, in other instances the introduction of salt water to inland areas by channel dredging might cause damaging saline contamination of extensive underground basins. Extensive studies have been made to alleviate the salt-water intrusion in the lower reaches of the Sacramento River.

REQUIREMENTS OF LOCAL COOPERATION

Authorization of a harbor project by Congress is usually contingent on local interests' bearing a substantial part of the cost of the proposed harbor. The allocation of costs between the Federal Government and local interests is based on policies established for different types of harbors. In general, local interests are required to bear the costs of rights-of-way; utilities, bridges, and highways, including their relocation; slips, moorings, and all other facilities for servicing small craft; piers, wharves, transit sheds, and allied structures in commercial harbors; dredging for slips and for mooring areas; landscaping; and administration buildings. Local interests are also required to assume the cost of any damage resulting from construction of the project. Other special requirements may be included as conditions warrant.

The willingness of local interests to assume their share of the cost of a project is sometimes the deciding factor in determining the location of a harbor, especially where a choice exists between two favorable sites. Under most circumstances, the harbor probably would be located at the site where local interests were willing and able to comply with the requirements of local cooperation. For example, a choice existed between Pierpont Bay and Port Hueneme as sites for a small-craft harbor in Ventura County. Pierpont Bay had widespread support in Ventura County until it became known that a harbor at Pierpont Bay would require bypassing of sand twice instead of once. The knowledge that either the City of Ventura or Ventura County would have to assume the cost of the second bypassing, which was estimated at $125,000 a year, eliminated Pierpont Bay from further consideration and concentrated support for a harbor at Port Hueneme.

NATIONAL DEFENSE

National defense is also an important consideration where a choice exists between two sites. It may even be the deciding factor in determining whether a harbor will be justified. The fact that one site was militarily more important than the second site would lead to either the elimination of the site with less military importance or to the construction of both harbors, should such action be found necessary and justified.

COSTS

With the exception of national defense, all factors discussed and all solutions of the problems they present are related to the cost of the project. They may affect costs of construction, rights-of-way, and maintenance. Where a choice exists between sites, the site that would provide adequate facilities and that would be economically the most feasible probably would be the site selected. A comparison of costs for various sites is made by reducing costs including amortization, interest on investment, operation, and maintenance to an annual basis.

APPLICATION OF THE FOREGOING FACTORS TO THE SELECTION OF SPECIFIC HARBOR SITES

As previously stated, the final selection of a harbor site is dependent on a combination of factors, although some may be more important than others. To illustrate how the importance of these primary factors varies at different sites, their effect on proposed harbors either under consideration or already authorized is briefly discussed in the following paragraphs.

The Cambria site was selected for further study mostly because of the requirements of fishing. Principal secondary factors were the proximity of population centers and the distance from existing harbors. The alternative site at San Simeon was selected because of the physical features of natural protection.

The improvement at Morro Bay resulted primarily from the bay's importance to national defense as well as its favorable physical features. Morro Bay is one of the few natural bays in southern California.

The Coxo anchorage site would be a harbor of refuge only. The site was selected for further study because of the distance between existing harbors and because of the natural protection at the site.

Port Hueneme was recommended for construction because of the effects of other proposed harbors in that vicinity on the adjacent shore line and also because of the needs of the tributary area.

The proposed Playa del Rey Harbor was recommended because (1) the site is near the largest concentration of population on the west coast; (2) ample space is available for a harbor of the proposed capacity; (3) the land has comparatively little value except for a harbor; and (4) rights-of-way costs, although high, would be relatively low for an area of its size close to the center of population.

Primary factors leading to the adoption of a project at Redondo Beach were (1) the effect of the project on the adjacent shore line and (2) the peculiar conditions of wave exposure. The head of a submarine canyon is in the vicinity of Horseshoe pier in Redondo Beach. Ten-second waves refract over the banks of the canyon in such a manner as to converge on the beach between Diamond Street and Fifth Place with highly destructive effects. At the same time, the divergence that occurs over the submarine canyon tends to create a zone of calm water at the proposed entrance. The proposed offshore breakwater would intercept the waves before converging and obviate the necessity for an extremely heavy breakwater section.

The site of Mission Bay Harbor, which is now under construction, was selected principally because of its favorable physical features. Mission Bay was a large natural bay being shoaled by depositions from San Diego River. A major factor in the authorization of this project was the voting of bonds by residents of the City of San Diego to meet the requirements of local cooperation before adoption of the project by Congress. The river is now being channelized direct to the ocean, and the bay is being dredged.

CONCLUSION

The foregoing comments may be summarized in the following conclusions:

a. Factors affecting the location of any harbor are many and vary at each site.

b. The location is seldom determined by any one factor, but by weighing the importance of many factors or combinations of factors.

c. No mathematical formula exists for solving harbor problems. Answers cannot be obtained by tossing these factors into a hopper and turning a crank.

d. Many aids, such as wave hindcasting from historical weather maps and construction of refraction and diffraction diagrams, have been developed recently for the determination of proper location. This research is still in progress.

e. Despite our scientific advancements, experience in analyzing the results of our studies is still the first requisite in determining the most favorable location for a harbor and the best plan for its optimum development.

CHAPTER 21

FACTORS INFLUENCING AND LIMITING THE LOCATION OF SEWER OCEAN OUTFALLS

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A sewer outfall, or as it is commonly called and will be hereinafter termed, an ocean outfall, is customarily a pipe line extending seaward from the shore and designed to convey sewage and industrial wastes, treated or otherwise, to such a location offshore as is hoped, or expected, will prevent contamination of the nearby littoral waters, protect recreational facilities in the vicinity, and result in a disposal of the contaminating wastes without nuisance or meance to public health. Protection of aquatic life is at times the most important consideration.

Unless the cost of construction is very great or the ocean outfall rests upon a physically insecure or inadequate foundation, this method of final disposal of sewage and industrial waste is probably the most economical to be found because the ocean water and its dissolved components, together with certain microscopic and macroscopic marine life, are able to complete the destruction of even the most noxious wastes without difficulty if afforded the properly controlled opportunity. Thus, the presence of the ocean in or near the "front yard" of a municipality offers an almost irresistible temptation to dispose therein of the sewage of the community.

The term ocean outfall is, at times, applied to a sewer outlet which discharges its contents at or near high water. Scores of such outlets along the Pacific Coast empty into bays, estuaries, harbors and at river mouths. Since these have little, if any, effect upon the objective of this meeting they will be disregarded here, with the comment that they are decidedly insanitary, and further discussion directed toward outfalls which extend seaward or away from shore an appreciable distance and, in general, to a depth which does not interfere with navigation.

Principal among the functions of an ocean outfall is the discharge of contaminating waste at such a location as will prevent its return to shore in such quantity, concentration, and elapsed time as to constitute a source of contamination to the shore waters. Thus, a location for the construction of the outfall which might be otherwise satisfactory could, and many times should, be ruled out because it is unsatisfactory from a sanitary standpoint.

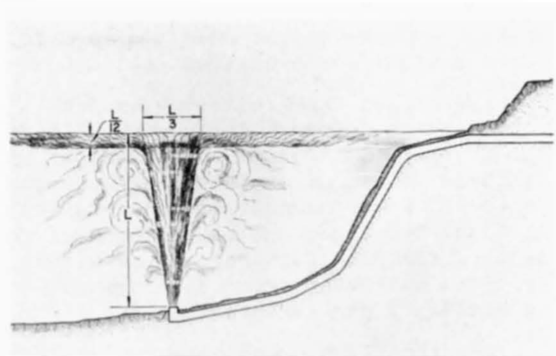
When fresh water or sewage is discharged at or near the floor of the ocean, its tendency is to rise with reasonable promptness to the surface. This is occasioned by the difference in specific gravities of the two fluids. In general, sewage may be expected to rise in an expanding column, the diameter of which at the point where the ocean surface is broken is about one-third of the length of the path from the point of discharge to the surface, and the thickness in which, as it spreads laterally over the surface, will approximate one-twelfth of the length of the rising path (Fig. 1). Factors likewise have been determined to indicate that the dilution of the fresh water or sewage with sea water will vary with the depth, direction, and velocity of the discharge at the ocean floor (Fig. 2).

Thus, one may predetermine with some degree of accuracy what will comprise the fluid mixture which, upon reaching the ocean surface, tends to spread laterally over it and which, correspondingly, will or may, be carried by the tide and wind induced currents into areas where it is not wanted.

From this it may be seen that the second series of factors to be considered are the wind currents and tidal currents because these have the controlling effect upon the time interval between the discharge of the water from the outfall pipe and its entrance in more dilute form into areas where it is undesirable. The limiting time factor from outlet to shore may be expressed as between three to ten hours, depending upon the degree of dilution, the presence or absence of large solids, and the extent of pre-treatment.

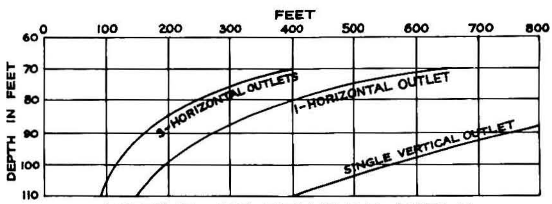
Irrespective of all other considerations those enumerated in the two paragraphs above should exercise the primary control in the location of the end of the pipe line from which the waste liquids are discharged, but from this another consideration springs. For instance.... a selected point of discharge may be such that partially treated wastes from it will travel with sufficient promptness and at sufficient concentration to adjacent shores so as to render the site objectionable. Nevertheless, the site may be utilized if the degree of treatment of the wastes prior to discharge is improved.

From this it follows that an otherwise unacceptable point of discharge may be rendered perfectly appropriate if treatment of the sewage onshore is more complete, leaving less of the work for the ocean to do; but it must be reckoned that the resulting costs for onshore construction and treatment will be materially greater and may outweigh other advantages which the site offers.



IDEALIZED SEWAGE FIELD
DISCHARGE FROM VERTICAL OUTLET

Fig. 1



RADIUS OF SEWAGE FIELD
100 C.F.S. - 1 PIPE

Fig. 2

Disregarding the possible destruction of an ocean outfall by dragging anchor flukes, sunken vessels or the like, it is probably physically possible to make a sound and permanent structure of such a pipe line almost any place. Construction may vary from the mere laying of pipe on a reasonably firm bed which apparently is, and in the past has been, undisturbed, to the resting of the line on piling at the ocean bed or above the water or excavation of a trench through solid material and the embedment of the pipe therein. Intermediate between the extremes are many combinations, and the use of many different construction materials, depending upon the diameter and length of the outfall and the forces, natural and otherwise, to which it is to be subjected.

There are two extremes offered in articulated pipe lines, one being the rigid type of structure designed to withstand all of the forces, natural and otherwise, to which it will be subjected without yielding at all, and the flexible type of structure, which may move reasonable distances laterally and vertically without leakage or rupture. Each has its place and a combination of the two may, at times, yield the most satisfactory structure.

An excellent example of the rigid type of structure is the Los Angeles City Outfall at Hyperion, wherein the reinforced concrete tubes, 100 ft. in length and 12 ft. in diameter, are set on concrete and steel piling and joined rigidly together. This outfall, only recently constructed, has not been in operation long enough to determine its permanency, but it appears to be capable of withstanding anticipated natural forces.

Two outfalls built by the Sanitation Districts of Los Angeles County at White Point are examples of the combination rigid and flexible type structures; the rigid portions of each being concreted-cased pipe laid in rock trenches from the shore some 2,000 ft. seaward, emerging thence onto the undisturbed sand floor of the ocean and laid to the end with flexible ball and socket joints at strategic intervals.

The outfall into San Francisco Bay now being constructed for the East Bay Sewerage Project by the East Bay Municipal Utility District will be critically observed as one which is moderately flexible to the end and one in which steel and rubber instead of cast iron and lead form the joints. All of these lines are of large diameter, the smallest being 60-in., the largest 12 ft.

For years the greatest weakness in articulated ocean outfall construction was found at the joints. These proved the undoing of two large diameter outfalls off the Southern California coast; one at Hyperion, and one at Santa Barbara. In each instance it was attempted to use the tongue and groove type of joint, concrete to concrete, with the annular space filled with plastic asphalt compound. Appraisal of these two lines indicated the need for a better and more secure method of connecting the pipe together and resulted in the construction of concrete pipe equipped with cast iron end rings, the rings joined to the pipe by welding to the longitudinal pipe reinforcement.

In the first open ocean outfall along the Southern California coast to show any degree of permanency, reinforced concrete pipe sections 60 in. in diameter and with a 7-in. shell thickness were equipped with mehanite cast iron end rings. The rings were welded to the longitudinal reinforcement and, for underwater connection, were so constructed that a pre-cast lead-caulking ring was driven into place from the inside of the pipe (Fig. 3). This method of joint construction has proved secure and sound in operation since 1935, but from the nature of the joint the

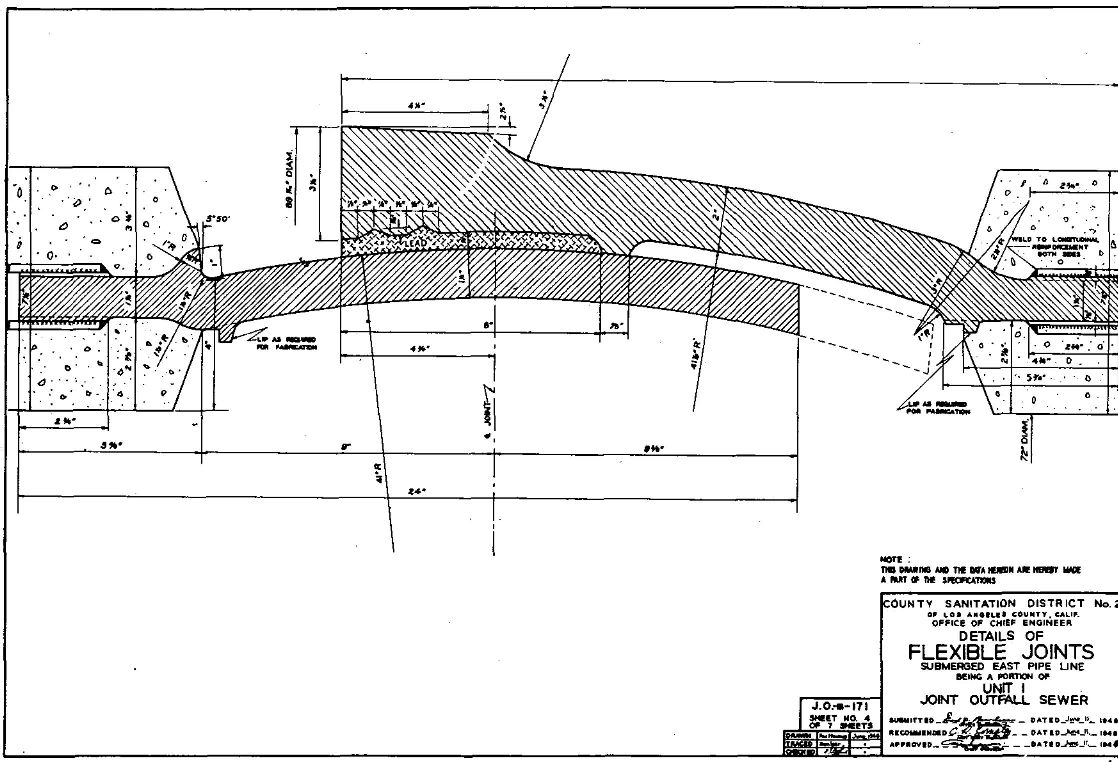
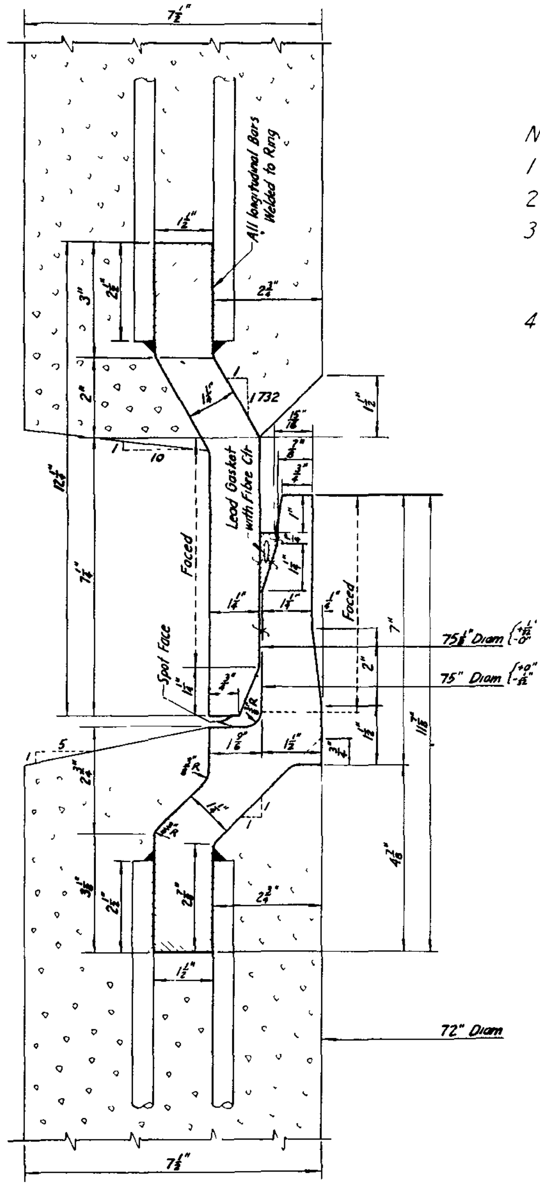


Fig. 3

interior of the line is not one smooth tube because of the annular indentations at each joint (Fig. 4). The pipe hydraulics are correspondingly influenced.

Cast iron pipe of 60 to 72-in. diameter is available but has grown increasingly difficult to obtain in the past decade and, for years during the 1940's it either could not be had at all or its delivery date set so far in the future as to discourage its use. Fortunately, there is no reason to require a sewer ocean outfall to be built of cast iron, as has been demonstrated by the use of concrete in such lines for more than the past quarter of a century. Failure has occurred in concrete outfalls but without exception those which have failed have done so for



NOTES

1. Poured Pipe to be furnished in 24'-0" lengths
2. The Joint Rings to be Meehanite Type GA
3. Twenty (20) sections of pipe to be beveled 3° on one end and the joint ring set at this angle. The beveled end to be plainly marked
4. Except as noted in (3), joint rings to be set at right angles to the axis of the pipe. Allowable tolerance to be 1/4" across the diameter of the pipe

Scale 3/4" = 1"

COUNTY SANITATION DISTRICT No 2
OF LOS ANGELES COUNTY CALIF
OFFICE OF CHIEF ENGINEER

DETAILS OF
INSIDE CAULKED JOINT
POURED PIPE WITH WELDED MEEHANITE JOINTS
SUBMERGED EAST PIPE LINE
UNIT 1 - JOINT OUTFALL SEWER

SUBMITTED — ACTING OFFICE ENGINEER — DATED Jan 31 1947
RECOMMENDED — ASST CHIEF ENGINEER — DATED Jan 31 1947
APPROVED — CHIEF ENGINEER — DATED Jan 31 1947

J O-m-171
SHEET No 5A
OF 7 SHEETS

DESIGNED	G POSTHUMUS	JAN 1947
DRAWN	G POSTHUMUS	"
TRACED	H WHITECOTTON	"
CHECKED	N ANDERSON	"

Supersedes Sheet No 5

Fig. 4

reasons other than decomposition or destruction of the concrete. The combination of reinforced concrete barrel with cast iron end rings and metal to metal joints, secured by lead caulking with a certain degree of flexibility, results in a totally adequate ocean outfall if otherwise adequately designed and supported.

An ocean outfall should be designed so that it may be entered through manholes provided during construction and caulked from the inside instead of the outside of the line. The reasons for the manholes are obvious and the ability to repair a damaged joint from the inside is readily apparent when one considers the difficulties attendant upon attempting to caulk a leaky joint from the outside.

An ocean outfall flowing under pressure, and all are under pressure to some extent because of the difference in specific gravity of the two liquids, will -- unless prevented therefrom -- scour an unconsolidated foundation at a leaky joint to an extent which may cause a non-metallic joint to yield, allowing the adjacent pipe to settle, thus creating an offset in the pipe line which is practically impossible to repair (Fig. 5). This was observed in the original Hyperion pipe line and also in the one constructed at Santa Barbara. Even though the joints are of metal, care should be taken to prevent this possibility. Prevention of scouring at the joints has been successfully accomplished by piling gravel of sizes varying from 2 to 6 in. against the sides of pipe for 3 to 4 ft. beyond each edge of the joint.

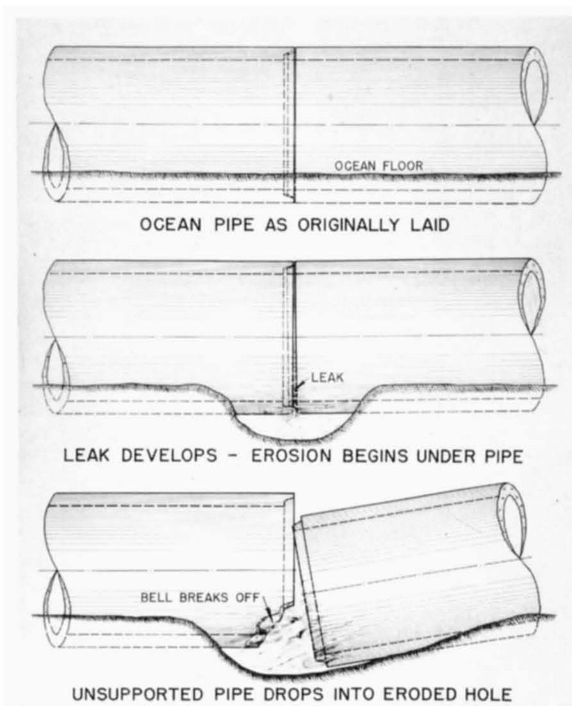


Fig. 5

Two forces will tend to scour a trench in unconsolidated material adjacent to the pipe and parallel to its long axis. One is the drag of the salt water resulting from the updraft at the outlet, and the other is the tendency for a current flowing normal to the long axis of the pipe to move seaward along a pipe line acting as a submerged weir. At times, this path, or trough, will attain such depth that the current normal to the pipe line will scour under the pipe and eventually undermine it (Fig. 6). This may be avoided by depositing rock from 2 to 6 in. in diameter from about the spring line of the pipe to the ocean floor on each side. Any scour after the rock is placed will be filled by the gravel rolling into it.

It can be shown that outfalls of material proportions have no place in inner harbors or where there is serious danger of damage from anchor flukes. In any harbor where ships swing at anchor an ocean outfall is in danger, even though the pipe be plainly marked on all navigation charts. In certain natural harbors, such as San Francisco and Puget Sound, it is obvious that the bay or sound waters must be treated as open ocean in the design of ocean outfalls.

The best foundation for an open ocean outfall pipe is afforded by a trench cut in rock as far seaward as the scouring effect of waves is felt on the ocean bottom into which the pipe line is placed and embedded in tremie concrete. When a depth is reached at or beyond which the most serious storms will not disturb the ocean floor, the best foundation is said, by marine experts, to be the undisturbed ocean floor. If the latter is unconsolidated, however, precautions against longitudinal and cross scour must be observed. If rock foundation for the shoreward section of the line is not available, it should be simulated by placing the outfall pipe on a permanent pile foundation sufficiently deep so that the normal ocean bed will completely cover the pipe.

Best protection to adjacent shore waters is obtained by discharging the outfall contents into currents which are predominantly along-shore and which are seldom, if ever, directed toward points onshore such as small capes, bays, or inlets along the shore.

Excepting under the most unusual circumstances, preliminary treatment, to the extent of primary sedimentation, and the best possible dispersion or diffusion of the sewage into the sea water at the outfall outlet is advisable and will materially assist in preventing shore contamination. Careful investigation may indicate that these two factors, plus reasonably favorable currents, may entirely eliminate all trouble in this regard.

It is our opinion that at least a year of intensive study should be a prerequisite to the selection of any ocean outfall site; that the study should include a precise and accurate measurement of wind and tide induced currents during all stages of the tide in each season of the year. Additionally, the ocean floor should be thoroughly investigated to determine its consolidation and composition in order that the type of construction most appropriate to the location may be selected. These factors, coupled with existing data relating to the effect of depth, direction, quantity and velocity of discharge from the pipe outlet, should result in reasonably accurate knowledge of what may be expected from the outfall under all conditions.

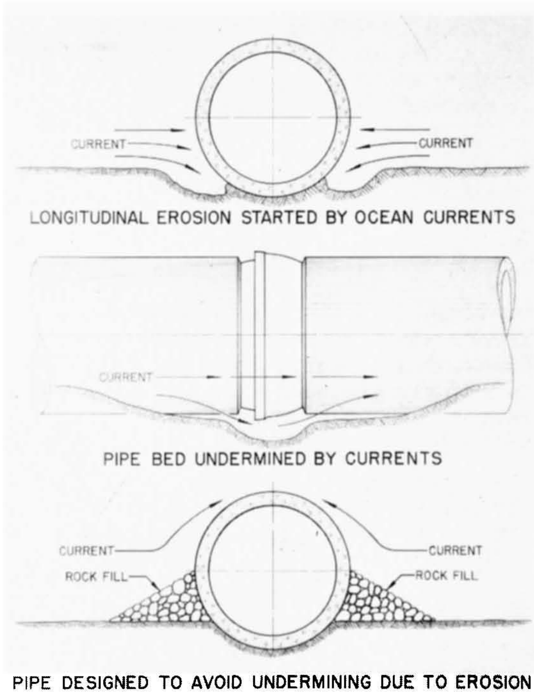


Fig. 6

One added observation about which little appears to be known, but which favorably influences the disposal of sewage into the sea, is that at depths of 100 ft. or so below the ocean surface the sea water is ordinarily not only quite cold but maintains a reasonably constant temperature throughout the year, while at the surface the temperature varies widely with the seasons. If the dispersion of the sewage into this cold sea water is carefully proportioned and controlled the rising column of mixed sewage and sea water at the point where the mixed liquor breaks the surface of the ocean will be a number of degrees colder and, consequently, of greater density than the surrounding ocean water in summer when recreational waters are most in use. The result of this is that the inertia of the rising column, having carried the mixed liquids to the ocean surface, is dissipated and the greater density of the mixed liquid causes it to plunge under the surface of the sea shortly after it begins to disperse laterally from the rising column. This phenomenon has not been carefully evaluated or rendered subject to measurement, but it is readily manifest because of the more frequent occurrence of shore contamination in the winter than in the summer.

CHAPTER 22

SEAWALLS AND BREAKWATERS

James R. Ayers
Head, Waterfront Structures
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Washington, D. C.

INTRODUCTION

Since the days of the Phoenicians and Egyptians, men have struggled to build harbor works capable of standing against the forces of the sea. Although the remains of Roman works have endured to the modern era, little progress in design was made until the early part of the last century. Modern developments have led to a better knowledge of wave pressures, but the principal source of guidance is still to be found by studying the causes underlying the disasters of the past.

This paper includes a brief outline of the principal structural types which have been built with varying degrees of success, a description of the results of certain model tests on a rubble mound breakwater, and a resumé of some of the most important lessons learned from the many failures which have occurred.

SEAWALLS

GENERAL

A seawall is a shoreline structure built for protecting and stabilizing the shore against erosion resulting from wave action. The design of seawalls is not susceptible to the degree of exactness which has been reached by the science of engineering in many other fields. The principle cause for this is the wide range in magnitude of the applied forces and the difficulties encountered in attempting to evaluate them. Since most seawalls are filled on the shoreside to a level approximating that of the top of the wall, resistance to wave force is provided by the mass of the wall and by the passive resistance of the backfill.

Frequently the seawall occupies such a position with respect to the high and low waterline that a wide expanse of beach and shallow water breaks the primary attacking waves at a distance seaward of the wall, in a succession of progressive steps. In such a case the attacking forces are due to breaking waves of greatly reduced height or to the onrush of water from broken waves. For locations with high tidal variations, seawalls are subject to a wide range in magnitude of the wave forces. At a certain tide stage, the forces may be those due to reflecting unbroken waves, whereas in other tide stages the full effect of breaking waves must be resisted.

These two types of wave action have long been recognized qualitatively. The theoretical basis for computing pressure due to reflecting unbroken waves has been developed by Sainflou (1928) and verified by several investigators. The existence of the second type of wave pressure, namely that produced by breaking waves, has become established as a result of several years of experimentation in wave pressure measurements by French and Italian investigators (De Rouville, Besson, and Petry, 1938). So far no theoretical method has been developed and accepted for computing pressures from breaking waves although Minikin (1950) has translated the model test work of Bagnold (1938-39) into a workable formula.

For a seawall project of sufficient magnitude, a model test is the most reliable and expeditious means of determining definite information concerning the behavior of the proposed design, subject to the attack of various assumed conditions of exposure. The prototype must be reproduced accurately in the model to achieve valid results.

As a consideration, second only to effectiveness and stability, the character of the property to be protected -- whether industrial, residential, or recreational -- should influence the selection of the type of wall and the architectural treatment of it.

TYPES OF SEAWALLS

The topography of the site, extreme tide range, wave characteristics, and foundation conditions will generally determine the type of structure to be built. Fig. 1 illustrates a version of the gravity section type of seawall. Walls of this type have been built to heights of 50 ft., with their bases extending to a distance of 30 ft. below extreme high water level. Early gravity section walls were of dry masonry construction. These were followed in succession by cut-stone or concrete blocks dowelled or keyed together. Still later, cut-stone facings set dry, or in mortar, backed with rubble concrete, or concrete, were tried and found to be a definite improvement. Modern practice is to make the structure as monolithic as possible, eliminating all openings, cracks, and irregularities in the facing.

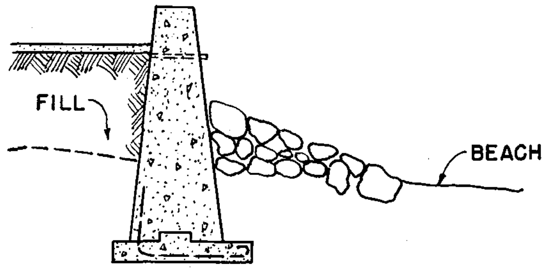

Fig. 1
Seawall -- vertical face.

Fig. 2 represents a minimum construction where poorer foundation conditions exist or where erosion of sand beneath the wall is likely. The sheet pile protection has a dual purpose, namely (1) prevention of erosion and (2) support of upper cantilever wall with continuity of bending strength. This wall is suitable for mild wave exposure. It is materially strengthened against vibration and settlement of the backfill by placing reinforcing steel in the paving slab adjacent to the wall and anchoring the slab to the wall.

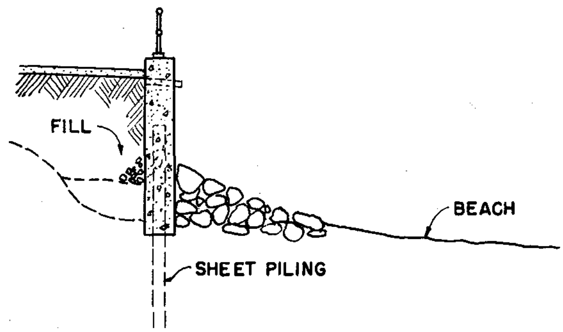

Fig. 2
Seawall -- vertical face.

The wall in Fig. 3 has a curved face. It is founded on piles as required by foundation conditions, and protected against erosion at the toe by sheet piles and rip-rap cover. This wall is suitable for locations having wide beaches with a relatively flat slope of the foreshore. It is effective under moderately severe wave action. In the design of curved face seawalls, the most satisfactory shape seems to be one where the wave path is turned upward at the beach surface and outward, just below the top of the wall. Experience has indicated that the curved face is not effective in turning waves whose height is sufficient to overtop the wall.

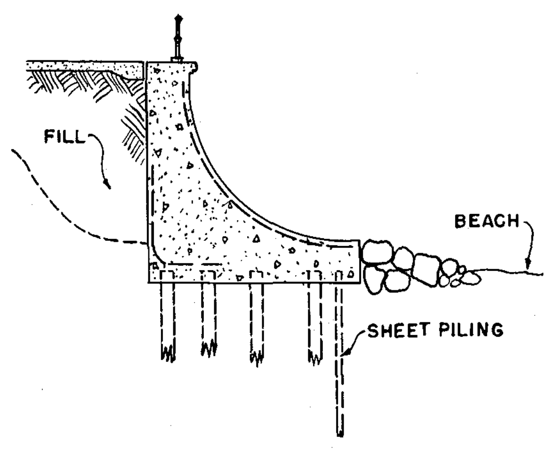

Fig. 3
Seawall -- curved face.

Fig. 4 is an example of a stepped-face seawall supported on piles and protected at the toe by sheet piling and rip-rap. This wall can be of relatively light construction and is suitable for moderate wave exposure. The stepped-face wall avoids the excessive shock pressures from wave action by forming eddies and air pockets which act as cushions to dissipate the wave energy in a series of successive stages.

The seawall shown in Fig. 5 is an example of a combination where the waves are dissipated to some extent on the set of inclined steps, and any higher wave motion is turned upward and outward by the curved face of the upper portion. This type is

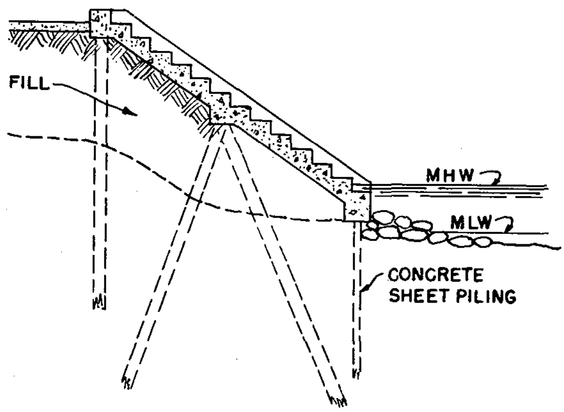

Fig. 4
Seawall -- stepped face.

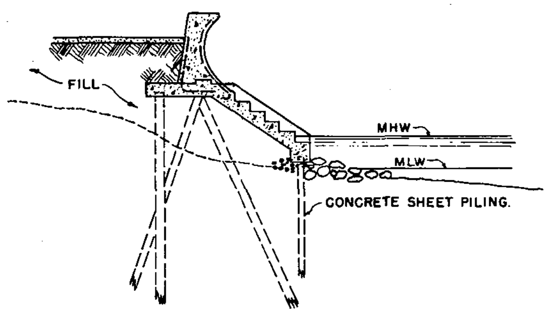

Fig. 5
Seawall -- combination.

particularly suited to locations where the forewhore is narrow and the wave attack moderate. It is adaptable to a wider tidal range than either the curved or stepped-face used alone.

CAUSES OF FAILURE

In the early days, when most seawalls were of the gravity type, the principal cause of failure was dislocation of the stones comprising the wall, followed by the washing out of the backfill and the resultant complete failure of the section. As seawalls have become more monolithic in construction, the principal cause of failure has been due to undermining of the toe or to the development of excessive hydrostatic pressure behind the wall. The latter produces excessive toe pressure followed eventually by settling, tipping and outward wall movement.

Failures have occurred involving modern type reinforced concrete seawalls due to lack of proper cut-off walls or rip-rap protection for the toe. Failures due to the impact of falling water behind the wall have eroded the backfill to such an extent as to leave the wall without benefit of horizontal support against the attacking wave forces. In other instances, the construction of a seawall has altered the natural forces in such a manner as to result in the erosion of the foreshore to a depth of several feet in front of and adjacent to the wall.

PROTECTIVE MEASURES

Experience has demonstrated the necessity for protecting certain vital points, which are most vulnerable to wave attack. The most effective primary protection is the provision of rip-rap of adequate size and extent to prevent the back wash of receding waves from eroding the foreshore. For locations with firm bottom, this may be sufficient. For locations with soft or sandy bottom, sheet piles of adequate length are required to prevent loss of material beneath the wall itself. Adequate toe protection is the most important single precaution which may be taken to prevent overturning of the wall seaward, although proper drainage of the backfill to prevent the development of serious hydrostatic pressure differentials must not be overlooked.

Since the principle resistance to the oncoming wave force is the passive pressure of the earth backfill behind the wall, it follows that erosion in this region must be prevented. The wave attack must be broken sufficiently to prevent throwing of large quantities of water into the air to fall behind the seawall. This is one of the principle reasons for using a stepped-face wall instead of a comparable vertical face. Although a stepped-face wall may be subject to greater wave force, as long as the passive resistance of the backfill is not reduced by erosion, the wall has adequate and lasting stability. Paving over the backfill is an effective means of preventing erosion of the filling material. Adequate protection of the backfill against erosion is the best insurance against overturning of the wall shoreward.

After completion of a seawall, the necessity for groins or other additional foreshore protection should be determined by periodic checking of the foreshore profile.

MAINTENANCE

A seawall is not a type of structure which may be built and left to perform its function for a long period of time without the necessity for frequent inspection and maintenance. Even after the knowledge of wave action and wall behavior has progressed much farther than at present, there will be the need for constant vigilance to detect and correct weaknesses after severe storms, which usually occur at first in the form of erosion. The best knowledge now available cannot always predict with certainty just how a wall built at a certain location along a particular alignment will alter the natural forces. Equilibrium may be reached only after the occurrence of several typical storms and the adjustment and replenishment of the foreshore protection. Seawalls built in accordance with knowledge now available, and maintained consistently, can be reasonably expected to have a long useful life.

BREAKWATERS

In contrast to seawalls, just discussed, which protect a shoreline with the benefit of continuous lateral support from backfill on the shore side, a breakwater is a free standing structure, located in varying depths of water, providing the primary protection for a harbor from the direct action of waves. Where these structures extend into deep water, they are subject to the full fury of the largest ocean waves occurring at the particular location.

GENERAL

The degree of exposure at a given site is a function not only of the general geographical location with respect to possible wave action, but also of the local hydrography and topography. These include the water depth at the structure, the slope of the bottom, and the tidal range.

The earliest breakwaters were unformed piles of stone of a size that could be handled with the limited equipment available at the time. It soon became evident that the sea slopes were not adequate or the stones of sufficient size to resist the forces delivered by storm waves. Heavy wave action lowered the top of the mound and flattened the seaward slope. It was necessary to constantly replenish the mound until an equilibrium slope was reached. This slope was often found to vary from 1 on 5 to 1 on 10 on the seaward side within the range of the worst attack. Below this level, the slope to the bottom was often as steep as 1 on 1.

The portion of the mound above low water is extremely vulnerable to injury by storm waves in either one or both of two different actions. The first is the raising and forward transport of the stone by the incoming waves. The second is the withdrawal and lowering of the stone during the back wash or recoil.

TYPES OF BREAKWATERS

Rubble mounds have been fashioned in an almost endless variety of cross-sections. In nearly every case, the original shape has been altered by heavy storms after which reshaping and replenishment of stone has been necessary in the damaged areas. An example of a modern type of mound breakwater is shown in Fig. 6. The large mass of stone is so arranged that the smaller sizes, forming the lower central portion of the core, are protected by the larger stones forming the exterior slopes and the upper portion, the latter being most severely exposed to direct wave action. The relatively large volume of Class B stone indicated is to provide adequate stability during construction.

A mound of rubble stone is indicated where there is an abundant supply of rock available. It is particularly adapted for locations with small tidal range and in depths of water, up to perhaps 60 ft. It has the advantage that storm damage or vertical settlement due to a poor foundation site may be repaired by renewing or replacing the dislocated stone.

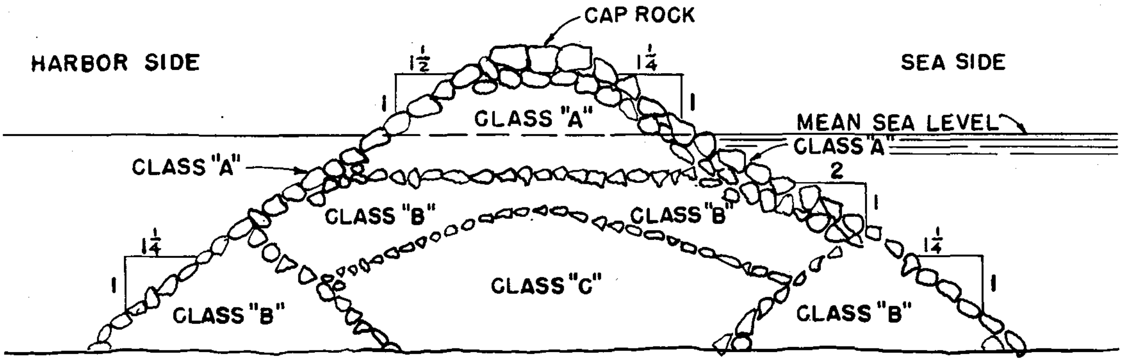

Fig. 6. Breakwater -- rubble mound.

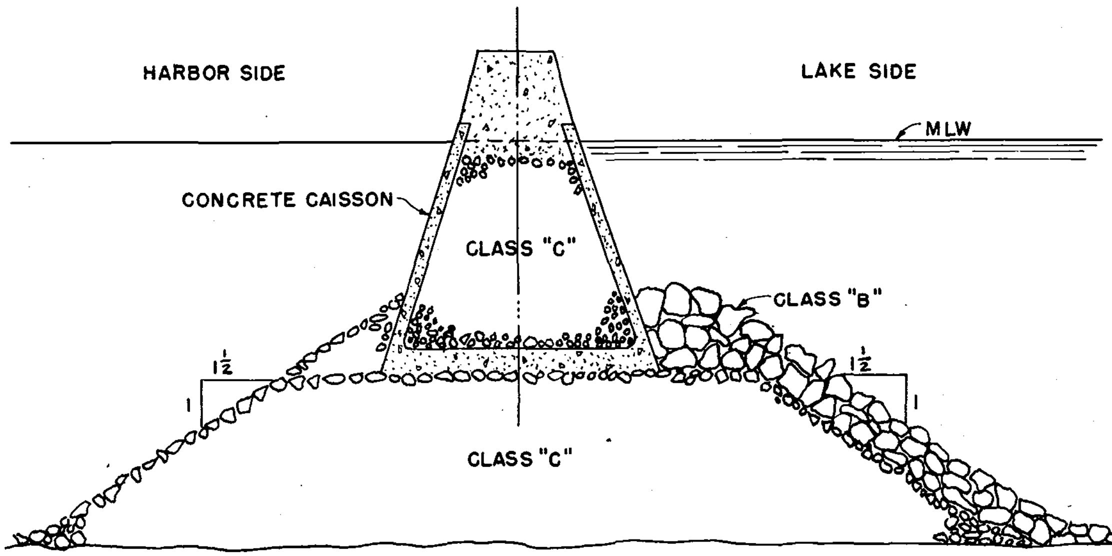

Fig. 7. Breakwater -- composite.

A composite type of breakwater is shown if Fig. 7. For deep water sites and at locations having large tidal variation, the quantity of stone required for a full height rubble mound is not economically feasible. Such a condition gives rise to combinations of rubble bases and various types of superstructure. Here the rubble mound provides the base which accommodates itself to the irregularities of the sea bottom, and may be deposited in deep water and allowed to stand for the purpose of obtaining a large part of the total settlement before placing the superstructure. Composite breakwaters of this type may be divided into two classes, namely those with superstructure founded at low water level, and those whose superstructure extends sufficiently far below low water to avoid the breaking of storm waves and disturbance to the rubble base. The class with superstructure founded at low water, most of which were built prior to 1900, has been located at sites having a great range of tide.

Vertical-face breakwaters have been used extensively in Europe with varying degrees of success. Fig. 8 is such an example. Many arrangements of blocking have been tried. The usual practice is to set the blocks in horizontal courses with joints crossing in all directions, or suitably keyed and dowelled together. This construction has been varied, where differential settlements were expected, by trimming the blocks in inclined layers whose slope is about 70 to 75 degrees with the horizontal. Blocks weighing up to 410 tons and extending throughout the full wall thickness of 12 meters have been used in the construction of some of the more modern vertical-face walls.

A modified vertical-face breakwater is shown in Fig. 9. The lower portion is a concrete caisson-type structure, built of prefabricated units and sunk into position on a prepared sea bed. After sinking, the interior is rapidly filled to water level with sand or gravel, and covered with a protective stone blanket. After initial settlements have occurred, openings between caissons are filled with concrete and a monolithic cap structure is cast on top. The stepped upper monolith will reduce the height and rise of the waves which would otherwise occur at a vertical face, at the expense of greater wave force against the breakwater. Therefore the stepped capping may be of reduced height as compared to a vertical-face superstructure for the same degree of harbor disturbance resulting from overtopping.

The efficacy of this design has not been definitely established. Model studies to determine the total force and height of wave rise against a vertical-face breakwater, compared to one modified at the top as shown, would establish the relative merit of the respective designs.

Figs. 10a and 10b illustrate a type of steel sheet pile breakwater adapted to fresh water sites and moderate seasonal wave disturbance. Numerous examples of this construction are found in the Great Lakes. The structure is vulnerable to

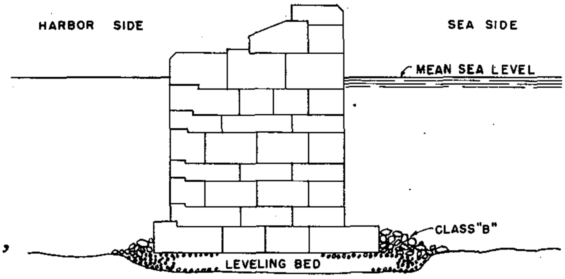

Fig. 8
Breakwater -- precast concrete or stone block -- vertical face.

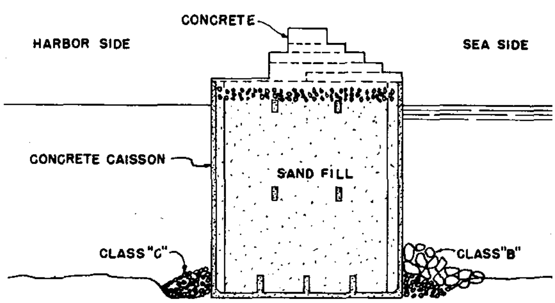

Fig. 9
Breakwater -- full caisson (modified vertifical face).

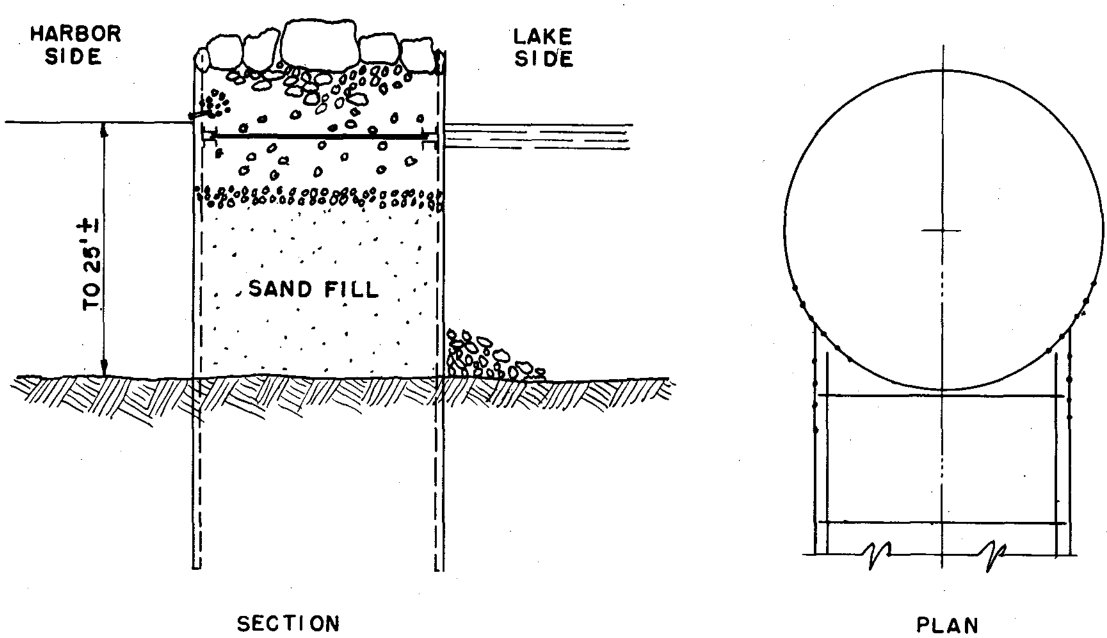

Fig. 10a. Breakwater -- steel sheet pile straight wall type.

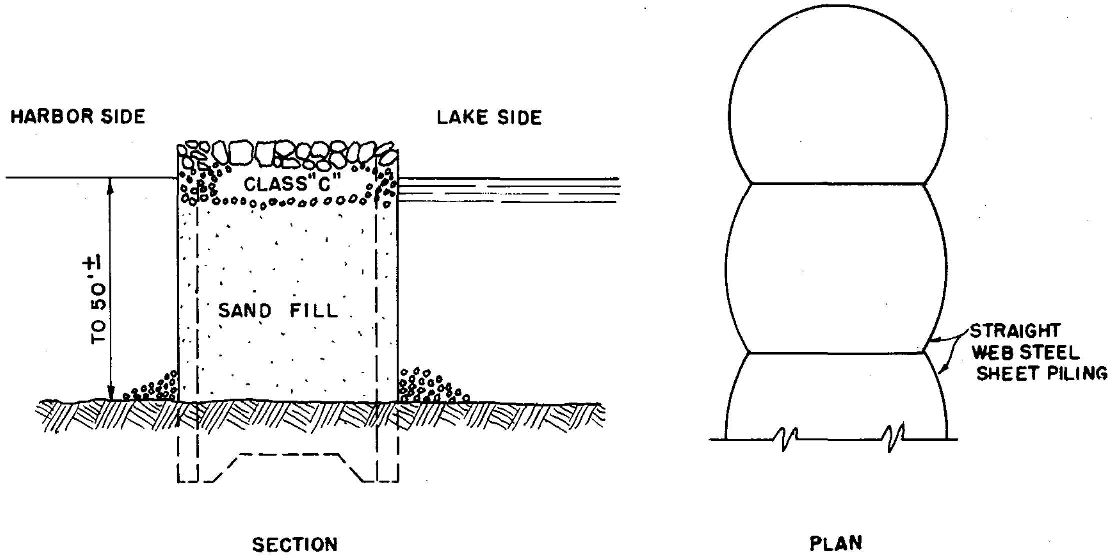

Fig. 10b. Breakwater -- steel sheet pile circular type.

storm damage before filling of the cells during construction, but this can be minimized by proper sequence of building operations.

Many other types of breakwater have been proposed and tried. Among them are pneumatic, or air-bubble breakwaters, floating breakwaters of both vertical and horizontal extent, and submerged barriers. So far as is known, no breakwater installations based on these principles have proven successful in the prototype.

WAVE PRESSURES

Many investigations have been made to determine the magnitude of wave forces against fixed objects. At present, two general types of wave pressure are recognized, namely first that due to reflected waves and second -- that due to breaking waves. Fig. 11 gives the general shape of these two types of pressure diagram. The methods of Sainflou (1928) and Molitor (1935) refer to a form of reflected

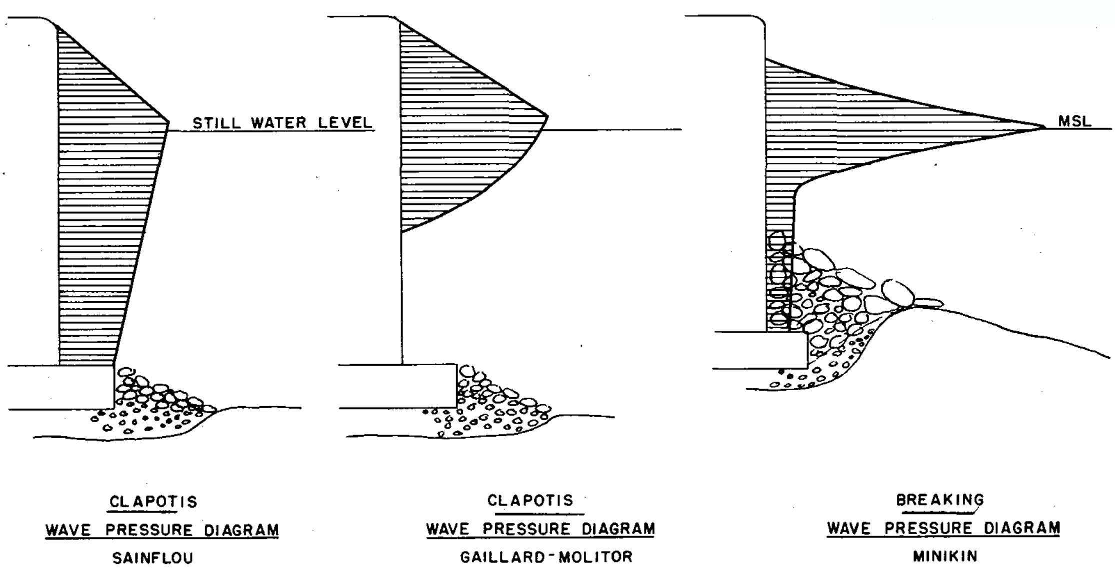

Fig. 11

waves, usually called a clapotis. The methods employed by Lira (1935) closely approximate the analytical solution of Sainflou (1928) while hydraulic model tests conducted in the University of Lausanne, reported by Cagli (1935-36), substantiate to a marked degree the validity of Sainflou's analysis. The method of Minikin (1950) refers to breaking waves and clearly shows the high intensity of pressure developed in the vicinity of mean sea level. The total wave force and overturning effect on a given breakwater are materially increased for the case of breaking waves. Further experimental effort in measuring wave pressures and translation of the results into usable form is most desirable.

MODEL TEST OF RUBBLE-MOUND BREAKWATER

The many uncertainties attending the study of wave pressure and its effect on breakwaters has led to a search for other methods of investigation. In recent years, the success of model testing in other fields has suggested the use of this tool to the problem of breakwater stability. Accordingly, the Bureau of Yards and Docks has sponsored a testing program at the Waterways Experiment Station, Vicksburg, Mississippi. The effort has been concentrated on two aspects of the breakwater problem, namely the stability of component materials during various stages of construction and after completion of a breakwater, and the relative stability of stones of varying size and density.

STABILITY OF MATERIALS DURING CONSTRUCTION STAGES

For the purpose of the model study, the ranges of stone weight in the various classifications were as follows:

Class A Stone

Percent of total	Prototype Weight
75	10 - 12 ton
20	3 - 9 ton
5	1 - 2 ton

Class B Stone

15	2 - 4 ton
30	1 - 2 ton
15	100 - 1000 lb
10	50 - 100 lb
5	20 - 50 lb
5	10 - 20 lb
10	5 - 10 lb
5	1 - 5 lb
5	less than 1 lb

Class C Material

50	0.50 - 1.00 lb
50	0.25 - 0.50 lb

The first tests were performed on models of partially completed breakwater sections representative of the various stages of construction on a prototype breakwater. Each tested condition of the model was subjected to wave attack until stability of erosion and displacement had been reached. These tests were limited to the water depth prevailing at the location of the proposed prototype, namely 58 ft. Specifically, it was desired that the model should yield information of value on the following points:

1. The height to which the Class C material could be constructed without being displaced by wave action before the protective covering (Class B) was placed.
2. The advantages to be gained by placing the Class B stone concurrently with the placing of the Class C core material.
3. The amount of covering stone (Class B) necessary to protect the core material (Class C)
4. The general stability of the completed breakwater section.

Class C material -- unprotected. In testing the stability of the Class C material, four different partial cross-sections representative of four stages of construction in the prototype, were used. These test sections had top elevations of - 49 ft., - 38 ft., - 29 ft., and - 24 ft., all referred to mean sea level. The model breakwater was subjected to waves of four sizes as follows:

Height	Length	L/H Ratio
7.5 ft.	210 ft.	28.0
10.5 ft.	210 ft.	20.0
15.0 ft.	270 ft.	18.0
21.0 ft.	300 ft.	14.3

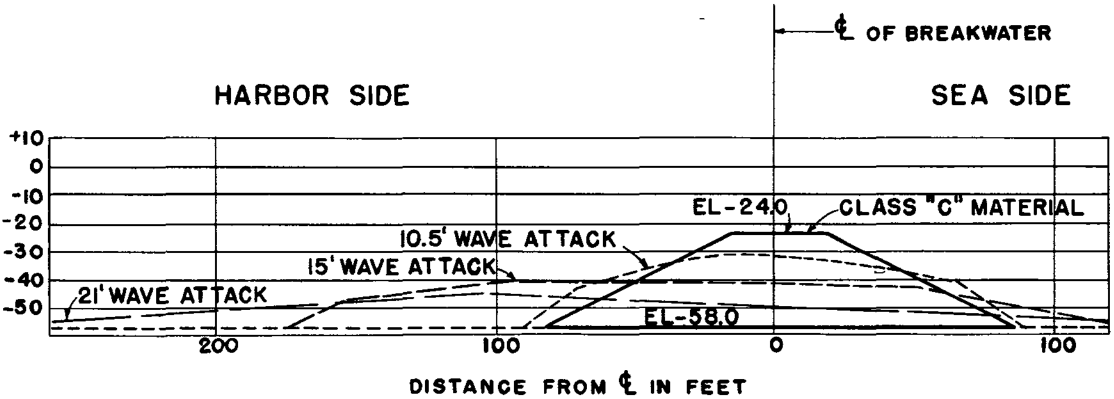

Fig. 12. Displacement of breakwater material by wave action.

Fig. 12 is typical for the tests of the C material without enrockment, with top elevation at - 24 ft., and indicates the outline of the damage to the mound by waves 10.5 ft., 15.0 ft. and 21 ft. high. Fig. 13 indicates the heights to which

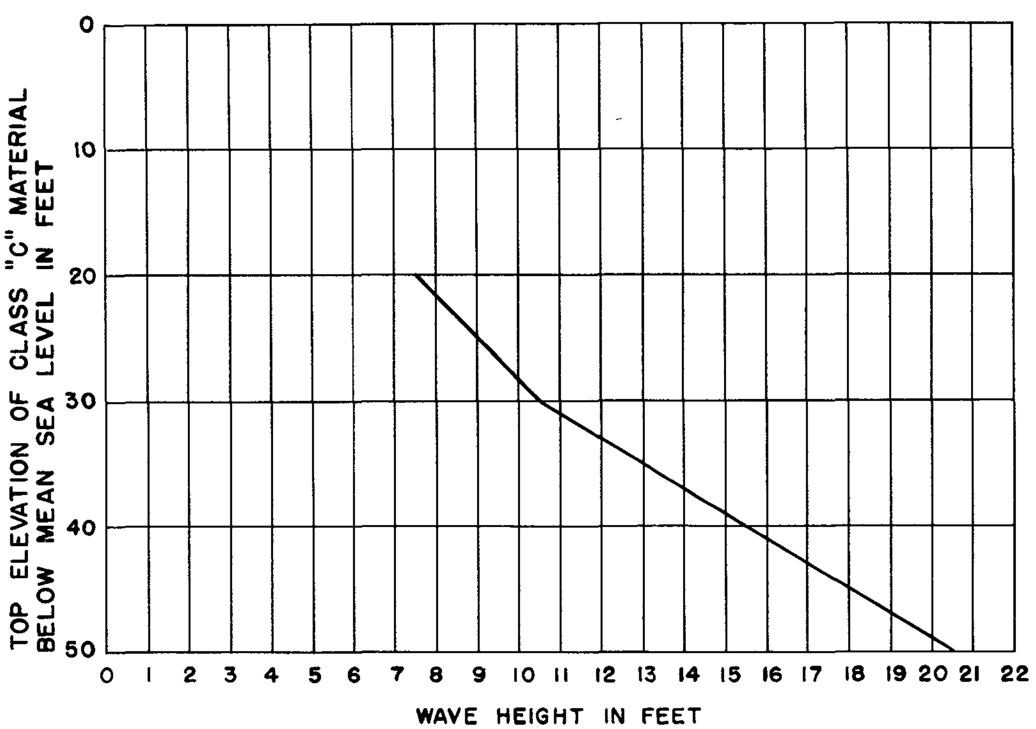

Fig. 13. Stability of class "C" material

the Class C material may be placed in 58 ft. water depth, without displacement of the material outside the design limits, for various wave heights. It will be noted that the wave heights and corresponding maximum top elevations are as follows:

Wave height	Maximum top elevation of unprotected Class C material
7 to 8 ft.	- 20 ft., mean sea level
10 to 11 ft.	- 30 ft.
15 to 16 ft.	- 40 ft.
20 to 21 ft.	- 50 ft.

Class C material with Class B stone as toe protection on one side only. Two series of tests on each of three partial breakwater sections having top elevations of 38 ft., - 29 ft., and - 24 ft., mean sea level, were made. One series had Class B stone protection on the harbor side only; the other series had protection on the seaward side only. A typical illustration of the results of the first series is shown on Fig. 14. It appears that there is no particular advantage in

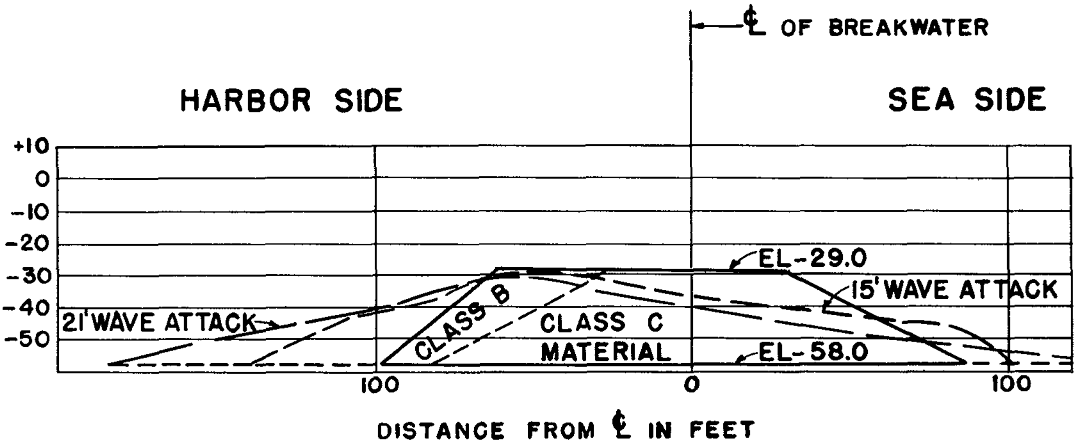

Fig. 14. Displacement of breakwater material by wave action.

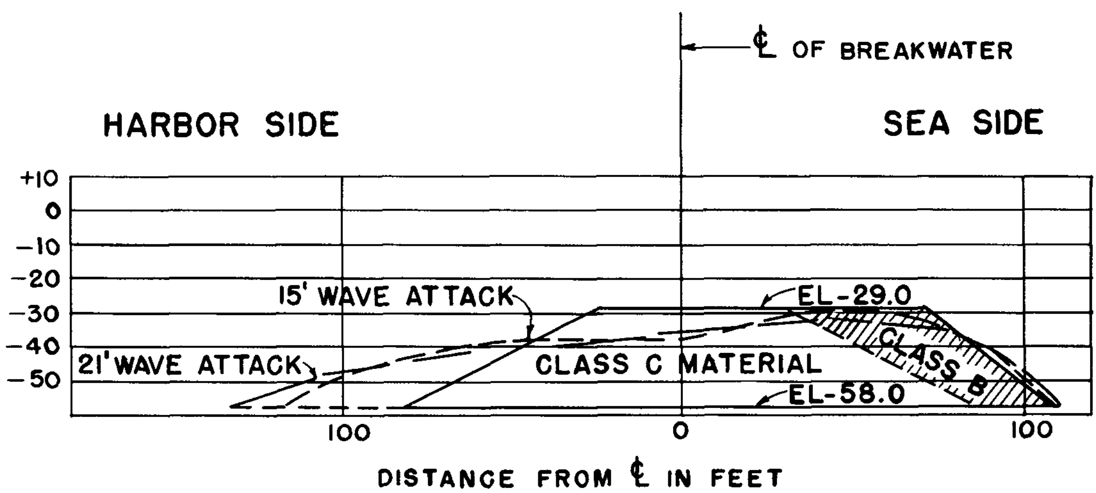

Fig. 15. Displacement of breakwater material by wave action.

adding toe protection on the harbor side only. The waves carried the unprotected Class C material over the Class B material to such an extent that no great saving could be realized by use of this method. Fig. 15 is typical for the results of the second series, where Class B protection is provided on the seaward side only. The damage is very similar in type and extent to that of Fig. 14, indicating no advantage over placing the Class B stone on the harbor side only and, for all practical purposes, no advantage over placing the Class C material without toe protection.

Class C material with Class B stone as toe protection on both sides. Fig. 16 shows typical results for tests of partially completed sections with toe protection on both harbor and seaward slopes. For sections of lower elevation, there was considerable displacement of Class C material due to the extensive area of this material exposed to the action of the waves. The resulting scour was concave in shape, with the deposition of the displaced material greatest on the harbor slope. As the top elevations of the sections were raised, the exposed area of the Class C material was decreased, and the displacement of material became progressively less. As a result, there was practically no displacement of the Class C material for the tests of the section with a top elevation of - 24 ft., even during the 21.0 ft. wave attack as shown on Fig. 17.

From a study of these tests, it is concluded that the greatest degree of safety with respect to displacement of materials due to wave attack, is obtained by placing the Class B stone on both landward and seaward sides simultaneously.

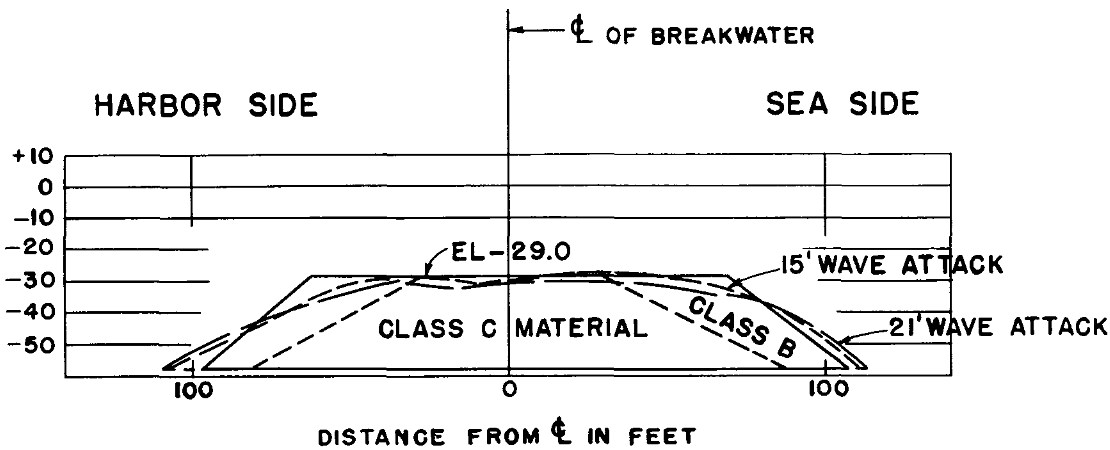

Fig. 16. Displacement of breakwater material by wave action.

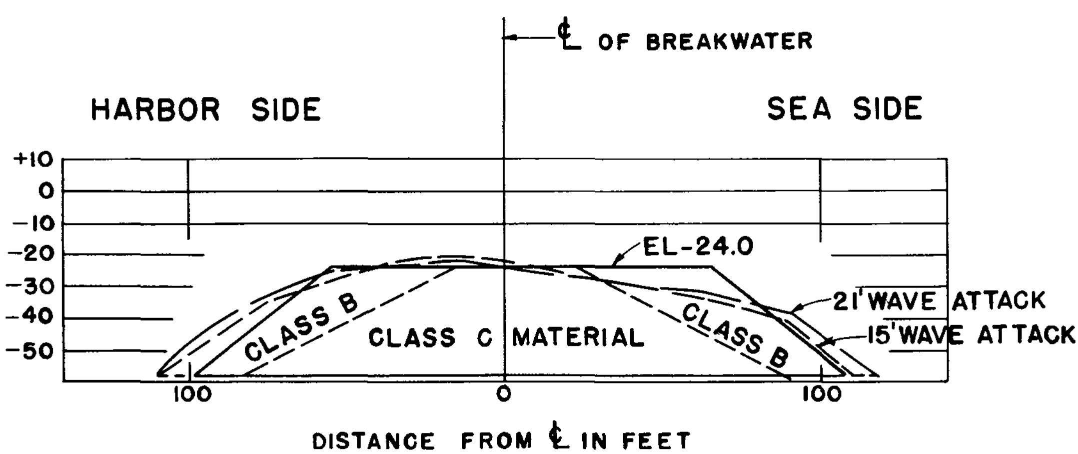

Fig. 17. Displacement of breakwater material by wave action.

At lower elevations, the sections would be endangered in severe storms, but the damage would not be entirely detrimental, as the Class C material displaced would be washed over the Class B protection on the harbor slope where it would not interfere with the future placing of materials. A distinct advantage results from the fact that as the sections are raised in elevation, the area of the Class C material exposed to wave action becomes smaller, thus reducing the displacement.

Completed Class B section. The completed Class B section extending to elevation - 10.0 ft. would not stand the attack of 15 ft. and 21 ft. waves. A prototype breakwater likely to sustain exposure to waves higher than 10 ft. during construction would have to be built with flatter slopes than those chosen for this breakwater.

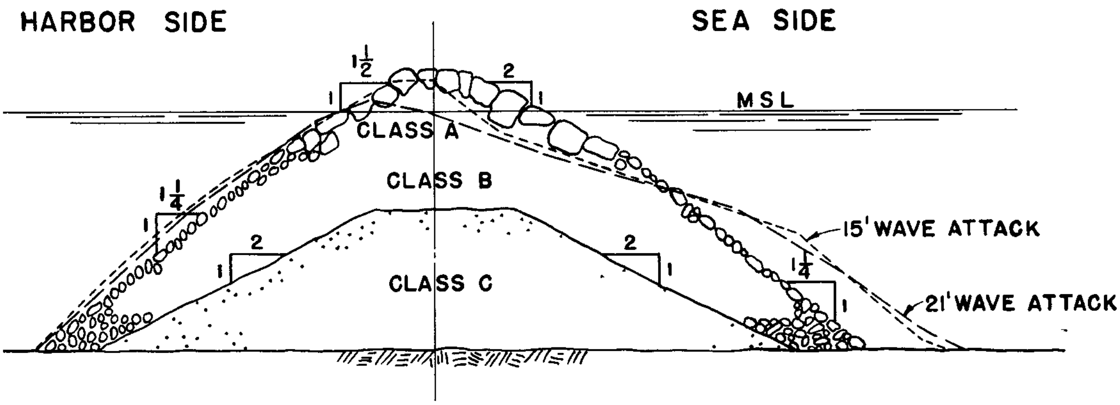

Fig. 18. Breakwater -- rubble mound (model test of stability).

Complete breakwater section. Fig. 18 shows the results of the tests on the complete breakwater section. Minor damage was suffered from the attack of 10 ft. waves, but the breakwater failed to maintain its design section under attack by 15 and 21 ft. waves. Thus for prototype locations where severe storms occur, the seaward slope should be flatter, with larger cap stone, or the top elevation of the Class B stone and Class C core material should be lowered to about - 20 ft. and - 30 ft. mean sea level, respectively, thereby increasing the amount of Class A stone.

RELATIVE STABILITY OF STONES OF VARYING SIZE AND DENSITY

After conclusion of the tests just described, the Bureau of Yards and Docks has sponsored a continuation of the testing program seeking an empirical formula for determining the weight of cap rock required to withstand design waves of various sizes, beginning with an experimental check of the accuracy of the Iribarren formula (see Chapters 23, 24, and 26). The water depth chosen was 90 ft., with a range in size of cap rock from 4-1/2 to 27 tons, wave heights from 5 to 31 ft., wave periods 5 to 13 sec., and side slopes of 1 on 1-1/4, 1 on 1-1/2, 1 on 2 and 1 on 3, specific gravity of stone 2.3 to 2.8. The results of these tests so far have not been completely analyzed. The indications are that the range of conditions covering the design of rubble breakwaters is so wide that separate formulas, or perhaps separate curves for corrective coefficients will be necessary to cover the conditions of (1) no waves overtopping the mound (2) varying depths of wave overtopping.

It is realized that the information gained from this set of tests is very limited, as it applies only to one depth of water and one cross section of prototype breakwater. No quantitative information was obtained regarding the wave forces, as the tangible results appear only in terms of amounts of damage to the section tested by a particular wave. Yet these are indicative of one means of approach through a relatively new medium of controlled study.

BASIC LESSONS FROM EXPERIENCE

The lessons learned from experience provide the principal source of present knowledge with respect to breakwater behavior. These lessons are all the more to be respected, as each one has been gained only at the expense of a total or partial failure of many actual structures.

The height and length of waves assumed for design should be sufficiently large to allow for exceptional storms as yet unknown to the locality.

Stones and blocks should be of adequate size; the smaller the stone, the flatter the sea slopes required for stability.

The destructive influence of the sea extends to considerably greater depths than was originally thought. The core protection must be of sufficient size and extent and carried far enough below water to prevent withdrawal of the smaller core material.

In breakwaters of composite construction, with rubble base and vertical wall, the top of the mound should be located sufficiently far below mean low water to prevent the breaking of the largest waves. The base of the superstructure should be protected by heavy blocks, or rubble, on the benching seaward of the breakwater.

In the case of easily erodible bottom material, a protective blanket, covering the bottom for a considerable width in front of the outer foot of the work, should be provided especially in shallower water.

Superstructures with exposed open joints are susceptible to severe damage from falling water and the pressure of trapped air.

Vertical face breakwaters should not be built in water of insufficient depth to maintain oscillatory wave motion. Those founded at the sea bed should be located in water at least twice the height of the greatest storm waves. Unless the material of the sea bottom is of a firm or rocky nature, an extensive rubble foundation is necessary to protect the sea floor from erosion for a considerable width in front of the toe.

CONCLUSION

The rate of progress in the science of breakwater design has been slow. Much remains to be discovered, especially in the realm of quantitative expressions for many of the combinations of primary variables. Methods of wave measurement and forecasting will do much to reduce some of the uncertainties of the past.

The science of testing by use of accurately scaled models, where the variables may be rigorously controlled singly and in groups, promises to become the most effective tool yet developed, not only for checking the stability and behavior of a given design, but also for leading the way to more perfect methods of breakwater analysis.

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CHAPTER 23

THE HYDRAULIC MODEL AS AN AID IN BREAKWATER DESIGN

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INTRODUCTION

The complexity of wave-action phenomena and the complicated geometry of most harbors usually make it impossible to obtain adequate answers to design problems by an analytical approach. Except for the most elementary wave-action problems, the general experience and judgment of the engineer cannot be utilized effectively. Indeed, this method of approach to harbor problems has led to designs which not only failed to alleviate wave-action conditions, but, instead, actually intensified them. It is the intent of this paper to show how the small-scale hydraulic model can be an effective tool in the planning of harbor development and in the design and layout of breakwaters to provide protection from wave action.

TYPES OF WAVE-ACTION MODELS

Wave-action models can be divided into two general types: (1) harbor models, and (2) breakwater stability models. Harbor models usually reproduce the entire harbor area and the various shore-line structures, together with sufficient area seaward to allow proper generation of waves. This type model is used to determine the best solution to wave- and surge-action problems involving the selection of the most efficient type, length and alignment of breakwaters, the location, alignment, shape, and width of navigation openings, the proper location and type of piers and spending beaches, and the effects of proposed dredging projects. Thus, the harbor model has to do with the solution of problems involving the effects of contemplated changes in static boundary conditions on wave reflection, refraction, diffraction, and attenuation.

Breakwater stability models are useful in selecting the most efficient design of breakwaters with respect to the forces imposed upon them by wave action. This type model is used to determine the shape and magnitude of wave pressure curves on impervious, vertical and inclined walls, the stability of caissons and cribs, and the proper size and density rock, degree of slope, crown elevation and cross-sectional shape of rubble-mound breakwaters.

DESIGN OF WAVE-ACTION MODELS

In nature, surface wind waves and long-period, surge waves are propagated by the restoring force of gravity and are classified, therefore, as gravity waves. The forces of surface tension and friction are not usually significant for nature waves and can be ignored. Waves in small-scale models are gravity waves also, and the model is designed, operated, and the test results converted to prototype values on the basis of transference equations derived from Froude's model law. Further, the model must be constructed geometrically similar to its prototype if dynamic similarity is required. After the linear scale (L_r) has been selected, based on considerations to be explained in subsequent paragraphs, the remaining scales can be derived from Froude's number and simple geometrical principles. For an undistorted model, these scales are shown as follows:

Characteristics	Dimensions*	Scales, Model to Prototype	Characteristics	Dimensions*	Scales, Model to Prototype
Area	L^2	$A_r = L_r^2$	Force	F	$F_r = L_r^3\gamma_r$**
Volume	L^3	$\bar{V}_r = L_r^3$	Weight	F	$W_r = L_r^3\gamma_r$
Time	T	$T_r = L_r^{1/2}$	Unit pressure	F/L^2	$P_r = L_r\,\gamma_r$
Velocity	L/T	$V_r = L_r^{1/2}$	Energy	FL	$E_r = L_r^4\gamma_r$

*In terms of force, length and time.

** γ_r is the unit weight scale of the liquid.

Although the force of gravity predominates in the propagation of nature waves, this may not be true for model waves unless the linear scale of the model is chosen judiciously. If this scale is too small, the forces of surface tension and friction may become of such magnitude, in relation to the force of gravity, that accurate reproduction of wave phenomena will not be obtained. In harbor wave-action studies the wave characteristics which must be reproduced accurately are wave patterns and corresponding wave amplitudes. Since the d/L ratio affects wave velocity, and thus wave patterns and amplitudes, over a considerable range of this ratio, it is not possible to distort the linear scale of this type model except in special cases.

Distorted-scale harbor models can be used if the waves in nature are either deep-water waves or waves of translation. The degree of distortion allowable is limited to this extent: If the waves are deep-water waves in nature, they must be deep-water waves in the model. Similarly, if the waves in nature are waves of translation, they must be waves of translation in the model. In general, short-period wind waves can seldom be studied by use of distorted-scale models, whereas in the case of long-period surge waves, scale distortion, if not too great, will not affect the accuracy of model results. Fig. 1 shows the effects of surface tension, internal friction, boundary friction, and scale distortion on the propagation of waves. These curves, the equations for which can be found in theoretical hydrodynamics texts (Lamb, 1932) and in papers by Keulegan (1950a, 1950b), are very useful in designing small-scale harbor models. Fig. 1(a) and 1(b) show that surface tension and internal friction are not critical and will not affect the selec-

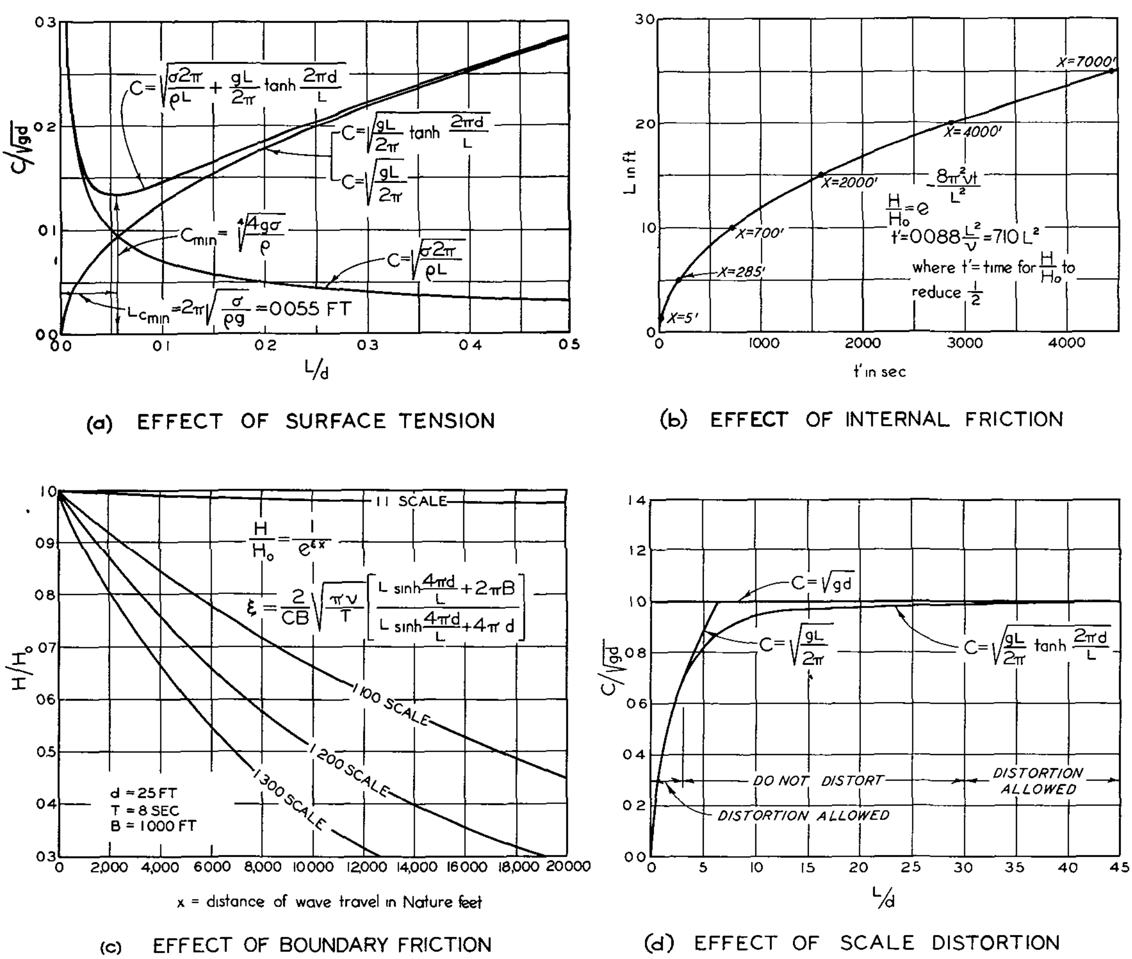

Fig. 1. Design of wave action models.

tion of scale unless very large prototype areas are to be reproduced in a very small scale model. Fig. 1(c) and 1(d) show, however, that boundary friction and scale distortion can affect appreciably the accuracy of model results, and these factors should be given careful examination before the scales of harbor wave-action models are selected. In breakwater stability models it is necessary to reproduce the dynamic characteristics of the waves and the resisting forces of the breakwater material. Therefore, scale distortion is not possible in these models.

Economical considerations require the selection of the smallest model scale consistent with the desired accuracy of model results. The degree of accuracy which can be obtained by use of scale models is determined by the laws of similitude, as explained above, the exactness of model construction and the accuracy with which the characteristic wave dimensions can be generated and measured. Therefore, these considerations, tempered by the funds available, determine the model scale selected.

TYPICAL HARBOR MODELS

The most common types of harbor models are those having to do with long-period surge waves and those in which the principal problem is caused by short-period wind waves. An example of the former is the model study of Long Beach Harbor, which was performed by the Waterways Experiment Station (1949c) for the City of Long Beach, California. Since this study was described by Thorley (1950), it will not be discussed in detail here. The main problem concerned the effects of contemplated pier extensions on surge action in the existing Long Beach Harbor and selection of the best plan for the proposed additional harbor basin immediately southeast of Long Beach Harbor. An existing distorted-scale model (horizontal scale 1:300 and vertical scale 1:60) of the San Pedro Bay area was utilized for this study. It was not possible to develop an ideal solution to the problem at Long Beach because of the wide spread in the spectrum of waves existing in the San Pedro area, which made it impossible to eliminate entirely the conditions of resonance between the periods of the waves and the natural oscillating periods of the basin waters. The most critical periods of resonance for each inner-harbor basin were determined by frequency-response tests in which the magnitudes of disturbances inside the basins were compared with those of the exciting wave trains for periods up to about six minutes. Waves with these periods were then generated in the model from various directions outside the outer breakwater, and the modes of oscillation in the outer harbor which resulted in the worst conditions in the inner basins were determined. The inner-harbor basins studied in this manner were the existing Navy-Long Beach Harbors and the southeast-basin harbor proposed by the City of Long Beach. Since the existing Navy-Long Beach Harbor is known to be very satisfactory with respect to long-period surge action, the results of new plans were analyzed by comparison.

A typical example of a harbor model concerned with the effects of short-period wind waves is that of Oswego Harbor, Lake Ontario, New York (Waterways Experiment Station, 1949a). The purpose of this model was to determine whether the proposed plan of harbor improvement would be adequate to protect the harbor from wave action, and, if it were not, to devise a plan which would provide sufficient protection at minimum cost. The study was conducted in a concrete model geometrically similar to its prototype with an undistorted linear scale of 1:100. As can be seen in Fig. 2, the harbor is exposed to wind waves generated by storms from all directions between west and northeast. Storms occur most frequently from the directions west to northwest, where fetches, wind speeds, and resulting wave heights are greatest. Because of the harbor's physical layout, however, storms from the area north-northwest to north-northeast are the most critical. The large waves from the west are refracted harmlessly onto a spending beach inside the harbor. The deep-water dimensions of model-test waves were selected by applying Sverdrup-Munk curves (Arthur, 1948) to prototype wind data; these test waves were then charted into the various wave-machine positions by refraction diagrams (Johnson, O'Brien, and Isaacs, 1948). Various lengths and alignments of breakwaters were tested. The existing arrowhead breakwater and an arrowhead wave-trap form of detached breakwater were found to be ineffective against northerly waves, due to a submarine ridge in the outer harbor which caused convergence of wave orthogonals at the navigation opening.

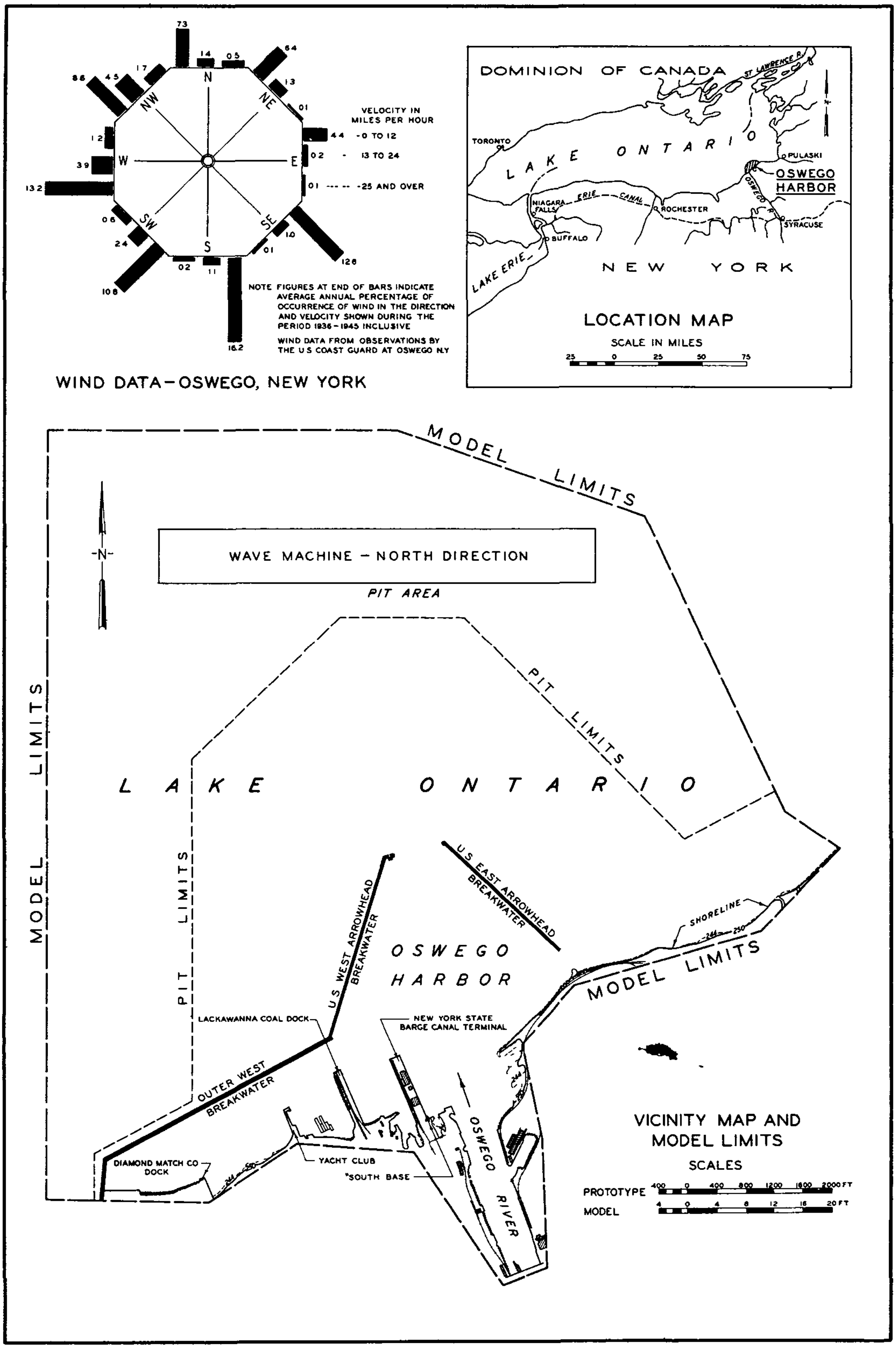

Fig. 2. Model study of Oswego Harbor

It was concluded from the results of the model study that: (1) the originally proposed plan would not have been adequate and, instead, would have intensified wave action conditions in the harbor; (2) a breakwater plan developed during the model study, similar to the originally proposed plan except for slight differences in length and alignment, would be satisfactory; (3) an artificial spending beach should be constructed at the southeast corner of the New York State Barge Canal Terminal; (4) an alternate breakwater plan developed during the model study would afford more effective protection from wave action than any of the other plans, but the cost of constructing this plan might be prohibitive; and (5) the existing impervious, vertical-walled wharfs in Oswego Harbor magnify the action of waves that gain entrance into the harbor through the navigation opening, making it desirable to avoid this type of construction for additional wharfs or other harbor structures unless the structures are to be located in harbor areas amply protected from wave action.

A TYPICAL BREAKWATER MODEL

A 1:30-scale model study was performed recently at the Waterways Experiment Station to determine the relative stability and maintenance costs of two types of rubble breakwaters proposed for use at East Beaver Bay Harbor, Lake Superior, Minnesota (Waterways Experiment Station, 1949b). Details of these breakwaters are shown in Fig. 3. Economy prescribed end and side dumping of rock quarried at the breakwater site. This in turn required stability tests to determine the most efficient breakwater section.

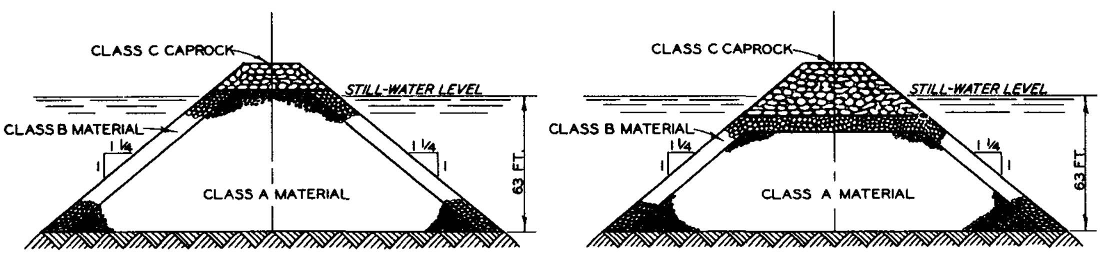

Fig. 3
Breakwater designs tested in model study of breakwater stability, East Beaver Bay Harbor

A comprehensive wind-wave analysis was conducted to determine the dimensions, directions of approach, and frequency of occurrence of waves which would attack the breakwaters. The data and information provided by the model tests and prototype wind-wave analysis were used jointly to estimate the quantity of breakwater material required for maintenance during breakwater construction and for a forty-year period after construction.

The model design was based on Froude's model laws, and the study conducted in a concrete wave tank, 5 ft. deep, 18 ft. wide, and 119 ft. long, with a 4-ft. by 9-ft. glass viewing panel in the wall adjacent to the model breakwater. The breakwaters were placed across the tank, normal to the longitudinal axis, approximately 90 ft. from the wave generator; the bottom area at the test section was molded in sand. Model waves were generated by a plunger-type wave machine, and wave heights were measured and recorded by an electrical apparatus designed and constructed especially for this purpose.

The model breakwaters were constructed in a dry tank, and, after the tank had been flooded to a given depth, the structures were subjected to attack by test waves of various dimensions until no further displacement of breakwater materials occurred. Soundings were taken frequently to determine the progressive displacement of the breakwater material. Replacement of displaced rock and further subjection to wave attack determined the increased stability gained through the process of damage and repair.

It was concluded from this study that the breakwater constructed by end-dump methods would require about three times as much material for maintenance during construction, and about two and one-half times as much material for maintenance over a 40-year period after construction, as would be required for the conventional-type breakwater. However, because of the difference in the unit costs of handling materials for the two different types of breakwaters, it would be necessary to compare total costs of construction and maintenance over a 40-year period in order to select the most economical breakwater design.

THE MODEL AS A TOOL IN RESEARCH

Although it is desirable that mathematical expressions, in which all the variables can be handled with relative ease, be available to govern the design of breakwaters, the action of wave forces on breakwaters has proven too complex in most instances to be resolved by derived equations of wave motion. Measurements of natural phenomena are desirable, but it is difficult and expensive to obtain adequate data of the required degree of accuracy. These facts suggest the use of models as a research tool to obtain corrective coefficients for available, yet inadequate, theoretical equations, or to formulate empirical equations and design curves. The value of scale models in the development of design criteria can be explained very readily by example. Two types of breakwaters have been selected for this purpose. These are (1) the sloping, rubble-mound breakwater, and (2) the vertical, impervious breakwater. A large number of other breakwater designs are obtained from different combinations of these two types of structures.

DESIGN CRITERIA FOR RUBBLE BREAKWATERS

Use of the small-scale model as a research tool excludes the testing of specific structures to obtain the most efficient design. Instead, generalized data are desired which encompass the complete ranges of all the variables involved. After the variables thought to affect the phenomenon have been selected, the research study can proceed in either of two directions: (1) Tests can be conducted in which each variable or dimensionless parameter is varied in turn with the remaining variables held constant. From the data obtained in this manner, design curves can be prepared which will allow determination of the effects of each variable. Or (2), one of the better available theoretical equations can be revised by application of corrective coefficients obtained from small-scale tests.

Because of the complex action which occurs when waves attack a mound of irregularly shaped and placed rock and the lack of an exact theoretical analysis of the forces involved, it is not feasible to perform tests in which wave pressures on the individual rocks are measured. A very simple method of obtaining the information desired is to perform stability tests of small-scale rubble mounds using different types of rock (size, shape and density), slopes and cross sections of the breakwater, depths of water and wave dimensions (H/L and d/L ratios), and, by trial and error, to determine the largest wave which does not cause displacement of rock. The research testing program can be guided by dynamic and resistance force expressions based on an idealized picture of the phenomena involved and aided by dimensional reasoning. Thus, by making the necessary simplifying assumptions concerning the forces involved when a wave impinges upon a breakwater rock, the following expressions for dynamic and resistance forces can be obtained:

$$F_d = k_1 \, w_1 \, \ell^2 H \tag{1}$$

and

$$F_r = k_2 \, (w_2 - w_1) \, \ell^3 \tag{2}$$

where w_1 is the specific weight of the liquid in which the rock is submerged, w_2 is the specific weight of the rock, ℓ is a characteristic linear dimension of the rock. H is wave height, and k_1 and k_2 are coefficients, the magnitudes of which are assumed to involve the characteristics of both the waves and the resisting rock structure. These coefficients may include wave steepness (H/L) and d/L ratio, breakwater slope ($\emptyset$), effective coefficient of friction between rock (μ), void ratio (v_r), and rock shape factor (Δ). For incipient instability, $F_d = F_r$, or

$$\frac{w_1 \ \ell^2 \ H}{(w_2 - w_1)\ell^3} = \frac{k_2}{k_1} = f\ (\emptyset,\ H/L,\ d/L,\ \mu,\ v_r,\ \Delta) \qquad (3)$$

Knowing that the individual cap rock weight is $W = \gamma_r \ell^3$, and substituting in equation 3 the value of ℓ obtained therefrom, it follows that

$$\frac{w_1\ (w_2)^{1/3} H}{(w_2 - w_1) W^{1/3}} = f\ (\emptyset,\ H/L,\ d/L,\ \mu,\ v_r,\ \Delta) \qquad (4)$$

The results of scale tests can be plotted to show the effects of each variable of the right-hand member of equation 4.

Of the existing theories on this subject, that of Iribarren is, perhaps, the best and most familiar. Iribarren's formula has appeared in several forms (Iribarren, 1949; Hudson, 1950; Epstein, and Tyrrell, 1949) but one which is more general, and dimensionally homogeneous is shown as follows.

$$W = \frac{k_3 \quad w_1{}^3 \quad w_2\ \mu^3 \quad H^3}{(w_2 - w_1)^3\ (\mu\cos\emptyset - \sin\emptyset)^3} \qquad (5)$$

in which k_3 is an undetermined, dimensionless coefficient.

Preliminary experiments at the Waterways Experiment Station are underway at the present time and have indicated that results might be plotted with d/L as abscissa, k_3 as ordinate, and $\emptyset$ as a parameter, producing a family of curves for various slopes. Further testing should indicate more completely the relative importance of variables omitted. In these tests the maximum (design) wave height which did not cause displacement of rock from the face slope was determined. If a slight amount of damage, insufficient to reduce appreciably the efficiency of the breakwater, is allowed, the design wave height can be increased several feet. Design latitude would obviously be increased if data were available for both the no-damage and slight-damage criteria.

DESIGN OF VERTICAL-WALL BREAKWATERS

Because the function of impervious vertical breakwaters is to reflect rather than absorb the incident waves, the overturning moments resulting from the pressure forces are one of the principal causes of failure. Here, again, there are existing formulas which give wave pressures with varying degrees of accuracy. Notable among these are the formulas of Sainflou, Gourret, Benezit, and Molitor (Hudson, 1950). Since none of these theories defines results of the phenomenon with consistency or with known degrees of accuracy, it is necessary again to modify one of the more accurate and preferably less complicated formulas by experimentation. With these considerations in mind, Sainflou's formula for standing waves is suggested (Sainflou, 1928).

For this type study, data concerning the magnitude and location of standing-wave pressures can be taken for various wave dimensions and water depths, and the ratio of measured moments to the Sainflou (1928) theoretical moments can be compared with the various parameters thought to affect the phenomena.

It has been established recently that vertical breakwaters situated in water depths generally considered free from breaking waves may be subjected to localized shock-type pressures more intense than, and possibly in addition to, standing wave pressures. Shock pressures are very difficult to measure in nature, since they are quite erratic and occur only over small areas under certain critical conditions. These critical conditions are more easily obtained in the model, although even this is difficult. Preliminary studies of breaking-wave pressures have been made in England by Bagnold (1938-39) and in the United States by Morison (1948). Additional studies are now in progress at the Beach Erosion Board Laboratory, Corps of Engineers, Department of the Army, Washington, D.C.

CONCLUSIONS

The experiences of the Armed Forces in World War II showed the inadequacy of the existing science of waves and wave action with relation to engineering structures in harbors or on beaches and gave impetus to the development of both theory and experimentation in this field. Although there are a considerable number of theoretical works available, as the result of the accelerated pace of investigations brought about by the military situation mentioned above, and although there are quite a number of investigations in progress at the present time, it is hardly possible, yet, to solve any but the most elementary harbor problems by analytical reasoning. Use of the small-scale hydraulic model for solution of specific problems and as a tool in general research is considered to be, therefore, the best approach to the study of harbor wave-action problems, and, in most cases, the costs of such studies are only a small fraction of the prototype installation cost.

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CHAPTER 24

DESIGN OF BREAKWATERS

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INTRODUCTION

As the name implies, a breakwater is a barrier constructed to break up and disperse heavy seas, to shield the interior waters of a harbor from winds and waves, and to provide shelter and protection for ships, shipping facilities, and other harbor improvements. Breakwaters are structures used to improve a naturally protected (sheltered) harbor or to create a sheltered harbor at locations required for shipping, refuge, recreation, etc.

Breakwaters may be roughly divided into two main groups, the vertical-wall type and the rubble-mound type. A possible third group, the composite type, consists of the wall-type placed upon a rubble-mound foundation. Since the experience of the San Francisco District, Corps of Engineers, has been limited to the construction of rubble-mound breakwaters and jetties and inasmuch as practically all breakwaters on the Pacific Coast are of rubble-mound construction, the second half of this paper has been limited to the consideration of the design of this type of structure. The first half of the paper discusses general subjects (choice of location and type of breakwater, etc.) relevant to both types.

Until recently, the design and construction of breakwaters was largely an empirical "art" based mainly on the designer's observations of the performance of previously constructed breakwaters. Great latitude was given personal discretion and judgment, since those factors which might influence or standardize design were little understood.

It was not until 1923 that the problem of wave forces on vertical-wall structures was effectively attacked, and not until 1938 was an adequate solution evolved for the same problem in relation to sloping-faced structures. Knowledge, both theoretical and empirical, of forces on the first type of breakwater has been extended by Benezit (1923), Lira (1927), Sainflou (1928), Molitor (1935), Cagli (1935), Gourret (Catena, 1941-43), Iribarren (1949), and Minikin (1950). The work of Iribarren (1949) on sloping-faced structures, with additions in collaboration with Nogales y Olano (Iribarren and Nogales y Olano, 1950a, 1950b), appears to be the most adequate formulation of this problem. Epstein and Tyrell (1949) also developed a formula very similar to that of Iribarren (1949) for sloping-faced structures. Recently, Mathews (1948) and Rodolf have each developed formulae for the solution of this complex problem (see Chapter 26).

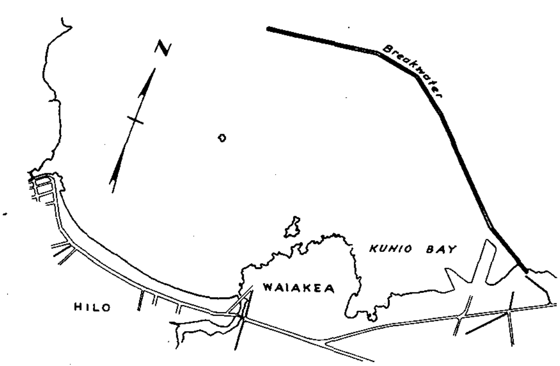
Conjecture in design consideration was further reduced when it became possible to forecast storm waves with a fair degree of accuracy (Sverdrup and Munk, 1947). This procedure, in conjunction with those for graphical construction of refraction diagrams (Johnson, O'Brien, and Isaacs, 1948) makes it possible to determine wave heights at the breakwater location, from which the "design wave," the wave which the structure is designed to resist, may be chosen.

At the present time, the "art" in breakwater design, i.e., that portion of the design problem not already fairly rigidly determined, has been restricted to choice of site, type of structure, selection of design wave, and materials to be used. Most other aspects of the problem have been more or less standardized. This paper will present fundamental principles and general and specific criteria to be applied in breakwater design with particular reference to the rubble-mound type.

SITE CONSIDERATIONS

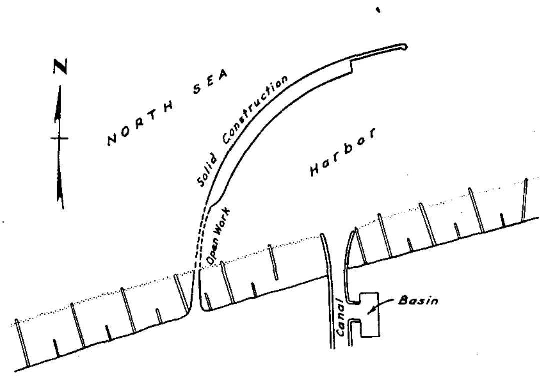
Protection offered and location of structures. The primary purpose of breakwaters is to protect harbor areas from the action of heavy seas and in locating them there is only one hard and fast criterion to apply; i.e., the structure or structures should offer the desired protection at the least possible cost. The seven following examples of types of breakwaters used at different locations present some general considerations.

a. A single breakwater extending from one shore such as Hilo Harbor, T. H. (Fig. 1-A). The harbor is a bay, or is at least so protected naturally that exposure to wave action is restricted to one direction. The breakwater may be straight or curved and extend either perpendicularly or obliquely from shore.



HILO HARBOR, HAWAII, T. H.

Fig. 1-A

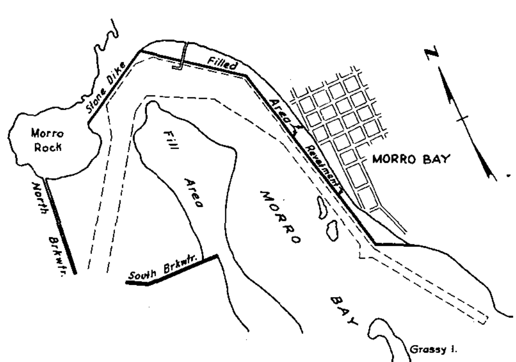


ZEEBRUGGE HARBOR, BELGIUM

Fig. 1-B

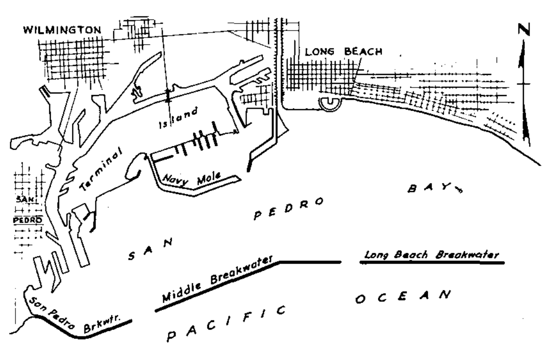
b. A single breakwater attached to shore by an open viaduct as Zeebrugge Harbor, Belgium (Fig. 1-B). The harbor is in a region of considerable littoral drift, the interruption of which is considered to be injurious either because of accretion in the harbor or shore erosion.

c. Twin breakwaters such as Morro Bay, California (Fig. 2-A). The harbor has little or no natural protection as in the case of an open roadstead or crescent-shaped bay.



MORRO BAY HARBOR, CALIFORNIA

Fig. 2-A

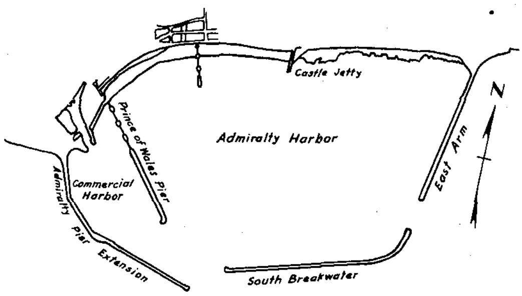


LONG BEACH HARBOR, CALIFORNIA

Fig. 2-B

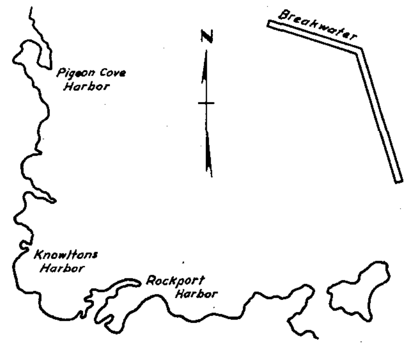
d. More than two breakwaters such as Long Beach Harbor, California (Fig. 2-B). Basic conditions are the same as in paragraph 3, but traffic and required harbor capacity considerations dictate multiple entrances. Colombo Harbor, Ceylon, or Dover Harbor, England (Fig. 3-A), are also examples of multiple breakwaters which completely enclose a harbor.

e. A single isolated (detached) breakwater such as Sandy Bay Harbor, Massachusetts (Fig. 3-B) and Santa Monica, California (Fig. 5). The Sandy Bay Harbor is located where the coast is indented somewhat and is fairly well protected. Cost and traffic considerations are factors which must be considered together with other design criteria.



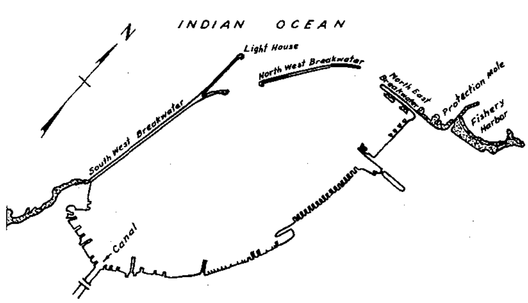
DOVER HARBOR, ENGLAND

Fig. 3-A



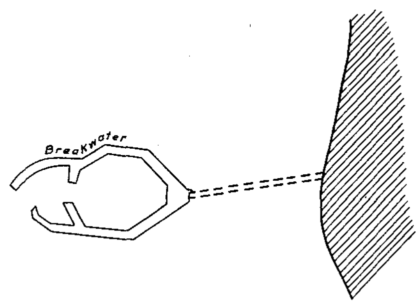
SANDY BAY HARBOR, MASSACHUSETTS

Fig. 3-B



COLOMBO HARBOR, CEYLON

Fig. 4-A



HUNDESTED ISLAND HARBOR, DENMARK

Fig. 4-B

f. Overlapping breakwaters such as have been constructed at Colombo Harbor, Ceylon (Fig. 4-A), and Genoa Harbor, Italy. The prime purpose of the overlap is to provide additional protection at the entrance or more protection inside the harbor.

g. Island breakwaters such as the Danish fishery harbor at Hundested Island (Fig. 4-B). Such breakwaters form a completely artificial harbor lying entirely outside the zone of littoral drift.

From the above examples it is seen that the "desired protection vs. least cost" rule is too broad to make the choice of site problem a simple and straightforward one. One must determine the general extent of protective works needed, the number, widths and clearances of entrance channels to be provided, and the necessity for additional protective works. Also one must weigh and balance initial costs with maintenance charges for a particular placement of breakwaters. For these determinations, studies must be made of the size of harbor to be provided or protected, the size and number of vessels using the harbor, the extent of protection from heavy swell needed for efficient harbor operation, and the maintenance problems a certain placement of structures would cause (i.e., interruption of littoral drift may cause harbor shoaling and require maintenance dredging).

Hydrography, topography and foundation investigations. After a tentative site has been selected (the necessity for construction already having been demonstrated), an intensive local-condition investigation must be made prior to any construction. Complete and detailed hydrographic surveys are necessary, not only of the tentative site but of adjacent areas. This phase of the investigation should be complete to the extent of supplementing standard survey procedures with bottom samplings, measurements of existing currents, and measurements, if possible, of amounts and directions of materials transported by these currents. These data will permit appraisal of possible alternate locations, will furnish information on foundation conditions, will provide a permanent record of conditions prior to construction, and will make possible -- through use of prior surveys, old charts, maps, etc. -- a study of any changes the bottom has undergone.

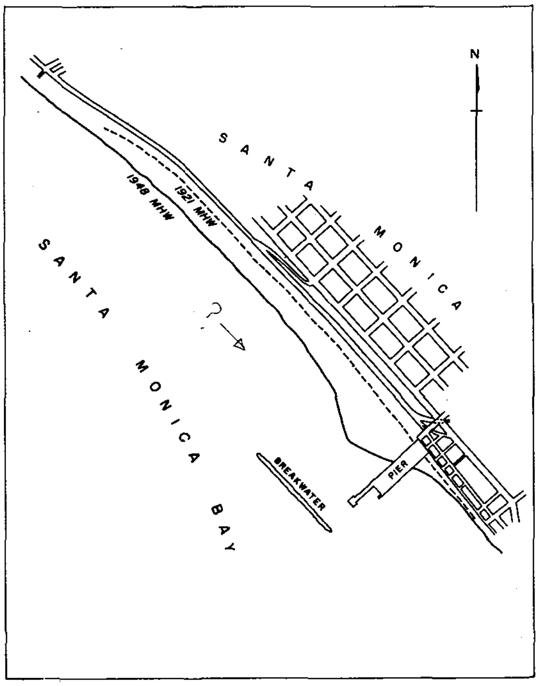


Fig. 5
Santa Monica Harbor, California

This last study, with the evaluation made of existing currents and material drift, should be used to determine, if possible, the effects of construction on shore erosion and deposition. A breakwater, in a sense, is a perturbation introduced into the natural state of a shore line, and, as such, will certainly change this state. As an example, a detached breakwater running approximately parallel to a beach will stop wave action shoreward of it, thereby interrupting, in this area, the natural state of material transport due to waves. A shoal may then form between the beach and the breakwater, and the beach downshore of the area will be eroded (see Fig. 5).

In addition to the hydrographic surveys, topographic surveys are needed showing land details in the vicinity of the breakwater and along the shore for a considerable distance on either side of the breakwater or breakwaters contemplated. These should be used as a record of shore conditions prior to construction to show the extent of any future changes the structures may cause, as well as for determining the location of the structure.

Foundation conditions must be known accurately for proper choice of the type of breakwater to be used at a particular site. The foundation analysis can be made by means of probings, washborings, or drillings. As discussed later the thoroughness of this investigation will be determined by the structural types contemplated, and the permissible settlement of these types.

Availability of materials. A factor to be considered in the choice of type of structure, as well as the exact choice of its site, is the availability of materials for construction. For instance, hydrographic and topographic surveys may indicate that construction of a breakwater is feasible at two locations, each of which would adequately provide harbor facilities desired for the area. Two or more types of breakwaters may be under consideration for each site. The cost of material transportation (a function of the distance it must be moved), and the type of materials readily available at each site then become determining factors in choosing between the two sites and between the breakwater types.

For a rubble-mound structure, the size of rock necessary to resist wave action is a function of the density of the rock, namely, $s/(s-1)^3$, where "s" is specific gravity. It is obviously desirable to use as dense rock as possible for maximum resistance. For maintenance considerations, the stone should be resistant to abrasion and deterioration by wave action. Of two sites, one may have nearby a quarry suitable for supplying only relatively small rocks, or rocks of low density. The other site's prospective quarry may be capable of providing higher density and larger stones but is located at a greater distance from the proposed harbor. A balance must be struck between the low initial cost but high maintenance costs at the first site, and the high initial cost but low maintenance charges chargeable to the second site.

For concrete or pile structures, similar cost balances must be made. The availability of aggregates of acceptable qualities will be a major factor in determining initial costs.

SELECTION OF TYPE

Types suitable for various conditions. As previously noted, breakwaters may be divided into two main groups: the vertical-wall type and the rubble-mound type. Under vertical-wall breakwaters, various sub-types exist, depending on methods and materials used in construction. These, with some general criteria of suitability, are listed below:

a. Masonry wall -- suitable in depths up to 65 ft.; requires a firm foundation; needs very little maintenance; and may be adapted for use as quays. They must not be exposed to breaking waves since the combination of high pressures due to the breaking wave (Iribarren and Nogales y Olano, 1950b) and only a small settlement of foundation can bring about total destruction (Catena, 1941-43).

b. Timber crib -- suitable in depths of from 10 to 40 ft. Cost of construction is high. Unsuitable in salt water.

c. Caisson -- suitable in depths of from 10 to 35 ft. Heavier than timber-crib structures. Suitable in both fresh and salt water.

d. Steel sheet pile -- suitable in depths up to 40 ft.; may be used in any kind of foundation into which steel piles may be driven.

None of these vertical-wall types should be used in regions where waves may break upon the structure.

Rubble-mound types are adaptable to any depth of water, are suitable on nearly all foundations, and may readily be repaired. However, they do require relatively large amounts of material, and are not suitable for use as quays without major modification.

Foundation considerations. Foundation materials vary from solid rock to soft mud, and each gradation must be dealt with differently. The quantities which need to be evaluated are firmness (compressibility), homogeneity, durability, and scourability.

A vertical-wall breakwater may be placed directly on the bottom if the bottom is firm, homogeneous, and not readily scoured. As the firmness and homogeneity lessen, a stone foundation must be put down to distribute the structure's weight; and as the bottom material becomes more susceptible to scour, rip-rap must be added to prevent scour at the toe of the structure itself. For very soft bottom materials, a pile foundation may be placed or a trench dug and filled with sand or rock. Particular attention must be paid to the possibility of settlement, for a masonry wall-type structure is subject to complete failure if its foundation settles any appreciable amount. Therefore, if a rubble substructure is used, sufficient time should be allowed for settlement before the superstructure is added. (The destruction of the Mustapha breakwater at Algiers in February 1934 is an example of complete failure of a wall-type breakwater, where "after the crest of an enormous wave passed over the breakwater without breaking, and before the passage of the following trough, the superstructure was clearly seen to have resisted the force of the wave. Then a slight trembling of the superstructure was noted, followed by the complete collapse of the breakwater as the trough of the wave passed . . .") (Catena, 1941-43.)

As noted before, a rubble-mound structure may be used on almost all types of bottoms. If the bottom material is extremely poor, it may be necessary to remove and replace it with sand or other suitable material to form a satisfactory foundation for the structure. A sand blanket may be used on certain unsatisfactory materials to form a spread foundation for the breakwater.

DESIGN OF RUBBLE-MOUND STRUCTURES

Forces. Iribarren (1949) published a formula for the calculation of stone size in rubble-mound breakwaters which, because of its rational basis, some verification, and its ease of application, has found wide use in design of these structures. It succeeded in relating stone size or weight to breakwater slope, stone density, and, most important, wave height at the breakwater.

Its use, though, was limited, for not until 1943 was any progress made toward predicting waves engendered by ocean storms. Prior to this, the only data available to predict wave heights were sporadic and generally valueless visual data gathered by unqualified shore observers. (Wave-height observations seem to double or treble the true size of swell when its magnitude is in any way out of the ordinary.)

With the advent of wave forecasting procedures and of wave refraction evaluation methods (see Chapters 4 and 8), the wealth of wave information contained in long-compiled synoptic weather maps become available to the breakwater designer, and the application of formulae, such as that of Iribarren (1949), dependent upon wave characteristics in determining structural characteristics, became more valid. Below is the Iribarren formula, in standard American units, for surface and above-surface stone weights:

$$W = \frac{k'_1 H^3 s}{(\cos\phi - \sin\phi)^3 (s - 1)^3}$$

For weights of stone at depths below the surface, H is replaced by H_1,

where $$H_1 = \frac{\pi H^2}{L_o \sinh^2 \frac{2\pi d}{L}}$$

W = weight of stone in tons (2,000 lbs.)

k'_1 = a constant = 4.68×10^{-4} for rubble
= 5.93×10^{-4} for artificial blocks

H = corrected wave height (ft.) (see below)

s = specific gravity of stone

ϕ = angle of the slope with horizontal

L = wave length (ft.)

d = depth below still water level (ft.)

In the original formulation, Iribarren (1949) suggested that H be the wave height expected at the toe of the structure. In the latest paper by Iribarren and Nogales y Olano (1950b) the determination of H is governed by a theoretical wave-steepening effect of the breakwater.

There are definite limitations to the formula. It is apparent that for slopes between 45° and 90° the $(\cos\phi - \sin\phi)^3$ term in the denominator is negative and therefore the stone weight also is negative. For a slope of 45° this term approaches zero and the stone size becomes infinite. Neither of these last statements has physical meaning. In general, though, seaward slopes are less than 45°, varying only between 1 on 1-1/4 and 1 on 1-3/4, and in this range of slopes the formula may be used.

Maintenance considerations. Unlike the rigid, vertical-wall type of breakwater, the rubble-mound type, when subjected to severe wave action, is not prone to complete failure. Rubble structures, not being monolithic, will follow more of a process of disintegration; that is, wearing away or dislodging stone by stone, rather than total collapse, and the damaged structures, if anything, will offer a more stable base for any repairs. This repairable feature makes necessary a decision between the relative costs of initial construction and maintenance in designing a rubble-mound breakwater.

It is possible, by use of synoptic charts, to predict with some accuracy a maximum design wave and, if the data are complete enough, to predict also the yearly frequency of the larger waves. It is also possible to design a breakwater to withstand, with minor repair, the largest wave expected. However, it undoubtedly would be less expensive initially, though more costly from a maintenance standpoint, to design breakwaters for a wave smaller than the maximum, and a breakwater so designed may show a lower total annual cost, including interest, amortization and maintenance cost, than one adequate to resist all storms.

We have a prime example in the San Pedro Bay breakwaters of the importance of the decision to be made between immediate and future costs. Between April 20 and 24, 1930, prior to the construction of the Middle (Los Angeles-Long Beach) detached breakwater, large waves entered the Bay and caused extensive damage to the inner Long Beach breakwater (O'Brien, 1950). In 1939, waves of destructive amplitudes caused great damage to the then partially completed detached breakwater and some damage to the San Pedro breakwater. In the first case, the swell was engendered by a southern hemisphere storm and in the second case the swell resulted from a tropical storm immediately to the south of Long Beach. (Note that wave forecasting techniques were not in usable form until 1943.) In both cases, the infrequency of occurrence of waves as destructive as these suggested that future designs be drawn for smaller but more frequent waves. The breakwaters were restored to their original conditions with no additional provisions for withstanding storms of the magnitude of these two.

Determination of crest width and elevation. When using the formula of Iribarren (1949), breakwater crest heights may be determined by using the technique of calculating breaker characteristics at a sloping face breakwater (Iribarren and Nogales, 1950b). In the case of rubble-mound structures where vessels are not likely to be moored at, or near, the structures, it is not always necessary to completely obstruct the waves, although the volume of water passing over the top should not be sufficient to cause undue disturbance in the harbor.

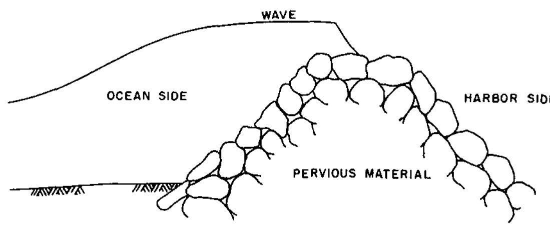


Fig. 6

Crest widths are determined more roughly. If the breakwater is so designed (for reasons of economy) that some higher waves will pass over the crest, sufficient width must be allotted to withstand forces caused by these waves. Reference to Fig. 6 will clarify the preceding statement. The cap and armor stone being pervious, an impinging wave will cause water to surge through the structure. At the harbor crest edge, there will be three concurrent phenomena:

a. The weight of the stone will be decreased by the buoyant force due to submergence, and

b. The surge through the breakwater will give rise to a force acting to dislodge the stone, and

c. The portion of the wave passing over the crest will tend to dislodge the stone.

Increasing the crest width acts to lessen these rupturing forces by first decreasing the magnitude of the surge through the permeable structure (friction and turbulence), and second, by decreasing the energy to dislodge available to the wave passing over the structure ("bottom" friction and turbulence).

Other factors in determining crest width are the method of construction decided upon, and the use to which the breakwater will be put in addition to its primary function of dispersing heavy seas. Placement by trestle requires dumping space aside the trestle. Placement by truck requires providing sufficient width for truck maneuverability. It may be desired to lay a road on the breakwater in which case its use will determine the structure's width.

Determination of slopes. Few specific criteria for determination of slopes exist, though for normal wave attack, slopes 1 on 1-1/2, to 1 on 1-3/4 will maintain their slope, with slight flattening, in deep water. It has been the practice to make the slope of the tip of a breakwater as flat as possible (up to 1 on 2) since this section is always exposed to normal (perpendicular) wave attack. Iribarren and Nogales y Olano (1950b) indicate that the stone-size slope relationships they have developed give stable configurations. This verification is based on Larras and Colin (1947-48) determination of stable slopes for the breakwater at Argel after repairs to numerous deficient slopes.

An interesting aspect of the slope determination problem comes to light when the mechanics of wave force exerted on either vertical-faced or sloping-faced structures are examined. For non-breaking waves, forces on vertical structures are a function of the height of the standing wave (or clapotis) created by complete wave reflection, whereas, mound breakwaters, because of their slope, are always subject to attack by breaking waves. The force caused by the impact of a fully breaking wave is much greater than the force arising from the same wave being completely reflected (Morison, 1948). As an extreme, if the slope of a breakwater were decreased to the point where it became relatively flat beach, wave breaking then would cause almost no reflection. Here we have two extremes of wave action; the vertical-wall causing complete reflection, and the horizontal-slope causing no reflection. Certainly, slopes in between these extremes cause a combination of breaking and reflected waves, with some intermediate slope causing a half and half division.

This division is of little importance, especially on the Pacific Coast, in the design of outer breakwaters where the depths of construction are such that waves are likely to break at, or even slightly before, the structure. Such a wave engenders very high shock pressure, and a discussion of the division of energy available in the breaking or reflected wave is academic. However, for breakwaters in the interior of a harbor, placed there mainly to further reduce wave action, the division between reflected and breaking waves is important. A vertical-faced structure will reflect unchanged a non-breaking wave and the successive interaction between reflected and incident waves will only add to roughness of sea in the harbor. A sloping-faced structure, in causing waves to break, will decrease the amplitude of those reflected. In a discussion of the problem by Iribarren and Nogales y Olano (1950a), two points of importance may be mentioned:

a. To the findings of workers of the Laboratory of Delft, giving ratios of reflected to incident wave heights, they apply the batter (inverse of the slope) as a parameter and find quite consistent results, i.e., the ratio of reflected to incident wave heights increases almost linearly with decrease in batter (increase in slope).

b. They propose the slope $\frac{8}{T}\sqrt{\frac{H}{2g}}$ as the limiting one between reflection and breaking. (T = period of wave and H = wave height.)

Zones of stone and stone classification. As pointed out previously, stone used should be dense and resistant to abrasion. Stone sizes are determined by design wave characteristics though absolute consistency in size and weight cannot be expected. General criteria for zones of stone and placement are fairly easy to establish.

The stone used may be roughly classified as follows:

a. (Armor stone). This is the principal protective covering, which is exposed to the most violent wave action. Its adequacy determines the success or failure of the structure.

b. (Secondary protective covering). The armor stone, being necessarily large, will make for large voids in the principal protective region. A breaking wave would tend to wash away the smaller and unclassified materials constituting the core if no secondary covering is used. This stone, then, consists of rock smaller than armor but still large enough to resist by itself the turbulent flow through the primary covering voids.

c. (Core). This serves as support for the protective cover, and in itself prevents the propagation of swell through the breakwater. It consists of rubble of different sizes so graded and placed as to present maximum compactness with a minimum of cavities. The minimum size of its constituents is limited since this portion of the work is "floated" in place and, therefore, must be of sufficient size to place itself by action of gravity alone -- in spite of currents or swell.

The thickness of the zones vary but again practice has set up general criteria. The armor stone should be a minimum of 2 layers thick (very roughly 10 ft. for 10 tons average weight of stone). The secondary protective stone should be 3 to 4 layers thick (about the same width as the principal protective covering for 5-ton average weight).

CONCLUSIONS

In a way, a paper such as this, dealing concisely with design consideration for breakwaters, can never be adequately prepared. The field is not a closed one. Each of the preceding paragraphs by itself would warrant at least as much discussion as this entire presentation. The selection of site offers numerous complexities which should be fully dealt with. There is much disagreement on methods of calculating stone sizes and slopes. The formulae of Mathews, Rodolf, and Epstein disagree with that of Iribarren, but the applicability of these formulae versus Iribarren's has not been touched on.

There are problems which, at the present time, have no solution. In the determination of crest width, for example, some factors contributing to instability of harbor-side stones have been mentioned but not evaluated.

However, the purpose of this paper is to present only a statement of the problems, techniques, and criteria now encountered in the field of breakwater design. If it in any way stimulates discussion of these factors, its hoped-for function will have been fulfilled.

ACKNOWLEDGEMENT

The authors wish to acknowledge the help received from the unpublished draft of the Corps of Engineers Civil Works Manual on "Breakwaters" prepared by F. W. Rodolf, one of the authors of Chapter 26. Thanks are also due D. S. Cruickshank and G. W. Stark of the San Francisco District, Corps of Engineers, for their suggestions and reviews during the preparation of the paper.

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CHAPTER 25

BREAKWATER CONSTRUCTION

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INTRODUCTION

Upon receipt of an invitation to participate in this conference and present a paper on the subject of Breakwater Construction, I was happy to accept, feeling that I would benefit to a far greater degree than my contribution to a subject in which I, and the concern which I represent, have had a deep interest for many years.

Our experience embraces some of the major jetty and sea wall construction in San Francisco Bay, viz: Treasure Island, Mare Island, Alameda Naval Air Station, Sacramento and San Joaquin River bank protection and other jetties along the Northern California Coast, including Crescent City.

This paper is limited to breakwater construction on the basis of our experience previously mentioned. The subject has been divided into two parts, namely, (1) construction of jetties, or breakwaters which can be built by use of floating equipment and (2) construction of jetties which are not practicable to be built by use of floating equipment.

I shall preface my presentation by presuming that we have before us a design of jetty, (1) indicating that borings show a substantial depth of mud that is unstable, but which -- by the addition of a blanket of quarry rock fines will stabilize sufficiently to carry the weight of the jetty without movement, (2) a portion showing mud to a depth that it will not stabilize requiring removal by dredging.

CONSTRUCTION OF JETTIES BY FLOATING EQUIPMENT

The first requirement is the location of a deposit of stone which will produce a sufficient quantity of rock that is durable, not subject to disintegration by the action of air and sea water, and will withstand the soundness tests specified for stone which will be used for armor or face and cap of the jetty. It is obviously desirable that the location of the deposit be as near as possible to a navigable waterway to reduce to a minimum transportation cost from quarry face to barges.

Production of stone. As all deposits vary in formation no uniform procedure will apply to all quarries or deposits. In the process of quarrying rock, care should be exercised to conduct the blasting operation in such a manner that will produce the maximum amount of the larger pieces of rock to be used for armor or face and cap stone.

The armor or face and cap stone should be stockpiled for use when required. Rock fines for blanket material can be produced by passing smaller material over a grizzly (the "thru" material being blanket, and the "overs" will combine with the core rock). Core rock is usually abundant in quantity due to the smaller size.

Loading onto barges. In our operations we have built adjustable loading ramps which are raised and lowered to compensate for tide, by means of electrically powered hoists designed to permit rock trucks to back up the ramp and dump directly onto the deck of the barges, which method is suitable for blanket, core and smaller sizes of armor or face rock. For rock that is larger and of such weight that dumping would result in damage to the deck of the barge, loading is accomplished by swinging the individual pieces aboard by crane with the use of slings. Skips have also been employed for loading. At our McNear Quarry where the core rock is the product of material passing through a 48 in. x 60 in. jaw crusher, scalped over a screen which removes non-specification fines, the resulting product about 24 in. maximum is loaded directly onto the barges by means of rubber conveyor belt.

Transportation by floating equipment. Transportation from quarry or loading point to site involves use of barges designed strong enough to withstand impact of dumping rock thereon, with decks protected by heavy wood sheathing. Tug boats are used to tow barges to the jobsite.

Placement from floating equipment. (Bottom dump barge.) Placement of blanket material and core rock by bottom dump barge is accomplished by securing it alongside a mooring barge equipped with winches and held by four anchors running from its four corners to enable accurate positioning of the dump barge. This procedure is desirable when the core is approaching the surface in order to obtain full coverage along the line of the jetty and maintain the slopes of the core. In deep water which affords a wide base, the major tonnage may be placed within the lines by dumping while the tow is under way.

Flat deck barges -- unloading by bulldozer. Another method utilized for placement of blanket material and core rock has been to employ a tractor bulldozer to push the rock over the side of a flat-deck barge. A mooring barge to moor and control positioning of the barge in the same manner as described for the bottom dump barge is required for this method. The tractor is kept on the mooring barge and walks aboard the rock barge to unload and then returns by means of a ramp designed to accommodate the height of the rock barge when unloaded.

Conveyor barges. In connection with the jetties built by our company in San Francisco Bay, we designed and built self unloading hopper-converyor barges, the conveyor of which discharges rock over the bow end of the barge, hinged to permit vertical movement which permits bringing the top of the core mound to a specified elevation. Positioning of the barge on the center line of the jetty is again accomplished by use of a mooring barge. Power for driving the conveyor or air equipment is either installed on the hopper-conveyor barge if by power units, of if electric by installing electric generating and air compressor equipment on the mooring barge.

The conveyor barge consisted of a steel hopper mounted on the dock of a steel barge. The bottom of the hopper being formed by removable panels. A 48 in. conveyor was installed under the panels. The operation consisted of removing the first panel at the bow end of the barge by means of a cable running from the control platform over a sheave on a movable bridge which travels the length of the hopper. The removal of each panel permits the rock supported by it to flow onto the conveyor belt out over the head pulley into the water, exposing the next panel and repeating until unloaded. The advantages of this method of placement appear to be; accurate control of positioning, building the core to full section rapidly thereby producing a rapid concentration of weight, which permits the mass to find its bottom where underlaid by mud. If there is settlement, the core can be built up by the addition of more material long before the armor stone is placed. In the placing of material by conveyor there is always the natural segregation, the fines remaining in the center of the core and the coarser material going to the outside of the mound which is the result desired.

Placement by this method is particularly desirable for building the core in comparatively shallow water and for completing the core to the desired elevation above water where bottom dumping must cease because of lack of floatation. It is a more rapid and accurate method than placement by derrick or crane.

Placement of toe, armor and cap stone. In order to withstand the action of heavy seas and afford the protection for which jetties and breakwaters are designed, the armor and cap stones even in harbors and bays, are specified as individual pieces of substantial weight ranging from 2 to 5 tons for harbor work and considerably heavier for jetties exposed to the open sea.

Placement of this rock therefore requires weight handling equipment consisting of derrick or crane barges of adequate capacity. The crane barge is anchored parallel to the center line of the jetty upon which the core has now been completed to the designed elevation. Anchoring is accomplished by placing two adequate anchors offshore from the jetty with anchor cables running through sheaves on the offshore corners of the crane barge to power driven winches. Two similar cables run from sheaves on the jetty side of the barge to dead-men placed in the core rock or to anchors placed on the opposite side of the jetty. The rock barge is moored alongside and placement proceeds by the crane lifting the individual pieces from the rock barge and placing toe rock along the line away from and parallel to the core. This line being marked by range stakes. Armor stone is next placed on the toe and core and continues up the slope of the core below water until it emerges.

Placement of armor stone above water is usually specified to provide a given thickness of armor or face, to be keyed in place and built to a specified slope with a minimum of voids as far as practicable within the range of the pieces of rock specified. Exercise of care in selection of size and shape of each piece is therefore required. Templates may be provided to guide the crane operator in maintaining thickness and slope.

Cap stone is finally placed across the top of the jetty to the specified elevation again in such a manner that as far as practicable keying into the armor and minimum of voids is maintained. The cranes are equipped with rock grapples which seize and lift an individual piece up to a weight and size limited by the capacity of the grapple and the weight lifting capacity of the crane. Larger stones are handled by use of steel wire rope slings.

It should be pointed out that the above methods wherein barges are moored alongside are only practical in comparatively quiet water. During stormy weather or when heavy swells are running which would cause the barges to pound against each other the work must be suspended in order to avoid loss of or damage to barges and equipment of substantial value. Fortunately after the core is built a lee is thereby provided which except in extremely heavy weather permits work to proceed on that side.

CONSTRUCTION OF JETTIES WHICH ARE NOT PRACTICABLE TO BE BUILT BY USE OF FLOATING EQUIPMENT

This situation occurs under two conditions, (1) the obvious one where insufficient or no water is available for floatation such as bank protection along shallow beaches and river banks, and (2) the other condition of building out into the open sea at locations where heavy seas or storms are to be encountered during the major part of the year, such as the condition to be found along the Northern Pacific Coast. Under this latter condition at locations where the weather records show that one may expect four or five months out of the year that swells have abated sufficiently to where they will not wash a 100 ton crane overboard, jetties have been built by use of the top of the jetty as the travelway for hauling and placement equipment.

This method entails building a roadway on the top of the jetty by filling the voids with whatever sized material will accomplish and provide a smooth surface as the jetty progresses seaward and possibly paving with a thick portland cement concrete slab. The crane must have sufficient capacity and reach to place the core rock along the centerline and to the specified distance from centerline, and to place the armor and cap stone. As the design of ocean jetties specify weight of individual pieces for armor to range from 7 to 15 tons, and larger for the seaward slopes rock grapples are used only on core rock. Armor and cap stone are handled with slings, fashioned so as to trip and release the rock when positioned.

The rock is transported over the top of the paved jetty to the crane at the end of the jetty by trucks. At intervals along the jetty it is widened on the lee side to provide turn outs for trucks to turn around after dumping at the crane, and for passing other trucks.

Another method of constructing jetties in the open sea has been to build a pile railroad trestle from the shore for the length of jetty to be built. Transportation and placement being accomplished by railroad flat cars and rail unloading and placing equipment.

Obviously the availability of deposit is desired to be close to the site of the jetty and the same method of quarrying previously described should be followed.

It will be appreciated that because of the many varying conditions of design required to meet existing physical conditions, my presentation of this subject may have overlooked some phases or methods not embraced in our experience.

CHAPTER 26

DESIGN AND CONSTRUCTION OF JETTIES

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INTRODUCTION

The purpose of this paper is to present a brief outline of the general engineering procedure for the siting and design of jetties and the methods of constructing such structures. After a general presentation of the formulae proposed by various engineers to determine the size and weight of individual pieces of stone or other material which should be used under various wave heights, this paper will be devoted principally to the construction of rubble stone jetties. This is the type principally used on the Pacific Coast of the United States. In this paper the word "jetty" designates a structure extending into a body of water to direct and confine the stream or tidal flow to a selected channel.

PURPOSE OF JETTIES

Jetties are usually placed at the mouth of a river or entrance to a bay to aid in deepening and stabilizing a channel for the benefit of navigation. Properly located jetties will confine the discharge area, promote scour, and extend into deep water the point where the current slackens and transported material is deposited. Jetties also protect the ship channel from waves and cross-currents and from longshore sand movements.

TYPES OF JETTIES

Factors which influence the type of jetty and design of the structure include: the physical characteristics of the site area and its exposure to wind, waves, currents, and tides; possibility of ice damage; meteorological conditions and their effect on water conditions and currents; sea conditions including the determination of maximum wave which should be designed for and littoral currents which may affect sand movements in the locality.

The cost of construction and maintenance is usually the controlling factor in determining the type. Different types may be practical in any locality and the cost of each type, as well as the annual cost of maintenance for different types, should be estimated for comparison before final decision is made. The availability of materials will greatly influence the cost and, for some types, would affect the cost of maintenance. Rubble-mound structures require periodic repair of portions damaged by storm waves but, even though damaged, the structure as a whole still functions, whereas breakage of a vertical face monolith structure generally leads to total destruction of portions of the whole structure and high cost of repair.

The seven general types of jetties are briefly described as follows:

(1) Random stone: A rubble-mound structure is in fact a long mound of random stone. The larger pieces are placed on the outer face to afford protection from destructive waves, and the smaller sized stones are placed in the interior of the structure. This type is adaptable to any depth, may be placed on any kind of bottom, and absorbs the wave energy with little reflected wave action. This type requires relatively large amounts of material. If not carried high enough, storm waves may sweep entirely over the jetty and cause a secondary wave action in the protected area, and if the voids between the stone are too large a considerable portion of the wave energy may pass through the structure. Cross-sections of two random stone structures are shown in Fig. 1.

(2) Stone and concrete type is a combination of rubblestone and concrete. This type ranges from a rubble-mound structure, in which the voids in the upper portion of the rubble are filled with concrete, to massive concrete superstructure on rubble-mound substructure. The mound is used either as a foundation for a high

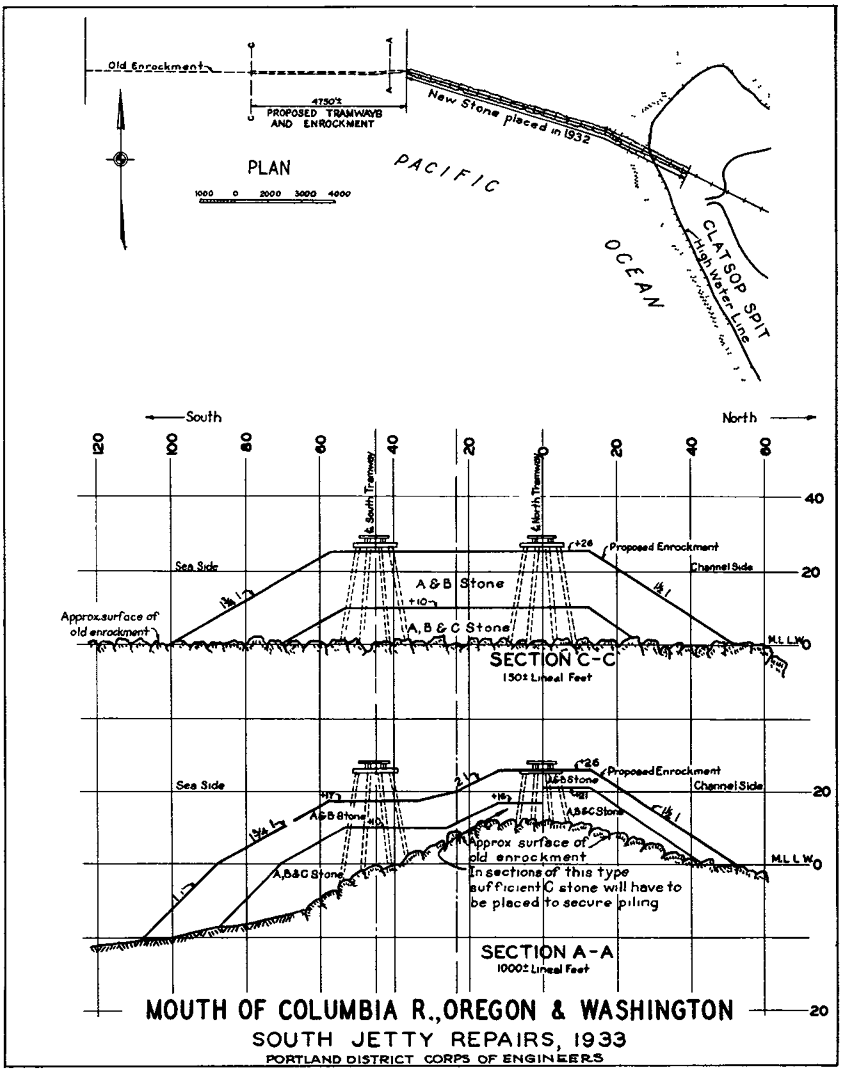
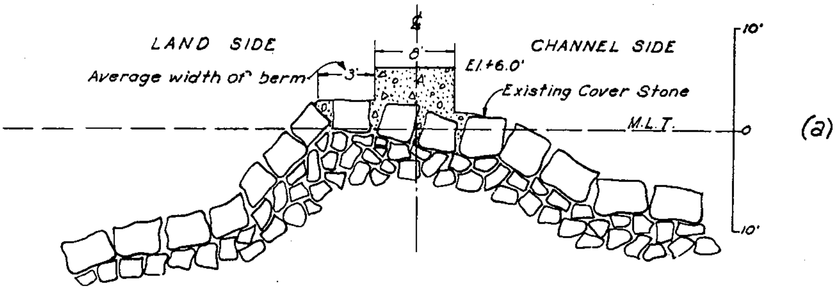


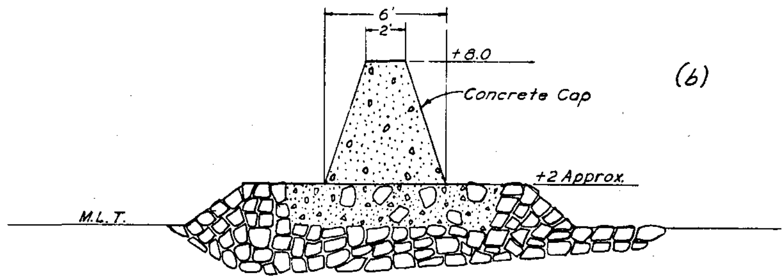
Fig. 1

concrete superstructure or as the main structure surmounted by a concrete cap with vertical, stepped, or inclined face. This type requires less material, and is used where the foundation is soft or subject to scour. The superstructure may be undermined by wave recoil down the face; rubble foundations require time to become permanently stable and should be placed years before the superstructure. This type of jetty, when properly designed and constructed, gives very satisfactory service. Cross-sections of stone and concrete jetties are shown in Figs. 2 to 7.

(3) Caisson type: The first caissons were built of iron but today they are built of reinforced concrete, floated into position, settled upon a prepared foundation, filled with stone to give stability, then capped with cap stones or concrete slab, and, occasionally, parapet walls are added. Some caissons have a reinforced concrete bottom which is an integral part of the caisson, while others, such as the ones used in constructing the Welland Ship Canal, are bottomless and are closed with a temporary wooden bottom which is removed after the caisson is placed on the foundation. Caissons are suitable for depths up to 35 ft. Foundations are either rubblestone alone or piling and rubblestone. Riprap of heavy stone is used alongside to prevent scour, to provide resistance against sliding, and to prevent weaving under wave action. On sand bottom, considerable riprap is required. The top



SECTION AT STATION 40+25 NORTH JETTY - SHOWING CONCRETE CAP



TYPICAL SECTION SO. WEST JETTY - STATION 0+00 to 15+90 - SHOWING CONCRETE CAP
FREEPORT HARBOR, TEXAS

Fig. 2

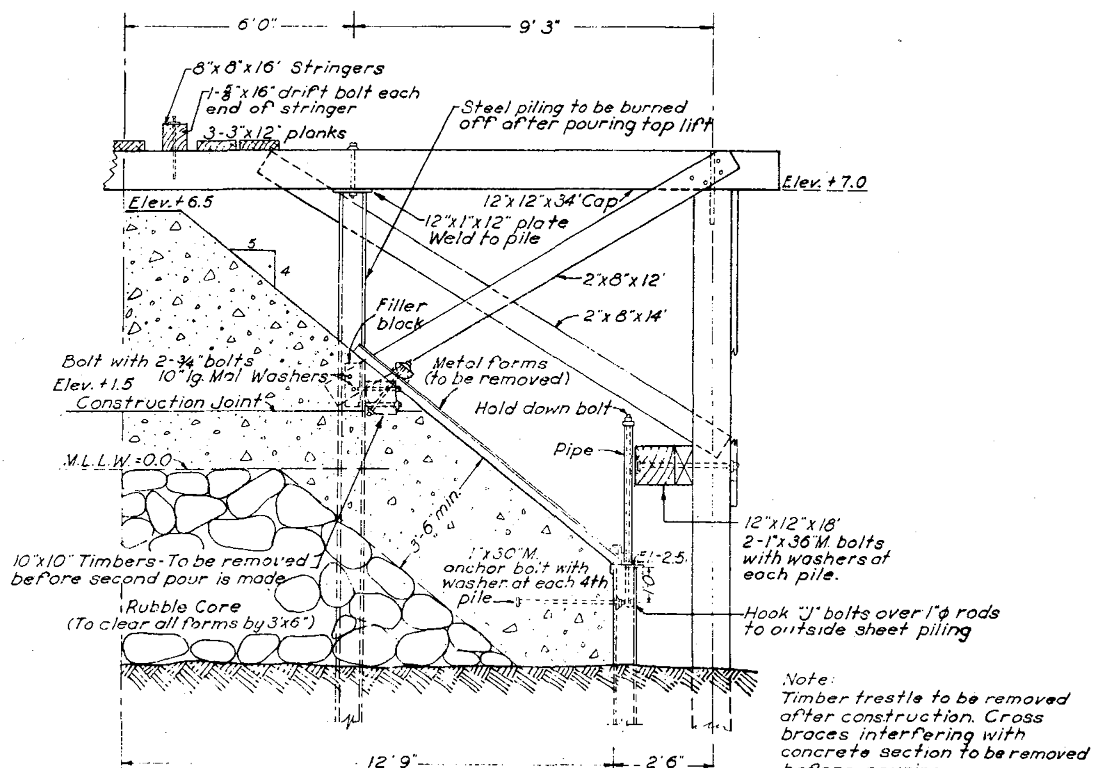


Fig. 3. Typical Section, Nome Harbor, Alaska -- West Jetty

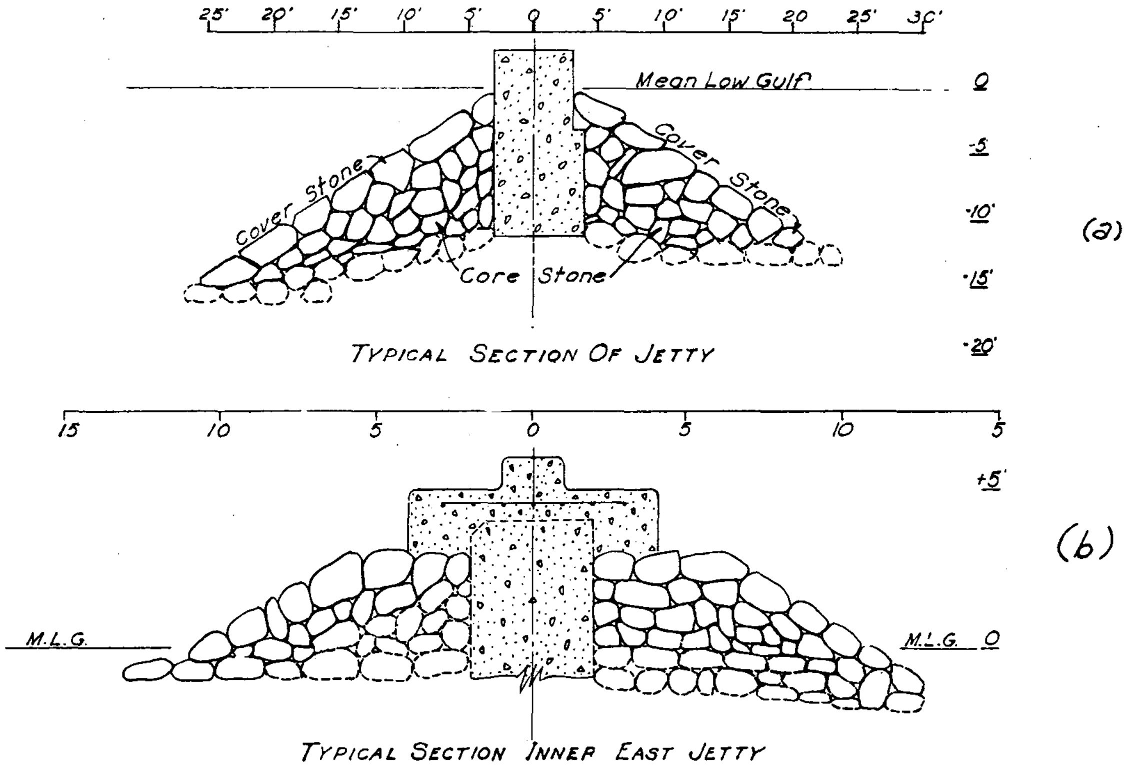


Fig. 4
Entrance to Mississippi River, Southwest Pass and South Pass

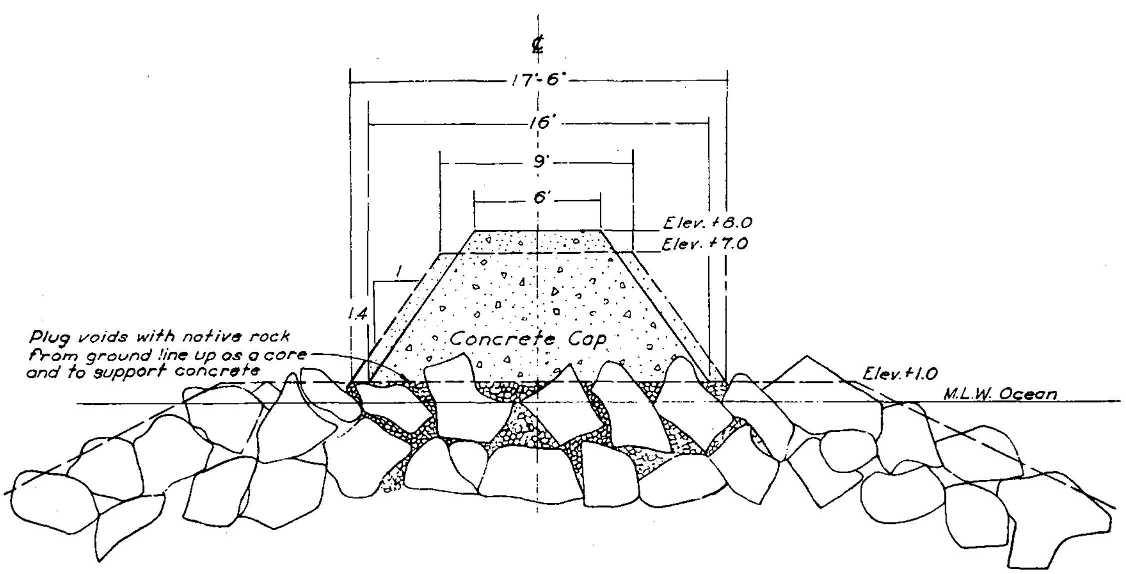


Fig. 5
Typical Section of Concrete Capped Jetty, Lake Worth Inlet, Florida

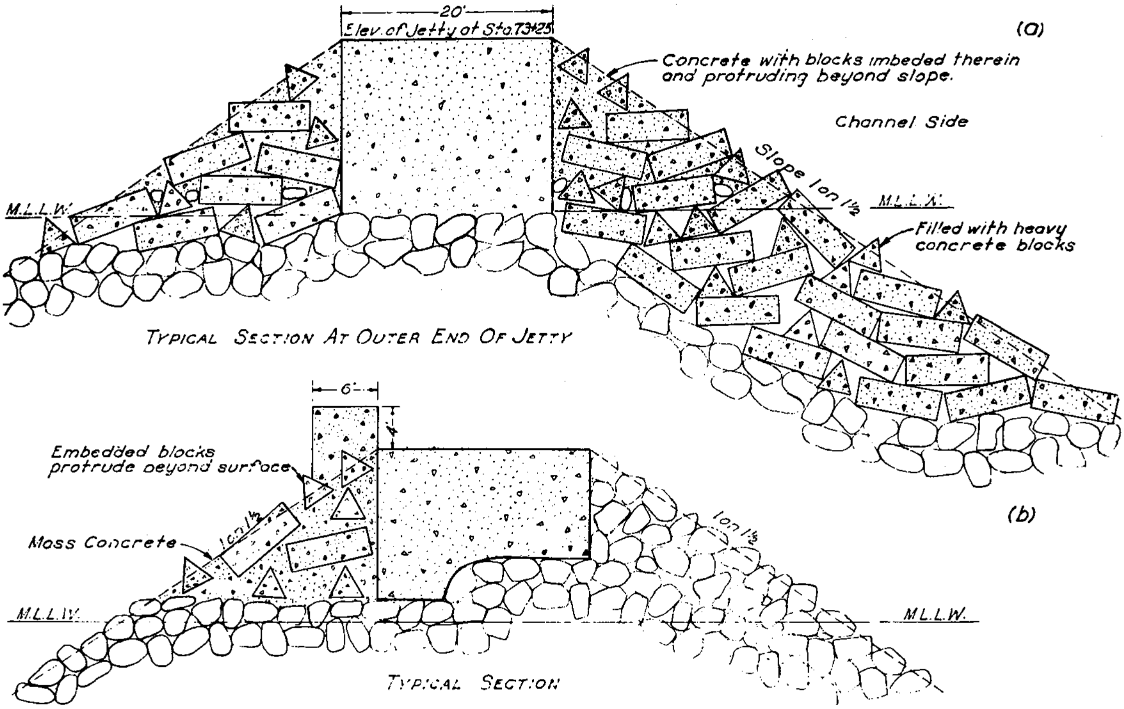


Fig. 6. North Jetty -- Humbolt Harbor and Bay, California

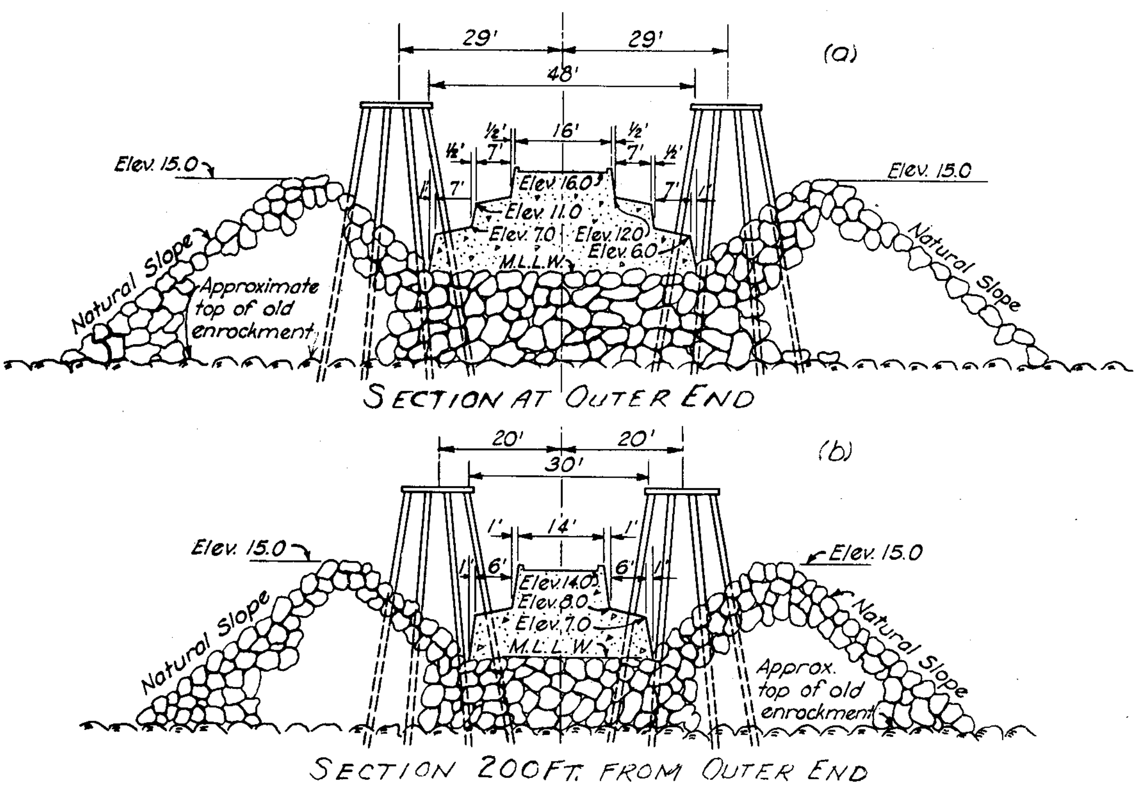
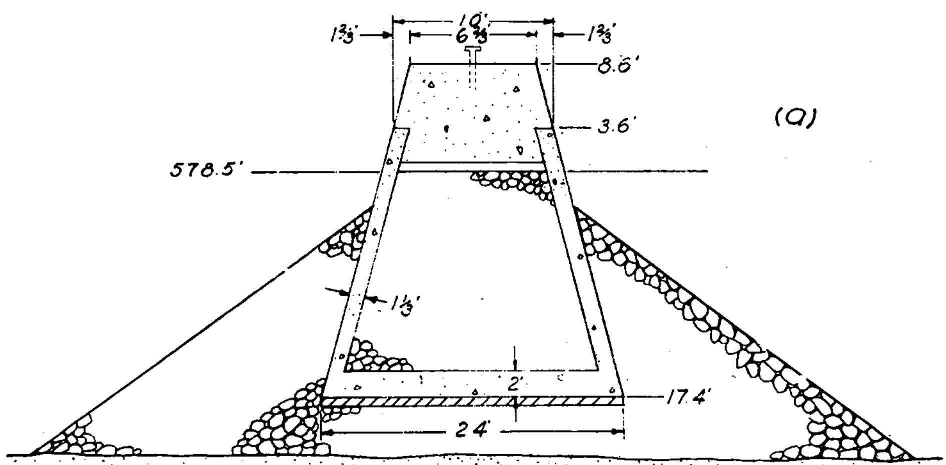


Fig. 7. North Jetty -- Umpqua River, Oregon

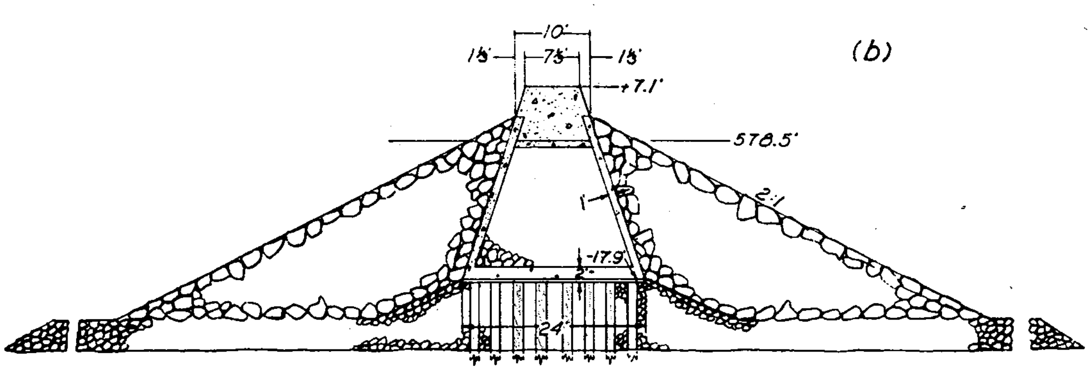
of the foundation rubble is dressed with crushed stone and leveled by a diver before the caisson is placed. Periods of calm water are necessary to float the caisson into position and sink on the foundation. If properly designed and placed, caissons are satisfactory. Cross-sections of concrete caisson structures are shown in Fig. 8.

(4) Sheet pile types include timber, concrete, and steel sheet pile structures. Timber is not suitable where marine borers can exist. Use of concrete piling is restricted by driving limitations. Steel sheet piling is used in several types of structures such as a single row of piling, with or without buttresses; two parallel rows with cross walls, and the cells thus formed filled with suitable material; and cellular steel sheet pile structures. The cellular type structure is widely used for breakwaters in the Great Lakes area. The life expectancy of steel piling depends upon water conditions at site. Steel piling may be used on any foundation where piling can be driven, permits rapid construction, but is subject to damage by sudden and unexpected storms during construction. Details and sections of steel sheet pile structures are shown in Figs. 9 to 11.

(5) Crib types are built of timber, and some of the compartments are floored. The cribs are floated into position, settled upon a prepared foundation by loading the floored compartments, after which all compartments are filled with stone. The structure is then capped with a timber superstructure which is usually replaced by concrete when the timber decays. Stone-filled cribs can withstand considerable



TYPICAL SECTION OF CAISSON JETTY ON RUBBLE FOUNDATION



TYPICAL SECTION OF CAISSON JETTY ON PILE FOUNDATION

Fig. 8

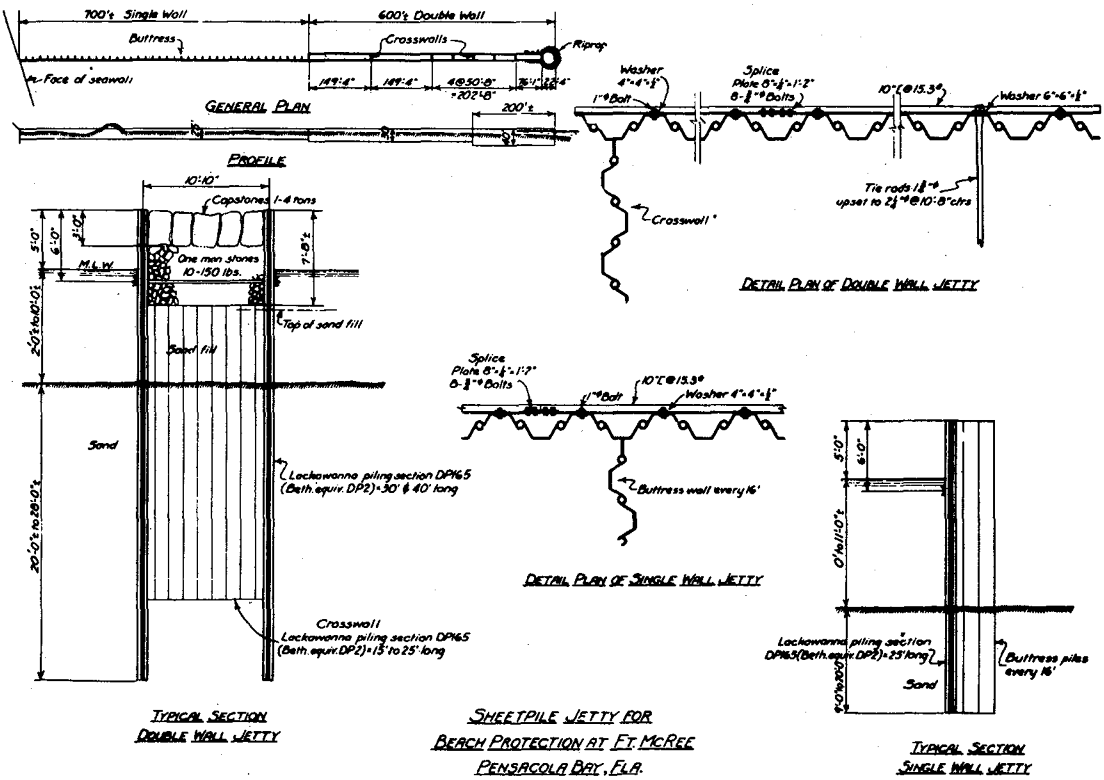


Fig. 9. Sheetpile Jetty for Beach Protection at Ft. McRee, Pensacola Bay, Fla.

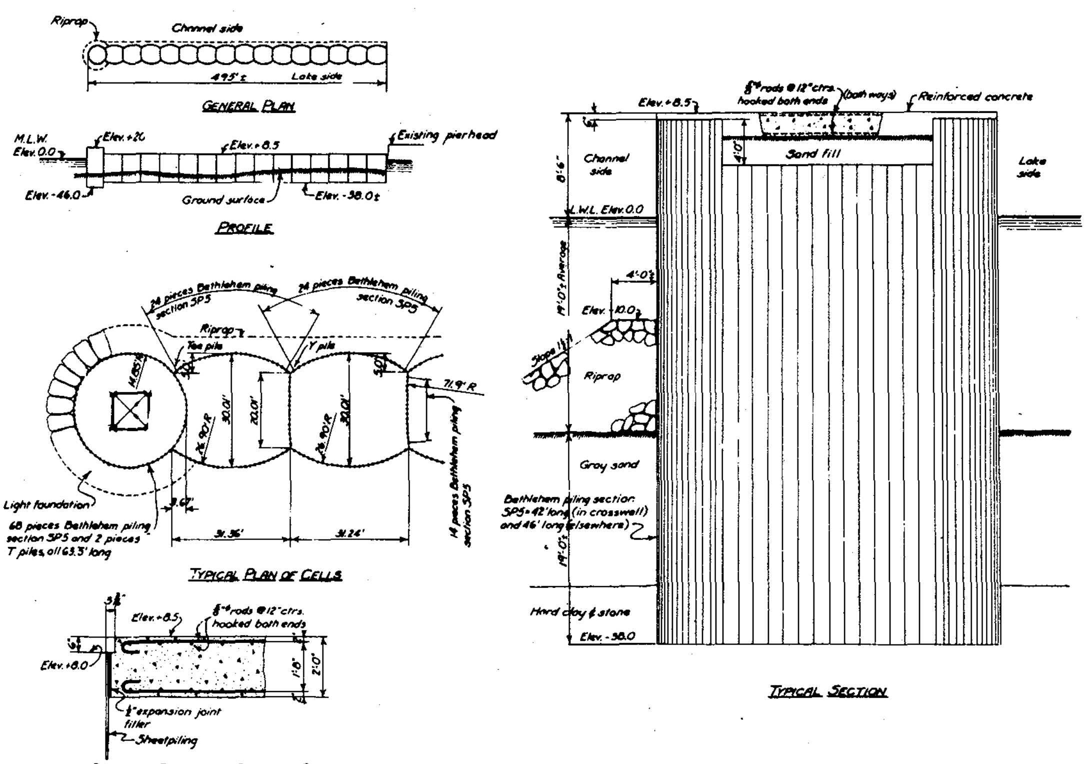


Fig. 10. Details of Steel Sheet Pile Jetty

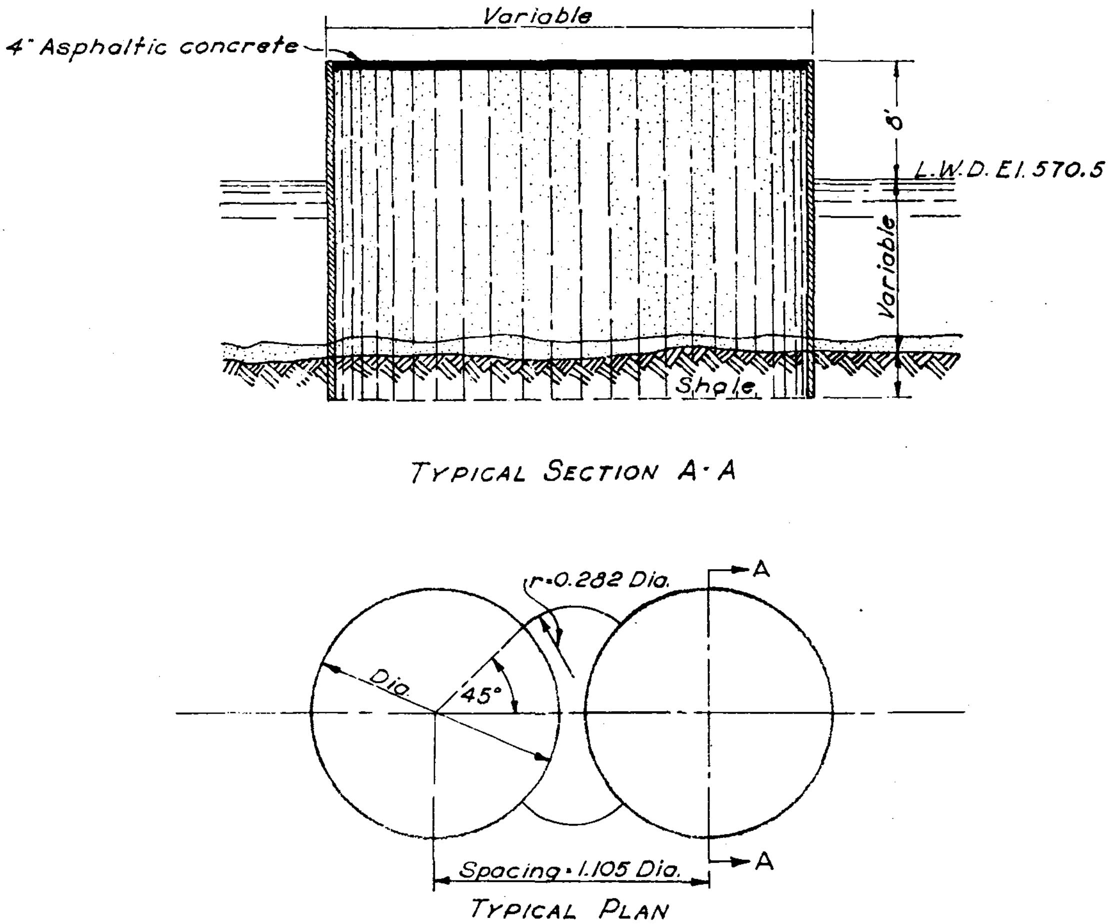


Fig. 11. Circular Type of Cellular Jetty

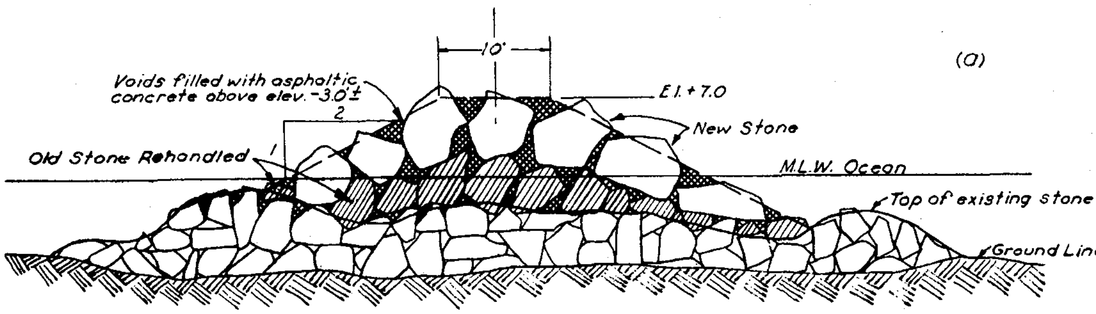
settlement and racking without rupture. Such structures are suitable for depths up to 50 ft. or more. Foundations are the same as for caissons but do not require such careful dressing. Settlement of foundation will bleed stone from the unfloored compartments but arching of stone over a cavity may prevent detection of stone loss by visual inspection. Timber structures are not suitable for salt water where marine borers may occur. In fresh water, timber-crib structures give long and satisfactory service.

(6) Solid-fill jetties are sometimes required to stop sand movement as well as direct currents. A core of well-graded stone, having a minimum of voids, with a cover of larger stone and an armour of heavy rubblestone is a common type. Caisson and sheet-pile structures are two other types of solid-fill structures.

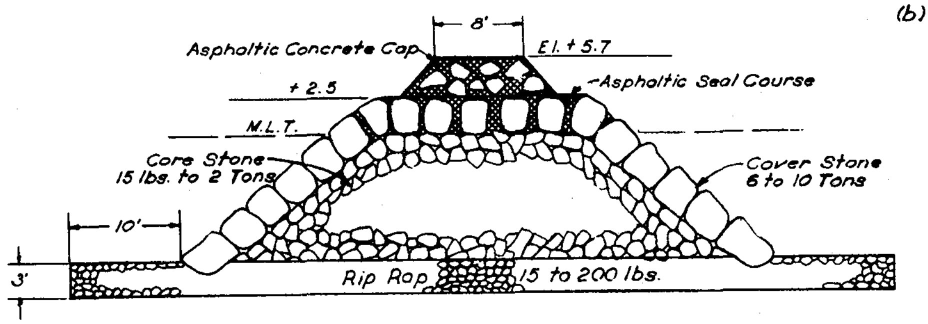
(7) Asphaltic materials have been used to fill the voids of rubblestone structures above the low-water line. The record of such structures is not impressive (see Fig. 12).

FACTORS AFFECTING DESIGN

The physical characteristics of the site must be determined by hydrographic and topographic surveys and sub-surface investigations. Surveys should extend up and down the coast to provide a complete record of conditions before construction, for future needs of comparison. Usually the fundamental problem is to create and maintain a satisfactory channel across the bar. Two jetties are necessary except under very unusual conditions. In determining the area of section between the jetties, requirements of navigation as well as tidal and stream flow must be con-



TYPICAL SECTION - STONE AND ASPHALTIC 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

Fig. 12

considered. On small streams or bays, navigation may require channels considerably in excess of the ideal hydraulic section. The natural movement of sand along the coast requires thorough study, as do wave forces developed at the site and their direction of approach. These subjects have been discussed at this conference and will not receive further consideration in this paper. Model studies of ocean entrances have not consistently given reliable results and, while the technic of making such studies is improving, their results must be accepted with caution. Selection of the exact site for the jetties is a big problem, often not given sufficient study. The cost of construction should not be the sole criterion of jetty location. Local sources of materials is a factor in design, often a controlling one.

DESIGN AND LAYOUT OF JETTIES

Design of a jettied entrance resolves itself into two problems; namely, first the fundamental one of siting the structures so as to create and maintain a satisfactory channel through the entrance, and second, to resist the forces created by ocean storms with a minimum amount of maintenance. Both problems involve many factors difficult to evaluate. As a preliminary step in making a layout for jetties at the entrance to any harbor, a careful study with field observations should be made of direction of tidal flow, both flood and ebb. On the north Pacific Coast where there is a wide dirunal inequality in the tidal range, the ebb is the stronger and predominant influence. The direction of storm waves should also be considered. In general, works should be laid out to conform with the main ebb current, rather than to oppose this, with a view perhaps to reducing wave action in the channel.

Several attempts have been made to determine the relation between the tidal prism and the sectional area of the entrance channel for natural maintenance of the channel without excessive currents. In a study of the relation of tidal prisms to channel areas of waterways on the Pacific Coast O'Brien (1931) proposed the formula:

$$A = 1000\ \bar{V}^{\,0.85} \tag{1}$$

where: $\bar{V}$ = volume of the tidal prism between MLLW and MHHW in square mile-feet.

A = area of the entrance channel section below mid-tide in square feet.

A study of the Sacramento, San Joaquin, and Kern Rivers in California (U.S. Congress, 1934) yielded a ratio of 1.08 square feet of sectional area per acre-foot of mean tidal prism for unrestricted estuaries and a value of 0.82 for restricted estuaries. Other values have been suggested for the ratios between the sectional area of discharge channel and acre-foot of tidal prism, but no one value is generally applicable to all entrances and results are often disappointing. As pointed out by M. A. Mason in a report of the Committee on Tidal Hydraulics (1950), "Study of the jettied entrances of the United States shows that the predicted results of the improvement as to entrance channel conditions were actually achieved in very few cases, either with respect to concentration of flow or protection."

Where a natural section can be found near the entrance, as is generally the case, more dependence should be placed in this than in theoretical or computed sections. The section produced by nature has been developed and maintained throughout the years and represents the effect of a summation of all the various influences, both favorable and unfavorable, which are at work in the vicinity. The main ebb velocity needed for the maintenance will be found to be much higher than is required for channel maintenance in non-tidal waters.

Navigation requires the channel to be of sufficient dimensions, proper direction, and to be easy to enter and follow. Vessels should not be required to move along a weather shore when entering or leaving the entrance, and they should not have a beam sea in a narrow channel. These requirements fix the direction of the channel in the quadrant of the prevailing heavy storms. The effect upon adjacent beaches must be carefully studied and, if possible, the final design should include features to prevent deterioration of the adjacent beaches. Artificial feeding of the down drift beach may be necessary. If the channel axis is approximately perpendicular to waves from the heaviest storms (may differ from wind direction), the tendency of the sand to be driven into the channel will be minimized. Severe storms from different directions will require a compromise. Papers on the natural and artificial movement of sediment have been presented in Part 3 of these proceedings and the subject will not be covered in this paper.

Since cost of transporting materials to the site is generally a large percentage of the cost of the work, a thorough search of the locality should be made for materials suitable to use, such as natural rock of sufficient hardness and strength to resist wave action.

Wave action is the most important source of the forces which a jetty must resist. The fundamentals of wave theory and the development of basic design data have been covered in Part 1 of these proceedings and further treatment of those subjects is considered unnecessary. With wave characteristics determined, the type of jetty and the depth of water at the structure both enter into determining the wave force applied against the structure. If the water is deep enough for waves to reach a vertical surface, approximately perpendicular to their line of travel, before breaking, a wave will rise against the vertical surface to about twice its normal height without breaking and be reflected back against the following wave with an apparent cessation of horizontal movement. The result is a standing wave which oscillates vertically against the surface with the same time period as the open-water wave but about double its normal height. This phenomenon of oscillating waves is known by the term "clapotis." At the International Navigation Congress held in Cairo in 1926, the term "standing wave" was adopted for the French term "clapotis" but since the term "standing wave" is employed to describe the wave of the hydraulic jump in stilling basins in dam design the term "clapotis" is generally used. Reference is made to Chapters 22, 23, and 24 for a discussion of the different methods of determining the forces of an oscillating wave.

In the United States jetties are seldom constructed with vertical walls extending to depths sufficient to avoid breaking waves. It is common practice in this country to construct rubble-mound jetties and most of those which have an upper section with a vertical or stepped face are of the stone and concrete type having a rubble mound extending from sea bottom to about mean low water with the concrete section above that elevation. Such jetties have generally given satisfactory service, especially if the rubble mound is able to resist the wave action. Occasionally, the rubble mound may need some repairs but if such repairs are made when necessary, the concrete super-structure will generally not be injured.

Most jetties are subject to breaking waves in some portion of their length. This may not be the case when the jetty is first constructed, but due possibly to the upsetting of natural forces, changes in the bottom configuration may produce conditions which will cause breaking waves. The extreme shock pressure which may result from a wave breaking against a vertical face may be avoided by a stepped face which breaks up the wave by absorbing the wave energy in a series of lesser impacts which are not simultaneous. A sloping wall may pass the wave over the super-structure and cause injury to the channel slope of rubble.

Rubble-mound types of marine structures have been constructed for thousands of years but there has been a dearth of engineering literature dealing with the design of such structures. Design of rubble marine structures has, in general, been based upon the behavior of existing structures and the experience of the designing engineer. European engineers have favored flat side slopes while American engineers have consistently used natural slopes ranging from 1 on 1.1 to 1 on 1.5. No attempt was made to theoretically determine the size of stone or the slope to be used until de Castro (1933) published the following formula to determine the weight of individual stones required for stability on the side slope of the structure considering the design wave.

$$W = \frac{704\, H^3\, s}{(\cot\phi + 1)^2 \sqrt{\cot\phi - 2/s}\,(s - 1)^3} \qquad (2)$$

in which:

W = weight of individual stones, in kilograms

H = wave height, in meters

s = specific gravity of the stone

ϕ = angle of side slope with the horizontal.

de Castro's formula is based upon the following theoretical considerations and approximations:

(a) The destructive action of a wave is proportional to its energy and assuming as a rough approximation, that storm waves heights are proportional to their length, uses H^3 as the value of the wave energy.

(b) The weight of the stone necessary to withstand the wave energy varies directly as the density in air, and inversely as the cubes of their density submerged in water or $\frac{s}{(s-1)^3}$.

(c) The stability of a stone subject to wave action is inversely proportional to some geometric function of the slope upon which it rests.

Subsequent to de Castro's work Iribarren (1949) published the following formula:

$$W = \frac{k_1\, H^3\, s}{(\cos\phi - \sin\phi)^3\,(s - 1)^3} \qquad (3)$$

in which:

k_1 = 15 for natural rock-fill structures, and

k_2 = 19 for artificial block structures

and all other symbols are as used in de Castro's formula (equation 2). Within the past few years there has been considerable discussion among coastal engineers regarding the merits of the Iribarren formula.

Presenting the formula in terms of units of measurement used in the United States, it becomes:

$$W_1 = \frac{k_1' H^3 s}{(\cos\phi - \sin\phi)^3 (s - 1)^3} \tag{4}$$

in which:

W_1 = weight of individual stone, in tons of 2,000 pounds

k_1' = 0.000468 for natural stones

k_1' = 0.000593 for artificial blocks

H = design wave height, in feet

s = specific gravity of the stone or blocks

ϕ = angle of the slope with the horizontal

In recent years Mathews (1948) of the Los Angeles District, Corps of Engineers, submitted for discussion, the following formula:

$$W_1 = \frac{6\,w\,H^2\,T}{(w - 64)^3 (\cos\phi - 0.75 \sin\phi)^2} \tag{5}$$

in which:

T = wave period in seconds

w = unit weight of the stone in lbs. per cu. ft., and all other symbols are as given for equation 4.

From observations on hydraulic placer mine operations Rodolf, co-author of this article, had concluded that the stability of a stone subjected to the action of a stream of water from an hydraulic "giant" was approximately inversely proportional to some power of a function of the slope that had been generally accepted for determination of earth pressure back of a wall, namely $\tan(45^\circ - \phi/2)$, and applying the results of his mining-experience, he submitted the following formula:

$$W_1 = \frac{H^2\,T\,s}{600 \tan^3 (45^\circ - \phi/2)\,(s - 1)^3} \tag{6}$$

This formula is intended to contain a small factor of safety to provide for the occasional individual wave which is higher than the highest wave.

The formulas of de Castro (1933), Iribarren (1949), Mathews (1948), and Rodolf are of a common type but show a considerable variation of results. As a means of verifying his coefficients Iribarren (1949) compared slopes determined by his formulas with a known example (Iribarren and Nogales y Olano, 1950). There follows a description of data used for this comparison:

> "Unfortunately those necessary details of each particular case, and especially the damages experienced, are difficult to obtain. Therefore, in this study, we are going to make special mention of one of singular interest.
>
> We refer to the interesting compilation on the part of Argel concerning weights of stones or blocks and their corresponding stable slopes at various depths, after repairs of numerous damages to deficient slopes. This outstanding compilation was made by Messrs. J. Larras and H. Colin in their article of December 1947 published in the periodical Travaux (see references at end of chapter).
>
> From page 609 of that compilation are obtained the following data, showing the corresponding batters, depths, and weights of blocks or stones.

Materials	Minimum weight (Metric tons)	Maximum batter	Minimum Depth (meters)
Artificial blocks	50	5/4	- 5
Natural stones	4	3/2	- 11
" "	1	3/2	- 14
Quarry waste	-	2/1	- 18

There can also be adopted, as a stable upper surface batter, that of 3/1 formed by 50-ton concrete blocks, adopted for strengthening the North dike which, according to the cited article, has withstood perfectly even the worst storm (3 February 1934) ever suffered by the port of Argel and which destroyed a large part of the Mustafa dike. Likewise, we can adopt the toe depth of 35 meters, which this North dike reaches."

To measure the accuracy of the other three formulas given above, slopes for the conditions used by Iribarren were computed by the formulas of de Castro (1933), Mathews (1948) and Rodolf and in the comparison of the results shown in Table 1, the profiles determined by each formula are arranged in the order of their accuracy as determined by comparison with measured profile (Fig. 13).

TABLE 1

COMPARISON OF CALCULATED AND MEASURED BREAKWATER SLOPES

Measured slope	Depths	Rodolf	Iribarren	Mathews	de Castro
1 on 3	+5 to -5	1 on 3.18	1 on 3.51	1 on 2.19	1 on 2.22
1 on 1.25	-11	1 on 1.37	1 on 1.83	1 on 1.32	1 on 1.18
1 on 1.5	-14	1 on 1.55	1 on 1.53	1 on 1.44	1 on 1.02
1 on 1.5	-18	1 on 2.85	1 on 1.67	1 on 2.11	1 on 1.07

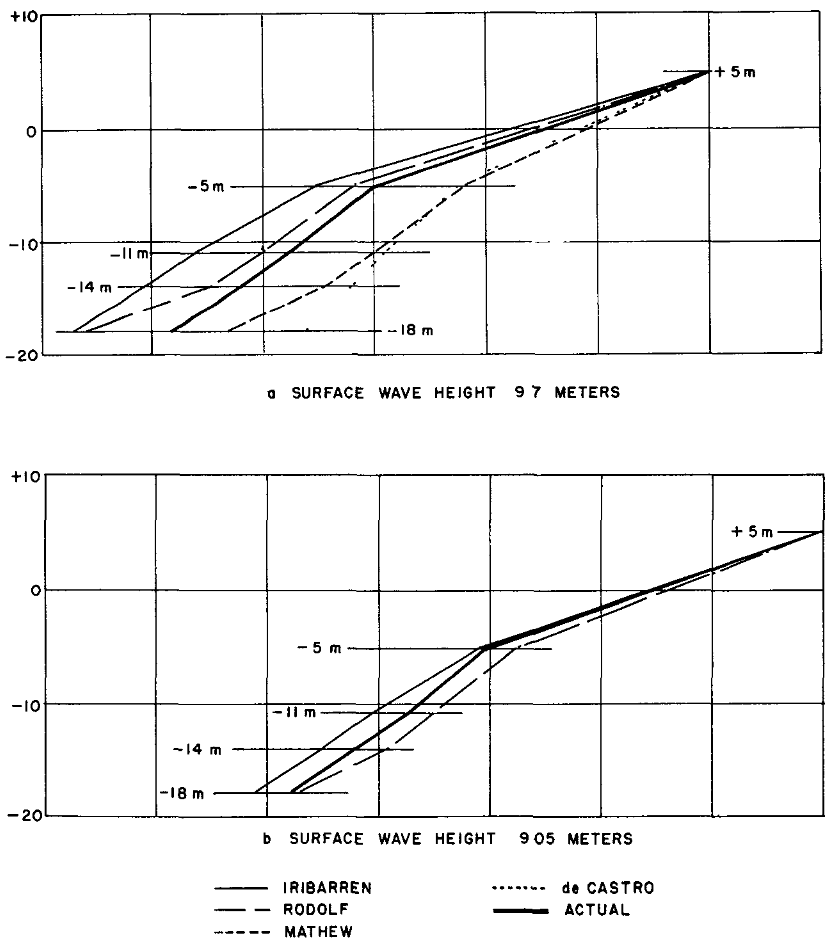


Fig. 13
Comparison of computed slopes with actual slope of rock dike at Argel, using formulas proposed by de Castro, Iribarren, Mathews, and Rodolf

The height of wave breaking on the dike had been reported as 9 meters, but Iribarren deduces that the wave was actually 9.7 meters and this height was used in computing the slopes shown above. Then, making new assumptions, a height of 9.05 meters is computed by Iribarren and he recomputes the slopes. The following tabulation shows the results from the formulas of both Iribarren and Rodolf.

TABLE 2

COMPARISON OF IRIBARREN AND RODOLF FORMULAS

Measured slope	Depth	Iribarren	Rodolf
1 on 3	+5 to -5	1 on 3.04	1 on 2.75
1 on 1.25	-11	1 on 1.68	1 on 1.21
1 on 1.5	-14	1 on 1.43	1 on 1.29
1 on 1.5	-18	1 on 1.51	1 on 2.04

From the above comparison it appears that both Iribarren's and Rodolf's formulas closely approximate the real profile and the following quotation from Iribarren can be applied to both formulas:

> "In this way we obtain a theoretical profile, shown in Fig. 3', similar to 3, even more closely approximating the real profile; but the really interesting fact is that both figures, whose calculated wave heights differ by less than 8 percent, a degree of approximation that we consider difficult for anything practical to really exceed, are now authentically confirmed by the very important direct observation from this interesting compilation. It is also now confirmed, undoubtedly, through authentic direct observation, despite the simplifications one is forced to introduce into the complex subjects of maritime engineering that the degree of approximation really obtained is superior to that of many calculations of engineering on terrestrial subjects, in which, even legally, are imposed large safety factors, generally greater than two and frequently approximating three."

A formula, similar to the above formulas is deduced by Epstein and Tyrell (1949). Tests now being conducted by the U.S. Waterways Experiment Station will investigate experimentally rubble-mound breakwaters. The formulas should be checked by the experimental determinations.

The following table gives the angle and value of the term $(\cos\phi - \sin\phi)^3$ for different slopes.

TABLE 3

Slope	Angle	$(\cos\phi-\sin\phi)^3$	Slope	Angle	$(\cos\phi-\sin\phi)^3$
1 on 1.5	33°41'	0.0214	1 on 3	18°26'	0.2530
1 on 1.75	29°45'	0.0515	1 on 3.25	17°06'	0.2898
1 on 2	26°34'	0.0894	1 on 3.5	15°57'	0.3238
1 on 2.25	24°58'	0.1308	1 on 4	14°02'	0.3853
1 on 2.5	21°48'	0.1729	1 on 4.5	12°32'	0.4375
1 on 2.75	19°59'	0.2139	1 on 5	11°19'	0.4825

There are several factors affecting the permanence or stability of rubble-mound structures which are not taken into account in the above formulas. Among these may be mentioned:

1. The angle or direction at which seas strike the breakwater. This vitally affects the slope presented to the sea.
2. The height of the structure.
3. The grading of stone sizes.
4. The manner of placing.
5. Shape of stones and texture or composition.

At the end of a jetty or breakwater the seas often strike in a direction almost at right angles to the line of the jetty and the result is a running sea parallel to the rock surface and a slope considered in the formulas as zero opposed to the force of the sea. In all the formulas proposed this calls for the smaller sizes of stone. As a matter of fact, however, the largest stone is required in such a position. At the end of the south jetty at mouth of Columbia, although built of the heaviest stone available, this cross sea caused a raveling or cutting off of the super-structure enrockment to low-water level, at a rate of about 300 ft. of jetty per annum, until such action was prevented by construction of a heavy concrete superstructure terminal.

Similarly, at the mouth of the Umpqua River, heavy "wing" blocks of concrete (136 tons) were poured on a level-rubble foundation at low-water elevation to protect the footing of a 1,700-ton main block, yet one of the 136-ton blocks 15 ft. x 15 ft. x 8 ft.) was shifted horizontally 20 to 25 ft. during the first storm. The other blocks could not be observed. These experiences show the fallacy of depending on small or moderate size stone on flat slopes if exposed to a sea which can break against or across it, and throws doubt on the use of functions of the slope of the structure as a principal determining consideration. If a breakwater or jetty is not high enough to prevent the seas from crossing over in force, stones will be removed from the crest (zero slope) and the back side of the structure also will be attacked. The back slopes of concrete capped structures of moderate height, are often attacked by such over wash.

It is, accordingly, evident that while a reliable formula for size of stone in breakwaters and jetties is most desirable, there are so many influencing factors and construction limitations involved, which cannot well be taken into account in a formula, that the problem usually resolves itself into the more practical considerations of the materials available at reasonable cost and the methods which can be used for construction; always bearing in mind that large stone of high specific gravity should be used, if available, for highly exposed structures. The larger the better.

In addition to currents caused by waves, littoral or alongshore currents must be considered when designing for protection against scour. Where river currents are present, special precautions may be necessary to guard against excessive scour along the toe and at the end of the jetty. If the proposed structure will obstruct or change the direction of river currents, scour at and around the end during construction may and probably will occur. Where original depth on the line of the structure is not sufficient to furnish protection to the base, the scour by currents around the end of the structure during construction may not be objectionable, as material will be removed without cost and thus permit the base of the structure to be placed at a depth assuring better protection, but will necessitate the use of more rubble than indicated by the original profile. Cases on record show that as a jetty was constructed outward from the land connection, scour around the end caused depths nearly twice those shown on the original profile, and necessitated the use of two or three times as much rubble stone as the original profile indicated. Possibilities of such scour should be investigated in determining the amount of rubble for the foundation. If advisable to control the scour, a mat of stone spread over the area in advance of jetty construction, or a rubble apron extending a long distance ahead of the main enrockment, are possibilities for controlling or preventing the scour. It cannot be stated with certainty that such scour can always be avoided.

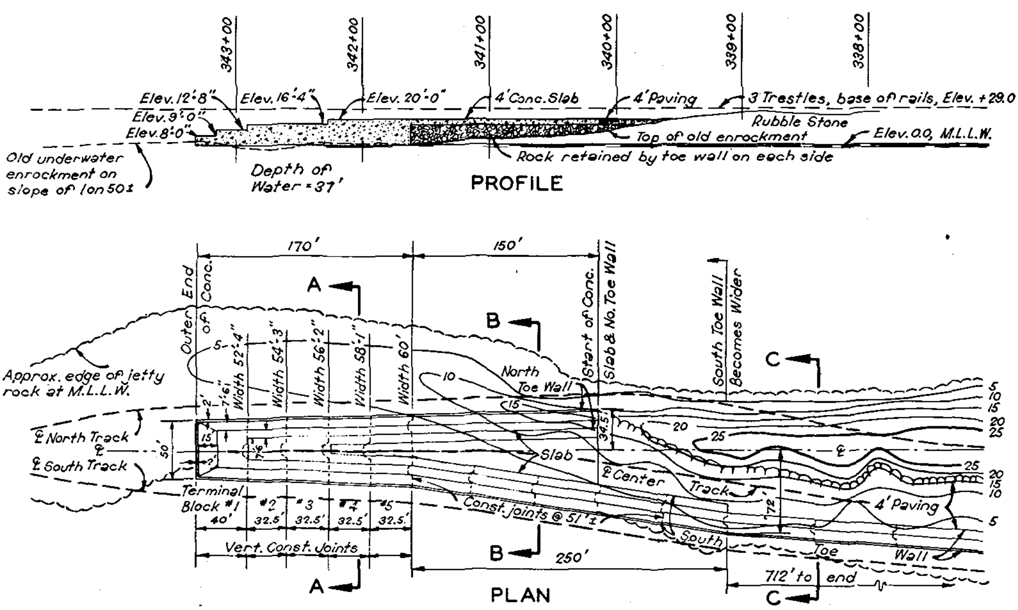
Analysis and design of a jetty structure follow the usual methods and procedures for design of any structure. Resistance to overturning, safety against sliding, and maximum pressure against the foundation are all investigated and designs are developed to satisfy the requirements as for a land structure, such as a dam or retaining wall. It is assumed that the reader is familiar with the usual design procedure and further discussion of the details of design in this paper is not considered necessary. The factors that must be considered in design of a jetty, which do not enter into design of a land structure, are: siting of the jetty to accomplish desired results yet provide for safe navigation, height of structure to provide protection to the sheltered area, avoidance of a location where the waves will build up, and the angle at which the waves strike the structure.

AIDS TO NAVIGATION

The District Commander of the U.S. Coast Guard should be furnished the following information: (a) advice as to authorization of a project involving construction of a jetty, (b) the proposed construction schedule, (c) maps showing the final location of the structure. During construction, temporary aids to navigation should be maintained, if necessary. Information as to installation or discontinuance of these aids should be furnished to the Coast Guard so that such information may be included in "Notice to Mariners" published by the Coast Guard. Changes in depths and location of channels should be reported to the Hydrographer, Hydrographic Office, U.S. Navy, Washington, D.C.

CONSTRUCTION OF JETTIES

Railroad trestles are often used for the construction of rubble jetties in areas subject to frequent heavy sea action, where the use of floating plant is impracticable. A single-track trestle will suffice for construction of a jetty having up to about 40 feet top width. The trestle should be several feet above finish grade of jetty to allow free dumping and placing under the trestle. Small rock may be dumped to reinforce the trestle bents. Larger rock is handled on flat cars and may be pushed over the sides by means of a crawler-type shovel, which moves from one flat car to the next, or standard dump cars may be used. The largest rocks used as a final covering, especially on the seaward side, are dumped in the same manner and placed in final position by means of a track crane. For construction by trucks the structure is built to approximate finished section as it progresses into the sea and a roadway is maintained on top of the jetty, or on a trestle roadway. The large stones are placed by crawler-type crane. Sections of a jetty section showing use of single and double trestle and track are shown on Figs. 14 and 15. Fig. 16 shows the detail of a trestle.

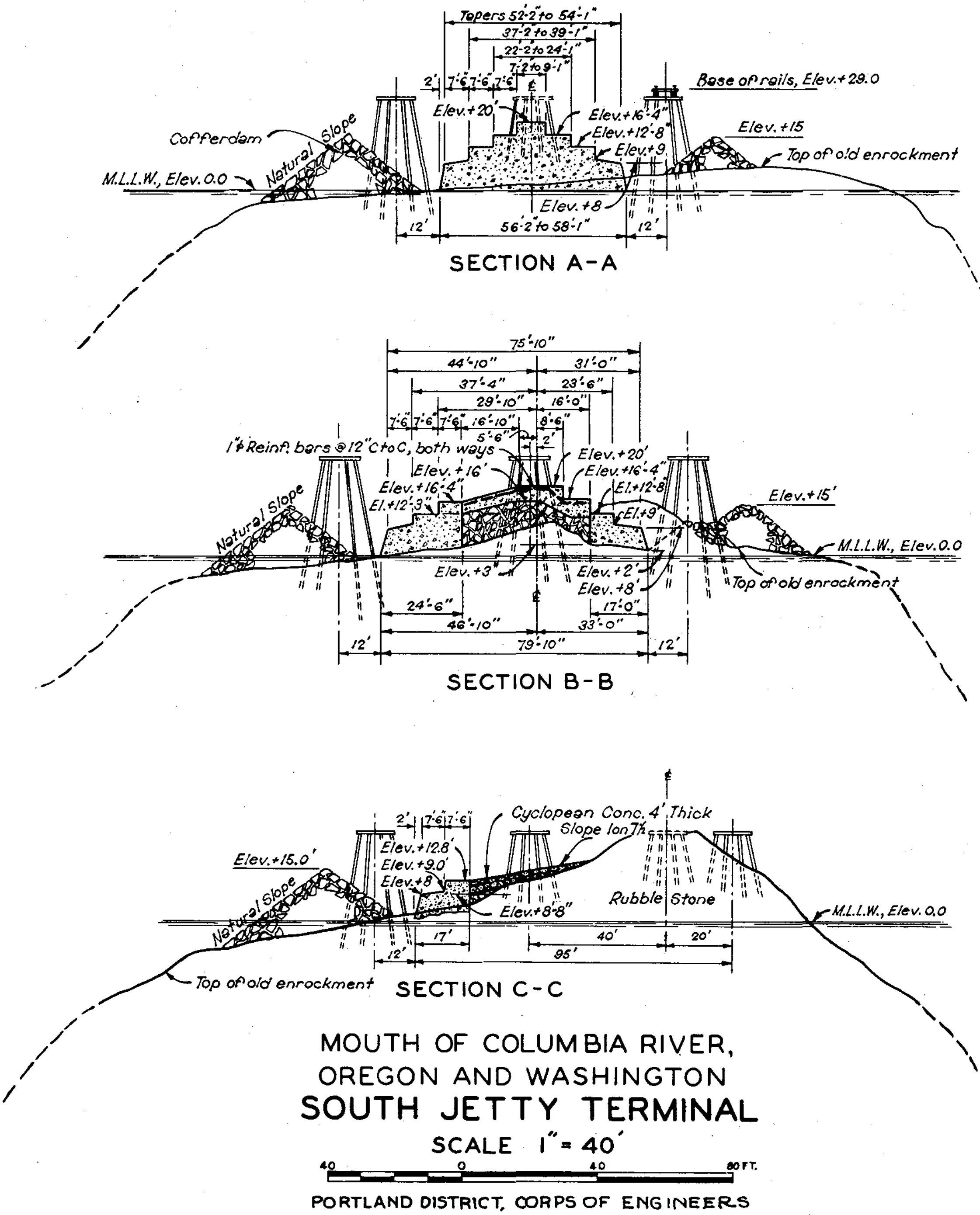


MOUTH OF COLUMBIA RIVER,
OREGON AND WASHINGTON
SOUTH JETTY TERMINAL

Scale in Feet

PORTLAND DISTRICT, CORPS OF ENGINEERS

Fig. 14



Tapers 52'-2" to 54'-1"
37'-2" to 39'-1"
22'-2" to 24'-1"
7'-2" to 9'-1"
Base of rails, Elev.+29.0
Elev.+20'
Elev.+16'-4"
Elev.+12'-8"
Elev.+9
Elev.+15
Cofferdam
Natural Slope
Top of old enrockment
M.L.L.W., Elev. 0.0
Elev.+8
56'-2" to 58'-1"
12'
SECTION A-A
75'-10"
44'-10"
31'-0"
37'-4"
23'-6"
29'-10"
16'-0"
16'-10"
8'-6"
5'-6"
1" Reinf. bars @ 12" C to C, both ways
Elev.+16'
Elev.+16'-4"
El.+12'-3"
Elev.+20'
Elev.+16'-4"
El.+12'-8
El.+9
Elev.+15'
Natural Slope
M.L.L.W., Elev.0.0
Elev.+3"
Elev.+2'
Elev.+8'
Top of old enrockment
24'-6"
46'-10"
17'-0"
33'-0"
79'-10"
12'
SECTION B-B
Cyclopean Conc. 4' Thick
Slope 1 on 7½
Elev.+15.0'
Elev.+12.8'
Elev.+9.0'
Elev.+8
Elev.+8'8"
Natural Slope
Rubble Stone
M.L.L.W., Elev. 0.0
17'
40'
20'
95'
12'
Top of old enrockment
SECTION C-C
MOUTH OF COLUMBIA RIVER,
OREGON AND WASHINGTON
SOUTH JETTY TERMINAL
SCALE 1" = 40'
40 0 40 80 FT.
PORTLAND DISTRICT, CORPS OF ENGINEERS

Fig. 15

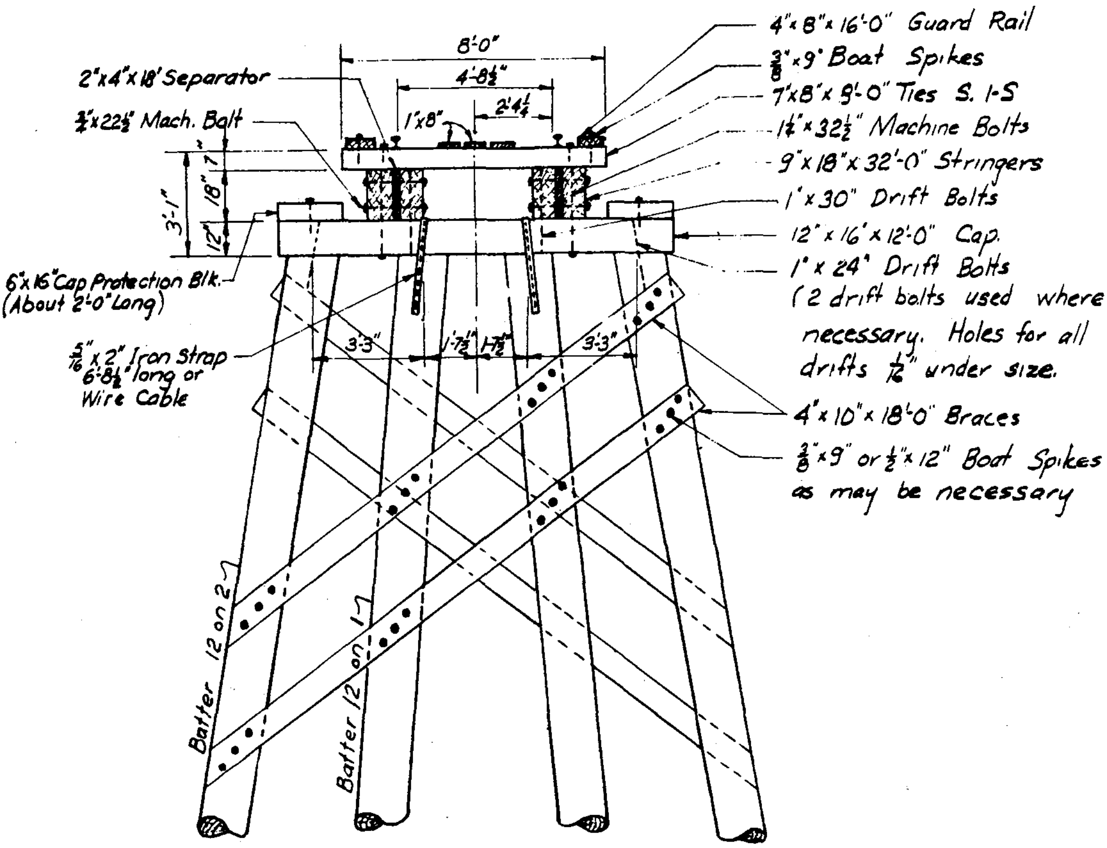


Fig. 16. Detail of Jetty Trestle, South Jetty, Columbia River, Oregon

Floating plant may be used for the construction of a jetty in waters where wave action is not too continuous or where the vessels may work in sheltered water on the lee side of the structure. This method consists of moving the material to the site in vessels and either dumping the material or placing by floating derrick.

Quantities of stone used in jetty construction may be determined by weighing, or by measurement of volume by cross-sectioning and calculation of weight. The former method is preferred. If the structure is built with floating plant the weight of the stone is usually determined by gages placed on the barges to measure the displacement of the vessel when loaded and the weight of the stone is calculated therefrom.

CONCRETING EQUIPMENT

The methods employed in placing the concrete portion of a composite breakwater or jetty vary with each individual project, being influenced by such factors as the magnitude of the project, plant readily available, transportation facilities, and weather and sea conditions. Depending upon the design of the structure, concrete must sometimes be poured below the mean water level, thus necessitating the use of extensive protective cofferdam arrangements on a wide stone base. Specifications usually require the continuous pouring of each monolith block to completion. This may call for fast, concentrated effort between tides.

In areas of heavy wave action the attack on the structure is often severe between low water and half-tide level. It is accordingly necessary to base the concrete superstructure at elevation of mean-lower-low-water or lower, if practicable.

The equipment and materials involved in concreting operations should be located conveniently close to the structure site. To accomplish the desired proximity, construction of access roads may be necessary. Stock piles of aggregate material in various gradations, as well as dry storage facilities for cement should be grouped about a central batching plant. On large jobs careful attention should be given to equipment and arrangements for high speed economical operation.

For the most efficient control of operations, the mixer should be reasonably close to the placing area. For small sections employing a single tramway, a side platform supported by piles has been used. For larger sections, with a tramway on both sides, the mixer usually can be supported by a platform between the tramways. On large jobs requiring large daily placement from trestle work it may be found impracticable to provide sufficient storage space at the site and mixed concrete may have to be transported a considerable distance (3-1/2 miles on railway cars at mouth of Columbia). The concrete is transported by dump buggies, trucks, or railway cars, using inclined chutes for placement to avoid a free vertical drop of more than about 5 ft.

In closing this paper it may not be amiss to again quote from Iribarren when he states:

> "However it is not logical to apply strict results, obtained by means of the application of theoretical formulas to sea conditions when it is the general rule to employ ample factors of safety for land conditions."

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CHAPTER 27

DESIGN AND CONSTRUCTION OF GROINS

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INTRODUCTION

Groins are frequently used for shore protection and improvement. Not infrequently the owner of shore property who has had groins built to protect or improve his property is disappointed with the results. More often than not this unhappy situation must be attributed to the fact that too much was expected by the owner.

The owner in such a case is not properly to be criticized, because a great deal remains to be learned about groins; their effects, their proper design and construction.

In the present state of the art of shore protection and improvement it is not possible to design and build groins without facing numerous uncertainties, particularly in the area of advance determination of the results which will be accomplished. This condition is faced frankly at the very beginning of this paper and should be kept in mind throughout the consideration of the subject of groin design and construction.

This paper presents a digest of what is considered by the writer to be the best current practice. No pretense is made for the development of original ideas on the subject. The writer is indebted to many engineers who have contributed accounts of their experiences to the literature, and to the members of the Beach Erosion Board and its staff, especially Dr. Martin A. Mason.

DEFINITIONS

In order to assure common understanding, it would be well to begin with the few definitions which follow:

Groin. A groin is a shore protection and improvement structure, narrow in width (measured parallel to the shoreline) which extends from a point landward of the shoreline into the water a distance (length) which may vary from less than one hundred to several hundred feet. Groins are usually built to trap littoral drift, or to retard erosion of the shore. Groins may be classified as impermeable or permeable. An impermeable groin is a solid, or nearly solid, structure. A permeable groin has openings through it.

Littoral current. A current moving generally parallel and adjacent to the shoreline.

Littoral drift. The material that moves generally parallel to the shoreline under the influence of hydrodynamic forces.

Berm. The nearly horizontal formation along the beach caused by deposit of material under the influence of waves.

PURPOSE OF A GROIN SYSTEM

The purpose of a groin system is to modify the rate of supply of material to the shore and the rate of loss of material from the shore, in order to retard erosion or to cause accretion.

The condition of the shore at any time depends on the material-energy balance. By material-energy balance is meant the relation between the shore material available and the energy, usually in the form of waves, which acts on the material.

A groin system modifies the material-energy balance. If the modification is a correct one, successful results are obtained, otherwise not.

A shore erodes when the rate of supply of material to the shore is less than the rate of loss of material from the shore. Equilibrium results when the rate of supply equals the rate of loss. Accretion occurs when the rate of supply is greater than the rate of loss.

If, under natural conditions, a shore is eroding, it may be possible for construction of groins to reduce the rate of loss to such an extent as to reverse the erosive trend and cause accretion. Alternatively, under less favorable conditions, groins could cause a condition of equilibrium or retard the extent of erosion.

In considering the use of groins to solve a particular problem, it is first essential to decide what it is desired that the groins should accomplish. Is the owner primarily interested in retarding the rate of future erosion? Or does he wish to try to obtain a wider beach?

LIMITATIONS IN APPLICATION OF THE GROIN METHOD

Having ascertained the desired goal, we can then proceed to a determination of whether or not groins are likely to be successful in accomplishing this goal.

Groin systems are unlikely to be effective in certain shore environments. The shore may consist of gently sloping rock surfaces, more or less vertical wave cut cliffs, or may be composed of gravel, boulders, sand, mud, and shell. Some shores are covered with vegetable growth, algae, grasses, reeds, or mangroves, whereas the shores of sand, gravel, and boulders are usually clear of such growth. Groins are usually considered for use only for shores composed of sand or gravel. Use of groins in any other shore environment is so rare as to be of little general interest and consequently will not be considered in this paper.

Proceeding, then, to consider other limitations in the use of groins, in the usual case, where wave action is the most important factor in moving shore material, groins perpendicular to the shore will not be successful if the waves which are most effective in moving shore material approach the shore with their crests parallel to the shoreline. The reason for this is that immediately after construction groins must modify the wave action in order to have any effect. Subsequently the material, if any, impounded by the groins, as well as the groins themselves, also modify the wave action. With groins perpendicular to the shoreline, waves must approach the shore with their crests at an angle to the shoreline in order for the groins to have any appreciable effect on the wave action. Studies to date have not indicated that construction of groins at other angles than at right angles to the shoreline would yield better results. The principal reason for this is that waves approach the shore from different directions at different times.

Another limitation is recognized in the case where the objective is to cause a substantial amount of accretion. In such a case, groins alone, no matter how they are designed or built, will not be successful if the natural volume of littoral drift is negligible.

In order, then, to determine whether or not groins are likely to be successful in accomplishing the desired objective, we must first learn something about the shore processes active in the problem area. The required information is usually classified in current practice under three major headings:

a. The natural source of the material composing the shore;

b. The rates of supply of material to the shore and of loss of material from the shore;

c. The manner of movement of material to and from the shore.

A comprehensive and complete investigation to obtain this information involves, in current practice, studies of the geomorphology of the problem area, petrographic investigations of shore materials, and studies of shoreline and offshore depth changes, the winds and waves prevalent in the area, tidal or other water level fluctuations, and the effect of existing shore structures.

Description of methods of conducting these studies is outside the scope of this paper. Some of them need improvement and work is underway in that field. Methods of analyzing the results of these studies in order to obtain data, particularly quantitative data, also need improvement and progress is also being made in this area.

At the present time, however, it is very difficult, utilizing currently accepted methods, to obtain the basic data required. Shore protection engineers are now, and will be for some years to come, confronted with the hazards of attempting to base determinations of groin behavior on shore process data which are incomplete, or inaccurate, or both. Considerable ingenuity is needed, therefore, in making the most of shore process data which it is possible to collect.

Analysis of the obtainable shore process data and the probable effect of groins will result in a determination that groins will either be effective in accomplishing the desired purpose or that they will not. If it seems likely that groins will be effective, it is then necessary to check whether some other method will produce equally effective results at lower cost. Depending on the desired purpose, alternative methods which may be considered are seawalls, bulkheads, revetments, offshore breakwaters, artificial nourishment.

Before recommending groins, then, we must determine that groins will not only accomplish the desired purpose but that they will cost less than any other equally effective method.

In studies leading to a tentative conclusion as to the acceptability of groins, certain assumptions have to be made as to length, spacing, profile, and construction materials. If the preliminary studies favor the use of groins, these design features are then reviewed carefully and appropriate modifications made in the final design.

For preliminary studies, assumed dimensions can approximate those most commonly found in existing groin systems. The groin profile should have an inner shore horizontal section, a connecting slope, and an outer horizontal section. The length of the groins may be assumed as that necessary to extend to the 6-ft. depth. A length spacing ratio between 1 to 1 and 1 to 3 should be assumed. The groin profile should be assumed to approximate the desired beach profile down to the low water elevation, thence horizontal to the end of the groin.

For final design, the preliminary assumptions of length, profile, and spacing should be modified by application of the considerations presented in the following paragraphs.

GROIN LENGTH AND PROFILE

In most shore problems encountered in current practice, wave action is the most important cause of movement of shore material. The shore material most frequently involved in groin studies is sand.

Individual sand grains are transported by sliding, rolling, saltation (jumping), or suspension. Once a sand grain is put into suspension, the slightest movement of the surrounding water will produce a corresponding movement of the sand particle. As long as the particle remains in contact with the bottom, currents of more than 1 foot per second will be required to move even fine sand. It is probable that fine sand can be moved by high, long waves in depths of water as great as 160 ft. However, the amounts of sand thus moved are small.

So far as now known most of the movement of sand by wave action takes place in the zone extending from the shoreline to the breaker zone.

In the usual case we are considering, a groin acts by interfering, directly or indirectly, with the movement of sand particles caused by wave action. Complete interference with movement of sand particles would require a very long groin, extending, depending on the magnitude of waves in the particular area, to depths of 18 to 160 ft. It is uneconomical to build such long groins because the effectiveness of the outer portions is not commensurate with their cost. It is also undesirable, where there is a pronounced predominance in direction of littoral drift, to build such long groins because of the danger of resultant starvation and erosion of the adjacent shore in the down drift direction.

Best current practice recognizes the intimate relationship between groin length and profile by studying both together. For convenience it will be considered that a groin will be made up of three sections:

I. The horizontal shore section;
II. The intermediate sloped section;
III. The outer horizontal section.

The following steps are involved in selecting controlling dimensions of a groin:

I. THE HORIZONTAL SHORE SECTION

(1) Elevation. (a) Case 1 - High groins desired to prevent movement of material over groin landward of low water line. Determine as equal to the level of maximum wave uprush during all except the least frequent storms. High groins are used in cases where conditions simulating small bay head beaches are desired. For example, for a narrow eroding beach fronting a high eroding bluff containing a considerable percentage of beach size material, retardation of the rate of erosion is desired. High groins could be used to hold material eroded from the high bluff on the narrow beach between groins.

(b) Case 2 - Low groins desired to permit some movement of material over groins landward of low water line. Determine by adding to the maximum high water elevation which occurs frequently, the maximum normal wave height (excluding infrequent storm wave heights).

(2) Length. The shore end of each groin should be securely fastened to a bulkhead or revetment or should be well keyed into high land to the rear in order to prevent the groin from being flanked as a result of storm action. Determine the length of the horizontal shore section measured from the bulkhead, revetment, or existing surface, as appropriate, as equal to the width (measured normal to the shoreline) of beach berm desired.

II. THE INTERMEDIATE SLOPED SECTION

The top of the sloped portion of the groin should be parallel to the slope of the beach normally attained by the sand which will be affected by the groin. The lower end of the slope should be at the elevation of mean low water or as nearly at this elevation as permitted by construction considerations.

III. THE OUTER HORIZONTAL SECTION

Whether this section is needed or not depends on the rate of littoral drift. If the drift is large the outer toe of the beach slope will be adequately maintained and the outer horizontal groin section will not be needed. If the drift is small the toe of the beach slope will have a tendency to recede with resultant loss of material from the beach unless the outer horizontal groin section is added. In this case the outer horizontal groin section should be of such length as to contain the extension of the proposed beach slope to its intersection with the existing bottom.

If the littoral drift is lean, groins alone may not impound sufficient sand to provide the desired width of berm referred to in the preceding paragraphs. In that case, artificial placement of beach material will be required to make up the deficit. The method of design of the groins will be the same as described herein, the artificial fill being utilized to create the desired beach profile.

GROIN SPACING

Groin spacing is determined by estimating the beach alignment that will result from the groins. When equilibrium is established in an impounding area, the alignment of the beach is perpendicular to the resultant of the forces acting upon it. This condition corresponds to a minimum rate of change, and consequently, the rate of loss will be a minimum.

When equilibrium is established in a groin system, the alignment of the beach between any two groins is perpendicular to the resultant of the forces acting upon it. For practical purposes this alignment can be considered to be nearly parallel to the alignment in nearby impounding areas.

If nearby impounding structures exist, the groin spacing may be determined in the following manner:

a. Determine the geographic alignment of the shoreline immediately adjacent to and updrift from the nearby impounding structure.

b. Project a line parallel to this alignment from the outer end of the expected beach at the down drift groin. The intersection of this line with the shoreline locates the base of the next groin.

If there is no nearby impounding structure, the alignment of equilibrium must be estimated by analysis of wave action. This is done by preparation of wave refraction diagrams (Johnson, O'Brien, and Isaacs, 1948).

TYPES OF GROINS

The most important types of groins may be classified, according to the principal construction materials customarily used, as steel sheet piling, timber, stone, or concrete. Current practice is illustrated by type designs prepared by the Beach Erosion Board and shown in Figs. 1, 2 and 3. The designs illustrated are all of impermeable groins.

Permeable groins have been built in a number of places, particularly in the Great Lakes area. Satisfactory results are claimed to have resulted from some permeable groin installations. The record shows that some owners of shore property who have had permeable groins built are satisfied that their shores are now in better condition than they were before the groins were built. Other owners are dissatisfied with the results obtained with permeable groins.

Sufficient data are not now available to establish conclusively the cause and effect relationships in these cases. Some of the difficulties involved in establishing such cause and effect relationships are well illustrated by a case observed in the Great Lakes area. A system of permeable groins was constructed along a portion of the shore of one of the Great Lakes. For a number of years thereafter, accretion occurred along this portion of the shore. The owner was naturally pleased and willing to give the groins full credit for the benefit. However, similar accretion occurred simultaneously along another portion of the shore about a mile distant where no remedial measures had been taken. It is not known, therefore, whether or not the permeable groins caused the accretion in the first instance.

In some cases where permeable groins failed to accomplish the desired objective, it is doubtful if impermeable groins would have yielded any better results. In these cases, it was hoped to trap sand when there was no sand to trap.

The real problem which is not yet solved is whether, in cases where it is reasonable to expect groins to accomplish a desired objective, permeable groins can accomplish this objective more economically and effectively than impermeable groins.

Pending further investigation of the permeable-impermeable groin problem, it is fair to say that the best current practice favors impermeable groins. For example, the Beach Erosion Board has not recommended permeable groins as a remedial measure for any problem area thus far studied.

DETAILS OF GROIN DESIGN

The details of groin design depend on the severity of exposure to wave action, the climate of the construction site, and on the construction material selected. Experience with similar structures in the locality provides the best current criteria for detailed design.

STEEL SHEET PILING GROINS

The design and construction of steel sheet piling groins should avoid the principal difficulties which have been experienced with such groins, namely:

a. Failure resulting from insufficient penetration of the piling;

b. Failure resulting from corrosion and abrasion of piling.

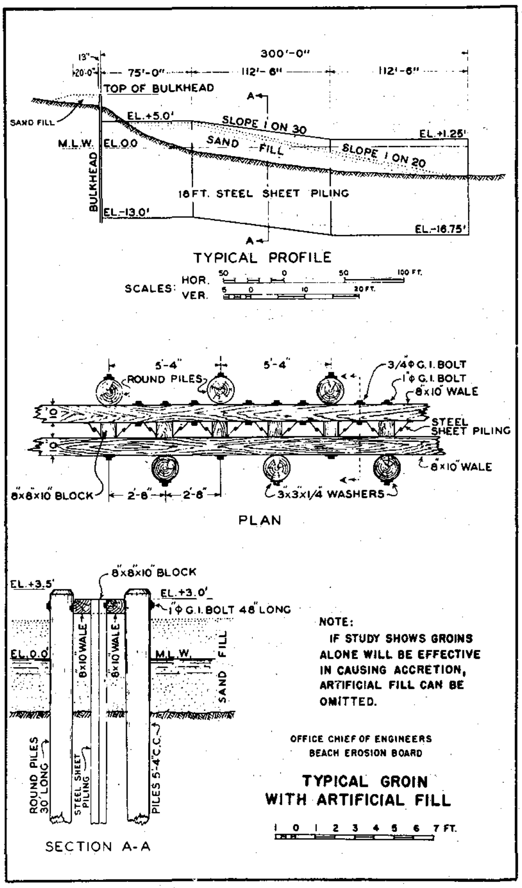

Fig. 1

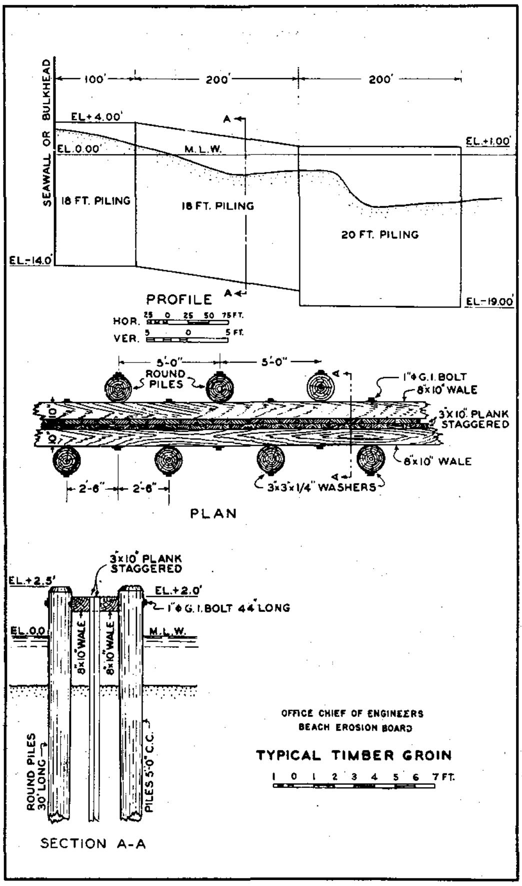

Fig. 2

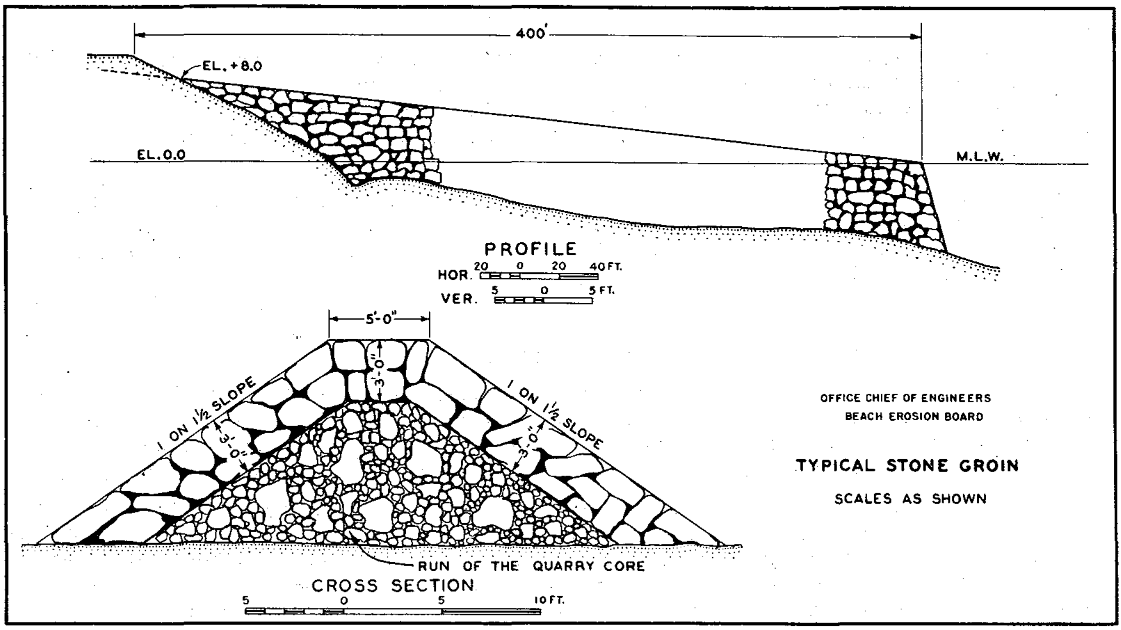

Fig. 3

The first type of failure can be avoided by providing two-thirds penetration under the most severe erosion expected to occur.

The second type of failure can be avoided in certain areas only by costly preventive or maintenance measures. It is probably wiser to avoid the use of steel sheet piling groins in those sections of the United States where rapid deterioration of the steel piling from corrosion and sand abrasion is experienced.

Under severe conditions the useful life of steel sheet piling groins is limited to only a few years. A Beach Erosion Board publication (1948) describes the effects of severe corrosion and abrasion. The following statement is quoted from this report:

> "The groins built from the arch section 0.375 inch steel piling were perforated in less than 4 years and in 7 years as much as 60 feet of a groin became useless for trapping sand. Straight section groins have somewhat longer life than do deep arch section groins of the same thickness, but holes were worn in straight section groins in less than 5 years, and in 7 years a considerable length of groin became useless as a sand barrier. Groins built from thicker sections of steel sheet piling may have proportionately longer life sufficient to more than amortize the added cost of the heavier steel when compared with groins built with thinner section piles. Holes did not appear in the groin at Palm Beach built with 0.547 inch piling until after more than 9 years."

In many localities, steel sheet piling groins have proved to be very satisfactory. In such areas, a useful life of 20 to 25 years or even more may be anticipated, if the groins are properly designed and built.

Experience with existing steel groins in the problem locality provides the best indication as to whether a reasonable length of useful life can be anticipated. In assessing experience with existing steel structures it should be noted that deterioration of steel groins results from corrosion and sand abrasion. The Palm Beach tests showed that the beach material moving under the influence of wave action was the primary factor in the rapid deterioration observed. The principal wear on the piling under similar conditions will occur in a zone a few inches above and below the sand line. Abrasion by the sand continually removes the rust covering and re-exposes the base steel to oxidation, thereby accelerating corrosion.

TIMBER GROINS

The design and construction of timber groins should avoid the major difficulties which have been experienced with groins of this type. As in the case of steel groins, provision should be made for sufficient penetration of the timber sheet piling. Another major difficulty to be avoided is deterioration of the timber resulting from marine borer attack.

Experience in the locality with existing timber structures provides the best indication of the severity of marine borer attack. At most localities in the United States where timber groins are likely to be built, the timber should be treated against marine borer attack.

STONE GROINS

Care must be taken in the design and construction of stone groins to secure the greatest practicable degree of impermeability in the core, and to use cover stone of adequate size, placed with the proper slope to withstand the wave action (and in some localities, the ice action) to which the groins will be subjected.

Experience in the locality still provides the best guidance for determination of the size of stone and the proper slope. In the absence of such experience, the formula for the calculation of rock fill dikes developed by Iribarren (1949) of Spain will be found useful. An English translation of the paper in which this formula was presented has been published by the Beach Erosion Board (Iribarren, 1949).

CONCRETE GROINS

Concrete groins have been successfully used in some localities, notably Palm Beach, Florida, and Waikiki Beach, Territory of Hawaii, but no satisfactory formulae for their design have yet been developed.

Existing concrete groins at Waikiki Beach are exposed to such light wave action that information concerning their design would not be of general value.

A number of concrete groins 140 to 200 ft. long at Palm Beach, Florida, were reported to be in excellent condition more than 10 years after construction. These groins were built of vertical concrete posts set on 12-ft. centers with horizontal precast concrete blocks between the posts. The posts are 2-1/2- to 3-ft. by 3-1/2- to 4-1/2-ft. rectangles in cross-section with widely beveled corners. The wall blocks are 1 ft. wide and 1 ft. or more high in cross-section. The ends of the wall blocks fit into recesses in the posts. The profile of these groins is made up of a series of horizontal steps with 1-ft. risers spaced 25 feet to 50 ft. apart horizontally.

SELECTION OF GROIN TYPE

The final choice between the several types of groins which may be considered satisfactory for a given problem area is based on cost estimates. Comparative costs of the acceptable types are estimated on an annual basis. Annual costs are determined from estimates of first cost, maintenance costs, and amortization over the probable useful life of the structure.

The types of groins which are found to be acceptable are for obvious reasons different in different parts of the United States and the Territories. The most economical type of acceptable groin likewise is not universally the same. Variations in costs will from time to time require changes in selection of the groin type which will be most economical.

Because of these variable factors each case must be studied individually in the light of conditions at the time of study. The problems studied by the Beach Erosion Board are not sufficient in number or scattered systematically enough geographically to justify preparation and maintenance of current data on the most economical and acceptable types of groins regionally.

In conclusion there is one point presented in this paper which, in my opinion, deserves the strongest possible emphasis. It is of the utmost importance to make every effort to make sure before recommending groins that they will actually accomplish the desired results in a particular problem area better and more economically than any other possible method. In some problem areas groins cannot achieve the desired results. In such cases groins will fail no matter how skillfully their detailed design and construction may be carried out. It is only common sense to ascertain first the desired objective and then to select most carefully the type of remedial works that will best accomplish this objective before proceeding to detailed design. In many cases, it will be found that one of the other types of remedial works should be selected in preference to groins.

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CHAPTER 28

DESIGN OF PILING

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A theory for the forces exerted on piling by waves has been developed and confirmed by model studies (Morison, O'Brien, Johnson, and Schaaf, 1950). The results obtained by the application of this theory are of the correct order of magnitude; however, additional laboratory and field experiments are desirable to define more accurately the values and limits of certain coefficients which enter the theoretical relationships. It is the purpose of this paper to present the possible calculations and the assumptions made with the limitation involved.

In the development of the theoretical relationship for the force on a piling the wave profile is the trochodial form and the particle velocity and particle acceleration are sinusoidal in form. The assumptions made in connection with this theory are that the water depth is large compared to the wave length and that the wave height is small compared to the wave length. However, the calculations using this theory are accurate wherever the sinusoidal particle velocity distribution shows good agreement with the actual distribution. This situation includes steep waves in deep water and waves of low amplitude in shallow water where the wave height is small compared to the water depth. The obvious exclusion is steep waves in very shallow water. The horizontal particle velocity and acceleration are given by the expressions:

particle displacement

$$x = \frac{-H}{2} \frac{\cosh \frac{2\pi(d+z)}{L}}{\sinh \frac{2\pi d}{L}} \sin \theta; \tag{1}$$

particle velocity

$$u = \frac{\pi H}{T} \frac{\cosh \frac{2\pi(d+z)}{L}}{\sinh \frac{2\pi d}{L}} \cos \theta \tag{2}$$

particle acceleration

$$\frac{\partial u}{\partial t} = + \frac{2\pi^2 H}{T^2} \frac{\cosh \frac{2\pi(d+z)}{L}}{\sinh \frac{2\pi d}{L}} \sin \theta \tag{3}$$

where

H = wave height
L = wave length
T = wave period
d = still-water level
z = depth below still-water level (negative)
t = time
θ = angular particle position in its orbit = $2\pi t/T$

The physical significance of these symbols is shown in Fig. 1.

The force (F) on a differential section (dz) of a piling is

$$F = \frac{1}{2} \rho C_D D u^2 dz + C_M \rho \frac{\pi D^2}{4} \frac{\partial u}{\partial t} dz \tag{4}$$

where

C_D = coefficient of drag
C_M = coefficient of mass

ρ = water density

D = pile diameter

The first part of the force equation represents the form drag caused by surface shear. The second part of the equation is the acceleration force on the displaced volume of fluid including the virtual mass effect. The term virtual mass may be explained as the increase in force caused by an apparent increase of the displaced mass of the fluid when an object is accelerated in a fluid as compared to in a vacuum. Thus, there is an apparent increase in displaced volume without an actual increase in the mass of the pile. The two effects of the force are shown illustrated in Fig. 2. The force (F) is a function of the depth below the still-water level and the position along the wave profile defined by θ. The position is given by the expression:

$$\frac{\bar{x}}{L} = \frac{\theta}{360} - \frac{H}{2L \tanh \frac{2\pi d}{L}} \sin\theta \qquad (5)$$

where $\bar{x}$ is defined in Fig. 1.

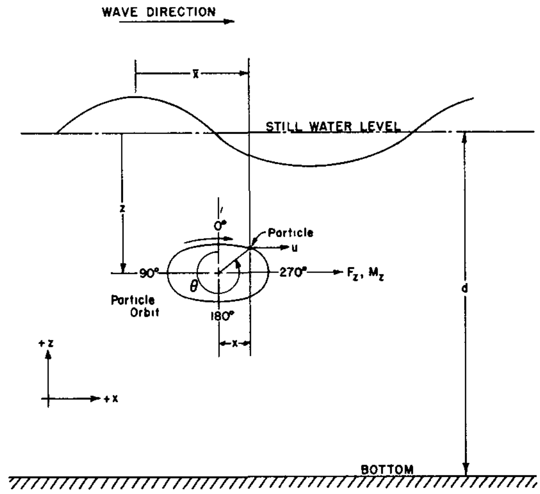


Fig. 1
Schematic diagram of particle motion

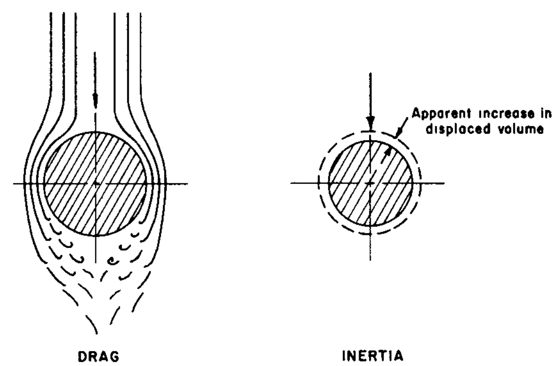


Fig. 2
Force components

The moment at any point on the pile is given by the expression

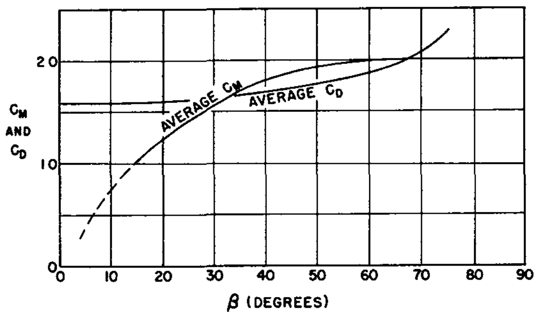
$$M_z = \rho \frac{H^2 L^2 D}{T^2} \left\{ \pm C_D K_2 \cos^2\theta + \frac{\pi D}{4H} C_M K_1 \sin\theta - (d+z)\left[\pm C_D K_3 \cos^2\theta + \frac{\pi D}{4H} C_M K_4 \sin\theta\right] \right\} \qquad (6)$$

where

$$K_1 = \frac{\frac{2\pi d}{L}\sinh\frac{2\pi d}{L} - \frac{2\pi(d+z)}{L}\sinh\frac{2\pi(d+z)}{L} - \cosh\frac{2\pi d}{L} + \cosh\frac{2\pi(d+z)}{L}}{2\sinh\frac{2\pi d}{L}} \qquad (7)$$

$$K_2 = \frac{\frac{1}{2}\left[\frac{4\pi d}{L}\right]^2 - \frac{1}{2}\left[\frac{4\pi(d+z)}{L}\right]^2 + \frac{4\pi d}{L}\sinh\frac{4\pi d}{L} - \frac{4\pi(d+z)}{L}\sinh\frac{4\pi(d+z)}{L} - \cosh\frac{4\pi d}{L} + \cosh\frac{4\pi(d+z)}{L}}{64\left(\sinh\frac{2\pi d}{L}\right)^2} \qquad (8)$$

$$K_3 = \frac{4\pi}{L}\left[\frac{\frac{4\pi d}{L} - \frac{4\pi(d+z)}{L} + \sinh\frac{4\pi d}{L} - \sinh\frac{4\pi(d+z)}{L}}{64\left(\sinh\frac{2\pi d}{L}\right)^2}\right] \qquad (9)$$



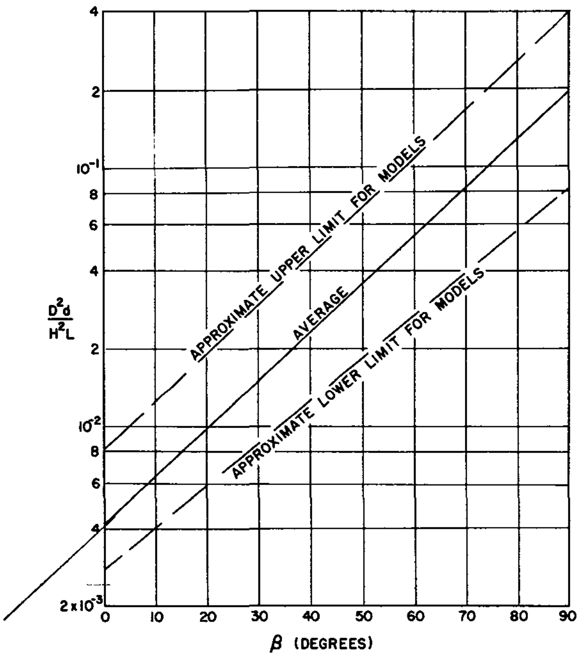


Fig. 3

Relationships of angular position of maximum total moment from model studies

$$K_4 = \frac{2\pi}{L}\left[\frac{\sinh\frac{2\pi d}{L} - \sinh\frac{2\pi(d+z)}{L}}{2\sinh\frac{2\pi d}{L}}\right] \qquad (10)$$

Equation 6 does not include the variation of the leverarm due to the surface elevation. Thus, the equation applies only in the case where the wave height is small compared to the water depth.

The combination of forces indicates that a maximum moment would occur at an angular position, β, other than the position of the crest. The expression for β is given by

$$\beta = \sin^{-1}\left\{\frac{\pi}{8}\frac{C_M}{C_D}\frac{D}{H}\frac{(K_1-(d+z)K_4)}{(K_2-(d+z)K_3)}\right\} \qquad (11)$$

The relationship of β to C_M and C_D is shown approximately by Fig. 3.

The angle, β, approaches zero for steep waves in shallow water when the pile diameter is relatively small compared to the wave height. It approaches 90^o when the wave length is long, the water is deep, and the pile diameter is of the same order of magnitude as the wave height.

As yet, no conclusive values of C_M and C_D for ocean condition have been obtained. These coefficients are empirical and can only be evaluated from measurements or by some relationship of oscillatory flow to steady flow, or by the extension of model study values.

The following sample calculations are presented to demonstrate the method and are based on results obtained by model studies.

SAMPLE CALCULATION I

TOTAL MOMENT ABOUT THE BOTTOM OF A PILE

Conditions:

H = 10 ft.	T = 10 sec.
d = 100 ft.	D = 1.5 ft.

Calculations (z = -100 ft.)

$L_o = 5.12\ T^2 = 512$ ft. (subscript o designates deepwater values)

Therefore

$d/L_o = 0.196$

From the Bulletin of the Beach Erosion Board, Special Issue No. 1, July 1, 1948,

$$d/L = 0.221 \quad \approx 0.2218$$

and

$$L = \frac{100}{0.2218} = 452 \text{ ft}$$

$$L = 452 \text{ ft.}$$

$$H/L = 0.02$$

$$D/H = 0.15$$

$$\frac{D^2 d}{H^2 L} = \frac{1.5^2 \times 100}{10^2 \times 452} = 4.97 \times 10^{-3}$$

corrections from page 370, IV th proc.

Hence from Fig. 3

$$\beta \approx 0 \text{ to } 15^\circ \text{ (assume average value of } 5^\circ)$$

Therefore

$$C_D \approx 1.6 \qquad C_M \cong 2.0 \text{ (use theoretical value of 2.0)}$$

~~$C_M \approx 1.0$ (a C_M less than 1.0 should not be used)~~

From equations 7 and 8 and the Bulletin of the Beach Erosion Board, Special Issue No. 1, July 1, 1948

$$K_1 = \frac{1.389\ (1.880) - 2.129 + 1.000}{2(1.880)} = 0.395$$

$$K_2 = \frac{1/2[2.777]^2 + 2.777(8.006)\ -8.068 + 1}{64\ (1.880)^2} = 0.0837$$

From equation 6 the terms,

$$(d + z)\ K_3 = 0$$

$$(d + z)\ K_4 = 0$$

From equation 11

2.0 ?

$$\beta = \text{Sin}^{-1} \frac{\pi(1.5)\ \cancel{1.0}\ (0.395)}{8\ (10)\ 1.6\ (0.0837)}\ \cancel{= 0.1737} = \underline{10^\circ} \quad = 20^\circ\ ?$$

$$= \arcsin 0.349$$

Equation 6 reduces to,

$$M_z = \rho \frac{H^2 L^2 D}{T^2} \left\{ \pm\ C_D\ K_2\ \text{Cos}^2\theta + \frac{\pi}{4} \frac{D}{H} C_M\ \text{Sin}\theta \right\}$$

$$= \frac{2.0(10)^2(452)^2(1.5)}{(10)^2} \left\{ +1.6(0.0837)(0.9848)^2 \right.$$

$$\left. + \frac{\pi}{4} \frac{1.5}{10}\ \overset{(0.395)(2.0) \approx}{\cancel{(1.0)}}(0.1737) \right\}$$

$$= 612{,}000 \left\{ 0.1294 + \overset{0.0162}{\cancel{0.0205}} \right\} = \underline{\underline{\cancel{91{,}600}\ \text{ft.lbs.}}} \quad 99{,}000 \text{ ft\#}$$

SAMPLE CALCULATION II

TOTAL MOMENT ABOUT THE BOTTOM OF A PILE

Conditions

$$H = 10 \text{ ft.} \qquad T = 10 \text{ sec.}$$

$$d = 100 \text{ ft.} \qquad D = 6 \text{ ft.}$$

Calculations (z = -100 ft.)

$$d/L = 0.221$$

$$H/L = 0.0221$$

$$D/H = 0.6$$

$$L = 452 \text{ ft. (same as for Sample Calculation I)}$$

$$\frac{D^2 d}{H^2 L} = \frac{6^2 \times 100}{10^2 \times 452} = 7.97 \times 10^{-2}$$

Hence from Fig. 3

$$\beta \approx 54^\circ \text{ to } 90^\circ \text{ (assume average value of } 70^\circ)$$

Therefore

$$C_D \approx 2.0$$

$$C_M \approx 2.0$$

Similarly, as in Sample Calculation I,

$$K_1 = 0.395; \qquad (d + z)\, K_3 = 0$$

$$K_2 = 0.0837; \qquad (d + z)\, K_4 = 0$$

$$\beta = \sin^{-1} \frac{\pi(6)\ 2.0\ (0.395)}{8\ (10)\ 2.0\ (0.0837)} = \sin^{-1}(1.11), \therefore \text{ use } 90^\circ$$

Equation 6 reduces to

$$M = \rho \frac{H^2 L^2 D}{T^2} \left\{ \frac{\pi}{4} \frac{D}{H} K_1 C_M \ (1) \right\}$$

$$= \frac{2.0\ (10^2)(452)^2(6)}{10^2} \left\{ \frac{\pi}{4} \frac{6}{10} (0.395)(2.0) \right\} = \cancel{2{,}310{,}000} \ \text{916,600 ft lbs} \ \text{ft.lbs.}$$

REFERENCE

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PART 5
CASE HISTORIES OF COASTAL PROJECTS

CHAPTER 29

HISTORY OF LOS ANGELES HARBOR

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The writer wonders if the person who assigned the subject of "History of Los Angeles Harbor" was aware that the conference was to be held in the Long Beach Municipal Auditorium. This latter city also has a harbor, the development of which is now so interwoven with that of Los Angeles that the Corps of Engineers in many official papers refers to "Los Angeles and Long Beach Harbors, California."

In the evolution of this large, modern, combined harbor with its present friendly internal rivalry, it has been designated by a number of names. Cabrillo in 1542 called the place "Bahia de los Humos." On the charts Vizcaino, 1602-1603, it appears as "Ensenada de San Andres." In 1734, the Spanish Admiral Gonzales gave it the name San Pedro, which still applies to the bay as a whole and to the community along the westerly side of the harbor.

In the state of nature, Point Fermin provided protection from westerly wind and wave. Catalina Island to the south intercepted some of the wave action from that direction. Thus, for all except southeasterly storms to which it was wide open, the roadstead offered reasonable protection. The bottom held anchors well. Inland lay a land-locked lagoon which was later to play a major role in the development of the port (Fig. 1). Concurrent with the founding of the mission of San Gabriel in 1771, and the pueblo of Los Angeles by a small colony of monks from that mission in 1781, came the regular use of the roadstead at San Pedro for loading and unloading of vessels.

Most of the nations of Europe had colonies throughout the world and were very jealous of them as outlets for home products and as sources of raw materials. Spain probably stood at the top in its all-out effort to keep foreigners out of her colonies and foreign vessels out of her colonial harbors. Probably the first American ship to come to San Pedro was the Lelia Byrd in 1805. Enroute back to Boston from the Hawaiian Islands, Captain Shaler, unable to get meat at Avalon on Catalina Island, came on to San Pedro and readily obtained hogs and sheep in exchange for Yankee-manufactured goods. Although a penalty of death and forfeiture of property faced the colonist convicted of trading with foreigners, this visit of the Lelia Byrd started a brisk contraband trade with Yankee and other foreign ships. First they came for otter skins and later for cow hides and tallow. They brought cloth and sugar and household goods of every kind. Very little money changed hands.

Rebellion against Spain was in progress during this period, and in 1822 came word of the relinquishment by Spain of her Western possessions. Under the succeeding Mexican rule all ports were declared open, but duties averaging about 25 percent were levied. San Pedro was made a port of entry in 1826, but with the collector for the port stationed in Los Angeles there was little, if any, payment of duty. In July of 1828, the port was again closed to foreign vessels and headquarters for these vessels were moved to Catalina Island. In the year 1835, the port was visited by Richard Henry Dana and later described in his famous book, "Two Years Before the Mast."

In 1827, a grant of several thousand acres of land, known as the Palos Verde Rancho, was made. This rancho followed the San Pedro Bay shore on the west, but reserved for public use about 42 acres of land with 1,400 ft. of water frontage where Fort MacArthur's lower reservation is now located. With the confirmation of this grant in 1846, the headquarters of foreign ships came back from Catalina and from this "Embarcadero" the business of the port was carried on for a number of years. With the return of the headquarters to the mainland came also the American occupation of the locality. In 1848 followed the Treaty of Guadalupe Hidalgo by which the United States acquired California. In 1850 California was admitted to the Union.

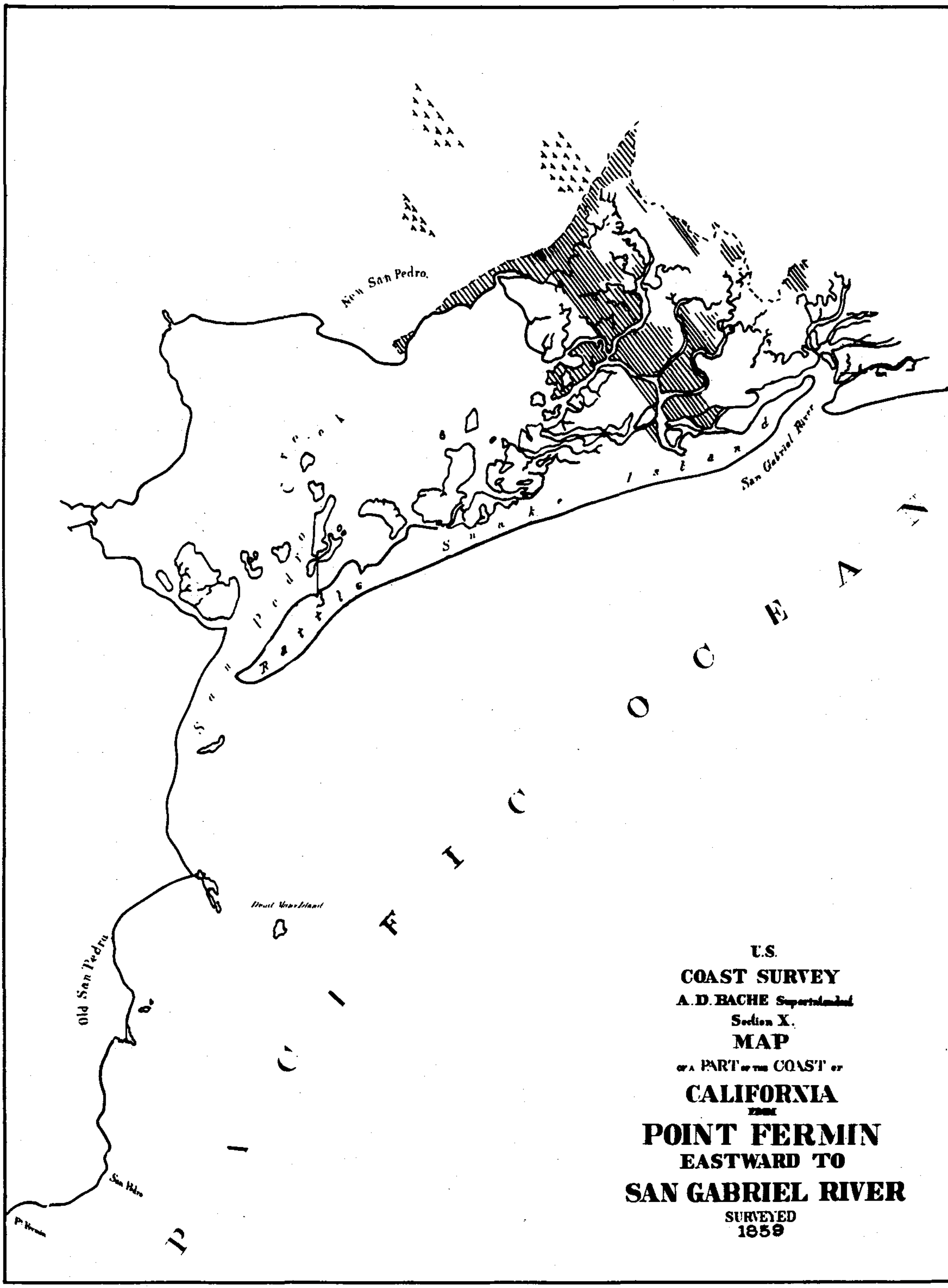
New San Pedro.
San Pedro Creek
Rattle Snake Island
San Gabriel River
PACIFIC OCEAN
Old San Pedro
U.S.
COAST SURVEY
A.D. BACHE Superintendent
Section X.
MAP
OF A PART OF THE COAST OF
CALIFORNIA
FROM
POINT FERMIN
EASTWARD TO
SAN GABRIEL RIVER
SURVEYED
1859

Fig. 1

Phineas Banning, who played a large part in the early history of the port, came to the county in 1851. His first position was clerk in a forwarding establishment at the "Embarcadero." He and his associates in 1853 bought 2,400 acres of land on the inner harbor from Manuel Dominguez, a grantee, for $1.10 per acre. On this land he laid out a town and named it for his birthplace, Wilmington, Delaware. To this inner harbor were shallow shifting entrances on both sides of Deadman's Island. One account says that about a mile north of Deadman's Island there were two or three entering channels having from two to six ft. of water at low tide, while to the west of this island were generally depths of only one to three feet. All these channels were very crooked and unstable, but Banning along with others found it profitable to take the lighters through these channels to and from a point about where the Catalina Terminal now stands. The cargoes had to be lightered anyhow and the added miles on a scow were directly toward the ultimate destination, Los Angeles, and into calm water. Wagon transport could in no way compete with water borne for as far as the latter could be effected.

One point of agreement in the Treaty of Hidalgo was that the United States would honor all grants. To do this literally would have been impossible, due to conflicting and overlapping grants and numerous claims, many of which were fraudulent. In order to settle the titles, a special Federal Commission was appointed in 1851 to determine the proper boundaries and rightful owners. The Federal District Court passed finally on the findings of the Commission in each case. A survey and description by metes and bounds followed, which was the basis for issuance of a patent to the owner by the United States.

When the Rancho San Pedro grant was surveyed in December, 1857, the outside boundary lines were described, after which that part of Wilmington Bay coming within these lines was specifically excepted in the following terms -- "Excepting, reserving and excluding from said tract, as thus surveyed, that portion thereof covered by the navigable waters of the inner bay of San Pedro, and which are included within the following described lines, to-wit ...."

In retrospect, it would seem that here was a natural for a publicly controlled port of large proportions. Here was 1,400 acres, capable of easy dredging and resultant fills, abutted by flat terrain for factories, railroads, etc., and with promise of a rich and populous surrounding territory. And just that has occurred, but not without struggles, first to get a harbor at all and then to get it under public control.

Perhaps an occurrence in May of 1860 was a precursor of one of the things to come. Hordes of sharks swarmed in the waters of the local bay and became the basis of a short-lived oil industry. After the sharks were speared and hauled ashore, use was made only of the livers, which were boiled down and thus yielded an oil which burned fairly well in lamps. This oil brought a good price in Los Angeles in those early days, when no one dreamed of the oil in the earth under these same waters.

Times were hard during the sixties. Following a long, dry spell, during which thousands of cattle died and many rancheros and businessmen failed, ranchos were offered for delinquent taxes. The 28,000-acre Los Alamitos ranch, including the area where the City of Long Beach now stands, was offered for sale in 1866 for delinquent taxes amounting to $152.65, with no takers. It later sold for sixty cents an acre.

In the late sixties, Banning who had become a state senator got the City and County of Los Angeles to finance a railroad costing $225,000 between the Pueblo and Wilmington. The road was 23 miles long and the only railroad in the southland. It began operation in 1868 and was an immediate stimulus to the commerce at the port.

Prior to 1871, all the improvements "up the creek" had been minor and privately financed. In this year, the people of Los Angeles became conscious of the harbor and held mass meetings to consider means for improving the harbor at Wilmington. A survey was made and data gathered to present to Congress. Banning made several trips to Washington. The idea advanced was to confine the tidal prism averaging some 250,000,000 cu. ft. to a limited single channel with a view to using the tidal flow as a dredging force.

On March 2, 1871, Congress adopted a project which contemplated straightening and deepening the channel between Deadman's Island and Timm's Point in San Pedro to 10 ft. This required closing the gap between Rattlesnake and Deadman's Islands. The estimated total cost of the project was $530,000, with the initial appropriation amounting to $200,000.

The closing jetty between the islands was built on a curve and in three sections. Its total length was about 6,700 ft. The initial 3,700 ft. beginning at Rattlesnake Island consisted of a single line of double sheet piling, well braced and rising about one ft. above high water. The next 1,000 ft. consisted of two parallel rows of 12-in. sheet piling 10 ft. apart, strongly braced and partially filled with brush, stone gravel, and sand. The 2,000 ft. next to Deadman's Island was rubble mound projecting several feet above high water.

It was assumed that sand would accumulate on the seaward side of the jetty in quantities and soon enough to make the wooden jetty permanent without repairs. This assumption was not well founded and there were a number of breaks especially along the lines of the old channels. Spurs and groins built out to intercept the sand helped but little. Original construction with concurrent repair ran from 1871 to 1881 and further repair ran on to 1903. By that time, some of the single sheet pile had been transformed into solid timber bulkhead consisting of timbers up to 18 in. wide securely drifted together. The entire length had been more or less reinforced with dumped stone.

While the jetty was not a total success structurally, it performed the function intended, and a good channel with a least depth of 10 ft. was scoured close to Deadman's Island. The use of this shallow channel was sufficient to warrant considering a deeper one and in 1881 a project to get 15 ft. of depth was adopted. This contemplated the extension of the east jetty beyond Deadman's Island to the 3-fathom curve and the construction of a correlating west jetty projecting 3,500 ft. from Timm's Point in the general direction of Deadman's Island and then southerly. Scour was to be given assistance this time by dredging the hardest portion. This work was completed in 1893 and was successful in giving a depth of 16 ft. across the bar. Further improvement, including dredging to 18 ft. across the bar and the straighteneing and widening of the channel inside, was authorized but the work delayed because of the question of where a breakwater should be built to create a harbor for commerce and refuge in the vicinity of Los Angeles.

In the same year that the little railroad between Los Angeles and Wilmington got into operation, a transcontinental railroad had been completed into San Francisco. Los Angeles and other intervening communities asked for the price of extending the railroad down state. The price given Los Angeles was a cash payment representing five percent of the total assessed valuation of the county, a right-of-way, 60 acres for depot purposes, and, as a bonus, the little railroad to Wilmington. The offer was accepted and in 1876 the harbor was connected to a major rail line.

The town of Santa Monica was founded in 1875 and the founders, including the idea of making it a seaport, had built the Los Angeles and Independence Railroad terminating in a wharf extending some 1,800 ft. into the ocean. The chief anticipated commodity was ore to be shipped to San Francisco for processing. This railroad, its wharf and holdings were absorbed by the Southern Pacific in 1876.

The population of Los Angeles in 1872 was 8,000, 20 percent being Americans. By 1886 the population had increased to 15,000 in the city and 50,000 in the metropolitan area. In less than 20 months these figures increased to 70,000 and 200,000 respectively. The commerce of the harbor had increased from 50,000 tons in 1871 to 450,000 tons in 1888. This was sizeable tonnage and yet the harbor had no completely protected anchorage. Occasionally a vessel was driven ashore. These losses in ships and in their time brought a demand for a breakwater to create a harbor of refuge.

In 1886 the first of four different projects for a breakwater in San Pedro Bay was submitted to Congress. This proposed project by the Engineer Officer in charge was to consist of two arms separated by a gap of 1,000 ft. The inner arm was to extend from the 3-fathom curve just off Point Fermin to about the 9-fathom curve, and the outer arm was to be a tangent approximately on that curve. The total estimated cost was $4,000,000.

The River and Harbor Appropriations Bill passed early in 1890, included $5,000 to be used to pay the expenses of preparing a project for a deep-water harbor "between Points Dume and Capistrano." The Board of Engineers appointed to make the report stated that the only possible locations were Santa Monica and San Pedro. This board recommended the latter location and proposed another breakwater of two arms. The inner arm of this breakwater was to extend from shore to the 6-fathom curve. A gap of 1,300 ft. permitted putting the outer arm in about the same location as that proposed in 1886, but made it 500 ft. longer. The total estimated cost was $4,500,000.

It was generally assumed locally that the question was settled and the Los Angeles Chamber of Commerce requested immediate construction. Recitation of certain events which transpired from 1881 on is needed to explain why the construction was repeatedly deferred. The Southern Pacific Railroad with their terminal in Wilmington became impatient in their wait for deep water to be brought into them and decided to go to deep water. Under state law with San Pedro unincorporated in 1881 they secured a right-of-way, reconfirmed in 1887 with some additions, which gave them almost the entire water front of San Pedro to the west jetty just inside of which was later constructed the Southern Pacific Slip. By purchase of land they also went on to Point Fermin and began the construction of a wharf approximately on the later site of the breakwater. This wharf was never completed or used. These combined moves gave this railroad a monopoly on the western side of the harbor. On the other side of the harbor, however, other interests in 1891 bought Rattlesnake Island for $250,000, changed its name to Terminal Island, and built a railroad via Long Beach to Los Angeles. Their terminal was on the westerly tip of the Island.

This opening for a competing railroad removed any possibility of a complete monopoly at the local harbor. At Santa Monica there was no competition. The railroad was extended a short distance to "Port Los Angeles" and construction of a second dock, known as the "long wharf," was begun. With the "long wharf" in operation deep inroads were made into San Pedro's commerce. However, breakwater protection was more essential at the new location than at Point Fermin.

Thus, it was that all attempts to get breakwater protection at Federal expense found the sponsors of Santa Monica striving to get the construction for their port. The controversy gained national prominence and much bitter argument went on both locally and in Washington.

As previously mentioned, an attempt to get an appropriation to carry on the authorized improvement of the inner harbor and its approach was denied until decision could be reached as to where the principal harbor with breakwater protection was to be located. To advise Congress in this regard another group of Army Engineers, known as the Craighill Board, was appointed in 1892. This board conducted a hearing in Los Angeles in July 1892. At this hearing, one of the points brought out was that the railroad had favored San Pedro and made it its headquarters for many years. The sudden change to Santa Monica was attributed to the ownership of all frontage at the latter location.

The Board's report was filed in October and presented to Congress in December of the same year. The report was unanimously in favor of San Pedro and proposed a continuous breakwater 8,500 ft. long on a curve to the line of the outer arm of the 1890 project and then a short distance along that line to the end. The estimated cost was $2,885,324.

Again the local reaction was that the question had been settled. The Chamber of Commerce sent a special delegation to try to get an immediate appropriation. The question didn't get out of the committee.

There followed several years of Congressional bickering, followed by preparation of bills calling for $392,000 to improve the inner harbor at San Pedro and $3,098,000 to build breakwater at Santa Monica. For a time it seemed that this would be the final decision, for the committee on commerce of the Senate voted to restore the Santa Monica item, which, in the interim, had been stricken from the bill. Senator White of California carried on the debate for five consecutive days and finally succeeded, with considerable help from newspapers and the general public, in getting an amendment providing for an appropriation of $2,900,000 and the appointment of a new commission of unprejudiced members. The decision of this board was to be final as to where the money should be spent.

President Cleveland in October, 1896, appointed the members of this board. He named Rear Admiral John G. Walker from the Navy, Augustus F. Rodgers from the Coast and Geodetic Survey, and William H. Burr, George S. Morrison, and Richard P. Morgan from civil life. This board made a thorough investigation, including soundings, and borings and prepared new maps. The conclusion of its report read: "Taking all considerations together, this board reports in favor of San Pedro as the location for a deep water harbor for commerce and of refuge in Southern California."

There was still some delay and threat of nullifying action, but in February, 1898, bids were opened. The low bid was for $1,303,198, less than one-half of the estimated cost. An effort in the Senate to get the indicated balance of about $1,600,000 expended pro rata upon both Santa Monica and San Pedro was promptly voted down.

On April 26, 1899, more than a year after award of contract, the first barge of rock from Catalina Island was dumped on the site. The event called for celebration, which was termed the Free Harbor Jubilee. This began with firing of cannon, speeches, and barbecue at San Pedro on the first day. On the second day, the scene shifted to Los Angeles where more than 100,000 persons watched a floral parade.

Progress of this contractor was so slow that his contract was cancelled and new bids invited. On May 14, 1890, another contract calling for $2,375,546.50 was let. Work was resumed in August of 1900 and proceeded without startling incident to completion of the contract. The first contractor brought by scow and dumped about 90,000 tons of rock, referred to as andesite, from Catalina. The second contractor used a double-track-standard-gauge railway trestle, building a considerable part of the substructure with sandstone from Chatsworth at the head of San Fernando Valley, and the balance with granite from several sites in the vicinity of Riverside. The contractor used two steam shovels converted into traveling cranes for unloading and disposing of the heaviest stones both superstructure and substructure. These cranes were stabilized as needed both by steel guys to the opposite track where the loaded cars were resting, and by rollers carried in heavy brackets on both sides of the front of crane which rested on a platform 4 ft. wide of 12 x 12 timbers, in turn supported by heavy timbers reaching from cap-to-cap of the trestle bents. Smaller stones were thrown and pried from the cars by hand. The substructure was built as they worked out to the end, and the superstructure as they worked back toward shore removing the interfering parts of the trestle.

Both the location and the design were included in the Walker Board report. The structure was to be detached, beginning in 4 fathoms of water about 1,900 ft. from shore. The first section was a 3,000-ft. tangent in a southeasterly direction followed by a curve 1,800 ft. long, with radius of 1,910 ft., and ending in another tangent 3,700 ft. long roughly along the 8-fathom contour. Even with a cancelled contract involved, funds were sufficient to permit extending the outer tangent 750 ft., making the total length 9,250 ft. instead of 8,500 ft. as planned.

As designed, the breakwater was to be all stone with a width of 38 ft. at mean lower low water, the dividing line between sub- and superstructure. Below this plane, the substructure was to be rubble mound with slope of 1 vertical on 1.3 horizontal throughout on the harbor side and below minus 12 ft. on the ocean side. The slope from low tide to minus 12 ft. on the seaward side was prescribed as 1 on 3. Stones were to range from 100 lbs. upward, with two-thirds averaging over 1,000 lbs. each.

The superstructure 14 ft. high and 20 ft. wide on top, was to consist of two walls and a roof of rectangular blocks of granite with interior solidly filled with random stone. On the ocean side the wall rises the 14 ft. in 4 courses, each 3-1/2 ft. thick, and retreating 3 ft. 4 in. in each lift for a total of 10 ft. No stone on the ocean side weighs less than 8 tons. On the harbor side the rise is made in 7 courses of 2 ft. each with retreat toward center of 1 ft. 4 in. in each lift for a total of 8 ft. No stone on the harbor side of the superstructure weighs less than 3 tons.

In the design the base of the superstructure was the same width as the top of the substructure. This lack of shoulder, particularly on the harbor side with its uniformly steep slope, caused concern early in the construction to end that a berm of about 4 ft. was used on practically all of the harbor side. A corresponding berm averaging perhaps 5 ft. was used on a large part of the ocean side. Along this side in many places additional unshaped stone was added on top of the berm with the intent of insuring the bottom layer of the shaped stone.

This detached breakwater was completed on September 9, 1910. During its construction, the advisability of the 1,900-ft. gap between it and shore was given reconsideration. It had been left as a secondary entrance and to encourage free circulation of water on theory that sewage and debris would leave faster and shoaling inside would be retarded. However, rocks and a kelp bed off Point Fermin made it uninviting as an entrance and storms drove the kelp inside and created a large area of rough water inside. By 1908 a project had been secured to connect the breakwater with shore. This section was undertaken shortly after completion of the detached section and completed in 1912.

The design of the Walker Board was not used in closing the gap. A random mound of the same top height and width was used because it was substantially cheaper. It withstood the seas equally well and less water passed over its top due to the extreme irregularity and the large voids receiving the seas on the installment plan and providing escape ways for much of their waters other than upwards. The special superstructure with the rectangular blocks under certain circumstances could prove the cheaper. It enables steep side slopes, which permit narrowing of the substructure. This could be the basis for a big saving particularly in deep water. However, unless stratified rock were available to make the blocks relatively cheap, the larger cross section required without them could still be filled and money saved. The second contractor on the detached section, largely due to the cost of preparing the rectangular pieces, needed the aid of creditors, profits from other jobs, and the savings of a lifetime to finish the job.

Congress charged the Walker Board with the duty of locating a harbor not only of refuge but of commerce. How farsighted its members were is well shown by the following paragraph quoted from the Board's report -- "At San Pedro a large expenditure has already been made for the improvement of the Channel leading into the inner harbor and in the inner harbor itself. The series of examinations made under the direction of this Board also show that any further improvement that may be needed can be readily made, and that the possibilities for the further development of the interior harbor are equal to any demand upon it which the future can be expected to make."

The Corps of Engineers created the Los Angeles District in 1899, and the first District Engineer, in compliance with the Act of March 3 of that year, had a new complete survey made of the whole harbor and recommended a project which called for the improvement of the whole inner harbor to a depth of 30 ft. This contemplated the dredging of 20,000,000 cubic yards at an estimated cost of $2,159,100. However, he proposed working first on that part extending up to and including the turning basin south of Mormon Island to a depth of 24 ft. and average width of 400 ft. at a cost of $550,000. When Washington assumed the attitude that this latter partial project, under way in 1903, was the project, people locally became alarmed and wondered if acquisition of title to seven-eighths of Wilmington Bay above and to the sides of the turning basin, by private interests under railroad control, was going to prevent full development of a free harbor, publicly controlled.

The title to this submerged, tide and marshland had been gained through perversion of law and later was declared invalid. But it posed a real threat at the time. In 1850 the Federal Government had granted by what is known as the "Arkansas Act," unappropriated swamp and overflow lands, including marshlands, to the States. The State patents in this Wilmington Bay case were obtained between March 5, 1880 and January 16, 1891 under the General Law of 1868, Chapter 415, as amended in

1870, for management and sale of "Swamp and overflowed, salt marsh and tide lands," the purpose of which general law was that swamp and overflowed lands should be reclaimed for agricultural purposes.

Local people felt the best way to bring the issue to a head was to press for the establishment of harbor lines. Thus, the rematch between the same opponents was on. The controversy did not gain the national prominence that the fight for the breakwater did, but locally and in Washington it was as tense and bitter. The good that came from the struggle was the time it provided and the interest it aroused in the plan of development for the inner harbor.

The Southern Pacific crossing to San Pedro in 1881 had severed the inner bay by trestle, which had been converted into fill. That part of the bay known as West Basin had been isolated. Immediately to the east of the railroad was the turning basin of the "partial project" and beyond that the East Basin. The Harbor Line Board appointed in response to the demand of 1903, delineated two large basins. The one east of the railroad varied in width from 1,500 to 2,400 feet and was about 3,200 feet long, providing room for ships to anchor as well as maneuver. The West Basin though smaller was outlined in the same general way. The distance between bulkhead and pierhead lines was 600 ft. to permit open-pile piers where there was lowland sufficient to warrant, and less at other places.

Following the submission of recommended lines in 1905, came the ruling from Washington that the War Department had no authority to establish harbor lines beyond the turning basin. The reasoning advanced was that beyond the project no harbor existed in which to establish lines.

In December, 1906, an application was made for permit to develop the inner bay east of the railroad by the digging of two channels each 1,000 ft. wide, with a reclaimed peninsula, varying in width from 1,100 to 2,000 ft. and 9,000 ft. long, lying between them. In the prolonged study of this application, the district office evolved plans of its own and in forwarding the application with unfavorable recommendation submitted a plan for the development for the entire inner harbor, both east and west of the railroad. This plan provided for the two basins, but reduced the water areas to that actually required for maneuvering and passing purposes and made pierhead and bulkhead lines only 40 ft. apart. A minor Y-shaped basin was proposed in front of the old landing in Wilmington. Areas to be reclaimed were so arranged that slips could be dredged into the land when more water frontage was needed. Various adaptations of the plan to show its possibilities fairly bristled with slips.

The previous conception of the outer harbor had been changed by advancing the bulkhead line into water sufficiently deep to allow reclamation of considerable areas so essential in the shore end of water commerce, and providing for two channels 400 and 600 ft. wide and 2,600 and 4,000 ft. long. The outer ends of the peninsulas thus formed remain unreclaimed but available today.

The guiding thought for the inner-harbor development and establishment of controlling lines was stated by D. E. Hughes at the time (February, 1907), in the following words:

> "Since neither tidal prism nor anchorage area would be of any use in this inner harbor, it would be an inexcusable waste to dredge more than is sufficient for the easy passage, berthing and turning of ships, and the rest of the reserved area should be reclaimed under Government control for the regulation of commerce. Such reclamation is also desirable for the reason that, through providing room near at hand for the disposal of dredgings, it would greatly reduce the cost of excavating the channels and slips."
>
> "The land reclaimed should never be alienated, but held as a public water front under control of Federal or State authority for the regulation of commerce."

What started as a paper is fast running into a book, so it will be necessary to omit recording of many moves and state that a joint resolution of Congress in March, 1908, caused the appointment of another Harbor Line Board in April. This Board submitted its report in July and the Secretary of War approved the lines the same month. Though they have been modified a number of times, particularly to permit widening of the Main Channel to 1,000 ft., the lines cast the die for the development of the harbor of commerce. The official approval also reaffirmed the authority to establish lines in navigable waters in advance of improvement in order to regulate it, and regardless of ownership, real or claimed, of the underlying land.

Suit was instituted by the State in 1908 to recover the patented lands in Wilmington Bay on the ground that the patents were invalid. A decision declaring many of the patents void was rendered in the Superior Court of Los Angeles County in January, 1911. This decision was confirmed by the Supreme Court of the State in 1912. Patents on two parcels were declared valid, but the ownership "subject to the public easements for navigation and fishery, and the State of California is declared to be the owner of all interest and title therein necessary to the support of said easements." In other words, these patentees had naked title.

For many good reasons the City of Los Angeles desired to bring the harbor within the city. From the time of authorization of the construction of the Panama Canal, the advantages in the matter of "terminal rate," new steamship lines, etc., could be foreseen, not to mention the prestige of having the city a seaport. Wilmington and San Pedro had seen and enjoyed enough of the political power of the city in the various controversies to appreciate that theirs were weak voices without it. They also were not unmindful of the necessity for large expenditures of local money for terminal and other purposes and realized the limits of their bonding capacities.

The first step of the city in its move toward the harbor was the annexation of a strip of unincorporated territory along Vermont Street from Manchester Street to the town of Wilmington. Sentiment in both Wilmington and San Pedro seemed to favor the consolidation, but state law provided that only cities of first class could consolidate. Los Angeles was first class, but San Pedro was fifth class and Wilmington sixth class at the time. Hence, legislative action was necessary. This action was blocked for a number of sessions, particularly by a state Senator who was from San Pedro, and who declared Los Angeles wanted to gobble his town. To circumvent this situation, proponents of consolidation circulated a petition for an election to exclude a part of San Pedro from the town, which would reduce its size sufficiently that the remaining portion could disincorporate under state law, and all would be eligible for annexation. After submission of the petition, the proponents got cold feet fearing that Long Beach might annex the excluded portion while the balance was getting disincorporated. A court order forced this election and those who had circulated the petition worked hardest to defeat the exclusion. The new consolidation bill passed the legislature a few days before the election and exclusion was voted down. Consolidation elections followed and the vote in both towns was favorable to the action -- Wilmington first and then San Pedro in August, 1909, by a 3 to 1 vote.

The improvement of the outer harbor began several years before consolidation. The Southern Pacific secured a franchise from the City of San Pedro and built its slip just inside the west jetty. Another franchise for the Miner fill and adjacent channels outside the west jetty was used and the improvement made. Huntington and associates had a concession for a mole also outside the west jetty between the Main Channel and the Miner fill, and intended using material dredged from the Southern Pacific slip to construct it. The excavation only provided about half enough material and the dredgings spread over the tideland unconfined. After consolidation, Los Angeles brought a consulting engineer from New York to advise upon the sequence and type of construction and upon his recommendation the Huntington concession was revoked and the city built the mole (known as Pier One) and a large concrete warehouse upon it. Here was recently established the foreign-trade zone authorized by the Secretary of Commerce.

On May 1, 1911, the state granted to Los Angeles all of the former's right and title to the tide and submerged land within the "consolidated city." These lands were, of course, to be held in trust for commerce, navigation and fishery. The leasing of land for other purposes for limited periods when not required for the primary purpose has been adjudged to be not inconsistent with the trust. The city's alert and vigorous prosecution of suits to recover all such lands erroneously alienated and the purchase of the Southern Pacific's right-of-way along the San Pedro water front has resulted in almost absolute control and ownership of the harbor and the water front by the municipality. In the successful suit to revoke the Miner fill franchise, the cost of the terminal facilities made in good faith became the consideration for a long-term lease to the builders. At Fort MacArthur and Reservation Point, the Federal Government, through state law ceding the foreshore of military reservations, is the owner of formerly submerged land now reclaimed.

The development of West Basin began in 1911, when the Federal Government not only ordered a drawbridge installed in the railway lines to make entrance into the basin possible, but also let a contract for a channel 200 ft. wide through the bridge and along the east side of the basin.

Development of Long Beach Harbor began in 1906, when the Los Angeles Dock and Terminal Company, a private corporation, bought 800 acres of marshland and tidal sloughs. This company secured a permit from the War Department to dredge certain channels. It also succeeded in forcing the railroad to replace a trestle across the old river mouth with a bascule bridge having 180 ft. of span. The plan of improvement consisted of an entrance channel protected by jetties on either side, an interior turning basin, and three channels extending northerly and easterly from the basin. All of this exists today as Long Beach Inner Harbor except the channel to the north, which was surrendered to induce industrial development.

In 1909, the City of Long Beach sold bonds and bought some frontage on its Inner Harbor and improved it with a terminal. In 1916, the city received a deed to the channels, turning basin, and about five acres near its pier from the company, which had experienced great difficulty in getting on with the dredging as the project was at the mouth of a river, which did have floods carrying large quantities of material in suspension which settled out in calm water.

Owning so little of the frontage in the Inner Harbor prompted Long Beach to turn in 1924 to the creation and development of an Outer Harbor with breakwater and moles. This Outer Harbor is still being improved with mole piers and fine transit sheds. The last bond issue for harbor purposes in Long Beach was in 1928. Oil was discovered in 1936, and the returns from oil development of city-owned property has carried the financial load very nicely since that time.

Los Angeles and Long Beach Harbors early had two common desires. One was to have river flood-waters permanently diverted. The other was to have direct water connection between the two inner harbors. River diversion was considered early in the development of Wilmington Bay and, had it been effected as then proposed, it would have consisted of a dike forcing all flood waters out where Long Beach Inner Harbor was later built. In this event there would have been no two harbors to connect.

A far-sighted District Engineer noting the narrow, sinuous natural channels legally subject to improvement for navigation, agreed with the railroad company owning the last land that in exchange for an easement with parallel sides to provide a direct connecting channel between the two harbors, he would recommend a permit to allow the company to reclaim the small and twisting tidal channels outside the easement. Initial connection was by combined effort of the Federal Government and the City of Long Beach. Later enlarging of this Cerritos Channel made possible by additional easement was done by the Government.

The floods of 1914 and 1916 hastened the construction of river diversion. The project consisting of an intercepting dike at high land at Dominguez and nearly five miles of channel to the sea between the City of Long Beach and its harbor was started in 1919 and completed in 1923. It was a Federal harbor project with many conditions of local cooperation including provision of right-of-way and bridges and assumption of operation and maintenance by a responsible agency. The created agency was the Los Angeles County Flood Control District.

In 1928, the Federal Government completed the removal of Deadman's Island and the reclamation of Reservation Point in a project which widened the Main Channel of Los Angeles Harbor to a minimum of 1,000 ft. up as far as the turning basin.

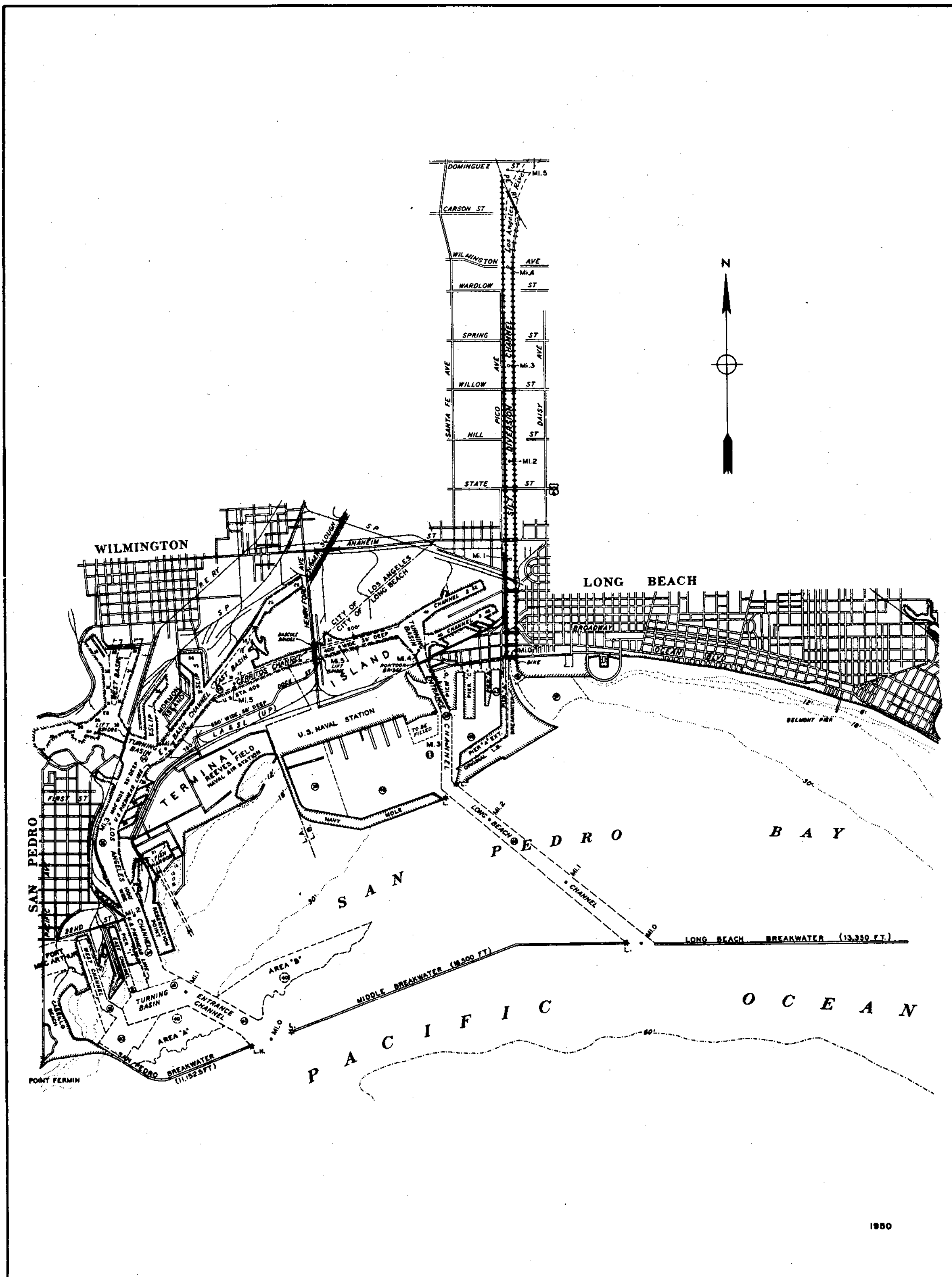
WILMINGTON
LONG BEACH
SAN PEDRO
N
U.S. NAVAL STATION
TERMINAL
ISLAND
REEVES FIELD
NAVY MOLE
SAN PEDRO BAY
PACIFIC OCEAN
LONG BEACH BREAKWATER (13,350 FT.)
MIDDLE BREAKWATER (18,500 FT)
SAN PEDRO BREAKWATER (11,152.3 FT)
POINT FERMIN
CERRITOS CHANNEL
BROADWAY
OCEAN BLVD
SANTA FE AVE
PICO AVE
DAISY AVE
DOMINGUEZ ST
CARSON ST
WILMINGTON AVE
WARDLOW ST
SPRING ST
WILLOW ST
HILL ST
STATE ST
ANAHEIM
TURNING BASIN
ENTRANCE CHANNEL
1950

Fig. 2

Breakwater extension to further protect the outer harbors and the face of Terminal Island was discussed for many years. The first project for this purpose was authorized in 1925, and provided for an all rock breakwater. Leaving an opening of 2,000 ft. at the end of the San Pedro Breakwater, the new structure was to continue in prolongation of the outer arm of the old breakwater for about 2-1/2 miles. After another opening, a second arm was to connect to shore at the west side of the flood-diversion channel mouth, making all discharge from that channel outside the harbor.

Many conditions of local cooperation were contained, including contribution of half the cost (estimated to total $14,000,000), unification of control of the two harbors, creation of a belt-line railroad, closure of a long length of Cerritos Channel -- Terminal would no longer be an island -- and others. Long Beach built its breakwater with inner arm in prolongation of the west bank of the flood control hoping to qualify its cost as part of the contribution. Other efforts were made to comply, but it became apparent that to meet them all would be impossible.

In 1930, a new project was authorized, which eliminated the arm to shore. Contemplating the use of dredgings from project areas in Los Angeles Outer Harbor in the base and hearting of the breakwater instead of making it all rock the total estimated cost was reduced to $7,000,000, the amount the Government was prepared to assume in the first project. This caused dropping of the local contribution condition. All the other conditions were also dropped. This composite breakwater, starting with an experimental earthen mound on the site in 1932, was completed in 1937. Extension of this breakwater chain with the same type of construction due east another 21,150 ft., including one 1,800-ft. opening, was completed in 1949 (Fig. 2).

As is evident, the writer, unable to include pertinent detail for the complete history of the harbor, has chosen to emphasize the early formative stages and has skipped lightly over the more recent expansion. To those interested, later history is readily available together with statistics of every sort. The roadstead in the lee of Point Fermin has developed into twin harbors whose combined annual tonnage has on occasion exceeded 32,000,000, and which together have an approximate 23 miles of improved wharves with more in process and projected.

CHAPTER 30

SANTA MONICA BAY SHORELINE DEVELOPMENT PLANS

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INTRODUCTION

The officially adopted Master Plan of Shoreline Development for Santa Monica Bay covers 13 miles of the shoreline between Topanga Canyon and El Segundo, with 9 miles in the City of Los Angeles, 3 miles in the City of Santa Monica, and one mile in unincorporated territory. It is planned to care for the beach recreation needs of 6,000,000 people, which is the estimated population for Los Angeles County in 1970.

$109,000,000 is the estimated cost of the proposed project, which includes beach development with an overall cost of $69,000,000, an amusement park at $10,000,000 and a yacht harbor, called Marina del Rey, with a capacity of 8,000 craft and costing $30,000,000. All the cost figures include acquisition.

Preliminary plans, not yet officially adopted, indicate a cost of $21,000,000 for improving the beaches along the southerly 8 miles of the Santa Monica Bay shoreline, with another $1,000,000 for Cabrillo Beach at San Pedro.

The beach development includes an ocean fill of 56,000,000 cubic yards, on which all the facilities will be constructed. These include scenic beach drives with divided roadways, promenades, areas for games of various kinds, bath houses, rest rooms, landscaping, restaurants, and last but not least, auto parking areas with a total capacity of 40,000 cars at one time. The amusement park and marina will also have parking fields with a capacity of 6,000 and 11,000 cars, respectively.

BACKGROUND OF MASTER PLAN

Such an ambitious program as this was not planned by someone deciding that we should have recreational beach facilities for 6,000,000 population and a yacht harbor for 8,000 boats and then having the details filled in. The Master Plan came about largely as a result of studies, begun in 1930, of beach erosion problems and of the many mistakes which had been made in shoreline developments in the past. Most of the erosion problems were the result of unwarranted or badly planned small boat harbor projects.

These mistakes of the past and some that are now being proposed bring to mind a quotation from Jonathan Swift:

"There are none so blind as they that will not see."

This quotation typifies the past and to a large extent the present attitude of many people, both in public and private life, toward shoreline development problems.

To understand the many varied problems which entered into the evolution of the adopted Master Plan it is necessary to review briefly the history of the coast of Santa Monica Bay.

NATURAL CONDITIONS

In 1542, when Cabrillo made the first voyage of discovery along the California coast, he gave the name of Bahia de los Fumos to Santa Monica Bay. The English translation is Bay of the Smokes, and the name was occasioned by the many smokes, visible along the shore, which came from the numerous habitations of the Indians.

It is presumed that the Indians, not being subject to pressure from real estate dealers, located their dwelling places beyond the reach of the highest tides and the effects of natural seasonal and cyclical erosion.

Beaches cut back in the winter time and are restored during the calmer periods of summer and early fall. The cycles of rainfall affect the beaches; narrower beaches resulting during dry cycles due to lack of flood-borne replenishments of sand and other erosion detritus.

The white man, building along the shoreline some 350 years later, ignored these basic facts. Subdivisions on the ocean shore extended to the mean high-tide line, legal boundary of the State-owned tidelands. On narrow lots, extending in many cases only 90 ft. landward from the mean high tide line, houses were constructed. The front yard might be under water at extreme high tide and during winter recessions of the beach. Houses were endangered and sometimes destroyed. Owners of these lots soon learned that the houses should be constructed on long piling.

Later came amusement piers, which had little effect on the shoreline. Still later came jetties and breakwaters which had a great effect on the shoreline, destroying many miles of beaches and filling the harbors the breakwaters were intended to create.

EFFECT OF BREAKWATERS AND JETTIES ON THE SHORELINE

There have been several such structures along the Southern California coast which have caused serious changes in the shoreline, at Coronado, Venice, Santa Barbara, Santa Monica, Seal Beach, Hueneme and Redondo Beach.

At Coronado, near San Diego, some 50 years ago, private interests constructed a curved jetty in an attempt to form a small boat anchorage. Serious erosion of the beach to the north took place, which necessitated construction of a heavy stone riprap seawall. Private interests at Venice, on Santa Monica Bay, in 1905 built a short breakwater to protect their pier. The structure caused the beaches to erode, with some damage to private homes.

By 1929 pressure for pleasure craft anchorages resulted in a long breakwater at Santa Barbara. It caused complete erosion of the beaches for 10 miles downcoast and destruction of a number of private homes (Fig. 1). The harbor by now would have been filled with sand but for periodic dredging.

In 1933 the City of Santa Monica built a breakwater which caused extensive erosion, mostly of public beaches (Fig. 2). By the middle of 1948 the harbor was half-filled with sand. In October 1948 a contract was let to dredge out 1,000,000 cubic yards. The erosion has been checked by developments which will be described later. However, the harbor will fill up with sand again and erosion downcoast will continue as long as the breakwater remains in place.

The City of Seal Beach, just south of San Pedro Bay, constructed a long jetty in 1936 which caused serious erosion downcoast.

In 1939 two more small coastal communities constructed small craft harbors on the Southern California coast -- at Hueneme, about 40 miles north of Santa Monica Bay, and at Redondo Beach, near the southerly end of Santa Monica Bay. The entrance jetties at Hueneme caused the downcoast shoreline to recede several hundred feet. At Redondo Beach the breakwater caused destruction of the public beach, and the promenade (Figs. 3 and 4). Over 30 buildings behind the promenade were destroyed or badly damaged. The cost of the property destroyed and of a stone sea wall constructed to prevent further damage is greater than the cost of the harbor and the harbor is practically useless as a boat anchorage.

LACK OF STATE CONTROL

It should have been apparent many years ago that small municipalities and other minor political subdivisions whose boundaries happened to include frontage on the ocean or a bay had neither the knowledge nor the means to cope with the ocean forces and should not have been entrusted with unrestricted control of tide and submerged lands within their boundaries.

Before 1931 there was no State agency empowered to regulate structures on tide and submerged lands. In that year the Legislature placed such power in the Division of State Lands. However, this did not apply to tide and submerged lands

Fig. 1

Fig. 2

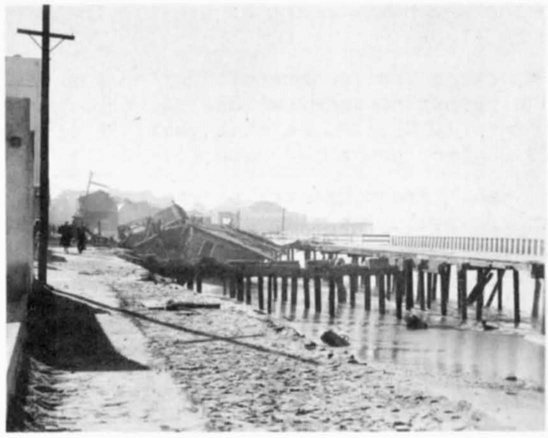

Fig. 3

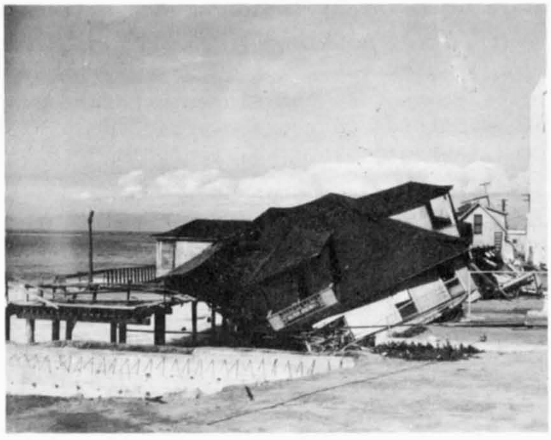

Fig. 4

Fig. 1
Private beach homes at Sandyland, Santa Barbara, California, destroyed in January 1940. Former wide beach at this location was lost through erosion caused by the breakwater at Santa Barbara Harbor, 10 miles west.

Fig. 2
Venice Beach; Los Angeles, California. Results of erosion caused by breakwater for small craft anchorage constructed at Santa Monica, 3 miles upcoast.

Fig. 3
Redondo Beach, California. Results of erosion caused by breakwater constructed to provide a small craft anchorage (January 1944).

Fig. 4
Redondo Beach, California. Results of erosion caused by breakwater constructed to provide a small craft anchorage (January 1944).

already granted to political subdivisions. It was another 12 years before the State recognized the need for erosion control study and created the office of State Beach Erosion Control Engineer.

Nevertheless, there seems little excuse for the promoters of these harbors, or at least their engineers, not knowing that the breakwaters and jetties would have the effect on the shoreline which did occur. These problems are as old as the Spanish Armada, at least, and research would have disclosed them.

A report entitled "A Discourse on Sea-Ports, principally of the Port and Haven of Dover" was made by Sir Walter Raleigh to Queen Elizabeth of England. It was published during the reign of Charles the Second.

SHORELINE PROBLEMS OF THE PAST

Breakwater building operations at the Royal Harbour of Ramsgate, England, from 1749 to 1768 resulted in the harbor being nearly filled with sand and silt. An engineer of that time predicted that, instead of an anchorage for ships, the harbor would become a field of corn unless recourse were had to some artificial means of clearing it. This brings to mind a remark made by the late Will Rogers who, when asked what he thought of Santa Monica Harbor, said, "I guess it's a mighty fine harbor but it looks as if it might need irrigating before long."

Two notable examples of the effects of harbor works on a sandy shoreline were at Madras, India, in 1877, and at Ceara, on the northeast coast of Brazil, in 1886. Extensive silting and erosion resulted in both places.

By 1908 these problems had become so important that a special section was devoted to them at the Eleventh Congress of the Permanent International Association of Navigation Congresses, held in Saint Petersburg, Russia, in that year. Experts from various countries, including the United States, presented papers.

Planners and others are still proposing small craft harbors at various places along the Southern California coast, apparently without consideration as to their effect on the shoreline. One such instance is at Redondo Beach. In spite of the disastrous results of the present breakwater, it is proposed to extend it to create a small craft harbor. If this is done I predict that serious erosion will occur downcoast from the proposed harbor in the southerly portion of Redondo Beach and possibly in Torrance, the adjoining municipality to the south.

SHORELINE HIGHWAYS

Another factor which increases the difficulty and cost of public beach development has been the practice of opening a coastal highway, which skirts the shoreline, and leaving a narrow strip of private land between the highway right of way and the mean high tide line. This usually results in unsightly buildings along the seaward side of the highway, destroying its scenic value and multiplying the costs of acquisition of ocean frontage which could have been acquired as a part of the right of way. Permissive but not mandatory legislation concerning such conditions was enacted by the Legislature a few years ago.

ENGINEERING STUDIES AND RESEARCH

In 1930 the City Council of Los Angeles instructed the City Engineer "to make a study and report on the best methods of temporary, as well as permanent protection and development of the City beaches." This initiated the research, studies and surveys from which grew the proposed $109,000,000 beach and marina project briefly described at the beginning of this paper. Since the ocean forces do not recognize political boundaries it was necessary to make surveys and studies of the entire shoreline of Santa Monica Bay, which required collaboration between the engineering and planning forces of both the City of Los Angeles and the County of Los Angeles.

Extensive research was made of data on shoreline problems in the United States and other countries. Studies were made of the original sources of beach sand and its replenishment and of wave, wind and current action, and the effect on the beaches of various structures along the tidelands, both those now existing and those which had existed in the past. Field surveys of the beaches and the adjoining ocean bottom to a half-mile offshore were made along most of Santa Monica Bay and at Cabrillo Beach at San Pedro. These have been repeated at intervals.

The importance of such surveys and studies cannot be emphasized too strongly. Without them, planning shoreline improvements and protective works is largely guesswork and serious mistakes may be made.

Because of the comparatively narrow beaches and the need for conserving and utilizing them to the fullest extent, the matter of erosion control was important. The City of Santa Monica was considering construction of a breakwater to create a small craft anchorage. Los Angeles had no jurisdiction in the matter and there were no legal means of preventing construction of the breakwater, which was certain to cause serious erosion of the Venice beaches. Therefore, the matter of erosion became a problem of paramount importance.

The erosion problem, the need for wider beaches, the need for a shoreline highway for direct access to the beaches and particularly the need for automobile parking space, combined with the fact that millions of yards of sand existed on the dunes located on city-owned property at Hyperion, led to investigation of the feasibility of solving these problems by utilizing the sand on the dunes for widening the beaches.

ENGINEERING REPORTS

In reports prepared in 1934 and 1935 and submitted by the City Engineer to the City Council it was recommended that the beaches be widened between Santa Monica and Hyperion with sand pumped from the dunes so that the area of public sand beach would be doubled, and sufficient space provided for a shore front drive, parking areas and other facilities. Due to the difficulties of financing such a project during the depression days no further action was taken.

By 1940 erosion caused by the Santa Monica breakwater had become quite serious. The City Engineer in another report to the City Council recommended that 500,000 cubic yards of sand from the Hyperion dunes be placed along the most seriously eroded frontage as an emergency measure and that an additional 12,000,000 cubic yards be later placed along the six miles of beach frontage between Santa Monica and El Segundo to provide beach facilities for the rapidly growing population. No action was taken.

The proposal to widen the beaches by pumping sand into the ocean evidently was considered extremely impracticable and visionary, judging by the many criticisms made both by public officials and private citizens. However, in 1943 when portions of Venice seemed about to meet the fate of Redondo Beach, an appropriation of $88,000 was secured to truck 150,000 cubic yards of sand from Hyperion and deposit it along the beach in the most threatened areas. In spite of dire predictions that all this sand would be washed away by the ocean waves in a single storm, the major portion of it still remained in place four years later. This small fill proved the soundness of conclusions based on studies and surveys made since 1930.

SEWAGE POLLUTION

Pollution of the waters of Santa Monica Bay caused by discharge of raw sewage into the bay at Hyperion has been a problem for many years. In 1943 the pollution became so serious that the State Health Commission quarantined ten miles of beaches in the central portion of the bay. In 1945 the Commission secured a court order requiring Los Angeles, and other municipalities using its outfall sewer, to abate the nuisance by constructing a sewage treatment plant.

Clearing the site of the proposed plant required excavation of 14,000,000 cubic yards of dune sand. A contract for the excavation was let in 1946 which required disposal of the sand to be made along the six miles of beaches between Santa Monica and El Segundo, as recommended by the City Engineer in 1940. The southerly three miles of the fill were placed to the full width proposed in the Master Plan and the remainder to about two-thirds the planned width. The fill was completed in December 1948 and resulted in a beach six miles long with an average width of 600 ft. The unit cost of excavation and disposal was 22.6 cents per cubic yard.

MAJOR FEATURES AND COST OF THE MASTER PLAN

The 56,000,000 cubic yard ocean fill required for beach development is the most spectacular feature of the Master Plan. This includes the 14,000,000 cubic yards already placed. The average width of the total fill extending along 13 miles of the shoreline will be 900 ft. and the maximum about 1200 ft. Only the outer one-third of the fill needs to be sand. The remainder can be material taken from high bluffs adjoining the beach near the northwesterly end of the project.

One-fourth of the entire fill has been in place for one and one-half years and many of the skeptics are convinced that the proposed project, from a construction standpoint, is not impracticable and visionary. Financing the project has produced just as much skepticism. The costs are great. A good idea of their magnitude can be secured from the following breakdown:

Acquisition costs	$ 21,000,000
Ocean fill	15,000,000
Groins	3,400,000
Sewers, storm drains and utilities	19,200,000
Highways and streets	17,500,000
Dredging and harbor improvements	13,500,000
Buildings and recreation facilities	8,400,000
Landscaping	5,200,000
Parking areas	3,700,000
Miscellaneous	2,000,000

Improvement of the beaches between El Segundo and Palos Verde is estimated to cost $21,000,000 and of Cabrillo Beach at San Pedro, $1,000,000.

COORDINATING AND FINANCING THE MASTER PLAN

This total figure of $131,000,000 may seem extravagant for recreation facilities until one gives consideration to the great population of six million people they will serve, and to the revenues that can be derived from operation of the various facilities, which would amortize the cost within a reasonable time.

In 1948 the City Council of Los Angeles retained the firm of Madigan-Hyland, nationally known consulting engineers of New York City, to make an engineering and economic study and report on the shoreline development plan. They approved the project and recommended that it be carried out through a regional authority and district, encompassing the area tributary to the development, under an enabling Act enacted by the State Legislature in 1947.

All operation, maintenance, bond redemption and interest charges would be taken care of by revenues plus a tax of only 9 cents per $100 of assessed value on real and personal property within the proposed district, with the entire capital cost amortized in 35 years. The tax rate of 9 cents would amount to slightly more than <u>two dollars annually</u> to the average home owner.

Madigan-Hyland analyzed other methods of financing the project, including revenue bonds. They found that the latter method was not in any way feasible for the entire project and would be feasible for the marina alone only if at least $16,000,000 could be secured from outside sources. They recommended against any attempt to finance the marina by revenue bonds.

The district tributary to the project comprises a portion of Los Angeles County and contains 30 municipalities and considerable unincorporated territory. The shoreline within the limits of the proposed project includes frontage, within 7 municipalities and a mile of unincorporated frontage. The Madigan-Hyland survey shows that 86.6% of beach visitors come from Los Angeles County, 6.1% from other counties of California, and 7.3% from outside California.

Under these conditions close coordination will be required if the project is to go ahead. It appears that the interests of the people will be best served through the medium of a regional shoreline park and recreation district.

CHAPTER 31

HISTORY OF OCEAN OUTLETS, LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

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INTRODUCTION

Los Angeles County has a number of watercourses which discharge into the Pacific Ocean. Three of these are of major importance in that they traverse the Coastal Plain area of the County. These three are the Los Angeles River, the San Gabriel River, and Ballona Creek. They have a combined drainage area of approximately 1,645 square miles, most of which is within Los Angeles County. Such area not only comprises over 40 percent of the land area of the County but, more important, includes within its boundaries, the great majority of the County's population.

The Coastal Plain area of Los Angeles County, prior to installation of flood control works, was probably subject to a greater potential flood hazard than any area of similar size and density of population in the United States. It has been subjected periodically to floods that, descending from the San Gabriel and Santa Monica Mountains, have rushed across the valley floor towards the Pacific Ocean altering topographic features and causing loss of life and property. As far back as 1815, floods of damaging character have been recorded. Within the last 90 years the San Gabriel and Los Angeles Rivers have changed their courses, creating new channels in materially different directions.

Prior to 1889, the floods, while causing extensive damage and inconvenience, did not create any general demand that remedial measures be adopted because channel encroachment had not yet developed to any marked degree and property values did not warrant the cost of coordinated protection from floods. However, between 1889 and 1914 a great industrial and agricultural expansion took place concurrent with a large increase in population. Property values boomed. Lands which lay adjacent to river and stream channels developed a market value, were sold and improvements constructed thereon, the purchasers in many instances not realizing that the property purchased was liable to flood damage.

Thus, the flood of 1914, while not the greatest of record, caused a property loss of over $10,000,000, made hundreds of people homeless, isolated communities and resulted in personal injury and loss of life. It was this flood which so forcibly brought to the attention of all County residents the necessity for a broad, coordinated program of flood control in Los Angeles County, a program which would, in part, insure defined and controlled waterways across the Coastal Plain to the Pacific Ocean.

Such a program has, in the intervening years, been conceived and is now in process of being carried out by the local and Federal agencies charged with this responsibility. The Los Angeles County Flood Control District, created by Act of the State Legislature in 1915, is the responsible local agency; the Federal agency is the Department of the Army which, through the Corps of Engineers, is responsible for carrying out the provisions of the Federal Flood Control Act of 1936 and later similar Acts whereby control of floods was recognized as a Federal as well as local responsibility.

It is the purpose of this discussion to briefly cover the history of each of the three major ocean outlets and the improvements which have been made thereto by the Flood Control District and the Corps of Engineers during the past 30 years of flood control activities in Los Angeles County. They will be discussed in order of geographical location from north to south.

BALLONA CREEK OUTLET

Ballona Creek discharges its waters into Santa Monica Bay at Playa del Rey about 26 statute miles by water northwesterly of the entrance to Los Angeles Harbor.

The outlet intersects a shallow lagoon approximately a mile long and 200 ft. wide lying parallel to and approximately 800 ft. inshore. Ballona Creek has its origin in the Santa Monica Mountains seven miles northeasterly from its mouth and has a tributary drainage area of approximately 129 square miles. The watershed includes the western part of the City of Los Angeles, the Baldwin Hills and the south slope of the Santa Monica Mountains to and including Sepulveda Canyon.

Early maps indicate that Ballona Creek outlet was located under the Playa del Rey bluffs at the southern edge of the tide lands into which Ballona Creek flood waters were discharged. It is quite probable that the natural outlet migrated during past times for some distance upcoast and back but no maps are available to verify this. During the period 1906 to 1908 the outlet was fixed at its location at that time, approximately one-quarter mile upcoast from the bluffs, by the construction of bulkheads, jetties and tide gates. This outlet, about 200 ft. in width and extending about 425 ft. seaward, served the dual purpose of providing not only an outlet for the Ballona Creek discharge of that period, but also tide water for the extensive system of canals, which had been constructed to the northwest in what was then the City of Venice.

With the increase in development and population in the Ballona Creek watershed, came an increasing need to improve the waterway of Ballona Creek, particularly the outlet to the ocean. This was primarily a problem of straightening and widening a not too well defined channel and of eliminating a right angle turn from the channel into the lagoon and from the lagoon into the outlet before flood waters could reach the ocean.

Following the formation of the Flood Control District, attempts were made to obtain participation of the communities interested in the channel and outlet improvement program. A partial improvement program, financed by Flood Control District funds, was carried on until 1936 when the Federal Flood Control Act was passed which provided sufficient funds for completion of the channel and construction of a new outlet. While a defined outlet for Ballona Creek was thus in existence from 1906 to 1936, in the latter part of this period, it became inadequate due to increase in discharge from the rapidly developing tributary urban area; to lack of a defined channel of sufficient capacity through the adjacent tide lands; and to the poor hydraulic conditions at the lagoon and outlet.

Under the Federal improvement program of Ballona Creek, the Corps of Engineers excavated a new outlet to the ocean early in 1937 at a location approximately 1400 ft. upcoast from the old outlet. This channel has a base width of 200 ft. and is trapezoidal in section with rock-faced levees. Parallel rock jetties, which extended seaward approximately 650 ft., were constructed by the Corps of Engineers during 1938 and later were grouted. The base width between jetties is 260 ft.

Following construction of the jetties, severe erosion occurred downcoast between the old and new outlets, since normal littoral drift in that vicinity is in a downcoast direction. This adverse beach condition continued until 1947 when the City of Los Angeles, as part of its beach improvement program, extended the jetties an additional 590 ft. and artificially replenished the beach, widening it to approximately 800 ft. with sand pumped from the site of the new Hyperion Outfall Sewer Plant.

Prior to construction of the present outlet of Ballona Creek, model studies to investigate the effect of outlet installations of various types on adjacent beaches were carried on by the California Institute of Technology in 1934 under agreement with the Flood Control District, and by the United States Waterways Experiment Station at Vicksburg, Mississippi in 1937 under Federal sponsorship. Both studies indicated that the beach between the new and old outlets would be subjected to extensive erosion when construction of jetties was completed with the Federal agency predicting that erosion would reach a maximum of 100 ft. It is of interest to note that beach erosion to approximately this extent occurred prior to the start of the beach widening program by the City of Los Angeles and that the former residence of Actress Mae Murray, located in this reach of beach, was partially undermined and had to be moved to a new site to prevent complete loss.

The construction of jetties has proven effective in maintaining satisfactory outlet conditions at the present location of Ballona Creek outlet. The elimination, for the time being, of the normal supply of sand from upcoast together with the flushing action of the ample tidal prism resulting from channel and lagoon storage capacity will doubtless maintain this favorable condition for some time to come.

LOS ANGELES RIVER OUTLET

The Los Angeles River has its origin in the Santa Susanna and Santa Monica Mountains bordering the westerly portion of the San Fernando Valley. It flows easterly about 20 miles along the south side of the valley, cuts six miles southeasterly around the easterly terminus of the Santa Monica Mountains to Los Angeles Narrows in the vicinity of Elysian Park, and thence flows in a generally southerly direction approximately 22 miles across the Coastal Plain, entering the ocean at Long Beach. It has a watershed area of approximately 818 square miles.

Early Californians, including Pio Pico, last Spanish Governor of Alta California, have been recorded as stating that prior to 1825 the Los Angeles River discharged southwesterly through Ballona Creek into Santa Monica Bay. A severe flood that year is credited with having changed the direction of flow and the discharge of the Los Angeles River has since been in a southerly direction.

Prior to January 1868, the Los Angeles River joined the San Gabriel River about seven miles north of San Pedro Bay. A flood which occurred during that month split the waters of the San Gabriel River above what is now known as Whittier Narrows and diverted a considerable portion into a new channel which discharged into Alamitos Bay, some six miles downcoast from the old outlet into San Pedro Bay. Thereafter, the name "Los Angeles River" was gradually applied to the lower reach of the old San Gabriel River and the new San Gabriel River became known as "San Gabriel River." Thus, the Los Angeles River acquired an official outlet to the ocean at San Pedro Bay, although still receiving through the interconnecting stream, the Rio Hondo, an appreciable percentage of the discharge from the San Gabriel River watershed.

The approximate location of the outlet of the Los Angeles River during the 90 years following the flood of 1825 was in the East Basin of Los Angeles Harbor near the easterly end of Terminal Island. During this time Los Angeles Harbor assumed a constantly increasing importance in the welfare of the County. Hence, when the flood of 1914 discharged several million cubic yards of silt in the dredged areas in Los Angeles Harbor and a smaller volume in the dredged areas in Long Beach Harbor, an appeal was made to the Congress of the United States for assistance in protecting the harbors from further damage of this nature. Under the Federal Government's responsibility for harbors and navigation, Federal funds were ultimately made available by Act of Congress in 1917, for the construction of a new channel to carry the discharge of the Los Angeles River to the ocean just east of Long Beach Harbor. This improvement, completed in 1921, provided a channel, trapezoidal in section, which had a base width of 530 feet and levees faced with heavy, rock riprap. The new channel was approximately 4-1/2 miles in length and extended due south through the City of Long Beach from its intersection with the natural channel upstream.

The construction of the Los Angeles River outlet in 1921 did not provide jetties to carry flows seaward to deep water. Consequently, deposition of sand and silt occurred. A survey in 1926 showed that in the lower 6,300 ft. of channel there was an accumulation of 612,000 cubic yards of sediment and a considerable delta formation seaward from shoreline. In order to protect the Long Beach Outer Harbor from intrusion of flood-borne debris, a stone breakwater, completed in 1929, was constructed on an extension of the westerly bank of the Los Angeles River. It extended southerly 4300 ft. into the ocean and thence in a southwesterly direction towards San Pedro.

Following completion of the breakwater, the sediments in the discharge of the Los Angeles River were deflected in a southeasterly and downcoast direction. The growth of the delta from 1923 to 1935 over an area which extended from the breakwater approximately 4,000 ft. downcoast and seaward from the Long Beach "Board Walk" a like distance was about 4,500,000 cubic yards. In 1938 this accumulation was greatly increased. Flood Control District offshore surveys taken in the summers of 1935 and 1938 showed that over 2,200,000 cubic yards of sand and silt had been deposited within a lesser area offshore during this 3-year period, the major part of which was the result of the March 1938 flood.

In 1943 and 1944 the City of Long Beach Harbor Department, as part of its harbor development program, extended the west bank of the outlet of the Los Angeles River seaward 3500 ft. to the southeast in a long-radius curve from shore by construction of a dike faced with rock riprap. The area westerly to the previously constructed breakwater was filled with material dredged, under supervision of the Corps of Engineers, from the delta at the mouth of the river. Additional material dredged from this area between 1943 and 1946 was distributed along the beach downcoast approximately four miles in a beach-widening program of the City of Long Beach. Over 6,000,000 cubic yards of sand and silt were dredged during this period to final depths varying from 25 to 45 ft. below mean lower low water. In 1946 the dike was extended another 700 ft.

Further improvement of harbor facilities was undertaken by the Long Beach Harbor Department in 1949 by extension of the rock dike 975 ft. farther seaward to a total length of approximately a mile and the filling of the reclaimed area to the west with material dredged from the Los Angeles River outlet. Approximately 8,200,000 cubic yards were removed to a depth of 70 ft. below mean lower low water during the dredging operations which were only recently completed.

In addition to these outlet improvements, a short length of jetty has been constructed along the extension of the east bank of the Los Angeles River by the Long Beach Harbor Department to protect the downcoast beach from flood damage.

The present conditions at the outlet are considered satisfactory and the dredged area should provide ample storage capacity for flood-borne sands and silt. The construction of Hansen, Sepulveda and Whittier Narrows Flood Control Basins within the tributary watershed and the improvement of the channels downstream therefrom should reduce materially the volume of silt and sands which will reach the ocean in the future, thus reducing outlet maintenance costs.

This reduction will, of course, be considerably less than would otherwise result should some of the sediments deposited in the flood control basins be sluiced to the ocean.

SAN GABRIEL RIVER OUTLET

The San Gabriel River, which drains the eastern part of Los Angeles County and a small part of Orange County has its headwaters in the San Gabriel Mountains which rise to an elevation of over 10,000 ft. above sea level. It flows southwesterly 14 miles across San Gabriel Valley to Whittier Narrows, thence southward 20 miles to enter the ocean at Alamitos Bay near the eastern boundary of Long Beach and within the City of Seal Beach in Orange County. It has a tributary drainage area of approximately 698 square miles. The low, marshy tide lands adjacent to the San Gabriel River outlet are susceptible to inundation during floods unless the runoff is carried to the ocean by an adequate channel.

During the period from 1868, when the San Gabriel River cut a new channel to the ocean, to 1931, the outlet migrated downcoast about 2800 ft. At the end of this period the outlet had reached the low bluff on which the Los Angeles Gas and Electric Corporation, in 1925, constructed a steam electric generating plant, which later became the property of the City of Los Angeles Department of Water and Power.

During the late twenties, the Flood Control District had improved the lower reach of the San Gabriel River by straightening and widening the channel terminating the improvement at the intersection with the natural channel of the river approximately 4,000 ft. northerly from the ocean. The natural course of the river below this point was tortuous and discharged into the easterly end of Alamitos Bay some distance westerly of the outlet to the ocean.

By 1930 a number of oil wells as well as other improvements had been located in the area adjacent to Alamitos Bay, which were subject to damage in case of inundation from flood waters of the San Gabriel River. It was, therefore, believed desirable that a new channel be constructed to provide a more direct and controlled outlet for San Gabriel River discharge.

The proposed project called for the construction of a channel having a base width of 260 ft. with rock-faced levees, extending from the terminus of the improved section upstream southerly to the confluence with Alamitos Bay, widening to a base width of 300 ft., thence to the ocean. The project also included construction of two parallel jetties 320 ft. apart and extending approximately 500 ft. seaward from Ocean Avenue, the shoreline drive between Seal Beach and Long Beach.

The original plans provided that the jetties be extended in units of 500 ft. each to a total length of 1500 ft. with the ends of each unit opposite each other in practically equal depths of water. This was to provide for the building of a symmetrical delta which, under favorable wave action, it was believed, would permit distribution of the delta deposits to both Los Angeles and Orange County beaches. Due to the location of the outlet, it was necessary that the several political subdivisions concerned approve the plans. In order to obtain the approval of the City of Seal Beach, it was necessary to revise the plans for jetties and construct the easterly jetty approximately 350 ft. longer than the westerly jetty, or to a length of 725 ft. The jetties were completed in 1933 and the channel excavation in 1935. The westerly jetty was extended in 1940 from an approximate length of 375 ft. to the same distance seaward as the easterly jetty. No extensions have been made to these jetties since then.

One of the important considerations in design of the outlet was that of safeguarding the supply of cooling water which was required to cool the condensers of the steam power plant of the Los Angeles Gas and Electric Company. The ebb and flood of tide water in Alamitos Bay had, at time of construction of the power plant, scoured a channel adjacent to the bluff on which the steam plant was located, and a satisfactory supply of cooling water had thus been available and was probably one of the major reasons for its location at that site. It was also necessary to provide a warm water by-pass in the east jetty to permit the water used for cooling purposes to be returned to the outlet.

In order to determine the most effective solution to the problems incident to the design of the outlet, model studies were undertaken by the California Institute of Technology in 1933 and 1934, financed jointly by the Los Angeles Gas and Electric Corporation and the Flood Control District. Based on recommendations resulting from the model tests, current deflector vanes were constructed on the downstream side of each of the 12 piers of the Second Street bridge across the newly-constructed section of the San Gabriel River. The levee on the west side of the channel was extended only a short distance south of Second Street, the first street upstream from Ocean Avenue, leaving an unrestricted opening into the channel from Alamitos Bay. The tests indicated that the flood flows would be directed, by the deflectors, toward the cold water intake of the steam plant which, in conjunction with the ebb and flood of the tidal prism in Alamitos Bay, would maintain satisfactory intake conditions at the steam plant.

By 1937, an extensive sand bar had formed on the west side of the channel, immediately upstream from Ocean Avenue, which thereby constricted the opening from the bay into the channel and, extending easterly into the channel, had also formed a constriction adjacent to the cold water intake. However, the depth of water adjacent to the intake was still comparable to that prior to construction of the new outlet. The flood of March 1938 scoured out a portion of the sand bar and reduced the constriction, thus permitting a less constricted flow from Alamitos Bay. By 1941, however, shoaling of the channel, resulting from tide-borne material, had become so serious that the Flood Control District constructed a 200-ft. groin of Wakefield sheet piling at the south side of the confluence of the bay and channel to deflect the tidal flow from the bay into a more restricted section hoping to thus maintain sufficient depth of water at the cold water intake. Conditions in the channel failed to improve and dredging of a training channel was undertaken during several periods in 1941 and 1942 to direct the discharge of the outlet along the easterly side of the channel adjacent to the steam plant. These measures were of only temporary benefit. In 1944 a separate entrance to Alamitos Bay was constructed under joint cooperation by the State of California, County of Los Angeles, City of Long Beach, Los Angeles Department of Water and Power, and the Flood Control District. Included as parts of the project were the separation of the San Gabriel River Outlet from Alamitos Bay by construction, by the Flood Control District, of a rock-faced levee on the west side of the channel, and the construction by the Los Angeles Department of Water and Power of a cold water intake on the bay side of the new levee and a connecting conduit under the river channel to the steam plant. This installation has provided a satisfactory supply of cooling water for the power plant since that date.

The new entrance to Alamitos Bay included construction of a stone jetty approximately 800 ft. long on the upcoast side, and the dredging of a channel varying from 80 ft. to 200 ft. in width which extended from the seaward end of the west jetty through the entrance and westerly into Alamitos Bay. Sand dredged at this time and during 1945 and 1946, amounting to approximately 800,000 cubic yards, was distributed in a beach-widening program upcoast from the west jetty of the bay entrance.

The shoaling of the outlet of the San Gabriel River has, during the past six years of deficient rainfall, and separation from Alamitos Bay, become more and more pronounced. The downcoast littoral drift has carried a large portion of the beach sand from upcoast of the bay entrance to the vicinity of the river outlet where it has been carried up-channel by flood tide. Ebb tide has failed to carry it back to the ocean. This has resulted in the formation of an extensive sand bar on the west side of the channel between Ocean Avenue and Second Street, and a still larger bar on the east side of the outlet seaward from the warm water by-pass to the extent of almost overtopping the east jetty.

Experience during past minor river discharges indicates that these sand bars will erode readily during floods. Ultimately, however, it may be necessary to dredge a basin seaward of the outlet or extend the jetties to deeper water. This eventuality may be delayed considerably, however, due to the dredging which the City of Long Beach has just undertaken at the entrance to Alamitos Bay. The basin to be dredged seaward of the entrance as well as the entrance channel itself will require the removal of approximately 450,000 cubic yards of sand which is to be distributed along the beach upcoast. This improvement not only will remove a part of the delta formed at the entrance to the bay, thus removing one of the immediate sources of supply to the sand bars in the river outlet, but it will entrap sand which will be moved downcoast by future littoral drift, thereby further delaying more serious shoaling of San Gabriel River outlet from tidal action.

CONCLUSIONS

In conclusion, and as of this date, the three ocean outlets, covered in this discussion, appear to be in a favorable condition in providing safe waterways to the ocean for flood waters. Beach damage resulting from installation of outlet jetties has become a negligible factor either by reason of the natural forces in play or because of artificial beach replenishment. The future requirements in maintenance of the individual outlets will, of course, depend upon future floods, the operation of upstream flood control works, and shore and offshore installations. By and large, the maintenance of adequate outlets should become less instead of more of a problem in the years ahead.

CHAPTER 32

HISTORY OF COLUMBIA RIVER JETTIES

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INTRODUCTION

Columbia River is the largest river on the Pacific Coast of the United States. It heads at Columbia Lake in British Columbia, about 80 miles north of the international boundary, and flows northward parallel to the summit of the Rocky Mountains for about 185 miles, thence turns back and flows generally southward through Upper and Lower Arrow Lakes and enters the United States about 25 miles west of the northeast corner of the State of Washington. Thence the river flows by a sinuous course southward, westward, and southeastward to the Oregon-Washington boundary, thence generally westward between the two states, discharging into the Pacific Ocean 583 statute miles north of San Francisco Bay and 154 miles south of the Straits of Juan de Fuca (distances computed from differences in latitude). The river has a total length of 1,210 miles, of which 750 miles are in the United States.

The river and its tributaries drain parts of seven states and a large part of British Columbia. The southern limit of its basin is in Nevada, and its eastern boundary is the Continental Divide; the area of the drainage being about 259,000 square miles, of which about 220,000 are in the United States. The watershed is generally rugged and precipitous, although many benches, flats, and valleys exist throughout the basin. The main tributaries are the Okanogan, Kootenai, Clark Fork, Snake, and Willamette Rivers. Of these, the Snake, which drains an area of 109,000 square miles, is by far the largest.

The extreme low water flow of Columbia River at its mouth, exclusive of tide water, is estimated at 65,000 sec.-ft., while the maximum discharge of record, during the flood of June 1894, was about 1,200,000 sec.-ft. Maximum discharges, due to melting snows in the high headwaters, occur during the period from May to July and the average freshet flow is about 660,000 sec.-ft.

Low flow generally occurs during November through February; the average low flow being about 70,000 sec.-ft. During the period of low flow, winter freshets occur, due principally to flood conditions in the tributaries west of the Cascade Range, and, occasionally, the winter freshets on the lower river exceed the average summer freshet.

The Columbia River is not classed as a sediment-bearing stream and for the greater part of the year the river is clear, but during freshets a small amount of material is carried in suspension. Measurements made below the mouth of the Willamette River in 1922 showed about 120 parts per million in suspension during average freshet stage. This, however, does not include the bed load of coarser sand, which, during the summer freshets, travels down the river bed in waves of material that range from 8 ft. to 14 or more ft. in height near the mouth of Willamette River but decrease in height as they travel down the channel. This bed load is the source of material forming the shoals in the lower river. Some of the sand finally and progressively reaches the ocean and is undoubtedly the major source of the beach sands extending on each side of the river's mouth.

Tides at the mouth of Columbia River have the diurnal inequality typical of the Pacific Coast of the United States with a long runout to lower-low normally following higher-high water. The mean range of tide is 6.5 ft.; the range from mean-lower-low water to mean higher-high water is 8.5 ft.; and extreme tides range from minus 2.5 ft. to plus 11.5 ft. Tidal effect extends upriver 145 miles to Bonneville Dam during periods of low-river flow. River discharge and tidal flow must be considered together as they are united in their action. The ebb discharge at the entrance averages about 1,350,000 sec.-ft., while the peak flow recorded during the freshet of 1933 was 3,065,000 sec.-ft. Data regarding flow at the mouth of the river, taken from the current survey of 1933, are shown in the following table:

TABLE I

Month	Number of tidal cycles	Average river discharge in 1,000 sec.-ft.	Average duration of tides in hours		Average tidal flow in 1,000 sec.-ft.	
			Flood	Ebb	Flood	Ebb
April	8	315	5.43	6.90	1,037	1,345
May	13	556	4.95	7.55	971	1,481*
September	7	125	5.81	6.44	1,257	1,306

*Maximum ebb discharge during this period was 3,065,000 sec.-ft.

MOUTH OF COLUMBIA RIVER BEFORE IMPROVEMENT

The mouth of Columbia River lies between a low sand spit, Point Adams, on the south, and a high, rocky headland, Cape Disappointment, on the north. Before construction of the south jetty, a sand beach extended southward 18 miles from Point Adams to Tillamook Head, the headland next south of the entrance. Extending north and west from Point Adams were numerous and changeable shoals known as Clatsop Spit.

Cape Disappointment, about 6 miles north-northwest from Point Adams, is the only headland on the low sand beach that extends northward from Tillamook Head to

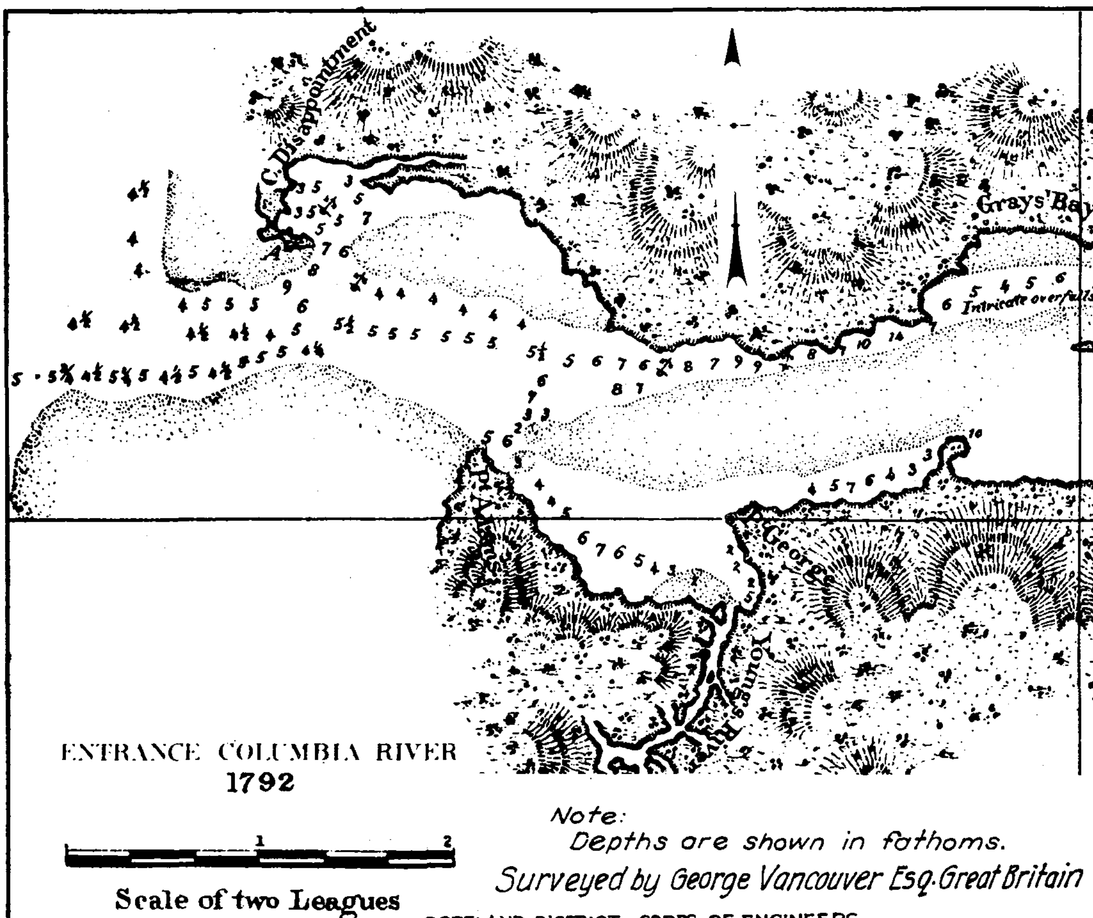


Fig. 1

Point Greenville, a distance of about 94 miles. The cape is the southern end of a coast range spur that swings in a curve, concave eastward, to form the west shore of Baker Bay, and consists of rounding hills covering an area about 3 miles long by 1 mile wide. The seaward faces of the hills are precipitous cliffs, with the extreme southern point, formerly called Cape Hancock but now called Cape Disappointment, rising about 190 ft. above the river; and North Head, the extreme western point, rising about 175 ft. above the ocean. The shoal area about the southern end of the cape is called Peacock Spit. The ocean bar connects Peacock Spit to Clatsop Spit by a curve, convex seaward, across the river entrance with its vertex, varying in location from year to year, and, before construction of the jetties, ranged from 2-1/2 to 4-1/2 miles west of a line connecting Cape Disappointment with Point Adams.

The earliest known chart of Columbia River at the mouth is a sketch made by Admiral Vancouver in 1792, shown in Fig. 1. The next known survey was made by Sir Edward Belcher in 1839, shown in Fig. 2. Condition of the river entrance in 1885, before construction was started on the south jetty, is shown in Fig. 3. The shoals, bars, and beaches at the river entrance are composed of a sand that is readily shifted in location by wave action, currents, and wind. Before construction of the jetties, both Clatsop Spit and Peacock Spit were constantly changing in position and depth. Inspection of Figs. 1, 2, and 3, shows that the direction of Clatsop Spit had varied from westward, with a length of nearly 8 miles in 1792, to northwestward, with a length of 3-1/2 miles in 1885. In 1839 its northern edge was but 1/2 mile north of Point Adams, while in 1885 it was 2-1/2 miles. The

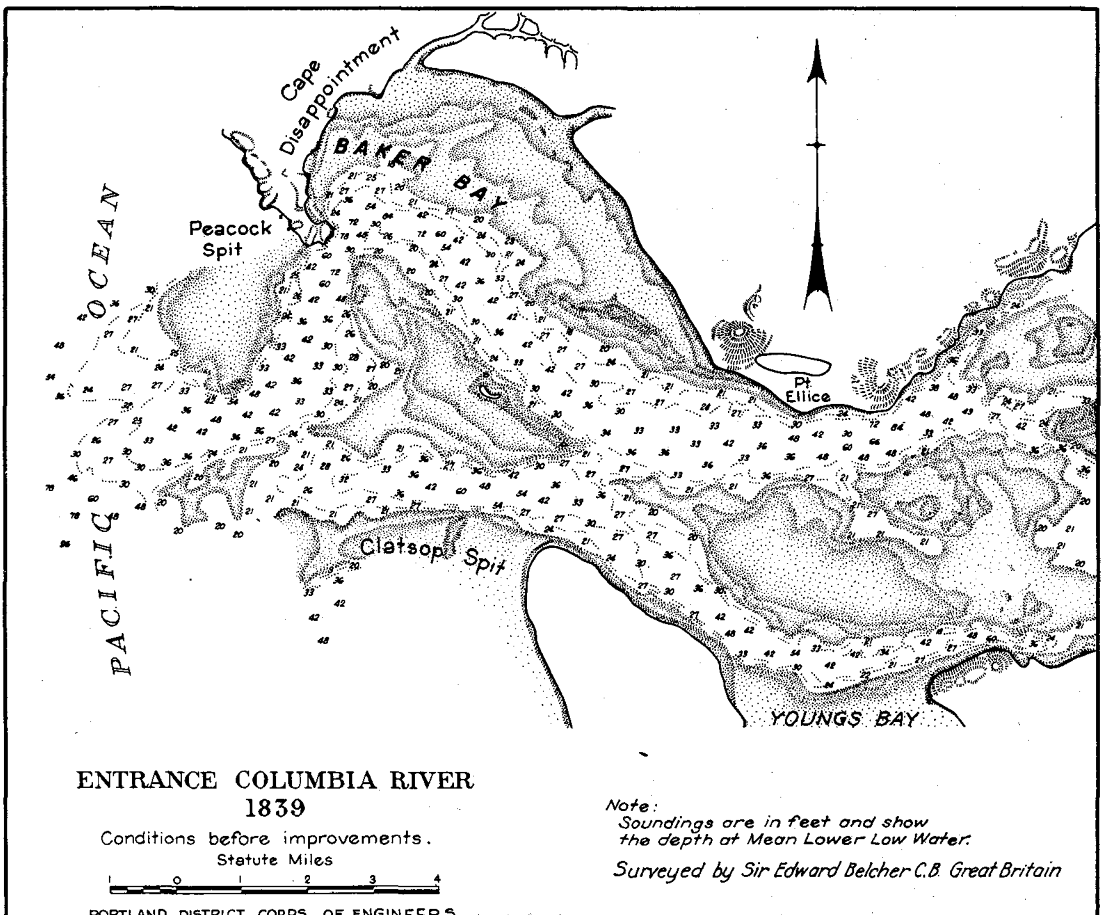


Fig. 2

channel side of Peacock Spit varied in direction from south to west, and its southerly limit ranged between 3/4 and 2-1/2 miles south of Cape Disappointment.

The steep offshore slope along the Pacific Coast in this area, combined with high wind velocities, results in heavy wave action which renders beach erosion and hydrographic studies difficult. The effect of the many forces and the resultant sand movements have been investigated and discussed in numerous reports, the most comprehensive of which are the following:

1903 report. The report of a board of engineers contained in the Annual Report of the War Department, 1903, Chief of Engineers, part 3, includes a series of charts showing changes at the mouth of the river up to that time, and analyzes the sand movement.

Current Survey, 1932-1933. A report by R. E. Hickson of the District Engineer's office gives the results of extensive current measurements and contains information on sand sizes, tidal flows, suspended load, and other general data.

Hodge report. A report on the mechanical and petrographic characteristics of the material forming the estuary and ocean beach, and gives consideration to the movement and sorting of beach sands by winds.

Technical Memorandum No. 20, December 24, 1936. This report, made by the U.S. Tidal Model Laboratory, Berkeley, California, covers studies made to determine the cause of recession of the shore of Clatsop Spit immediately south of the south jetty.

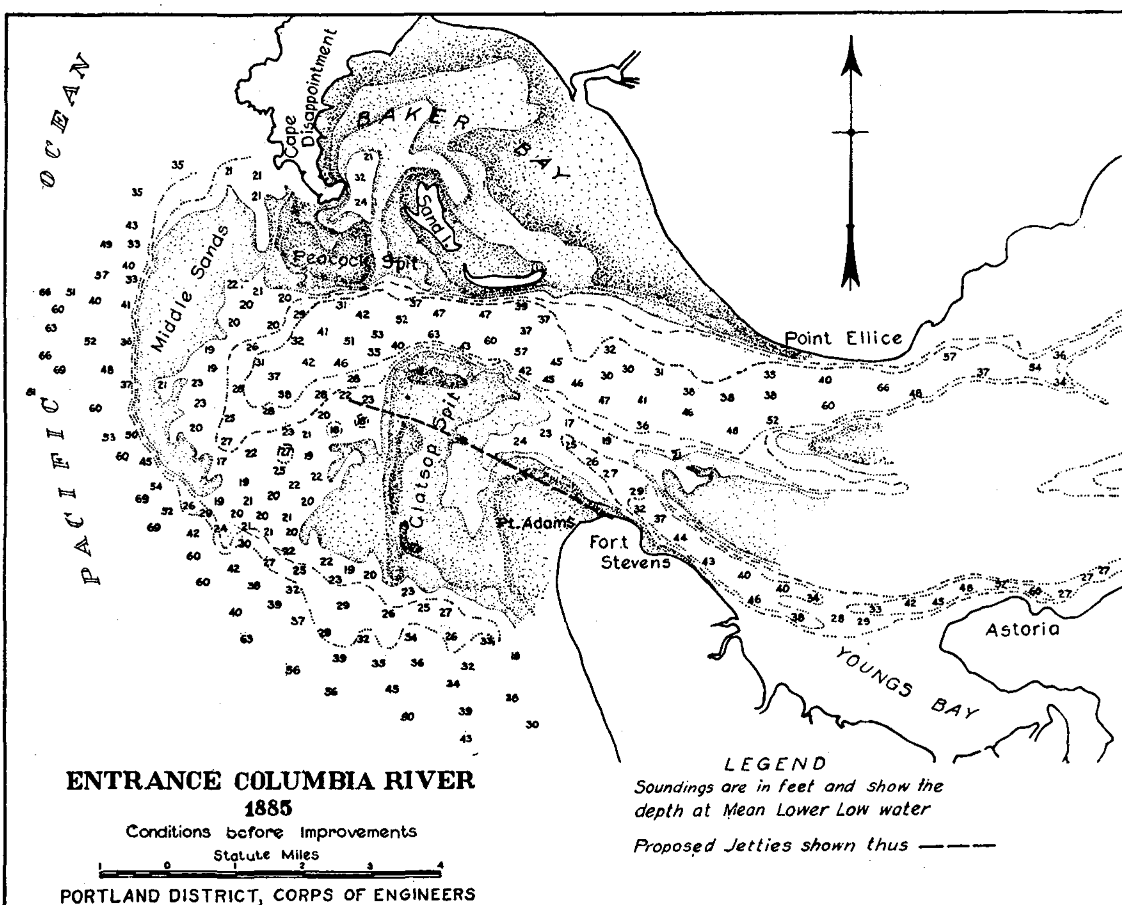


Fig. 3

These reports, in general, agree that sand of the beach south of the entrance, combined with sand brought in from the floor of the ocean, steadily moved to the north past Point Adams in a prolongation of the beach itself. Under the dispersing effects of currents and wave action, this sand had been scattered to form Clatsop Spit. Irrespective of the source, continuous accretions to the north side of Clatsop Spit steadily encroached upon the channel and pushed it to the north, accompanied by a wearing away of the opposite bank and deepening of the channel. The point of greatest erosion was at the extremity of the spit's crest, and here was the river gorge, the deepest and narrowest section of the entrance channel.

Conditions at the entrance in 1895, when the original south jetty was completed, is shown in Fig. 4. Deterioration of the channel by 1902 is shown in Fig. 5. Improvement of the channel by the time the extension of the south jetty was completed in 1913, and when the north jetty was 90 percent completed in 1916, is shown in Figs. 6 and 7, respectively. The condition of the entrance in 1950 is shown in Fig. 8.

The set of the main channel ebb tide is to the southwest at all seasons, and the channel to the ocean has consistently remained across the southwest quadrant of the bar; the only exceptions to this rule having been the channel shown by Admiral Vancouver's sketch of 1792, and deteriorating channels that have worked around to a northerly direction. The northerly movement of Clatsop Spit and the river gorge, while the channel across the bar remained in the same general location, increased the length of the main channel until it was much greater than the distance between deep-water curves measured directly across the root of Clatsop

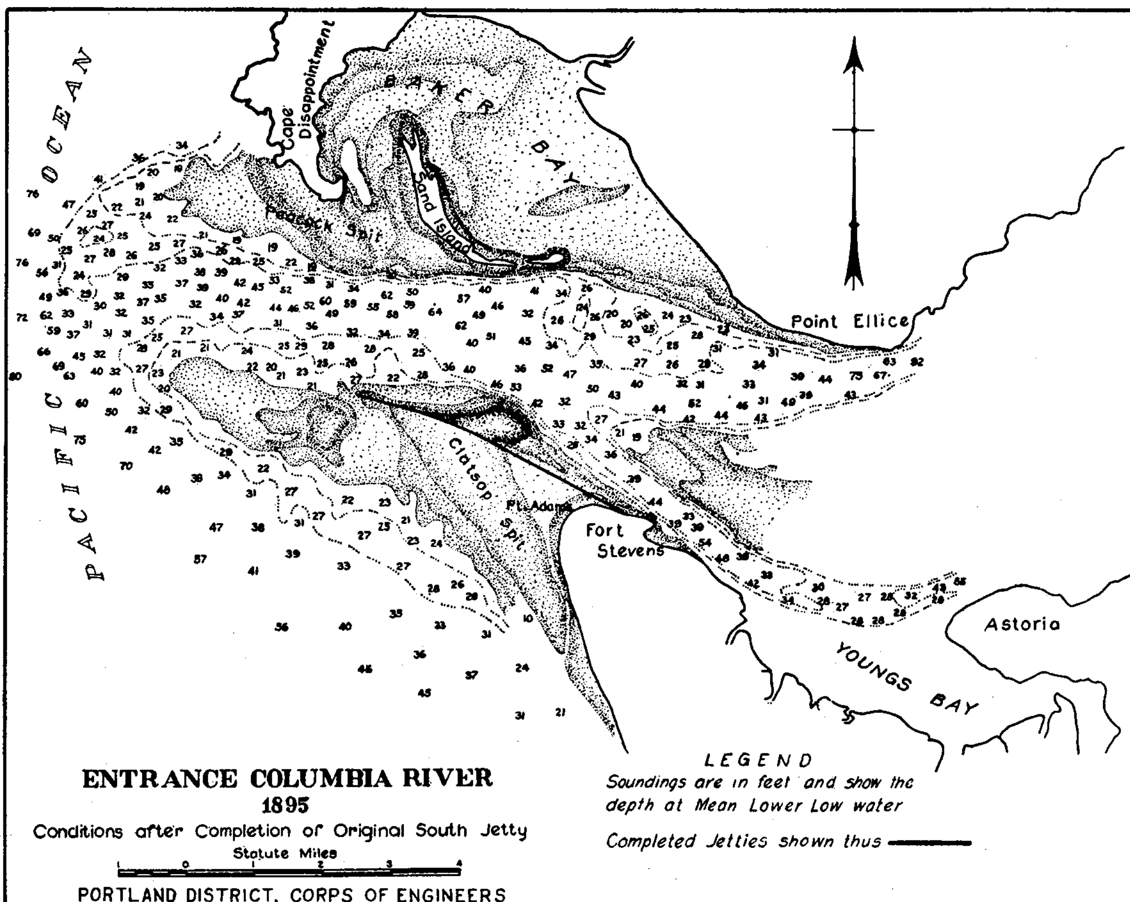


Fig. 4

Spit, and the shorter route became the easy route for the tidal flow. When the difference became sufficient the tidal flow would cut a swash channel across the spit. Under favorable conditions the swash channel would gradually increase in depth, and the main channel, partially robbed of the tidal flow, would begin to shoal so that there would be two inferior channels. Gradually, the channel across the root of Clatsop Spit would become the main channel, leaving the northern portion of Clatsop Spit as a middle ground or island. An examination of the old charts shows that Sand Island, which today separates Baker Bay from the river, probably was cut off from Clatsop Spit between 1792 and 1839 and gradually shifted north and east to its present location.

The many changes in the shoals at the river entrance added greatly to the hazards of navigation. In order to improve the channel, Congress, by the River and Harbor Act of August 2, 1882, authorized a board of engineers "to examine in detail the mouth of the Columbia River, Oregon, and report such plan, with estimates, for its permanent improvement as they approve." The board submitted its report on October 13, 1882, and it was printed in Senate Executive Document No. 13, 47th Congress, 2nd session.

The board of engineers used the river section between Chinook Point and Point Adams to determine the area of a section which would maintain itself, decided it was impracticable to diminish the area between Cape Disappointment and Point Adams by a detached structure on the bar, and recommended ".....the construction, at as early a day as possible, of a jetty, slightly convex to the north, extending from the shore near Fort Stevens (on Point Adams) in a northwesterly direction towards

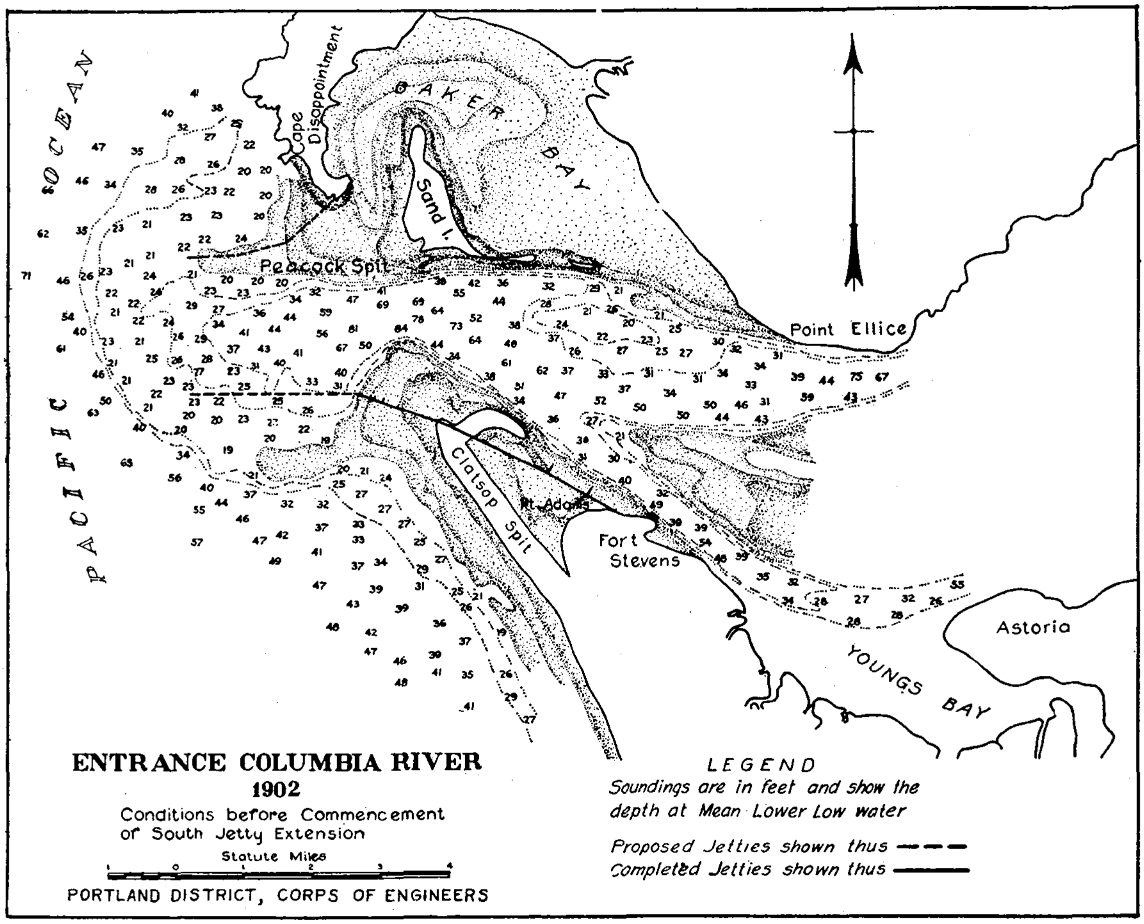


Fig. 5

a point about 3 miles south of Cape Disappointment, this jetty to stop short of that point or be prolonged beyond it, as experience may indicate to be necessary." The board also mentioned the possibility of a jetty on Peacock Spit. In view of the exposed location and the difficulty in maintaining any structure the board proposed the jetty be brought to the height of low tide, or higher, if experience showed necessity therefor.

HISTORY OF THE SOUTH JETTY

Construction of the south jetty was begun in 1885 but proceeded so slowly that it did not affect the condition of the bar channel until 1889. Rapid construction commenced in 1889 and its effect on the bar was immediately noticeable. In 1893 a board of engineers was convened to consider results attained and to report a plan for final completion. This report, submitted in May 1893, recommended construction of four groins on the north side of the jetty (shown in Fig. 5), and that the jetty at the shore end should be carried to 12 ft. above low water, should slope to plus 10 ft. at 1-1/8 miles from shore, whence it should slope to plus 4 ft. at its outer end, the total length to be 4-1/2 miles. All of these recommendations were carried out and the jetty was completed in 1895. The jetty was of rubble-mound type, built from trestle work.

The changes in the channel induced by construction of the jetty were large and important. The channel depth, which was 20 ft. in 1889, increased to 31 ft. in 1895, remained at 30 ft. through 1896 and 1897, then began to decrease. The direction of the channel, which had a bearing west of south in 1885, gradually

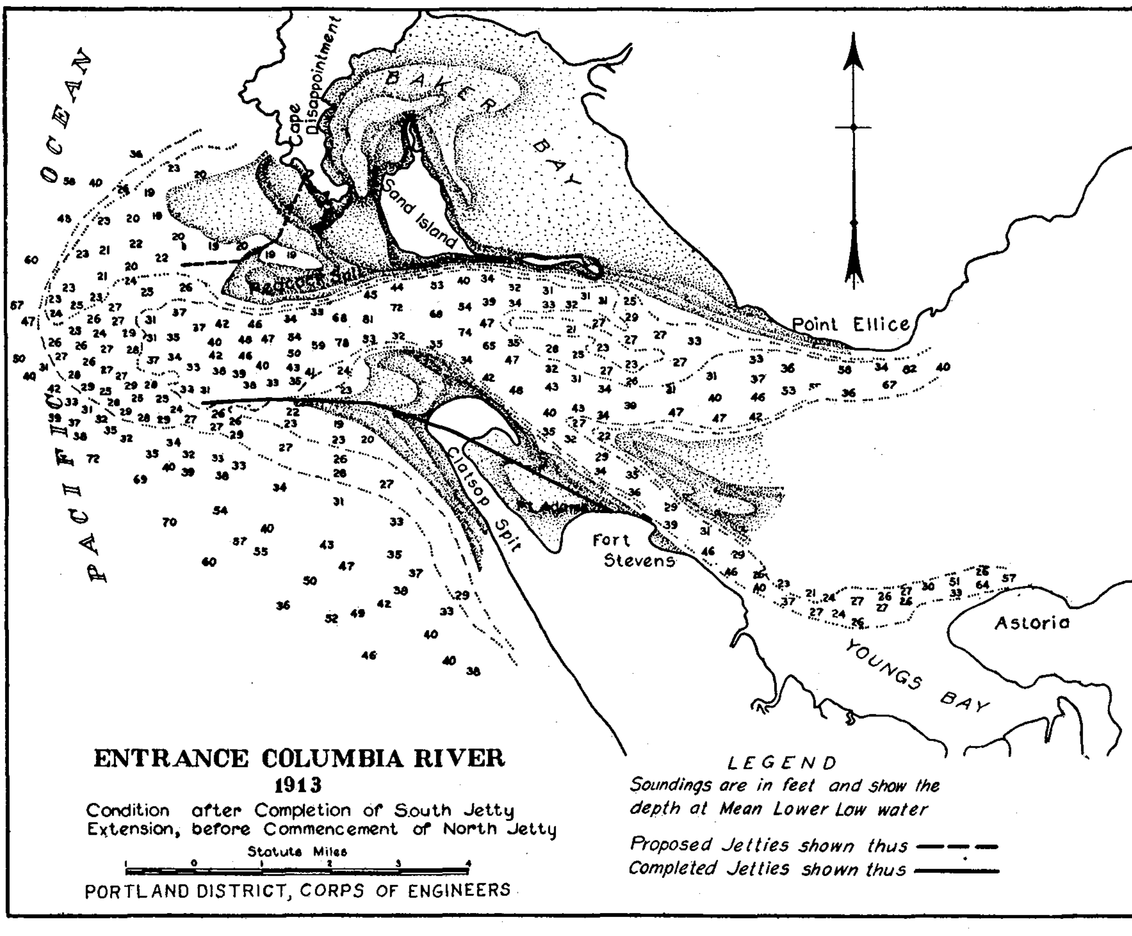
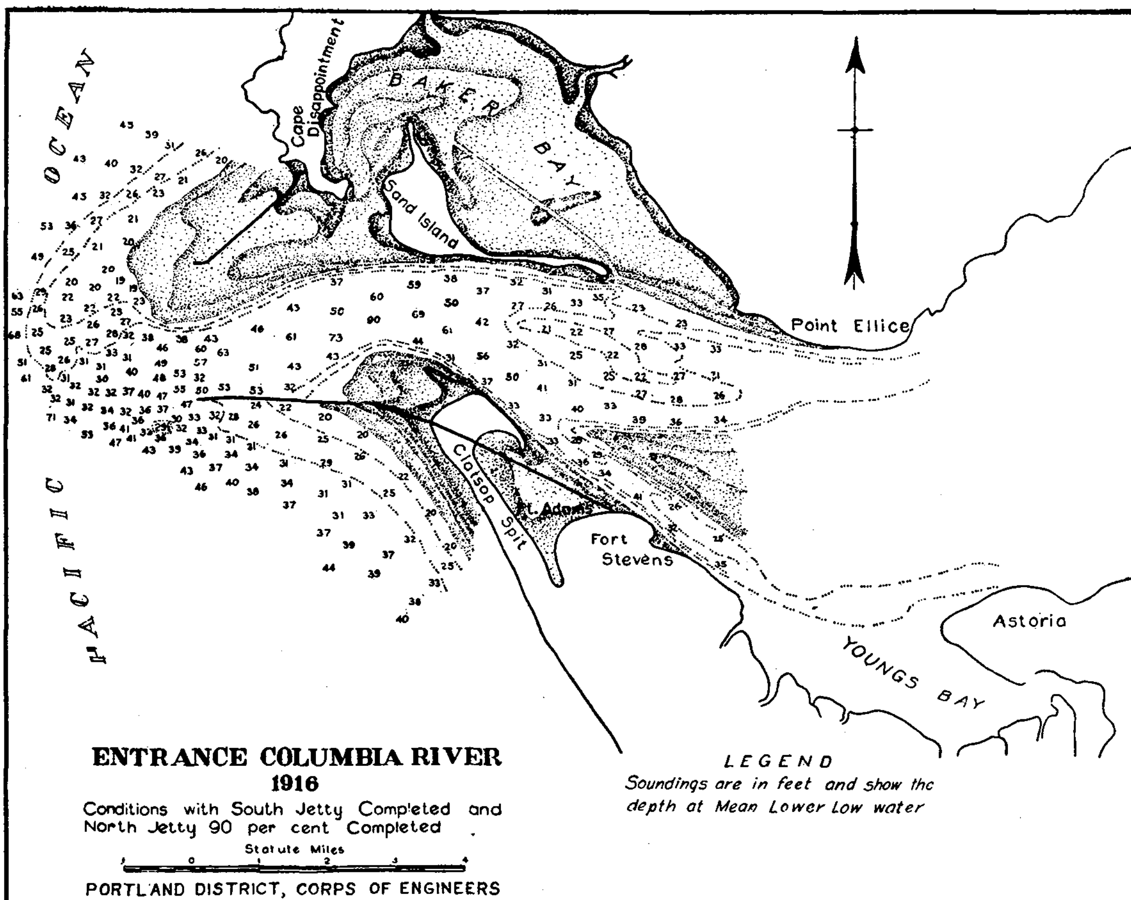


Fig. 6

swung around to the north as construction of the jetty proceeded, and by 1895 the channel ran almost due west and was in splendid condition. The swing to the north continued, however, and the depth increased to 22 ft. in 1902, when the remains of the old bar channel pointed nearly due north and there were two new channels of almost equal depth across the western sector of the bar. It was evident that further improvement of the mouth of the river was necessary.

The River and Harbor Act of March 3, 1899, authorized examination and survey of the mouth of the river "with a view to obtaining a channel 40 feet deep at lowest water, and a report as to the desirability of such improvement." The report is contained in House Document No. 94, 56th Congress, 1st session. This project was referred to a board of engineers who, on January 24, 1903, submitted a report that provided for extending the existing south jetty due west for a distance of 2-1/2 miles, construction of a north jetty from Cape Disappointment to a point 2 miles north of the outer end of the 2-1/2-mile extension of the south jetty, and dredging.

The extension of the south jetty was started in 1903 and completed in 1913. The jetty was constructed of rubble stone, and, because the exposure was so great and the incessant wave action prevented repair operations by floating plant, it was necessary to delay maintenance until the amount of work required would justify the cost of the necessary trestle and plant. No maintenance was done on the jetty until the fall of 1931, and by that time the sea had flattened the enrockment to about 102-water level and spread out the stone so that the width of the outer 2-3/4 miles was about 200 ft. at low-water level. Under three contracts, 2,200,000



ENTRANCE COLUMBIA RIVER
1916
Conditions with South Jetty Completed and North Jetty 90 per cent Completed
PORTLAND DISTRICT, CORPS OF ENGINEERS

Fig. 7

tons of stone were placed in the superstructure, which was carried out to within approximately 3,300 ft. of the outer end of the jetty as completed in 1913. This was believed to be the limit of the superstructure required. The reconstruction of the south jetty was completed in 1936, but the action of waves across the end face of the new work started piecemeal disintegration of the outer end. During a normal winter season the superstructure would ravel back 300 ft. or more. The outer end was impregnated with 12,737 tons of hot mixture of asphaltic mastic (85 percent sand and 15 percent asphalt) in an attempt to prevent raveling. While computations and later observations indicate that the asphaltic mix completely filled the voids to about low-water level (26 ft.), it did not prevent breakdown of the end, and raveling still continued. A solid concrete terminal was then constructed above low-water level and has proven effective. The concrete terminal is about 3,900 ft. shoreward from the end of the original jetty as conpleted in 1913.

The south jetty as now constructed is a massive structure (see Figs. 9, 10, and 11). The top width varies from 45 ft. to 70 ft., with an elevation of 26 ft. above mean lower low water. The sea slope is approximately 1 on 1-1/2 consisting of stone weighing up to about 25 tons, with 45 percent of entire enrockment having an average weight of 10 tons to the piece. The base width of the outer portion is approximately 350 ft. and the total height from bottom ranges up to 76 ft. The outer 3,900 ft. have been beaten down to 10 or more feet below low water and rebuilding is not believed to be necessary. This outer section, in depth of 70 ft., serves as a protective apron.

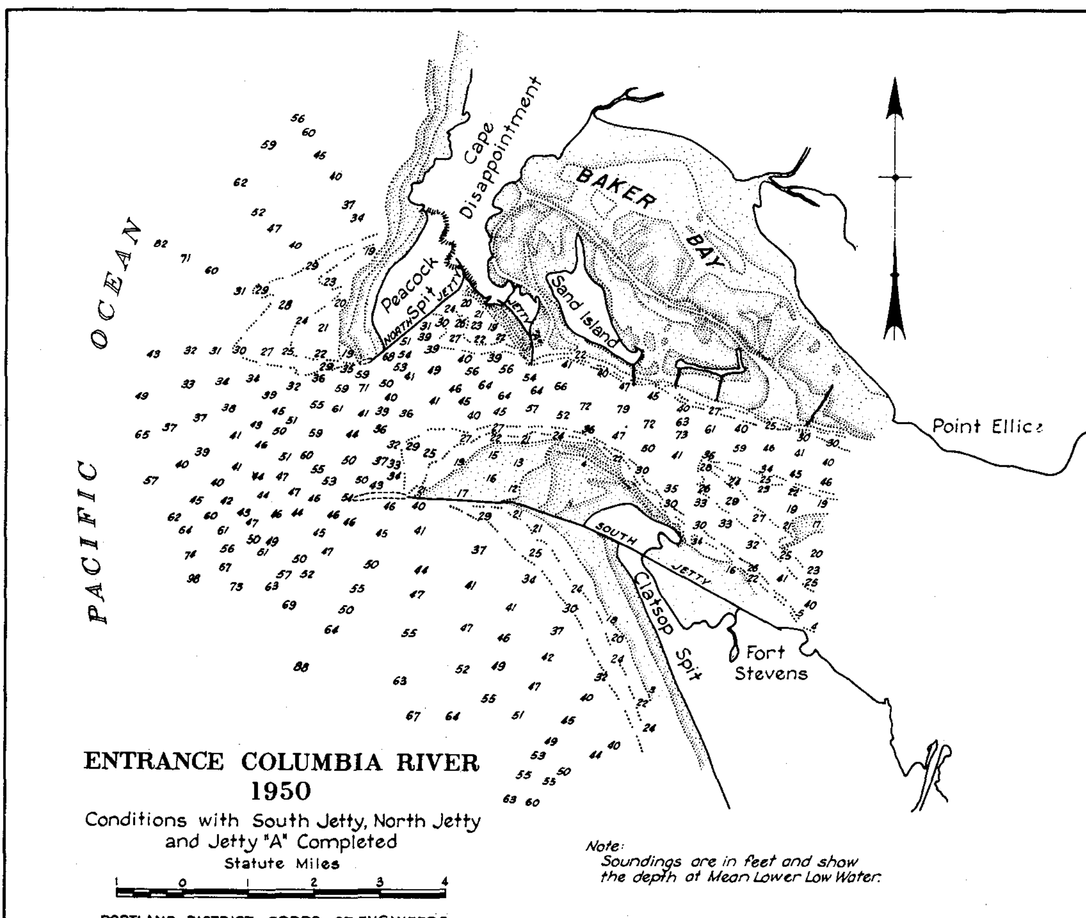
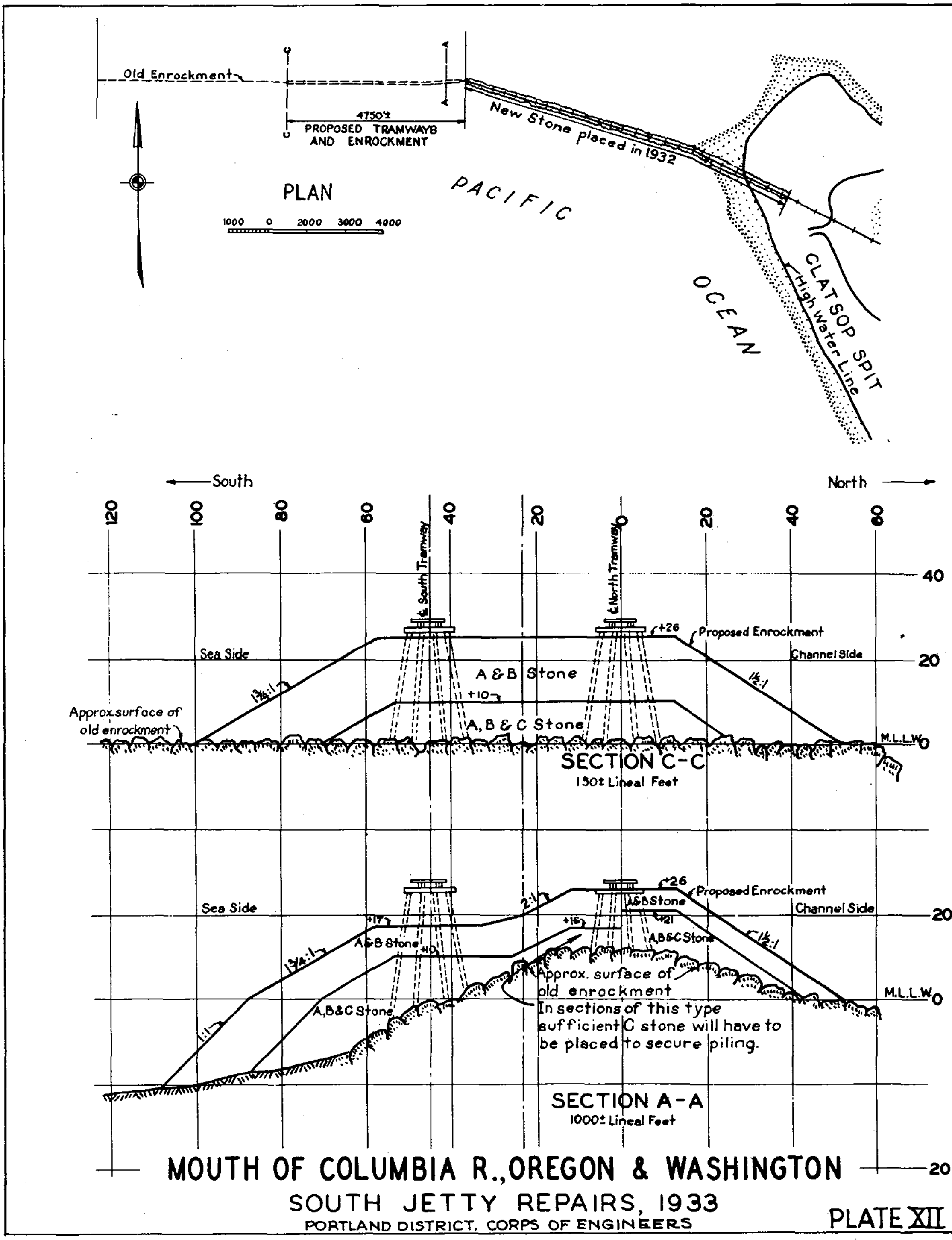
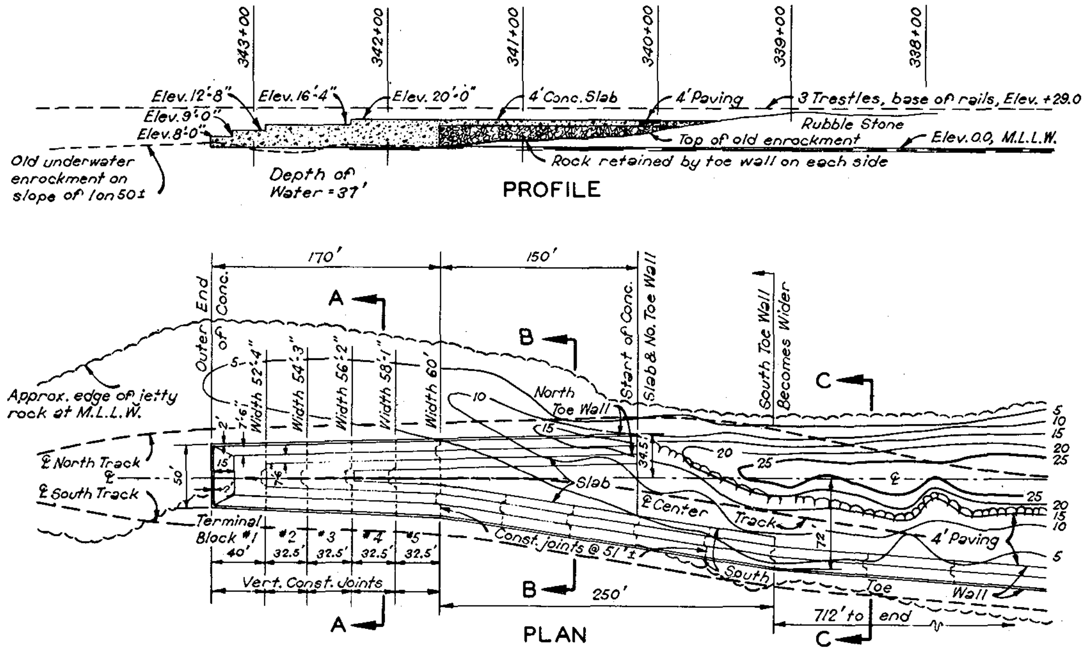


Fig. 8



Old Enrockment
4750'±
PROPOSED TRAMWAYS AND ENROCKMENT
New Stone placed in 1932
PLAN
1000 0 2000 3000 4000
PACIFIC
OCEAN
CLATSOP SPIT
High Water Line
South
North
Sea Side
Channel Side
Proposed Enrockment
A & B Stone
A, B & C Stone
Approx. surface of old enrockment
SECTION C-C
130± Lineal Feet
M.L.L.W.
In sections of this type sufficient C stone will have to be placed to secure piling.
SECTION A-A
1000± Lineal Feet
MOUTH OF COLUMBIA R., OREGON & WASHINGTON
SOUTH JETTY REPAIRS, 1933
PORTLAND DISTRICT, CORPS OF ENGINEERS
PLATE XII

Fig. 9



MOUTH OF COLUMBIA RIVER,
OREGON AND WASHINGTON
SOUTH JETTY TERMINAL

Scale in Feet

100 50 0 100 200 FT.

PORTLAND DISTRICT, CORPS OF ENGINEERS

Fig. 10

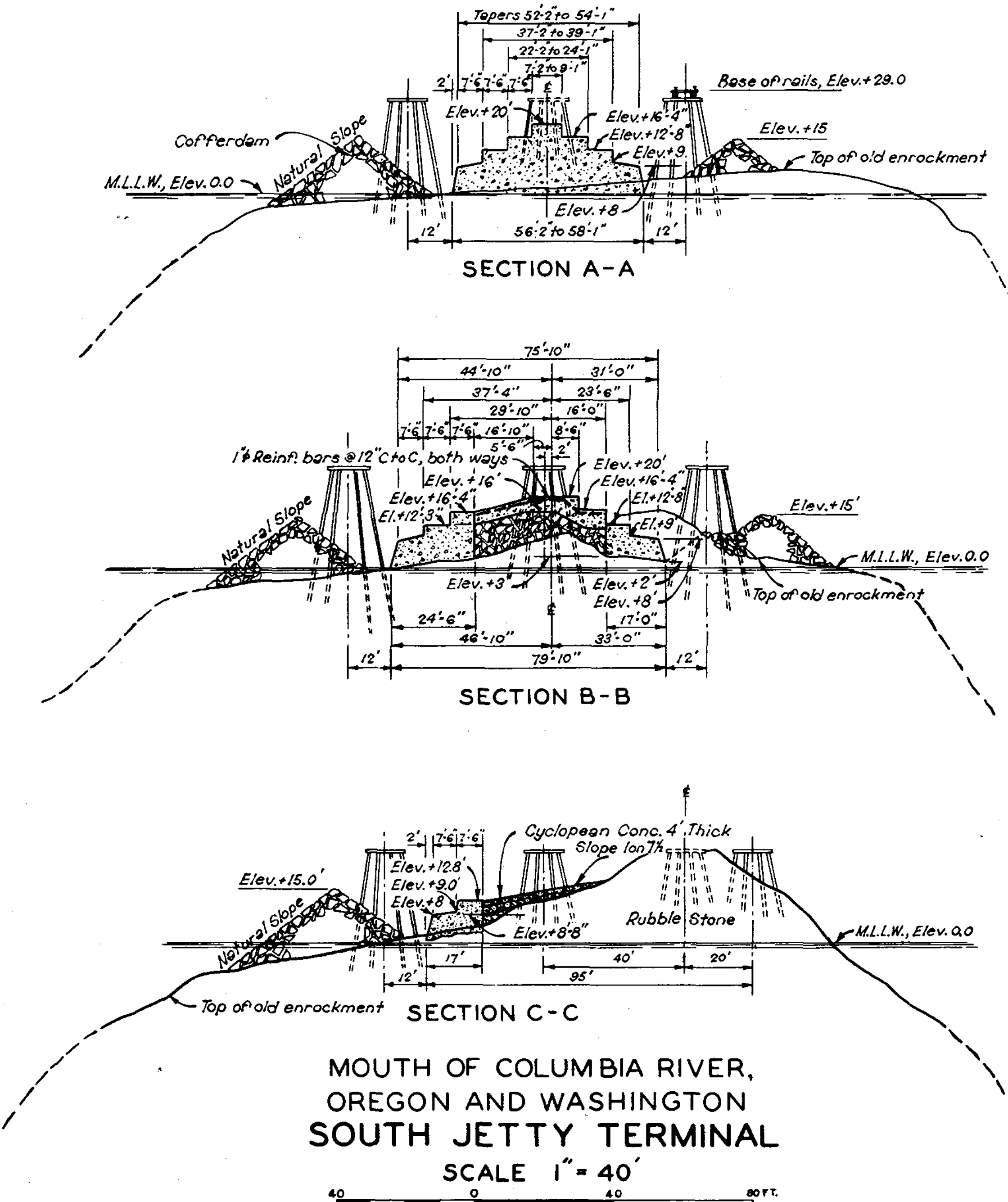
HISTORY OF THE NORTH JETTY

As construction of the extension of the south jetty progressed it became apparent that a north jetty would be necessary to stabilize the entrance and secure and maintain a channel 40 ft. deep. During 1912-1913, while the extension of the south jetty was still in progress, preparations were made for construction of the north jetty. Actual construction started in September 1913 and was pushed to completion in 1917, by which time the controlling depth on the bar had increased to 37 ft. The root of this jetty is on the west side of Cape Disappointment at the west end of a swale which cuts through the cape from Baker Bay to the ocean front, about 3,000 ft. from the southern end of the cape. The jetty was constructed 25 ft. wide on top with side slopes about 1 on 1-1/2, and elevation of the top from 28 to 32 ft. above mean lower low water. The jetty extended southwestward for about 2 miles, to a point 2 miles north of the south jetty, then turned westward for about 1,700 ft. Nearly 3 million tons of stone were placed in the structure.

The outer portion of the north jetty was subject to wave action across the end and by 1930 was flattened to low water. The part of the jetty extending southwestward from the shore for 2 miles, however, had been backed up by a natural sand fill on the northerly side and strongly resisted destruction by wave action. Some damage resulted from undercutting along the southeasterly or river face, and in 1938-1939 the north jetty was rehabilitated and a concrete terminal block was placed at the outer end. The outer portion extending westward from the turn, which had been flattened to low-water level, was not reconstructed. This outer section serves as an apron on the sea slope.

SHORE CHANGES SOUTH OF THE SOUTH JETTY

Between 1880 and 1883 a swash channel was cut across the root of Clatsop Spit about 1 mile northwestward of Point Adams. When construction of the south jetty blocked wave action across Clatsop Spit, sand accumulated on the south side of the jetty, raising the elevation of that portion of Clatsop Spit between the ocean and the swash channel and holding the channel against Point Adams beach. A new beach was thus established between the south jetty and the southern end of the swash channel which was gradually filled with the encroaching sand. By 1899 the southern out-



PORTLAND DISTRICT, CORPS OF ENGINEERS

Fig. 11

let had been closed off and the northern portion of the swash channel became a lagoon separating the new beach from the beach along the ocean side of Point Adams. Fig. 3 (1885) shows the swash channel before the south jetty was built across it. Fig. 4 (1895) shows the sand accumulated on the middle ground and the southern portion of the channel narrowed to near closure. Fig. 5 (1902) shows the remaining portion of the swash channel as a lagoon in the triangle formed by the old beach

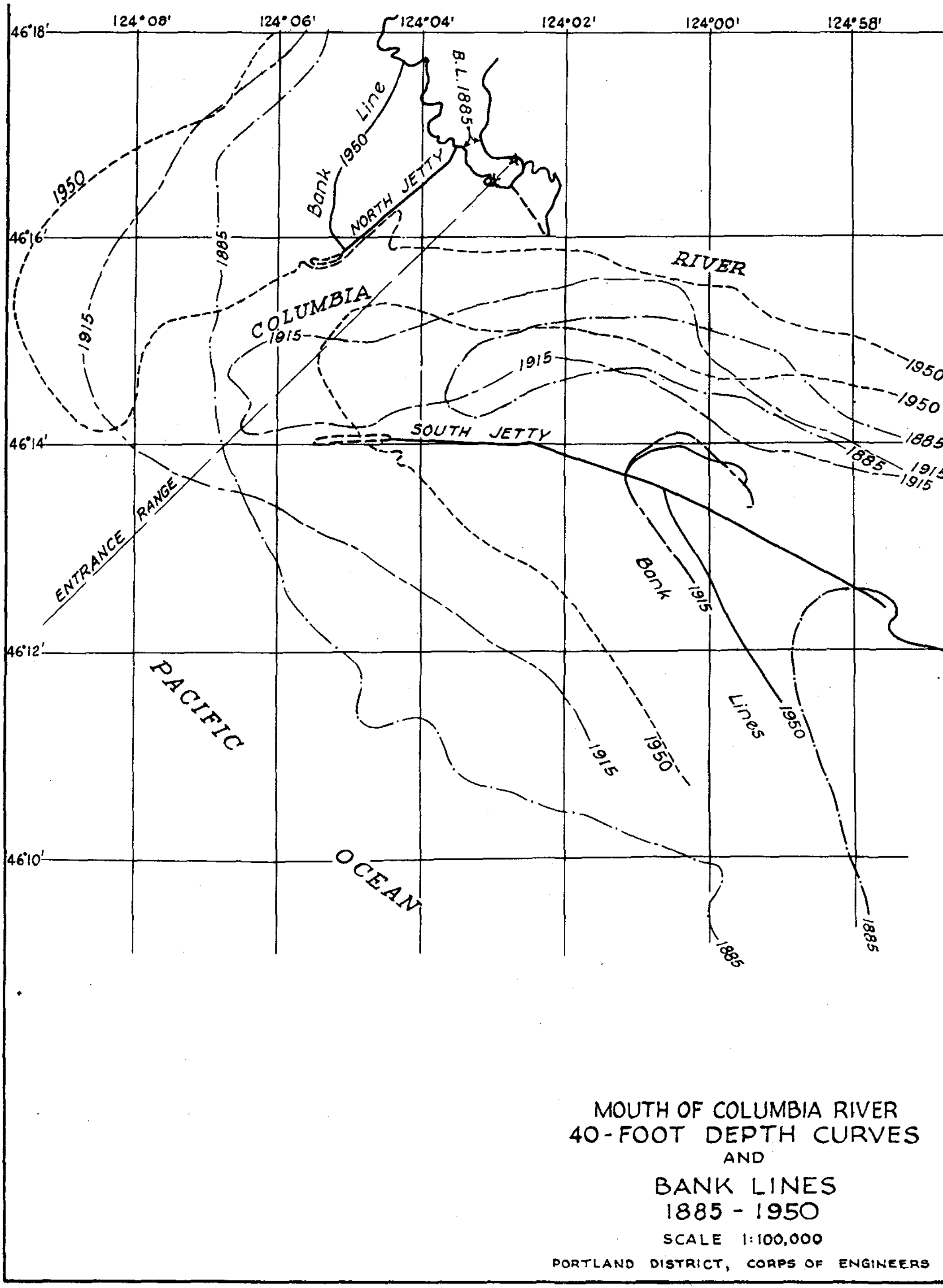


Fig. 12

along Point Adams on the east, the jetty on the northeast, and the landward side of the new beach on the southwest. As the new beach of Clatsop Spit continued to increase in height and advance seaward along the south side of the jetty, it created a new south point to the entrance, which, by 1900, was about 3 miles northwest of Point Adams. This was the outer-most position reached by the beach and since then it has receded about 1/2 mile.

The south jetty was built out across the submerged sands of Clatsop Spit, and a considerable portion of the spit lay on the northerly side between the jetty and the river channel. After 1889, when construction of the jetty was accelerated, the shoals on the river side of the jetty started to erode and, apparently, a considerable portion of the eroded sand was redeposited on the outer bar so that Clatsop Spit continued along the south side of the jetty and extended beyond, forming the crest of the ocean bar (see Figs. 3 and 4). As the accretion to Clatsop Spit had forced the river channel north before construction of the jetty, accretion to the crest of the bar now forced the channel crossing to the north until, in 1902, the channel headed nearly due north. The channel lost depth and definition as it was forced northward.

Before construction of the south jetty the sand of Clatsop Spit, combined with sand from other sources, was washed into the ocean by tidal currents and, transported shoreward by ocean waves, eventually landed on the beach. Thus the south beach was continually being fed by sand washed into the ocean from the mouth of the river. The south jetty disrupted this cycle of sand movement, and the 40-ft. depth contour off the shore began to move shoreward over an area extending about 5 miles south of the outer end of the jetty. At the southern end of this area the 40-ft. contour remained in approximately the same location but, at the midpoint, the contour receded shoreward over 1 mile between 1885 and 1900 and has continued to recede so that by 1945 it had receded nearly 2-1/2 miles. At the northern end of the area the 40-ft. depth contour receded 1.7 miles between 1885 and 1950 (see Fig. 9). (Attempts to feed beaches artificially by dumping dredged material off the beach in depths less than 40 ft. have not proved successful.)

As the new beach line of Clatsop Spit was raised above high water, the sand surface was exposed to erosion by wind. Following the initial accumulation and advance of the shore line seaward along the south jetty, as previously alluded to, recession of the shore line has proceeded steadily up to the present time. Wind transportation is an important factor in eroding the beach, but sand fences and grass plantings are now successfully used to limit, to some extent, the amount of sand transported by winds and the distance the sand is transported. There appears to be a southerly littoral drift of sand inside the line of breakers during the summer months and some sand has accumulated on the southern end of the beach in front of the city of Seaside since the jetty was constructed. The recession of the beach has been mostly limited to an area extending about 3 miles south of the jetty. The total recession, normal to the beach line, has been about 2,000 ft., just south of the jetty, and about 1,000 ft., 2 miles south (see Fig. 9).

SHORE CHANGES NORTH OF THE NORTH JETTY

The north jetty was constructed across Peacock Spit where depths ranged from 9 to 16 ft. and the low-water contour was along the base of Cape Disappointment. Depths of the water between the jetty and the south side of North Head ranged from 9 to 18 ft. As the jetty was built out from shore, the portion of the spit north of the jetty began to rise in elevation and, by 1917, when the jetty was completed. the spit was above low-water elevation in places along the jetty. The area between the jetty and North Head continued to rise in elevation and by 1930 the entire area from near the outer end of the jetty to North Head was filled to heights of 12 to 17 ft. above low water. About 48 million cubic yards of sand have been deposited in that area since construction of the north jetty was started in 1913. This trapping of sand has apparently resulted in starving the north shore line of the river all the way to the east end of Sand Island, with resultant erosion.

CHANGES IN THE ENTRANCE SINCE CONSTRUCTION OF THE JETTIES

Inspection of maps of the mouth of Columbia River shows that, in general, changes over short periods of time are of the same pattern as changes over longer periods, although the rate of change may be different. After completion of the jetties in 1917, depths on the southwest sector of the outer bar increased and by 1921 there was a least depth of 43 ft. for a width of 4,500 ft. and of 40 ft. for a width of over 6,000 ft. Since that time a channel in excess of project dimensions has been maintained without dredging.

Inside the outer bar, between the jetties, the deep water was on the south side, in 1917, with depths in excess of 50 ft. Built up by continuous accretions, Clatsop Spit grew to the north and west. Near the outer end of the south jetty the 40-ft. depth contour advanced 5,000 ft. westward and 4,000 ft. northward, and opposite the "knuckle," about 2-1/2 miles in from the outer end of the south jetty, the 40-ft. contour advanced 2,500 ft. northward between 1917 and 1932. This steady encroachment upon the deep water pushed the channel north, accompanied by a corresponding cutting away of the north bank. The 40-ft. depth contour along the north side of the channel receded northward 1,500 ft. opposite the south end of Cape Disappointment and about 3,700 ft. opposite the outer end of the north jetty. During this period of time the volume of sand in that portion of Clatsop Spit north of the south jetty was greatly increased. The source of the sands which made the increase of Clatsop Spit is not easily determined, but during most of the time between 1917 and 1932 the south jetty was in a condition which permitted sand to pass across it, and it is assumed that a large part of the sand came from the south. This material was carried across the jetty by the flood tides entering the river.

The deep water in the south side of the channel in 1917 provided an extremely satisfactory inner channel but controlling depths on the outer bar were only 40 to 42 ft. and presented a hazard to vessels when there was rough water on the bar. However, the depth on the bar gradually increased and by 1925 there was a channel 1/2 mile wide with depths of 46 to 48 ft. In 1925, Clatsop Spit had not encroached upon the inner channel sufficiently to cause undesirable alignment, and shoal depths on the westerly sector of the outside bar protected the channel from westerly waves. The combination of deep water on the outer bar, excellent alignment of the entrance channel, and protection from westerly waves afforded by shoal depths on the western portion of the bar provided the most satisfactory condition ever attained at the mouth of the river. The splendid entrance conditions continued throughout 1925 and 1926 but by 1928 Clatsop Spit had advanced into the river, and the navigation channel was being forced to the north. The westerly section of the outer bar continued to erode until depths increased from 20 ft. to about 38 ft. at low water so that now (1950) they are only 7 to 8 ft. less than depths in the ship channel across the southwesterly sector of the bar.

As Clatsop Spit advanced into the channel the portion of Peacock Spit lying south of the north jetty deepened and the low-water line receded back to near the base of the cliffs of Cape Disappointment. This permitted tidal currents to develop along the south side of the north jetty, and a deep trench was cut along the outer portion of the jetty.

It is very desirable to secure again the channel conditions that existed in 1925 and 1926 and while this may take a long time, it appears that it may be accomplished by the natural scouring of the southerly bank of the channel upstream, the construction of a stabilizing jetty between the north jetty and jetty "A", supplemented by dredging.

During the period from 1932 to 1938 the south jetty was repaired and improved. To prevent further shifting of the channel to the north with resulting erosion of Sand Island and Peacock Spit, four permeable dikes were constructed out from the south side of Sand Island, and jetty "A" was built southward from Cape Disappointment. By 1938 these structures had stabilized the north side of the channel east of jetty "A". (See Fig. 8 for location of these structures.)

The 40-ft. depth contour along the north side of the channel remained in approximately the same location except west of jetty "A", where it receded to the north a distance ranging from zero at spur jetty "A" to about 3,000 ft. near the outer end of the north jetty.

The volume of material above the 40-ft. depth, in that portion of Clatsop Spit lying north of the south jetty and west of the north and south line 10,000 ft. east of Cape Disappointment lighthouse, increased by only 1,200,000 cubic yards during the six-year period, but the sands shifted in location and, along its west side and the west half of its north side, the spit continued to advance into the channel, which caused the north side of the channel to recede as previously described. The encroachment upon the channel from the south also caused additional scouring off the top of Peacock Spit, which permitted an increased flow of the ebb tide across to the north jetty with a resulting current along the jetty, and considerable scouring occurred in the deep channel along the outer end of the jetty and extended in a projection of the jetty alignment. Spur jetty "A" caused erosioh of the south side of the channel as planned, and the adjacent reach of the channel has been practically stabilized since 1941.

Clatsop Spit continued to advance into the channel at its northwest extremity but the rate of advance gradually lessened and since 1947 the bulge to the northwest has remained in approximately the same location. Greater than project dimensions have been maintained round Clatsop Spit by the tidal and river currents, but the alignment of the channel is objectionable. Starting in 1939, material was dredged from the northwest portion of Clatsop Spit to better the ship channel alignment.

In 1885 the outer face of the bar was approximately 3 miles west of Cape Disappointment. Construction of the jetties forced the bar further into the ocean so that by 1900 the outer face had advanced 1 mile, and by 1950 the extreme western position of the sea face of the bar was over 5 miles west of Cape Disappointment. As the outer perimeter of the bar advances into the ocean the original depth of water increases and the rate of advance becomes less. Fig. 12 shows the advance seaward of the 40-ft. depth contour since 1885.

CHAPTER 33

HISTORY OF NEW JERSEY COASTLINE

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INTRODUCTION

The New Jersey coast probably is the most important recreational asset in the nation. This is due in part to the nearby densely populated metropolitan areas that experience unpleasantly hot and humid weather during the summer months. New York and its satellite communities, having a combined population of approximately 13 million, is only 50 miles from the nearest and 160 miles from the most remote of the 57 resort towns that dot the 125-mile length of New Jersey seashore. The Philadelphia metropolitan area, with a population of approximately 4 million, lies 60 miles from the nearest resort and only 86 miles from the farthest. Fig. 1 shows the geographic setting of the seashore area.

But it is not merely geographic proximity to large numbers of people and the compulsion of uncomfortable weather at home that attracts 4 million vacationers and a great many one-day excursionists to the New Jersey seashore resorts each year. Nearly all of the 125 miles of shoreline is a satisfactory sandy bathing beach, and about 80% of it is open to the public at no charge. The ocean is not polluted, its temperature is approximately 70° throughout the summer months, and its surf is not dangerous. The 57 resort communities collectively offer a great variety of accommodations ranging from luxurious hotels to modest boarding houses and tourist camps, and the surroundings include highly developed areas, as at Atlantic City, as well as localities remaining in a natural condition.

The development of this shoreline as a recreational resource began nearly two hundred years ago, at Cape May. By 1801, there was evidently at least one establishment there that was large enough to be considered a hotel. The following advertisement appeared that year in a Philadelphia newspaper:

> "The public are respectfully informed that the subscriber has prepared himself for entertaining company who use sea bathing, and he is accommodated with extensive houseroom, with fish, oysters, crabs and good liquors. Care will be taken of gentlemen's horses.
>
> "The situation is beautiful, just at the confluence of Delaware Bay with the Ocean, in sight of the Lighthouse and affords a view of shipping which enters and leaves the Delaware; Carriages may be driven along the margin of the ocean for miles, and the wheels will scarcely make an impression upon the sand; the slope of the shore is so regular that persons may wade a great distance. It is the most delightful spot the citizens can retire to in the hot season.
>
> "A stage starts from Cooper's Ferry on Thursday in every week, and arrives at Cape Island on Friday; it starts from Cape Island on Friday and Tuesday in each week, and arrives in Philadelphia the following day."

A large and fashionable clientele was built up by 1855, many coming from the southern states. The accommodations available included a huge structure, even by 1950 standards. It contained 482 rooms, each with a private bath, and a dining room seating 3,000.

Long Branch and Atlantic City began to attract visitors in 1819 and 1854 respectively, and the smaller resorts gradually came into being. Fewer work hours for most people, budgets permitting larger expenditures for recreation, and the family automobile inevitably resulted in the growth of these recreational communities along the seashore to the extent that now it is estimated that 800,000 visitors can be housed at one time. At many of the resorts, boardwalk promenades are located close to the beach fronts. Along the landward side of these popular

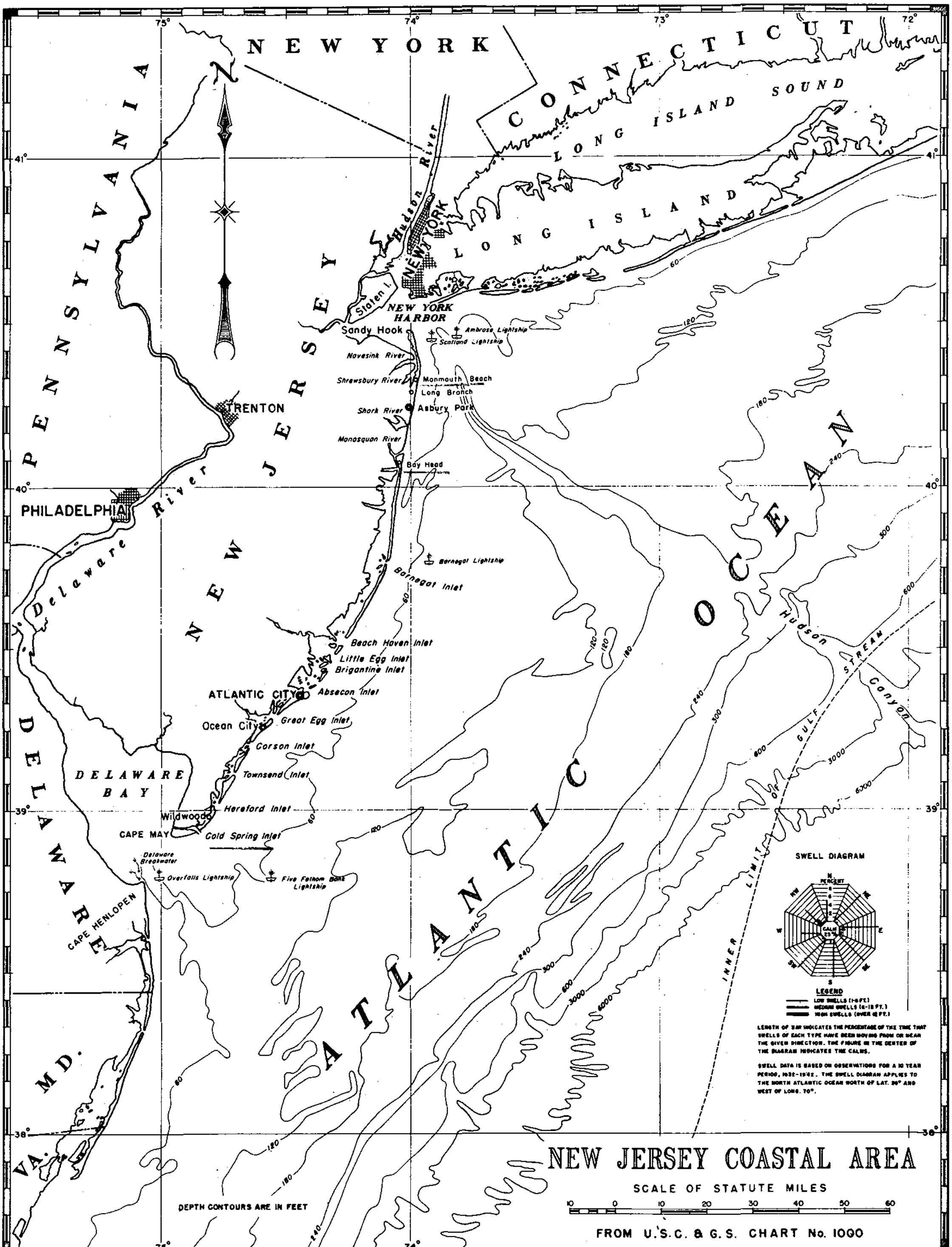

NEW YORK
CONNECTICUT
LONG ISLAND SOUND
LONG ISLAND
PENNSYLVANIA
NEW JERSEY
Hudson River
NEW YORK
Staten I.
NEW YORK HARBOR
Sandy Hook
Ambrose Lightship
Scotland Lightship
Navesink River
Shrewsbury River
Monmouth Beach
Long Branch
Shark River
Asbury Park
Manasquan River
Bay Head
TRENTON
Delaware River
PHILADELPHIA
Barnegat Lightship
Barnegat Inlet
Beach Haven Inlet
Little Egg Inlet
Brigantine Inlet
ATLANTIC CITY
Absecon Inlet
Ocean City
Great Egg Inlet
Corson Inlet
Townsend Inlet
Hereford Inlet
Wildwood
CAPE MAY
Cold Spring Inlet
DELAWARE BAY
DELAWARE
Delaware Breakwater
Overfalls Lightship
Five Fathom Bank Lightship
CAPE HENLOPEN
MD.
VA.
ATLANTIC OCEAN
Hudson Canyon
GULF STREAM
INNER LIMIT OF
SWELL DIAGRAM
LEGEND
NEW JERSEY COASTAL AREA
SCALE OF STATUTE MILES
10 0 10 20 30 40 50 60
FROM U.S.C. & G.S. CHART No. 1000
DEPTH CONTOURS ARE IN FEET

Fig. 1

features are located shops, restaurants and amusement places presenting a varied array of attractions. The larger resorts have piers extending out over the ocean, featuring convention halls, commercial exhibits, theaters, concert halls, and other attractions. Other piers provide facilities for sport fishing.

The social and economic significance of this unique seashore recreational area is enormous. It has become part of the pattern of life for thousands of people living as far as 400 miles from its center of gravity, Atlantic City. Families look forward to their vacations at the seashore as the bright spot of the year, and to their brief, week-end sojourns there as respites from their daily routine. The seashore provides the facility that translates their available leisure time and their financial latitude for expenditures beyond bare necessities to wholesome recreation. Likewise, it provides a livelihood for a great many people who furnish the facilities and cater to the needs and desires of the visitors. Their business has been valued at one billion dollars annually, making it the largest in the State of New Jersey.

Unfortunately, nature does not recognize the social and economic need for beach stability along the New Jersey shoreline. Changes are occurring presently that threaten the very existence of some of the resorts by destruction of their beaches and their shore front developments. In many cases, it is a matter of record that the shoreline location has varied over a wide band, including the present heart of the community, prior to its having reached the present state of development. These facts were forgotten in the excitement of building for ever-increasing numbers of visitors, and the beach line at the moment was accepted as the point where it was expected to remain. Minor variations occurring subsequent to such optimistic appraisals greatly reduce the beach areas available for bathing and sunning; overcrowding results, and soon the city fathers either are driven to taking steps to restore the beach or accepting the inevitable loss of patronage.

Some of the resorts for years have been fighting a losing battle with erosion, or merely worrying over the condition, while others have enjoyed some measure of success in their efforts to maintain reasonably stable beaches. It is the purpose of this paper to appraise the overall, general situation, and certain specific problem areas.

GEOLOGY

The 125 miles of ocean shoreline in New Jersey represents three geomorphic types: A sand spit about 5 miles in length at the northern extremity; 24 miles of headland beach; and 96 miles of barrier beach. The sand spit at the north end of the State, known as Sandy Hook, projects northward and westward into Lower New York Bay in continuation of a barrier beach which extends northward about 5 miles from the headland at Monmouth Beach. This spit cuts off the direct entrance of Shrewsbury River and Navesink River into the ocean. They enter the ocean to the north, back of Sandy Hook.

Southward from Monmouth Beach to Bay Head, a distance of approximately 19 miles, the upland terrain, with elevations at 15 ft. to 25 ft. above sea level, extends oceanward to the general line of the beaches where it terminates in an abrupt drop to the ocean strand, the width of which generally is about 50 ft. As the southern limit of the upland frontage is approached from the north, the terrain elevations decrease and the width of strand increases to about 200 ft. Emerging through the upland frontage, Shark River and Manasquan River enter the ocean at distances of 20 and 26 miles, respectively, south of Sandy Hook. Both of these rivers have been improved for navigation and their channel entrances are now protected by jetties. Several other water courses that once drained the upland now exist as lakes with their outlets to the ocean controlled by works installed for the purpose of regulating the levels of the impounded waters.

From Bay Head southward to Cold Spring Inlet, a distance of approximately 91 miles, the formation is a barrier beach lying from 2 to 5 miles off the mainland and varying in width from 550 ft. to 1 mile. Sand dunes ranging from a few feet to as much as 30 ft. in height are found along a few sections of this reach. At Cape May the mainland extends to the ocean front for a short distance.

The continuity of the barrier beach is broken by ten inlets which afford tidal connection between the ocean and the bays and sounds lying between the beach and the mainland. Three of these inlets have been improved for navigation; at Barnegat and Cold Spring, jetties have been built, while the third, Absecon Inlet, is improved only in the sense that the entrance channel is periodically dredged.

Since the earliest recorded observations there have been numerous inlets along the New Jersey coast, all of which have been generally unstable as to both location and cross-sectional area. A number of inlets which once existed have closed, leaving no physical trace of their existence. The general trend of the inlets through the barrier beach south of Bay Head is to migrate southward. Records indicate that some have shifted as much as 1 mile. From Absecon Inlet south, the southward migration of the inlets is generally accompanied by a seaward building of south shore. This has resulted in the south side of the inlets off-setting the north side; at Absecon Inlet, the south side extends a mile farther seaward than the north.

The formations along the New Jersey coast are of the cretaceous and more modern ages, and are composed of practically level unconsolidated strata of gravel, sand, silt and clay. They are flat sheets which slope gently to the southeast, extending out under the ocean, and presumably crop out on its bottom at the edge of the continental shelf. The total thickness of the strata is great near the shoreline. A well driven at Atlantic City to a depth of 2,305 ft. did not reach the ancient surface of hard rock underlying the oldest deposits.

The offshore hydrography is not complex. The formation resembles a plateau extending seaward 80 to 90 miles from the shoreline and gently sloping to 300 ft. below sea level in this distance. Beyond here, the depths increase in a precipitous manner, attaining 6,000 ft. approximately 100 miles from the shore. In general this description applies to the area extending from Cape Cod to the Virginia Capes, the outer "bluff" line curving to generally parallel the Long Island-New Jersey-Delmarva Peninsula shoreline. The plateau is cut by several submarine canyons, however all but one are so insignificant on this enormous stage as to have little bearing on shoreline processes in New Jersey. The exception to this generalization is the Hudson Canyon, which dissects the plateau in a northwest-southeast direction beginning at the entrance to New York Bay. It is as much as 100 ft. deeper than the adjacent plateau in its inner reaches, and near its outer extremity it plunges to depths 2,000 to 3,000 ft. below the surrounding ocean floor. It appears reasonable to suppose that so mighty a gash as this plays a significant part in the processes operating along the New Jersey shoreline.

As stated previously, the shoreline and presumably the offshore plateau is devoid of rock formations for great depths below the surface. Likewise, the New Jersey hinterland southeast of a line extending between Trenton and Staten Island (the Fall Line) almost entirely consists of unconsolidated sediments extending to great depths. This area, part of the Coastal Plain, is seen to include all of the ocean shoreline. The deposits are largely the result of erosion of the upland now included in the Appalachian Mountain terrain, which once was much higher than it is presently. Some of the material is marine in origin, as it is commonly accepted that the Coastal Plain has experienced several cycles of uplift and submergence; and a smaller portion of the sedimentary deposits were derived from the glacial moraines, the southernmost of which is found just north of the Fall Line. The latter class of material evidently is found mostly in portions of the Plain adjacent to the Delaware estuary.

It appears that the material composing the New Jersey ocean beaches is derived from the deposits found in the portions of the Coastal Plain adjacent to the beaches. Neither the beaches nor that part of the Plain contain the minerals associated with the morainal deposits, which were derived from the ancient crystalline rocks of New England. On the other hand, both contain cretaceous and tertiary sediments with paralleling mineral compositions. The predominant mineral is quartz.

The particle size on the beach as reflected by various criteria increases rather progressively from Sandy Hook to Manasquan River, then decreased from the latter point to Cape May. The median diameters at these places are 0.37 mm, 0.78mm respectively, and the percentages held on the number 28 sieve are approximately 30, 73 and zero.

THE SHORELINE REGIMEN

General description of shoreline changes. The earliest reliable survey of the New Jersey coast is that made by the Coast Survey in 1839-42. In the more than a century of record accumulated since then, it is found that the net change represents a loss of beach, but the differences along the shore between the earliest and the latest surveys of record are very irregular. In some places, there have been extensive accretions, in others the beach has been essentially stable, and at some localities the losses have been great. The record also shows that the beach frontage does not change between surveys in the same way in all its parts, nor is the rate or even the direction of change constant at any given point during the period of record. For example, the northern extremity lost heavily between 1839 and 1873 while the middle section and parts of the southern generally were accreting. There are instances of erosion and accretion occurring simultaneously in adjoining sections of a unit barrier beach (i.e., one lying between two inlets) during the period between two record surveys, followed by the diametric opposite in the next period for which comparisons of shoreline locations are possible. Where there was erosion in one section between two surveys, there is accretion between the next two surveys; in the neighboring section, gains were experienced in the first period and losses in the second.

Thus, while the overall result in the more than a century of record has been erosion, checking with the geologists' concept that nature is working to extend the continental shelf at the expense of the land mass, it appears that in the process an erratic or perhaps a cyclic shifting of masses of material takes place on the beach. There is also a possibility that the pattern of net losses to the sea is not a steady or even unidirectional process.

There is reason to believe that the present pattern of shoreline processes in New Jersey has existed for a long time, perhaps throughout the entire period during which geologic conditions similar to those that now obtain have prevailed. That it has been erosive in the net is evidenced by the existence of ancient marsh formations on the sea face of eroding barrier beaches at several points along the coast. Such formations were most certainly not laid down in the open sea; they must have been established during a period when the areas they occupy were protected by barriers much farther seaward than the present beaches. It is also known that inlets have closed and reopened prior to the earliest reliable survey. This indicates that the variableness that is characteristic of the present shoreline changes must have existed prior to 1839 also. Inlets exist as a result of a balance between the supply of materials moving along the adjacent beaches and the inflow and outflow of their tidal prisms. An increase in the supply may choke the entrance with eventual complete closure; a decreased supply may result in such serious erosion that a storm will reopen an inlet that can cope with the existing littoral drift.

Principal factors. The New Jersey shoreline regimen is a function of the supply of New Jersey beach material, and the waves, currents, tides and winds that shift it about, and the inlets, jetties, groins and other features that affect the processes.

Supply of beach-building material. It has already been stated that materials typical of morainal deposits are not present on the New Jersey beaches. As such minerals are present on the Long Island beaches, it is probable that these beaches do not constitute a source supply of material for the New Jersey beaches. The Hudson River and its deep canyon doubtless are effective barriers to such a movement, and the direction of drift in northern New Jersey is towards, not away from New York Bay, as evidenced by Sandy Hook. Likewise, the New Jersey beaches are effectively isolated from the Delaware beaches by the Delaware estuary, which debouches into the Atlantic through an entrance 12 miles in width. While it spreads its currents fanwise before its entrance, the materials discharged by the Delaware evidently are not carried eastward up the Cape May Ocean frontage. Groins and jetties here do not trap material on their westward sides, and the beach material is quite different from the sands composing the bottom of Delaware Bay.

The beaches receive little nourishment from stream detritus. Excluding drainage to New York Bay and Delaware Bay, the area discharging into the Atlantic or its coastal bays and sounds amounts to only 1,700 square miles approximately; of this total, only 121 square miles are drained directly into the ocean. The approximately 40-in. annual rainfall is well-distributed throughout the year, the terrain does not exceed 100 ft. in altitude, and there are numerous swampy areas. As a result, the water-courses are sluggish and carry little sediment. It is emphasized that 93% of the coastal drainage is discharged into the bays and sounds. Even if the streams carried much detritus, it would be trapped in these bodies of water and not become available for beach nourishment.

Thus, the New Jersey beaches probably receive no material from the shorelines to the north and south of the state, and little or no contribution from the New Jersey hinterland. Also, material brought down by the Hudson and the Delaware is not made available to nourish the beaches. It follows that the New Jersey ocean beaches as an entity have no source of supply of beach-building material unless the ocean floor itself constitutes a source. In the face of the record surveys obtained at intervals since 1839 indicating a net loss of beach, it must be concluded that the ocean does not supply as much material as it receives, and consequently does not qualify as a source of supply.

It is clear, if this concept is sound, that sand in littoral transport on the New Jersey beaches consists of a redistribution of the volume on the beaches. In the process, the sea exacts a charge for its services as the principal transporting medium, as does the wind for its work, with the result that there is generally a smaller volume moving along the beach than was eroded from some updrift location.

It is considered that beaches like those along the New Jersey coast are rarely static, although the shoreline may not be changing in a given period of time. A stable beach probably is experiencing an exchange of sand, the supply equalling the losses. Where a beach is accreting, the supply is in excess of the quantity carried away, and conversely, an eroding beach is losing more sand than is fed to it. The forces that effect this movement of sand to and fro are the waves, tides, currents, and winds. They may, and frequently do, act in concert, to produce the observed result.

Waves. The waves are doubtless the principal tool in nature's hands for molding and remolding the sands that comprise these New Jersey beaches. In general, it may be said that their action is less violent on this coast, where the sea floor slopes gently for a considerable distance, than in localities where greater depths exist close to the shore.

Observations by the Beach Erosion Board at Long Branch during the 20-month period extending from April 1948 to October 1949, reveal that 71% of all the waves are two feet in height, or lower. 94% are four feet in height or lower, and 98% are six feet or lower. The highest wave observed was about 12 feet; waves in this range occurred during only 1/10 of one percent of the observation period, or about 12 hours.

Observations by the Philadelphia District, Corps of Engineers, at Atlantic City during a 19-month period in 1935-1937 disclosed that 65% of the waves there are two feet in height, or lower, 90% four feet or lower, and 97% six feet or lower. The greatest wave observed measured about 13 ft. from crest to trough. It is apparent that the wave experience at Atlantic City is similar to that at Long Branch; these data, supplemented by less extensive observations at Ocean City, Wildwood, and Cape May, lead to the conclusion that in general the waves all along the New Jersey coast are generally similar in their height characteristics. This is probably true only for fairly long periods of observations. The waves on a given day may be quite different at one locality than those occurring at another.

Methodical observations of wave directions are available only for Atlantic City, Ocean City, Wildwood and Cape May. The data show that waves approach these beaches from directions north of normal to the general set of the shoreline much more often than from points south of the normal. Considering the swell data shown on Fig. 1 and the configuration of the shoreline, this condition is to be expected. Swells from the northeast, east, and southeast, accounting for 43% of all the observations north of latitude 39° and west of longitude 70° approach New Jersey south of Barnegat Inlet in such directions that their impingement on the beach are from directions north of the normal lines.

It may be inferred that the northeast swell is considerably reduced in significance in northern New Jersey due to the lee provided by Long Island, also that the southeast swell is likely to impinge on the beach at an angle to the south of normal to the northern New Jersey shoreline. The pronounced change in shoreline orientation north of Barnegat Inlet is clearly shown on Fig. 1. Accordingly, it is inferred, with support from casual observations, that the most common direction of wave approach at points along the shore north of Barnegat Inlet is south of normal.

Currents. It is generally believed that waves approaching a beach at an angle generate a longshore current away from the angularity. Thus, the more oblique approaches produce the higher longshore, or littoral, current velocities. Waves hitting a beach normal to its alignment would not produce a littoral current under this concept. Applying this to the New Jersey coast, it is seen that a wave-induced littoral current flowing towards New York Harbor is likely to be encountered north of Barnegat Inlet, and that a similar current setting towards Delaware Bay should exist south of Barnegat Inlet. Direct observations covering a considerable period are available at Atlantic City, where a recording current velocity and direction apparatus was maintained in operation for about two years. The data obtained showed the existence of a non-tidal current generally flowing towards Cape May. This current attained a velocity of nearly three miles per hour on one occasion.

Other currents existing in the locality that are, or may be, factors in the changes occurring on the New Jersey beaches include those generated by the discharges and inflows of New York Harbor and Delaware Bay, also the coastal inlets and tidal entrances; oceanic currents other than tidal; and tidal oceanic currents.

For a distance of at least a mile south of the tip of Sandy Hook, the normal ebb flow from New York Harbor generates a current of nearly 2 miles per hour velocity generally paralleling the ocean shore and setting away from the Harbor. During the normal inflow to New York Harbor, the current along Sandy Hook attains approximately the same velocity as is found during the ebb, but flows toward the Harbor. Information as to the distance south of the tip of Sandy Hook that a reversing current is experienced is not available, but it seems reasonable to expect that currents as strong as those encountered a mile from the tip would not disappear in less than five miles. At the entrance to Delaware Bay, reversing currents have been found as far as Wildwood, seven miles above the tip of Cape May, and at a point sixteen miles south of Cape Henlopen on the opposite shore of the bay. These currents had strengths of about 1.5 miles per hour, increasing to about 2.5 miles per hour closer to the entrance. It is important to bear in mind that these are normal velocities; during times when extraordinary ranges of tide are occurring, the tidal prisms of estuaries are proportionately larger, as are the current velocities throughout the current pattern. Without doubt, the Delaware estuary and the New York Harbor tidal complex are significant factors in the New Jersey ocean shoreline regimens within their respective zones of influence.

The ten tidal inlets and the two tidal rivers encountered along the shoreline are minor estuaries with effects on currents similar to those at New York Harbor and Delaware Bay just discussed, but obviously their areas of influence are much smaller. As an example, a carefully analyzed long series of observations at a point less than a mile from Absecon Inlet revealed a very feeble reversing current -- so feeble, in fact, that there are grounds for skepticism that such a current exists. Current velocities in the entrances, and through their typical bar channels, are on the other hand frequently quite strong. At Barnegat Inlet, currents having a velocity of nearly 4 miles per hour have been observed. Such currents without question are very important factors in shoreline development along the inlet shores.

While the general statement is true that the 12 coastal tidal entrances do not generate currents over large zones, it is important to remember that their bar channels sometimes are found close to the downdrift ocean beach. When this condition exists, the ocean shoreline may be subjected to swift main-thread inlet currents for as much as a half-mile. It is usually found that this section of shoreline is subject to violent changes, indicating clearly that the inlet currents are the principal factor in the regimen of the locality.

The non-tidal oceanic currents, other than those generated by waves, include the northeastward-flowing Gulf Stream, the inner edge of which is 140 miles off Atlantic City and 175 miles off Sandy Hook, a slow counter-current extending from the 20 fathom contour nearly out to the Gulf Stream, and wind generated currents. The Gulf Stream seems too remote from the shoreline to be a factor; the counter-current is extremely feeble, and also can be discounted; only the wind-generated currents merit further discussion.

From a long series of observations made by the U.S.C. & G.S. at the five lightships near the New Jersey Coast, it is found that the current velocity in knots generated by wind is 1.3% of the wind velocity in miles per hour. A 50-mile per hour wind velocity evidently would cause a 0.65 knot current (1.1 ft. per sec.). The data also show that the current would set about 13 degrees to the right of the direction towards which the wind is blowing. Currents of this order of magnitude acting alone would be of little significance in the New Jersey beach regimen. However, acting in concert with the waves and wave generated currents, their significance becomes real. It is generally accepted that detritus possessing given characteristics will begin to move when subjected to a certain current velocity. Under some conditions, that velocity will not be attained by wave-induced currents alone, but when these currents are reinforced by the wind-driven current, the marginal velocity may be reached and sand movement will begin.

Ocean tidal currents (the so-called rotary current) exist off the New Jersey coast but doubtless have no significance in its shoreline processes. The velocity at strength of flow is reported to be about 0.5 ft. per sec. at the Lightships; analyses by the Beach Erosion Board of current observations off Long Branch, and by the Philadelphia District off Atlantic City resulted in much lower values closer to the shore.

Tide. The tide in the locality is of the semi-diurnal type; that is, morning and afternoon tides resemble each other closely. The tide-defining data are tabulated below:

		Time of Tide*	
	Mean Range	H.W.	L.W.
Sandy Hook	4.6 ft.	12.74 hrs.	6.68 hrs.
Atlantic City	4.1 "	12.24 "	6.12 "
Delaware Breakwater	4.2 "	13.45 "	7.13 "

*Referred to upper and lower transits of the moon at Greenwich.

Mean ranges are often greatly exceeded; meteorological disturbances have raised the ocean to as much as 5.4 ft. above mean high water and lowered it to 3.5 ft. below mean low water at Atlantic City and presumably elsewhere along the New Jersey coast. The extreme high stages are usually accompanied by strong winds and great waves. Due to the increased depth of water over the shallow off-shore areas, the waves deliver more energy at their impact on the beach, and affect portions thereof normally free from the effect of waves.

Another characteristic of the tides that possibly is of significance in considering their relationship to shoreline changes is the variation in mean range that occurs over a 19-year period due to astronomical factors. At Atlantic City, the range of tide in 1933, the low point of the cycle, was 3.9 ft.; in 1940, the peak of the cycle, the range was 4.3 ft. A variation of only 0.4 ft. might seem inconsequential, but on the gently sloping beaches that characterize most of the New Jersey shoreline, the mean high water line would vary from eight to ten feet without any change in absolute elevations of the beach. It must also be borne in mind that every factor in shoreline processes, excepting the direct effect of the wind, would vary somewhat in effectiveness in consonance with the variation in tide. In the case of a beach in a delicate state of equilibrium during the low point of the 19-year cycle, it is conceivable that the high water mark would be advanced shoreward at the high point of the cycle far more than the 8 or 10 ft. referred to above.

While variations in sea level are not due to astronomical factors, it is germane to the general subject of tides to refer to the rising elevations that have been noticed in recent years. Whether this is due to a subsidence of the land mass or a gain in the volume of water present in the ocean is not important in these considerations; the bare fact that sea level relative to the land is now about 0.5 ft. higher than it was in 1925 is important, however. This change in sea level has moved both the mean high water and the mean low water lines landward about 20 ft. on a 1 on 40 sloping beach without the assistance of any accompanying erosion. Obviously, such a change increases the effectiveness, even though possibly very little, of the factors in shoreline processes. Again, a small increase in their strength may be sufficient to produce a significant change in the overall regimen.

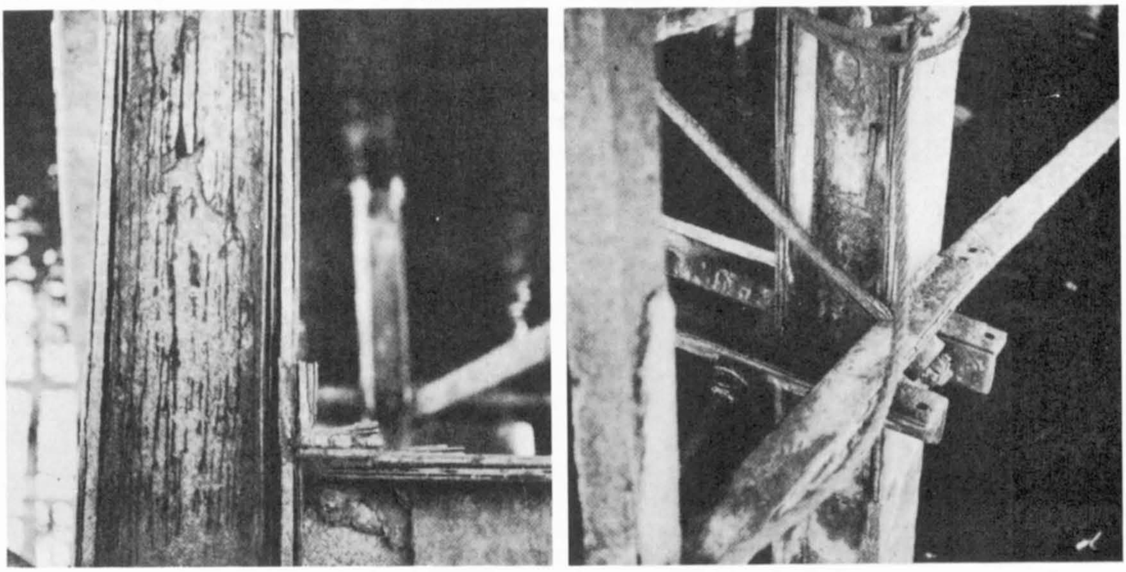
Inlets. The existence of ten inlets and two tidal rivers on the ocean shoreline of New Jersey has been mentioned previously. These tidal entrances are very important factors in the shoreline regimen, much more so than would be their role if they were mere generators of coastal currents. Some students of sandy coasts conceive them to be barriers to the orderly movement of the littoral drift. They divert part of the drift far seaward, perhaps beyond recovery, carry another part into their interior lagoon system, and temporarily store the remainder in outer bar formations.

As the bar develops in the direction of the littoral drift, springing out from the windward beach, the channel is forced towards the leeward beach, which then suffers erosion unless it is strongly protected. Unstabilized inlets migrate in this manner, and numerous fine examples of such movements exist in the history of the New Jersey coast, the best being the shift of Barnegat Inlet to the south amounting to a mile in about 100 years. Finally, the bar channel is forced into so unfavorable a position that a natural tendency exists for it to seek a more direct route to the ocean. With the help of a storm, or perhaps an unusually powerful ebb discharge, the bar is breeched, and a deep channel develops with the consequent rapid deterioration of the former location of the channel close to the beach. A section of the bar, perhaps containing hundreds of thousands of cubic yards of sand, then is detached from the main stem and is no longer barred from the leeward beach by swift inlet currents. The material thus freed gradually moves to that beach, repairing the erosion caused by the inlet channel when it was close at hand, and in some recorded instances producing a very considerable accretion. The beach immediately south of Barnegat Inlet was very lean in 1932, so much so that stabilization measures taken by the State a few years earlier to save the historic lighthouse there were threatened with an outflanking maneuver in the inlet's effort to resume its migration southward. The channel had been forced far towards the leeward beach, and an extensive bar formation was dangling like a ripe fruit. By 1937, it was clear that the inlet and its vicinity had passed through a particularly interesting phase of its development. The channel had shifted to the east, and the formerly lean leeward beach had experienced an accretion amounting to several hundred feet. Such events doubtless have occurred at the other inlets, fairly certainly at Absecon Inlet and Great Egg Inlet. Close study of the recorded changes in the ocean shoreline of the State reveals that the greatest erosion or accretion has occurred adjacent to the inlets and tidal rivers.

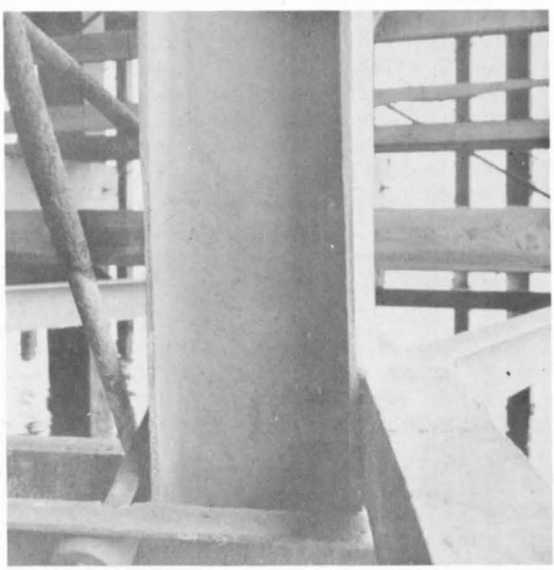
Shark and Manasquan Rivers, also Barnegat and Cold Spring Inlets, presently have jettied entrances. Absecon Inlet has a strongly protected shoreline on its down drift side, and an entrance channel 20 ft. in depth that is maintained by dredging; without maintenance, this channel would shoal to about 8 ft. over the bar.

The south jetties at Shark River and Manasquan River accumulated fillets of beach material extending to their outer ends years ago, and it is believed that the littoral drift is now by-passing these entrances, in part naturally and in part as a result of the direct deposit on the north beaches of material dredged from the channels. The north beaches suffered erosion in the early years of the existence of the jetties, but they are apparently relatively stable now. The Barnegat Inlet jetties have not had a well-defined effect on the adjacent beaches as yet, although they have been in existence for ten years. Apparently, there is a tendency at this inlet for fillets to accumulate north of the north jetty and south of the south, although it is considered certain that the long-term prevailing direction of littoral drift is from north to south.

Fig. 4. STRESS CORROSION



\- A -



\- B -   - C -

-A- Typical Corrosion in vicinity of restraint showing laminations of rust. In service 16 years. No coating protection.

-B- Typical Corrosion in vicinity of restraint with rust removed. Note loss of section on the flange. In service 16 years.

-C- Note absence of above conditions where steel was coated with Asphalt Chromate Emulsion. In service 21 years.

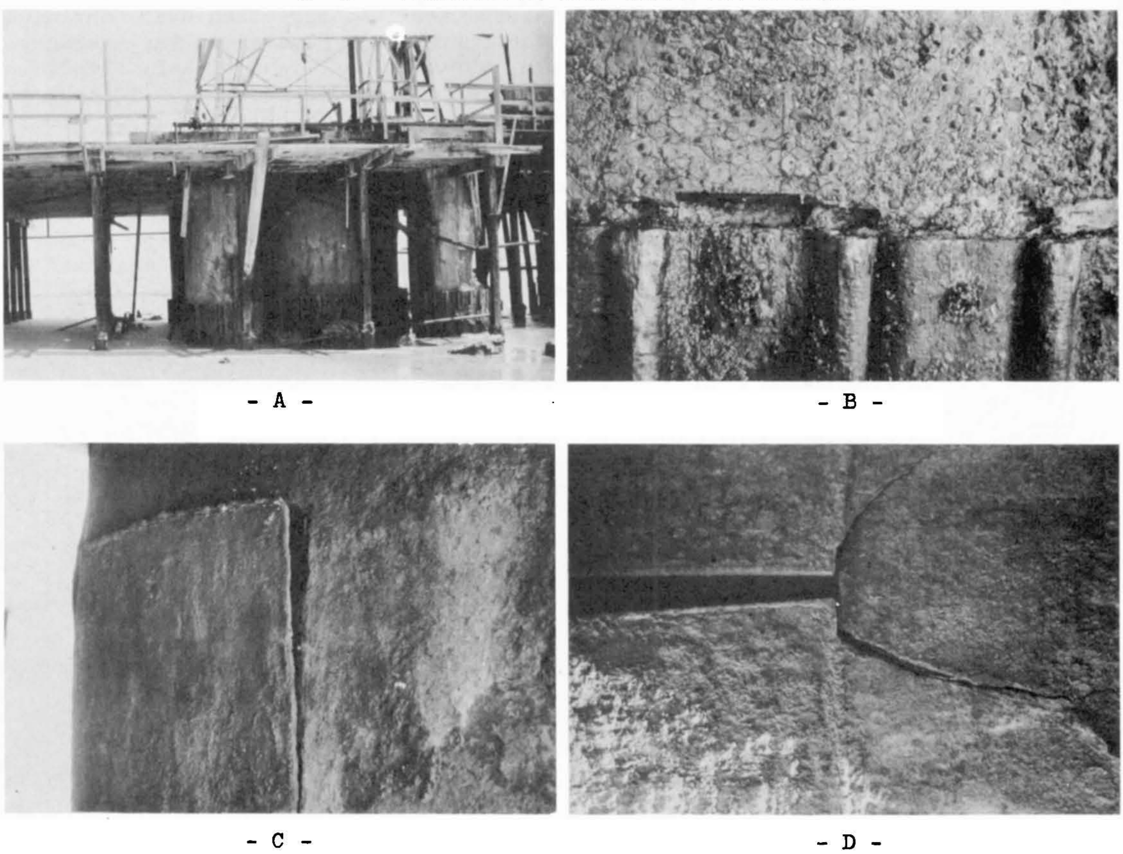
<u>Bracing.</u> Bracing should be of a minimum, located above normal wave heights and in areas where kelp is prevalent, consideration should be given to kelp loading. The section, where possible, should be round to reduce the drag coefficient of waves and the ease of re-coating. The principal objection to round section is fabrication.

Timber. The subject of wood preservation is a science by itself and all I can relate here are the precautions taken on our works to try to insure our superstructure and the results obtained. Again, the choice of untreated timber over treated lumber, even for fifteen years, is a matter of environment. All lumber was rough structural grade. All bearings were mopped with hot creosote. All laps were mopped with hot creosote. All lumber was spaced or gauged insofar as possible to provide free air circulation. Deck planks were gauged 3/4 in. All nails, drifts, bolts, etc., were galvanized.

After twenty years, aside from a few planks, the original deck and the original joists remain; some few caps, possibly sixteen percent, have been reinforced due to heart rot from water entry through checks.

By all means, provide as much air circulation as possible; keep the top surfaces of caps cleaned of all earth or wood fibre that might sift through the deck spacing and form a damp mat and induce decay from the top. When fungi or dry rot have taken over and auxiliary members are required, remove the ailing timber. The presence of such timber is a source of fungi spore and it is no longer a structural component of the work.

Fig. 5. CONCRETE IN SALT WATER ENVIRONMENT



- A - - B - - C - - D -

-A- Mass Concrete in Wave Area.

-B- Vertical Rise from Sheet Pile Forms.

-C- Mass Concrete in 1/4 in. Steel Shell above high tide line. Note weld failures.

-D- Mass Concrete in 1/4 in. Steel Shell above high tide line. Note shell failure.

some of the structures were of no value as protective devices but their removal was not considered justifiable or expedient. In the case of others, it is conceivable that the structures were initially successful but that complacency, budget problems, and short memories combined to prevent routine maintenance until the structures had lost their effectiveness.

If the concept that the sandy beaches of New Jersey are not static is sound, then it follows that a stable beach exists only where the rate of movement of sand away from that beach is balanced by the rate of arrival of other sand there. Thus, if works can be designed to reduce sufficiently the rate of loss at a beach where an unsatisfactory balance exists, erosion will be supplanted by stability. Obviously, such works will not <u>preserve</u> a beach when there is no supply of sand, unless they have succeeded in reducing the loss rate to zero, an unattainable goal. Even more obviously, they will not <u>build</u> a beach when no beach-building materials are furnished.

This simple, rational concept is widely accepted; yet, when beach protective works are being considered, it is sometimes either forgotten, or it is assumed that an adequate supply of material is reaching the problem beach when in fact this assumption is unsound. The alternative in such circumstances would consist of increasing the supply rate artificially. When the net loss rate is low, the economical solution to the problem might be this measure alone; beach protective works, which are always unsightly, might not earn their way. When the net loss rate is high and the cost of an artificial increase in the supply rate is great, it might be sound to include beach protective works in the project. Another important reason for careful consideration of the artificial nourishment type of project for the eroding beaches lies in the great variations in the supply volume. Artificial nourishment of a beach having such a history would carry it through an erosive cycle perhaps far more economically and certainly more successfully than a project for beach protective works alone. These principles will be discussed further in the presentation of the four outstanding problem areas that exist now.

<u>Long Branch and Vicinity.</u> Long Branch occupies the northerly portion of a 19-mile section of mainland ocean frontage. The littoral drift is northward, as evidenced by the accumulations at the numerous groins, but it is of negligible volume at Long Branch presently. This is due in part to the vigorous efforts of communities to the south to retard their erosion, thereby reducing the supply for Long Branch, and in part, in all probability, to a naturally small supply throughout this section of shoreline. The resorts to the south appear to be nearer to the kitchen, so to speak, and they seek as great a portion of the available supply as their groins can retain. The shoreline north of Manasquan River is entirely utilized, and there are few sections free of groins. It may be concluded that the supply, after running the gamut of so many structures designed to trap it, has become quite thin before the needs of Long Branch and its neighbors to the north can be met.

In addition to loss of beach, the locality in recent years has suffered loss of slices of headlands, which are composed of unconsolidated sediments. The headlands are occupied by pretentious mansions, and real estate values are extremely high. The problem has been attacked by the construction of strong bulkheads, fronted by heavy revetments of stone at their toes, together with high groins. Some of the groins have offshore breakwaters extending upcoast and downcoast perpendicular to the groin, making a structure resembling a capital T. The bulkheads have been successful in eliminating further erosion of the shoreline, but the groins, as may be expected, have not built much beach to serve either for bathing or as a buffer to protect the bulkhead. Where the T groins were built, the offshore breakwater serves the latter purpose, and there have been small accumulations of sand adjacent to their stems, paradoxically to a greater extent on the downdrift than on the updrift sides. Survey data are not available on which to premise an explanation of the results of these T groins, but it is reasoned that the accumulations are either the result of a redistribution of the ocean bottom enclosed by the adjacent Ts, or is material scoured from the bottom at the sea face of the offshore portions and carried through the gaps left between adjacent Ts. The sand that is thus trapped is native to the immediate locality. Its mineral composition includes much glauconite, giving it a darker appearance than the sands on the beaches to the south and the north of the T groins. Evidently, the sand trapped by the Ts is not the material in general littoral transport in the locality.

The Beach Erosion Board, in cooperation with the New York District, has recently completed a study of the use of dredged material deposited offshore by a hopper dredge to nourish the beaches of Long Branch and the resorts to the north. The material, removed from New York Harbor entrance channels, was placed during the summer of 1948 in a ridge about 1/2 mile from shore in 38 ft. of water. The volume deposited amounted to approximately 600,000 cubic yards. Careful surveys were made of the beach and offshore areas before, during, and after the dumping, the last survey being made 14 months after the last load was deposited. Tide and wave observations were made continuously throughout the experiment.

The data show that the shoreline receded during the study. In the last 12 months of observation, three of the four sections into which the beach was divided for convenient reference eroded 14 ft.; the fourth section, nearest to the stockpile, receded 43 ft. The area shoreward of the 18-ft. depth contour lost 222,000 cubic yards of material during the 12-month period, while the stockpile gained 39,000 cubic yards. The area seaward of the 18-ft. contour, excluding the stockpile, gained 182,000 cubic yards, indicating that the entire study area experienced a very close balance of gain and loss. The conclusion is inescapable that the offshore stockpile did not nourish the beach. It could not have been placed closer to the beach without endangering the dredge.

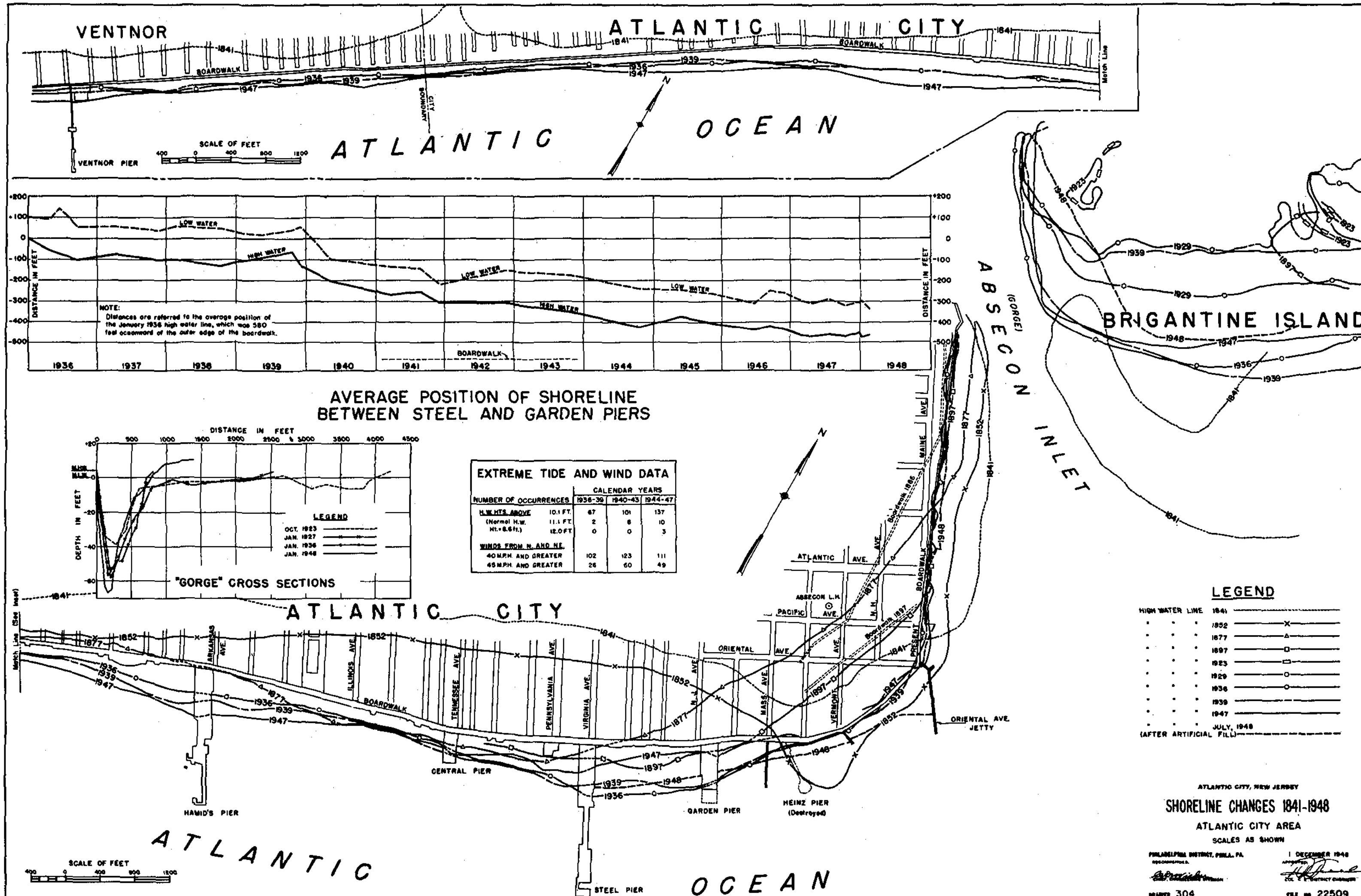
As stated previously this section of the New Jersey coast evidently does not enjoy an adequate littoral drift volume. Beach protective works cannot be expected to stabilize or build beaches without benefit of such a supply. Offshore deposits do not appear to solve the problem, leaving no alternative but the artificial placement of sand directly along the shore if an adequate bathing and protective beach is to exist there. This work could be accomplished by a hydraulic dredge pumping from Shark River, three miles to the south of the southern limit of the critical area, or from Shrewsbury River, seven miles to the north of the same point. It would be costly sand, but probably commensurate with the value placed on this frontage. It is reported that the existing, relatively new, beach protective works in some areas have cost approximately $1,000,000 a mile.

Atlantic City. Conditions at Atlantic City are quite different from those at Long Branch. This section of the shoreline is near the middle of the belt of barrier beach, which is composed of islands of various lengths separated from each other by inlets connecting the ocean with tidal lagoons lying between the mainland and the barrier beach.

Atlantic City occupies the northern 3-1/2 miles of Absecon Island, which extends eight miles from Absecon Inlet on the north to Great Egg Inlet on the south. Absecon Inlet, which is very important to the economy of Atlantic City, has been under improvement by the Federal Government since 1910. The existing project provides for a channel 20 ft. deep and 400 ft. wide. It is maintained by dredging alone; there are no jetties to assist in the stabilization of the entrance.

Fig. 2 shows that shoreline changes at this most important resort have been dramatic during the long period of record, although only those subsequent to about 1854 have been of much economic interest. Prior to this date, the island warranted the historian's pronouncement that "the undisturbed isolation of the island must have made it an attractive spot for refugees from war or justice." Fortunately for the growing resort, it appears that the first reliable survey recorded the deepest invasion of the sea. Between that survey, made in 1841, and about 1925, Atlantic City experienced a net accretion, although it was not a steady gain nor is this general statement true along the entire frontage. The inlet shoreline, for example, was farther seaward in 1841 than at any subsequent date, and there are areas that have experienced alternating gains and losses.

According to local people, the 1925 shoreline was quite similar to that existing in 1939, and it is also their belief that the intervening years were not marked by any notable changes. This happy condition was of course extremely important to the resort, which has grown to a resident population of about 50,000, and an assessed valuation of nearly $100,000,000.



EXTREME TIDE AND WIND DATA				
		CALENDAR YEARS		
NUMBER OF OCCURRENCES		1936-39	1940-43	1944-47
H.W. HTS. ABOVE	10.1 FT.	67	101	137
(Normal H.W.	11.1 FT.	2	6	10
Ht.=6.6 ft.)	12.0 FT.	0	0	3
WINDS FROM N. AND NE.				
40 M.P.H. AND GREATER		102	123	111
45 M.P.H. AND GREATER		26	60	49

Fig. 2

The bulk of this wealth is concentrated close to the shoreline along the famous boardwalk, and provided a setting for the most important asset of the resort, its beach. Any recession of that beach would result in overcrowding; the city plays host to 13,000,000 visitors each year, and the beach area is barely adequate for peak days. It was indeed a precarious situation in view of the recorded violent shifts of the shoreline.

In 1939, a new phase in the cycle of shoreline events was evidently entered. Beginning that year, perhaps even a few years earlier, the beach began to shrink steadily along the northward 1-1/4 miles, the heart of the frontage. By 1947, nearly 500 ft. of width had been lost leaving a mere 100 ft. for the thousands who had become accustomed to bathing in this particular locality. In the south portion of the city and along the frontages occupied by its sister resorts of Ventnor, Margate and Longport, accretions were occurring simultaneously that matched the losses in northern Atlantic City volumetrically.

The City's officials and its influential citizens prior to 1947 evidently appraised the situation as an unpleasant transitory condition that soon would be supplanted by the recuperative phase. However, they prudently took steps to reinforce the inlet shoreline to prevent a recurrence of the severe recession of the ocean shoreline that occurred between 1852 and 1877, in which action the inlet was doubtless an active participant.

In 1947, when it was evident that no further loss of the main ocean frontage could be accepted without suffering irreparable damage, the City fathers reacted with vigor. They sought the views of every individual and organization qualified to give advice on the matter at hand, and entered into a cooperative study with the Corps of Engineers. In 1948, they concluded a contract with a dredging company to pump approximately 1,250,000 cubic yards on the affected beach. This work restored the strand to a satisfactory width, and provided time to continue the appraisal of the problem.

A great quantity of information on the basic factors that enter into the shoreline processes at Atlantic City was available prior to the initiation of the Corps of Engineers cooperative study, and additional observations were made during its course. These data show that the basic difficulty was not a reduction in the rate of supply of beach-building material, but an increase in the rate of loss along this particular section of the island. It has already been stated that the occurrence of northeasterly storms does not follow a regular pattern of frequency of occurrence. Studies of the records revealed that such storms, during the period of erosion beginning in 1939, were much more frequent than during the years when a stable beach existed in this northeasterly salient of the island. Evidently, they are particularly effective here, a concept that is reinforced by reference to the historical changes. It also appears that the salient beaches on the downdrift sides of other inlets experience cycles of great accretion and great loss. It is probable that these beaches wax when nature is in a "routine" mood, and wane during periods of unusual frequency of occurrence of violent northeasters. Examination of the table on Fig. 2 entitled "Extreme tide and wind data" shows that the erosive period certainly could be characterized as stormy in comparison with the preceding period of stable beaches. While the condition doubtlessly was a transitory phase, there was no basis for a prediction as to when it would terminate. It was also clear that it continued throughout the course of the cooperative study, as the artificially placed beach also was eroding. Accordingly, the basic remedy prescribed in the report, which has been accepted by the city, consisted of measures to reduce the rate of loss supplemented by such repetition of the artificial nourishment procedure as is found necessary to counterbalance the remaining loss. Five new groins were recommended for the ocean frontage in addition to the existing two effective structures. It was further recommended that one of these be extended to fit into the resultant pattern of beach protective works. As the inlet is the important factor in the shoreline, additional works to stabilize it were recommended also.

Ocean City. Absecon Island's neighbor on the south, across Great Egg Inlet, is occupied by Ocean City. This resort, while not so populous as Atlantic City, has grown rapidly in recent years, attracting a clientele which prefers its quieter atmosphere to that of Atlantic City. One of its characteristics is the large number of well-maintained, attractive cottages that are occupied by their owners during the entire summer, or are available for leasing. The greater part of the community has developed on the northern portion of the island.

As Absecon Inlet is of great significance in the shoreline development at the northern end of Absecon Island, so Great Egg Inlet, which is larger than Absecon Inlet, plays an important role in the changes in configuration of Ocean City's island. Unlike Absecon Inlet, it has not been subject to improvement for navigation at any time, and until a year ago, its Ocean City shoreline has not been effectively stabilized.

Scrutiny of general maps of the locality leads to the concept that Great Egg Inlet has not always passed the tidal prism of the bays behind southern Absecon Island as well as those behind Ocean City. The lagoon pattern behind the southern half of Absecon Island suggests that there was an inlet about a mile north of the present southern end of Absecon Island which subsequently migrated southward eventually to merge with Great Egg Inlet. Fig. 3 shows that a large middle ground in Great Egg Inlet existed in 1842; the unnamed inlet to the north was evidently about ready to merge. The next survey shows that the tip of Absecon Island had extended southward to incorporate the middle ground in its land mass, forcing the entire flow of the merged inlets against the north end of Ocean City. This condition was found in the 1886 survey, when much of the present-day area of Ocean City was under the waters of the inlet. During the next 38 years, the inlet migrated to the north, against the apparent littoral drift, tearing off a large area of southern Absecon Island to the accompaniment of accretion of the northern end of Ocean City. Evidently the inlet was reorientating itself to serve as the single entrance for two systems of tidal lagoons; the time seized by it as opportune for this effort must have been one of relatively small littoral drift at the southern end of Absecon Island. The remainder of that island was accreting rapidly, probably utilizing most of the drift in the process.

In 1920, Longport secured its rear face and short inlet shoreline, and built a massive groin at its southern tip. Great Egg Inlet's migration to the north was at least temporarily thwarted, but apparently the south tip of the inlet continued to stretch itself northward for 20 years more. Part of this growth is credited to material once included in the bar formations within the inlet that was made available for beach-building as the inlet bar channels readjusted themselves to the changing regimen. It also happens that during this period, which was one of particularly rapid development of the resort, the area was filled by hydraulic dredging and dunes were leveled, perhaps making more material available for the development of the shoreline. After 1944, evidently the favorable balance between supply and loss was replaced by a serious deficit, and the erosion that resulted by 1949 carried the shoreline back to where it had been 30 years earlier. Unfortunately, there had been considerable construction on this land in the intervening years.

As the tip of the island accreted from 1886 to 1944, so did the important northerly two miles of the ocean frontage from 1886 to 1928. The commercial development of the resort kept pace with the accretion; new structures were erected on the gains, always leaving adequate beach, however. Subsequent to 1928, the erosive phase quickly eliminated much of the gain along the ocean frontage, leaving no beach in the heart of the resort. The middle 3 miles of the island generally have enjoyed continuing accretion.

In about 1935, a system of groins perpendicular to the shoreline were built along the eroding ocean frontage; they were rebuilt a few years later to incorporate a curving outer end. These structures have accumulated very small fillets of beach on their northerly sides.

With the arrival of the erosive phase at the tip of the island in 1944, the construction of a system of groins, some of which are anchored to a stout revetment, was undertaken; at the present time, this work is still in progress. The system was started at the southern end of theproblem area and as the work progressed northward, increasingly grave erosion problems preceded it. Due to the draw of the inlet and the set of the shoreline with respect to the wave direction, the littoral drift is toward the inlet, and the groins accumulated fillets on

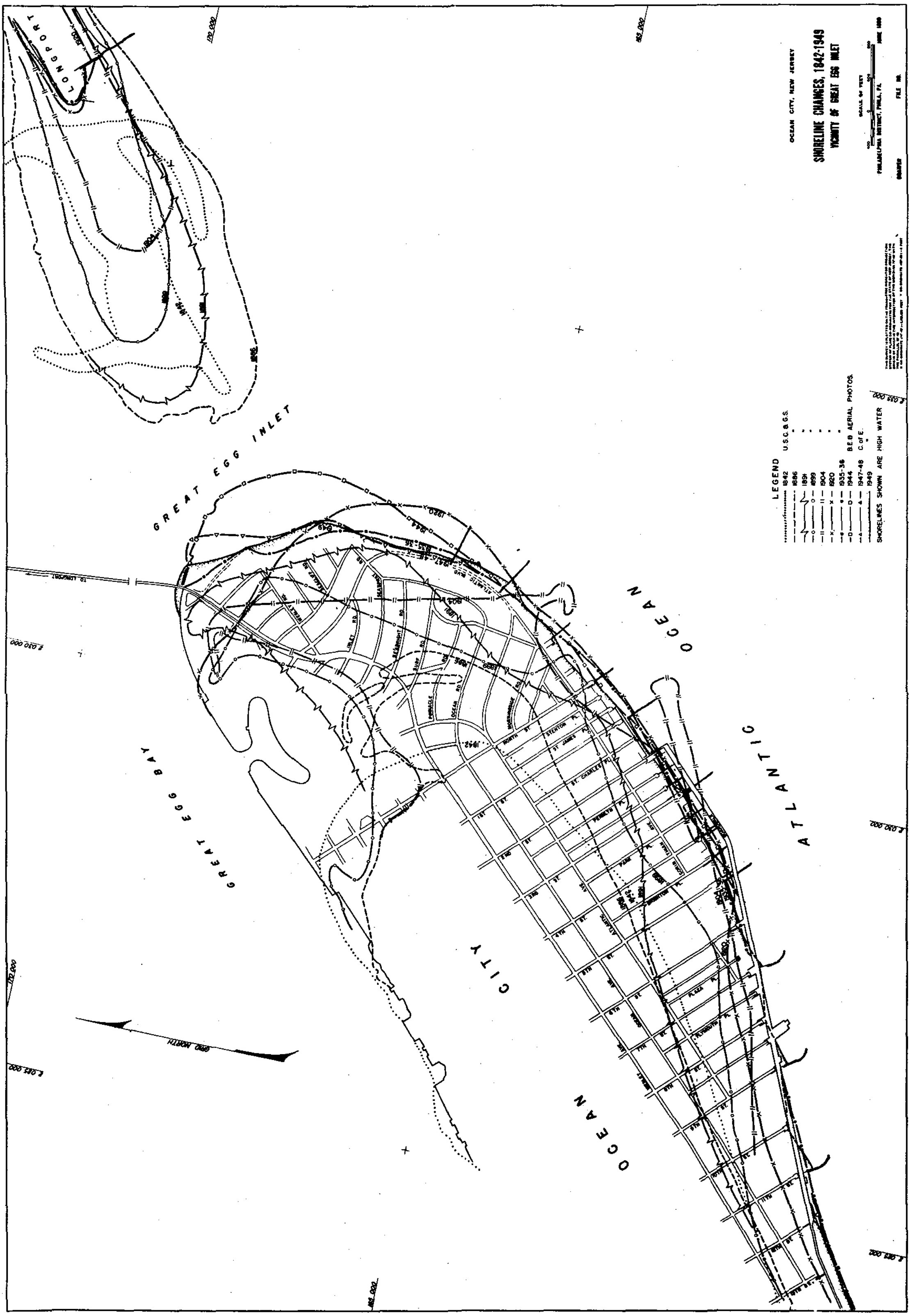

GREAT EGG INLET
GREAT EGG BAY
ATLANTIC OCEAN
OCEAN CITY
LONGPORT

Fig. 3

their southern sides. It remains to be seen whether these works will eventually aid in the establishment of a stable regimen in the vicinity.

Ocean City had no shoreline problem at its northern end until 1928. Its history during its growth as a resort had, until that date, been one of continuous accretion. The source of the later accretions clearly was the great mass of sand that had been taken progressively from the southern end of Absecon Island from 1886 to 1920. The accretions prior to 1886 can only be explained as the result of a greatly increased rate of supply in the general locality. Not only at Ocean City, but also along Absecon Island, was there an advance seaward of the shoreline between 1841 and 1886. The year 1920 saw the stabilization of the north shore of the inlet and also the beginning of a period of stability along Absecon Island. The advanced Ocean City shoreline no longer was receiving the large supply of sand necessary for its maintenance, and in only a few years the long, happy period of accretion was ended and serious erosion had taken its place. Beyond a reasonable doubt, Ocean City owed its accretions to an extremely favorable rate of supply, and its present-day erosion problem to such a decreased supply that the shoreline cannot maintain itself. The only visible supply is the present general littoral drift in the locality extending from at least Absecon Inlet to Great Egg Inlet, as erosion at one point of the present shoreline of Absecon Island is balanced by accretion elsewhere. Absecon Island thus is not presently serving to enrich the volume of littoral drift reaching Ocean City.

Under this new condition, maintenance of a beach at Ocean City at the locations desired by the city fathers evidently cannot be attained by beach protective works alone. Such works, which reasonably can be expected only to reduce the rate of loss, must be supplemented by an increased supply of sand. This can be secured by artificial means only, so long as the present shoreline regimen exists. The prudent engineer would proceed in the expectation that existing conditions might last at least as long as the resorts to the north strive to maintain their beaches in their present states.

Cape May. Fig. 4 shows what has happened at Cape May since the earliest survey of record, that of 1842. As may be expected at a community which was a going concern as early as 1801, the survey data can be supplemented to carry the story to an earlier date with reasonable assurance. The evidence indicates that the shoreline in 1804 in the midsection of the Cape, about half-way between Cold Spring Inlet and Delaware Bay, was about midway between the shoreline of 1842 and 1948. Erosion continued from 1804 to 1850, cutting back into what is now Cape May City to a point beyond the mapped location of the 1842 shoreline. Evidently the erosion in this section ceased in 1850, and for 29 years, there was accretion. Simultaneously, there was erosion to the east and west of the midsection, and by 1879 nature had produced a shoreline devoid of salients and embayments, curving gently from Cold Spring Inlet to Delaware Bay. This adjustment was merely a minor activity in the shoreline development in the locality; the net result throughout the period of record has been a loss of beach in all the mapped area of the Cape west of Cold Spring Inlet.

The littoral drift in the locality moves to the west, which is a continuance of the general movement towards Delaware Bay from the nodal area between Manasquan and Barnegat Inlet. The evidence to support this statement is the accumulation of material east of the east jetty of Cold Spring Inlet, the migration of the inlet before it was stabilized, also the accumulations at the numerous groins that exist at Cape May City and at Cape May Point. The fillet at Alexander Avenue in Cape May Point, on the Bay frontage, emphasizes this statement. This leads to the positive conclusion that has already been made; Delaware Bay does not constitute a source of supply for either the New Jersey beaches in general, or for Cape May in particular. Other evidence bearing on this point includes the character of the sand on the Cape May frontage. It is mineralogically similar to that on the beaches to the east and north, and its size suggests a progressive sorting of the same source to the end product observed. It is also quite different from the coarse, sharp sands of Delaware Bay.

The Bay is a powerful factor in the symphony of forces that act on the Cape May beaches. It has a tidal prism approximating 2 million acre feet, which

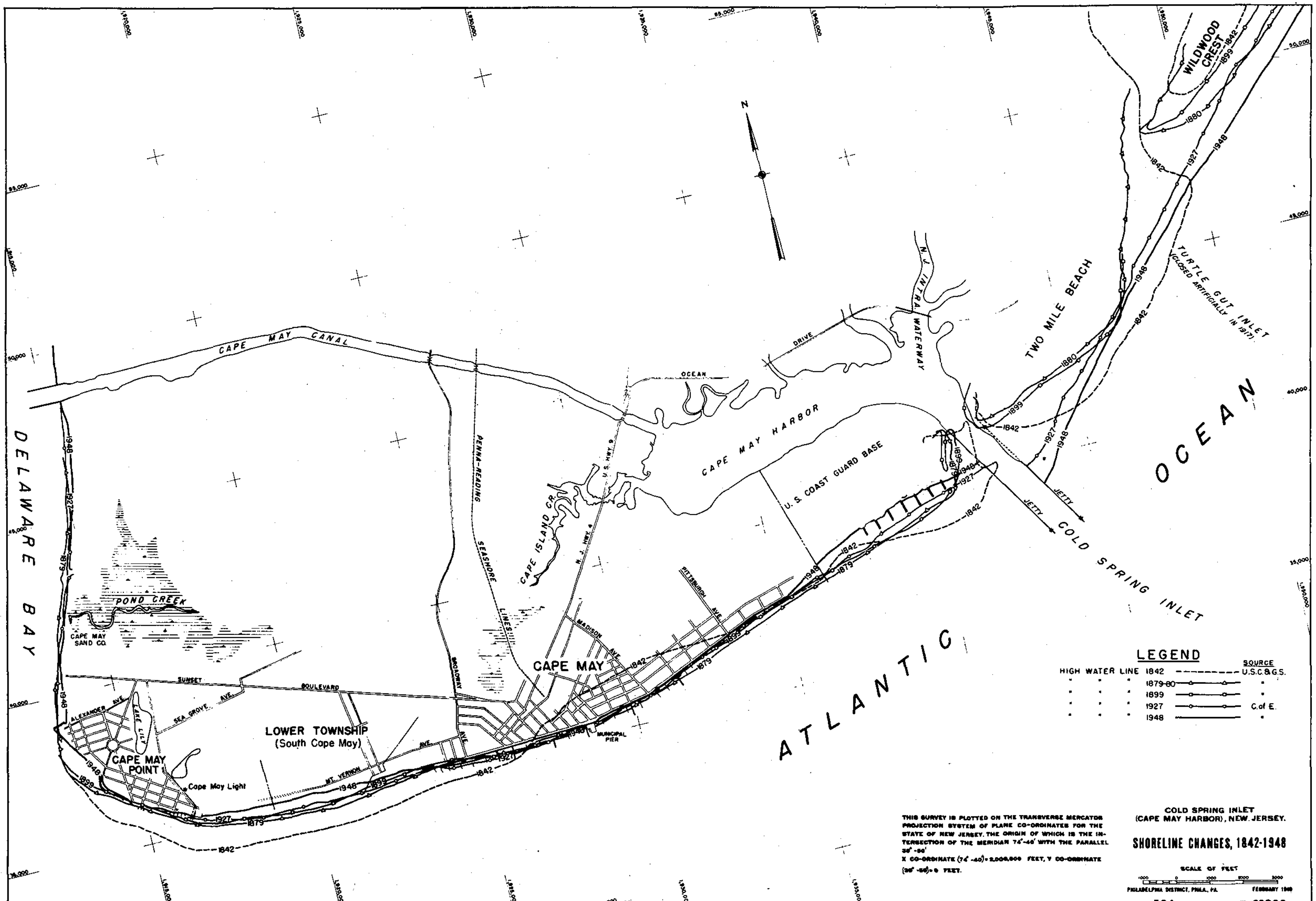

CAPE MAY CANAL
DELAWARE BAY
POND CREEK
CAPE MAY SAND CO.
CAPE MAY POINT
Cape May Light
LOWER TOWNSHIP
(South Cape May)
CAPE MAY
CAPE MAY HARBOR
U. S. COAST GUARD BASE
N. J. INTRA WATERWAY
TWO MILE BEACH
WILDWOOD CREST
TURTLE GUT INLET
(CLOSED ARTIFICIALLY IN 1917)
COLD SPRING INLET
JETTY
ATLANTIC
OCEAN
LEGEND
HIGH WATER LINE 1842
1879-80
1899
1927
1948
SOURCE
U.S.C.&G.S.
C. of E.
COLD SPRING INLET
(CAPE MAY HARBOR), NEW JERSEY.
SHORELINE CHANGES, 1842-1948
SCALE OF FEET
PHILADELPHIA DISTRICT, PHILA., PA.
DRAWER 304
FILE NO. 22820

Fig. 4

generates reversing currents throughout a fan shaped area centering about its mouth. As far upcoast as eight miles, such currents are known to exist. Along the Cape May frontage, they attain normal peak velocities of more than three feet per second, and under abnormal conditions of tide and weather, they are doubtless even stronger.

As stated before, the direction of the littoral drift is towards the Bay despite the reversing current that is characteristic of the current pattern developed by the Bay. This is evidently brought about by the work of the waves, observations at Cape May City revealing that about 90% of them approach the shoreline there in a direction that generates westerly currents. This overwhelmingly predominate angularity evidently offsets the easterly bay-generated current, and strongly reinforces the westerly bay current.

Cold Spring Inlet in its unimproved state apparently was as important in the regimen of shoreline development as Great Egg Inlet and the other inlets along the New Jersey shoreline are of great significance in the changes adjacent to them. The variations that occurred prior to its stabilization in 1907-11 are quite similar to those found near other unstabilized inlets. It can easily be conjectured, based upon known variations at other inlets, that the bulging west lip of the inlet in 1842 was of recent origin. Perhaps its accumulation required so much of the littoral supply that the beach at the mid-section of the Cape was deprived of the quantity of nourishment it evidently required for a balanced regimen between 1804 and 1850. When the lip subsequently receded to the shoreline of 1879, the released material restored the eroded mid-section.

Since the stabilization of the inlet in 1907-11 as part of a Federal navigation improvement project, the east jetty has trapped a large quantity of material. In the first few years following the construction, the rate of accumulation amounted to 100,000 cubic yards per year. Later, the rate decreased greatly, and in recent years, it appears that very little, if any, material is accumulating. Evidently the capacity of the jetty has been reached, or to express it differently, the shoreline to the east has reached a condition reflecting a state of equilibrium between the rates of supply and loss.

While the east jetty was trapping the supply, clearly the beaches to the west were being deprived of an equivalent volume of their nourishment. However, after the jetties were constructed, sand derived from the initial dredging and subsequent maintenance operations in the amount of 1,300,000 cubic yards was deposited directly on the west beaches. Clearly, artificial nourishment furnished the equivalent of more than 13 years of accumulation; the accumulation proceeded at the 100,000 cubic yards per year rate for only a few years, then decreased to approximately zero at present. Despite this nourishment, the Cape May beaches continued to erode, the rate being 4.2 ft. per year from 1899 to 1948 as compared with 4.1 ft. per year from 1842 to 1899. There was no general survey between 1899 and 1911 to separate the periods of before and after construction of the jetties exactly, but it is considered unlikely that conditions during the 12-year period 1899 to 1911 were greatly different from the years preceding and following. Evidently, the effect of the Cold Spring Inlet jetties was balanced out by the stockpiling in the earlier years of their history, and they are presently being by-passed by the supply from the east. The present regimen on the west beaches is unsatisfactory immediately to the west of the jetties, along the frontage owned by the Coast Guard, probably due to the "shadow" of the jetties; one of reasonable stability in the mid-section due to the arrival of the littoral supply to the east of this area and the existence of groins there; and an unfavorable balance of supply and loss from Cape May City westward around Cape May Point. Scant comfort can be taken from the fact that the erosion rate here was 12.3 ft. per year from 1842 to 1899 and only 3.7 ft. per year from 1899 to date. However, much of this beach is undeveloped and unused, and the consequences of the continuing erosion are not of great importance economically.

RESUME

The writer concludes that the New Jersey shoreline has no natural source of supply in the long-term sense. It may, and probably does, receive contributions from the sea, but these are more than offset by the takings; there is no upland supply of detritus, nor does New Jersey receive a supply from the shorelines of its neighboring States of New York and Delaware due to the existence of New York Harbor and the Hudson Canyon to the north and the Delaware estuary to the south.

The situation in detail has been exceedingly complex. The beaches gain and lose sand apparently whimsically. The sudden and sometimes dramatic changes are due to the pattern of occurrence of storms and the effects of inlets. They have lead to disappointing or even financially disastrous consequences, as resorts built structures and produced a high state of development of advanced shorelines on the assumption that the happy, temporarily accreted condition was permanent.

Maintenance of many of the beaches with reasonable stability is possible only by means of artificial nourishment. A few areas troubled with erosion problems have success with measures designed to reduce the rate of loss to a quantity commensurate with the rate of supply then current. As more beaches endeavor to solve their problems in this manner, more areas will be confronted with erosion. Inevitably, the task will become one of balancing the net long time average annual loss of material to the sea with an equivalent volume of artificially deposited sand.

NOTE: The opinions and conclusion expressed by the author are not necessarily those of the Corps of Engineers.

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CHAPTER 34

BY-PASSING SAND AT SOUTH LAKE WORTH INLET, FLORIDA

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INTRODUCTION

This paper describes the installation of and results obtained with the sand pumping plant located on the north jetty of South Lake Worth Inlet, Florida. This pumping plant was installed in order to pump drifting beach sand past the littoral drift barrier formed by the creation of South Lake Worth Inlet with its protecting jetties. It was anticipated that the pumping of the sand would relieve the erosion of the shore south of the inlet and cut down on the rate of shoaling in Lake Worth at the inner end of the inlet channel. A discussion of the results obtained with the installation is also included in the paper.

LOCAL CONDITIONS PRIOR TO BY-PASS PUMPING

Prior to 1918, Lake Worth had no direct access to the Atlantic Ocean and communication with the ocean was through a somewhat tortuous system of tidal channels running north and south from the lake. In 1918, local authorities began the work of dredging through the barrier to create Lake Worth Inlet near the north end of Lake Worth as a navigation channel for both pleasure and commercial craft (Fig. 1). The inlet has from its inception been protected by two jetties constructed for that purpose. The resultant littoral drift of beach sand is from north to south along this section of the coast, and, as could be expected, sand was impounded by the north jetty and erosion set in along the coast to the south of the inlet. The principal interest of this inlet to this paper is the fact that the measurements of the sand impounded north of the north jetty at Lake Worth Inlet enabled a fairly reliable computation of the average rate of littoral drift at this point to be made; such a computation shows the average drift to be about 225,000 cubic yards per year. The navigation channel, now a Federal project, has a depth of 25 ft.

In 1927, the South Lake Worth Inlet District was created by local authorities and South Lake Worth Inlet was dredged through the barrier near the south end of Lake Worth (Fig. 1). This inlet was dredged primarily to create a circulation of water in the southerly end of Lake Worth to relieve the stagnant condition of these waters; although pleasure craft drawing up to 6 or 8 ft. find it possible to use the inlet. Two stabilizing jetties were constructed as the inlet was being dredged; these jetties are much shorter than the jetties at Lake Worth Inlet, extending seaward only to about the 6- or 8-ft. contour and being only about 250 ft. in length. The mean tide range in the Atlantic Ocean at Palm Beach is 3.3 ft. and flow velocities in the inlet (as measured in February 1949) at strength of flood and ebb are in the order of 5 ft. per sec.

As stated previously, the littoral drift along this section of the coast is from north to south; this drift quickly filled the impounding area of the north jetty and worked its way around the seaward end of this jetty and into the inlet. The tidal velocities in the inlet were sufficient to sweep this sand through the inlet and into Lake Worth where the sand accumulated as a large shoal area (Fig. 2). The unrestricted growth of this shoal probably would bring about the eventual closure of the inlet, or at least an almost complete loss of its effectiveness. Measurements of the shoal area showed that it gained approximately 1,000,000 cubic yards between 1931 and 1937, an average growth of 165,000 cubic yards per year.

Concurrently with the filling of the impounding area behind the north jetty at South Lake Worth Inlet and the creation of the Lake Worth shoal, erosion of the beach south of the inlet was becoming noticeable. This was of concern to the property owners along the eroding area and some took steps to prevent or decrease the rate of erosion. One property owner who owned a considerable length of shore

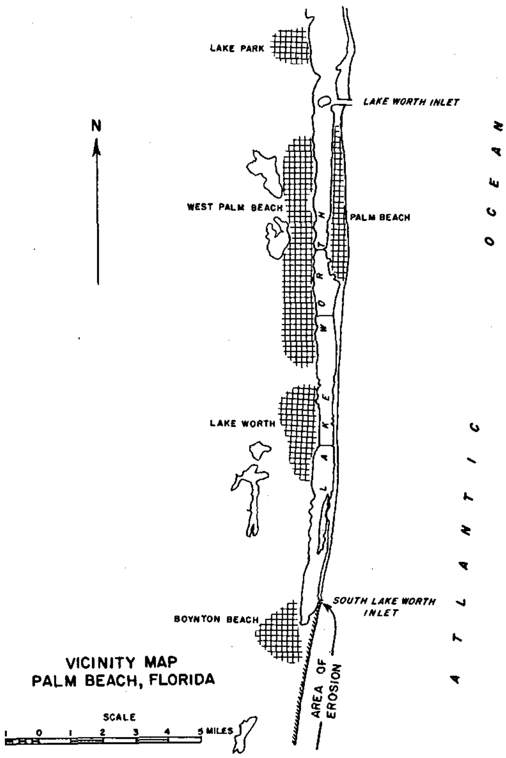

Fig. 1

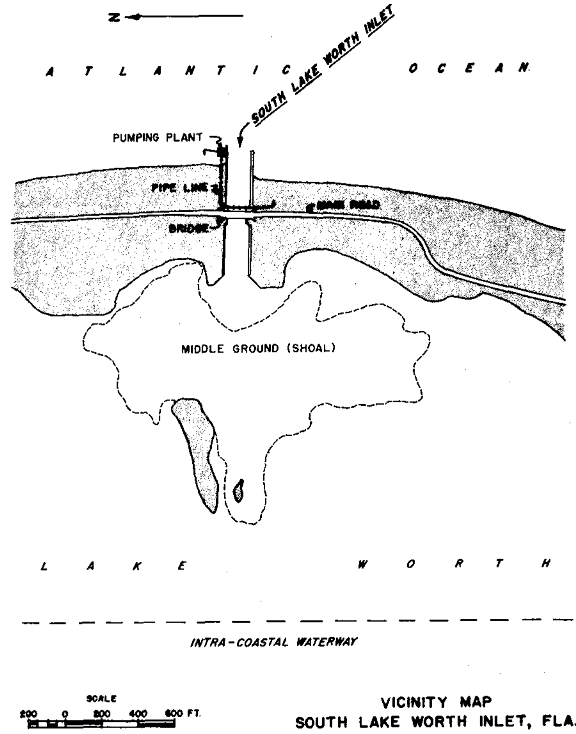

Fig. 2

property about 1/2 mile south of the inlet recognized the danger to his property within 4 years after the completion of the inlet in 1928. Starting in 1932 this particular property owner spent $160,000 to construct 2,000 ft. of seawall along the ocean frontage of his property and by 1936 found it necessary to spend an additional $40,000 to construct seven groins to support and protect the seawall from undermining. The owner had hoped that the groins would trap whatever sand was moving along the beach and thereby create a protective beach for the seawall; it was also hoped that the protective beach would also serve as a recreation beach. The quantity of drift was so small during this time (1932-37) that no effective beach was formed and the undermining of the seawall appeared probable even after the groins had been installed.

The concern of the South Lake Worth Inlet District over the shoaling of Lake Worth adjacent to the inlet and the concern of the downdrift property owners over the erosion resulting from the creating of the inlet, led to the decision to establish a sand pumping plant at the inlet as a means of pumping the sand from the north beach onto the south beach; it was hoped that both the Inlet District and the aggrieved property owners would benefit by such action. The operation was a joint undertaking on the part of the Inlet District and the property owner who had spent $200,000 on protection.

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, in fact the rate of littoral drift did not enter the initial computation. The design was based entirely on the anticipated needs of the groin field in front of the $160,000 seawall. The designing engineers had calculated the amount of sand they thought was needed to give adequate protection to the seawall and to fill the groins, and the pumping plant was designed to transport this quantity of sand in a period of about 2 years. The pumping was started in 1937 and was continued until about 1942. At this time the beach in front of the seawall had been built up to the desired level, and as the fuel oil shortage due to the war was becoming acute, it was decided to discontinue the pumping for the time being.

The pumping plant as installed in 1937 was placed on the north jetty about 50 ft. from the end. The pumping plant consisted basically of an 8-in. suction line, a 6-in., 65 hp. Diesel-driven centrifugal pump and about 1,200 ft. of 6-in. discharge line. The discharge line was carried across the inlet on the nearby highway bridge. An A-frame derrick on the roof of the pump house enabled the operator to swing the intake in a horizontal arc and to raise and lower it as required to reach the sand. The cost of the plant as installed was reported to be $15,000.

During the first year of operation (1937) the pump operated about 1100 hours, and moved about 60,000 cubic yards of beach material. This represents an average pumping rate of about 55 cubic yards per hour and an average operation time of 21 hours per week. The cost of moving the sand, including operation, maintenance, and depreciation, was in the order of 9 cents per cubic yard of beach material moved across the inlet. During the four years 1938-41, beach material was pumped past the inlet at the rate of about 48,000 cubic yards per year, or a total of about 250,000 cubic yards to 1942 at which time the pumping was discontinued due to the war.

RESULTS OF INITIAL PUMPING

The effects of the pumping operation was felt almost immediately along the shoreline south of the inlet. A noticeable accretion was said to be in evidence after 1 month's operation and within six months the 2,000-ft. length of seawall referred to previously was found to have a protective beach 120 ft. wide at high tide, whereas it had been almost devoid of beach material before pumping was started. Information is available which indicates that the rate of shoaling in Lake Worth decreased during the period the pumping plant was in operation, though this information is not sufficiently precise to enable quantitative statements to be made.

At the end of 5 years (1942) it was found that the beach had been completely restored for a distance of at least a mile south of the inlet as was evidenced by the fact that the groin field in front of the 2,000-ft. seawall had been filled with sand and the beach sand reached almost to the top of the seawall which was at an elevation of about 12 ft. above mean sea level. Unfortunately, no definite information is available on the comparative condition of the beaches for more than one or two miles south of the inlet.

RESULTS OF DISCONTINUING PUMPING

As noted previously, the pumping plant was shut down in 1942 due to war conditions. During the interval 1942-45, severe erosion of the beaches south of the inlet again set in and the Lake Worth shoal began to build rapidly threatening to nullify the effectiveness of the inlet channel. As a result of these conditions, the South Lake Worth Inlet District and Palm Beach County decided to resume pumping operations. Accordingly, in 1945 the Inlet District re-installed the 6-in. pump and 6-in. pipe line and resumed operation, the cost of the operation being borne by Palm Beach County. By 1948 it was evident that the 6-in. pump was not removing the material fast enough to prevent rather rapid shoaling of the inlet and in June 1948 an 8-in. pump and pipe line were installed in the hopes that this would meet the demands at this locality. Observations up to the present time indicate that an even larger pump is needed if the pumping rate is to equal the rate of littoral drift reaching the pump intake during northeast storms, and the installation of a 12-in. pump and line is under consideration.

PRESENT INSTALLATION

The present installation consists of a 10-in. intake mounted on a swinging boom of 30-ft. radius with a flexible rubber sleeve at the center of the turning radius (Fig. 3). A jet for agitating the sand is placed along the side of the intake. The pumping plant consists of an 8-in. centrifugal pump driven by a 275 hp. Diesel motor. The pump operates at about 600 rpm. The pump discharges into an 8-in. line about 1200 ft. in length which transports the sand across the inlet and discharges it on the beach to the south. The discharge end of the line is about 16 ft. above mean sea level while the high point in the line, where it crosses the inlet on the bridge, is about 24 ft. above mean sea level. The line appears to flow full at the outfall during normal pumping rates (Fig. 4). The pump is rated to have a 135-ft. total dynamic head for normal operation, and pumps at a velocity of about 12 ft. per sec. with from 10 to 20 percent solids (by volume). The beach material is about 60 percent shell and about 40 percent medium to coarse sand. The consensus is that the pump places about 100 cubic yards of beach material per hour of normal operation.

Fig. 3. Pumping plant near outer end of north jetty.

Fig. 4. Discharge line south of south jetty.

The operating schedule of the pump is flexible. Two full-time operators are assigned to the pumping plant for maintenance and operation. During periods of relatively calm weather, two or three hours pumping each day is found to remove all the material in reach of the 30-ft. boom. The boom has provisions for vertical movement of about 12 ft. and for horizontal movement of about 30 ft. around the arc of the boom radius. In effect it is capable of digging a circular trench some 30 ft. long and 8 or 10 ft. deep. The depth to which it digs is limited by bed rock or hard pan, and the amount it pumps is limited by the quantity available in the pumping area from day to day. During periods of northeast weather it is found that pumping on a 18-hour-per-day basis is hardly sufficient to remove the sand at the rate it accumulates in the impounding area, and it is believed that under these conditions much material moves around the outer end of the north jetty and into the inlet channel.

The eye of the pump is some 6 ft. above mean sea level and the priming of the pump is accomplished by a check valve in the suction head and a small auxiliary priming pump. No difficulty was reported in starting the equipment. As the 8-in. pump had been in operation less than a year at the time of last contact, the engineers concerned were not prepared to give an estimate of the yearly amount of material moved with the larger pump. However, they stated that over the period 1945-48 it was estimated that 70,000 cubic yards per year had been moved by the 6-in. pump previously installed, though it should be recognized that even the 6-in. pump was not operated on an average of more than about 25 or 30 hours a week. The annual operating costs of the present 8-in. installation, including depreciation, is in the order of $15,000 to $20,000 based on 1950 prices.

DISCUSSION

The results of the operation of the pumping plant at South Lake Worth Inlet present several aspects which are of interest to engineers concerned with shoreline processes. The fact that a pumping plant has been in operation for a number of years and has successfully replenished the downdrift beaches by pumping a portion of the littoral drift past the inlet is itself significant. Of corallary interest is an evaluation of the by-passing operation to determine what light this particular operation might throw on the general picture of shoreline erosion; this discussion will be concerned chiefly with this latter point.

It is to be recognized that the drift reaching the inlet from the north is split three ways at the inlet, part going within reach of the pump and being pumped past the inlet, part going into the inlet and shoaling the middle ground in Lake Worth, and part by-passing the inlet entirely and reaching the shore to the south. In practice, a measurement on the first two portions of the drift is feasible; however, the third portion -- that by-passing the inlet in deep water -- cannot be assessed by present means unless it can be assumed that the normal rate of drift at South Lake Worth Inlet is 225,000 cubic yards per year as it is at Lake Worth Inlet about 15 miles to the north. The 225,000 cubic yards per year figure obtains some support from the fact that the shoal ground in Lake Worth gained 1,000,000 cubic yards or so during the no-by-passing period, 1931-37, or an average of 165,000 cubic yards per year.

In event the 225,000, or even 200,000 cubic yards per year could be accepted as the normal rate of drift at South Lake Worth Inlet, it would then be possible to assess the magnitude of the three portions of drift described in the preceding paragraph. An assumption of 200,000 cubic yards per year might be considered to carry with it the assumption that the deepwater drift is in the order of 35,000 cubic yards per year; deepwater in this case is taken to be anything outside about the 6-ft. depth contour.

In view of the above statements, it becomes significant that the beaches for a mile or two south of the inlet have been restored and maintained by the 50,000 to 70,000 cubic yards per year which has been pumped past the inlet. The fact that this beach eroded when this 50,000 cubic yards per year or so of material was denied to it by the construction of the jetty indicates that the material traveling in the shallow water inside the 5- or 6-ft. contour is the key to successful beach maintenance downdrift from South Lake Worth Inlet. The other portions of the drift, the deepwater drift which by-passes the inlet and the material which moves into the inlet even during pumping operations apparently play little part in the determination of the foreshore beach profiles downdrift from the inlet. This is shown by the facts: (1) that the beaches eroded severely when the inshore drift was stopped even though the deepwater drift presumably continued to move past to the south, (2) the beaches have been reestablished without recovering the material swept into the inlet and caught in the shoal in Lake Worth over the past twenty years.

The reasoning in the preceding paragraph would lead to the tentative conclusion that, so far as beach maintenance downdrift from the Lake Worth Inlet is concerned, the transfer of the material normally traveling inside the 4- or 5-ft. contour is all that is necessary. This is admittedly a rather tenuous conclusion and should be considered more as a hypothesis to be examined in the light of any additional studies made at South Lake Worth Inlet.

If the tentative conclusion set forth in the preceding paragraph were found to be valid and could be expanded to apply to other inlets as well as to South Lake Worth Inlet, the problem of restoring downdrift beaches would be greatly simplified. This simplification would be evident from the fact that only about 1/3 or 1/4 of the total drift had to be pumped past the littoral barrier and that a stationary pumping plant was found to be capable of reaching the requisite amounts of material, i.e., the littoral forces brought the required material within reach of the 30-ft. swinging boom intake of the pumping plant. Estimates of the cost of pumping sand past inlets in other localities have indicated almost prohibitive costs in many cases; these prohibitive costs were generally the result of anticipating the moving of the entire quantity of drift by the pumping plant, which resulted in large pumping rates and also necessitated the design of some form of mobile pumping plant. For example, annual cost of 5 to 25 times the annual South Lake Worth Inlet costs have been estimated for by-passing at six other inlets for which careful estimates have been made.

Some of the excessive cost at other inlets is due to the fact that a submerged or floating pipe line would have to be provided as there is no bridge to carry the pipe line across the inlet. However, the principal cause of the higher estimates was the assumption of the necessity to pump practically all the drift and to have a movable pumping plant.

Possibly the favorable results at South Lake Worth Inlet may be in a measure due to the existence of a layer of resistant material which crops out at frequent intervals along this section of coast and may be delaying or preventing any noticeable erosion of the offshore zone seaward of about the 6-ft. contour. Unfortunately no precise hydrographic surveys are available which would enable the offshore zone change over the past 20 years to be studied with any exactitude.

CONCLUDING REMARKS

The problem of protecting or restoring beaches downdrift from littoral barriers is a recognized problem in coastal engineering. The sand by-passing plant at South Lake Worth Inlet has apparently been successful in that it has restored and maintained the beaches downdrift of the inlet without excessive installation or operation costs. Since it is the first installation of this nature, the results obtained therefrom should be of interest. Even though the basic hydrographic data which describe the results of the pumping are rather limited, the tentative conclusions which may be drawn from these data appear to stand against some of the previous thinking on the subject. These tentative conclusions should serve to provoke additional consideration of the practicability of by-passing sand at littoral barriers by means of fixed pumping plants.

CHAPTER 35

EROSION AND CORROSION ON MARINE STRUCTURES, ELWOOD, CALIFORNIA

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INTRODUCTION

The sole purpose of this paper is to present to the public more than twenty years' experience acquired in designing for, and eventually combating, the unusual forces of nature acting on marine structures in exposed waters. The structural design will naturally be determined by the use the project is called upon to serve and the standards of the designer. This particular phase of the overall picture, with the aid of available meteorological and oceanographical data for a specific area offers no particular difficulty in structural design.

The subject matter presented herein refers to observations made among the various pier and oilwell foundations constructed in the Elwood Field, California, during the interval between 1929 and 1935, and major maintenance problems to date.

Where protective measures made at the time of installation are explained, they refer to our own specific works, while the general observations in the field are confined to no one particular structure or type of design.

LOCATION

For several miles in the Elwood area, the coast line is marked by a very steep shale escarpment, 30 ft. to 90 ft. high, rising from the beach. At intervals this escarpment has been eroded by streams from the mountains which have cut canyons that are persistent well out to sea, although filled with sand and silt to the level of the present ocean floor. These former canyons, as far as we have tested with jets, indicate depths up to 90 ft. from the ocean floor down to original shale.

The beach line, extending seaward, is a relatively smooth shale surface, dropping seaward about 40 ft. in depth in 2500 ft. From the base of the escarpment this beach is intermittently covered with a mantle of sand varying in thickness up to 8 ft. from season to season. Beyond the breaker line to approximately a depth of 60 ft., there are extensive beds of kelp.

DESIGN

On the first 1650 ft. seaward of the pier, a three-pile bent of 8 in. x 8 in. --32 lb. "H" section was used. Previous to driving, the piles were wire brushed to remove mill scale, washed to remove dust and loosened scale, and given two coats of asphalt chromate emulsion, having a minimum one-sixteenth of an inch of coating.

The first thirteen bents seaward, or those in the normal breaker line, were protected by driving a 14 in. diameter 3/16 in. thick 8 ft. long steel cylinder over the pile through the sand into the shale, jetting the interstitial space clean of sand and filling the same with a 1:2 cement grout.

The reasoning behind this deviation from standard practice in the field was the experience of the designer, who had seen steel piles of structural shape cut off in the breaker line by abrasion of sand in suspension, whereas round shapes survived, such as the old Olympic Club pier on the San Francisco beach.

This detail of design proved to be very effective as the adjacent piers have had to have piling in the breaker range replaced or protected within five years after installation.

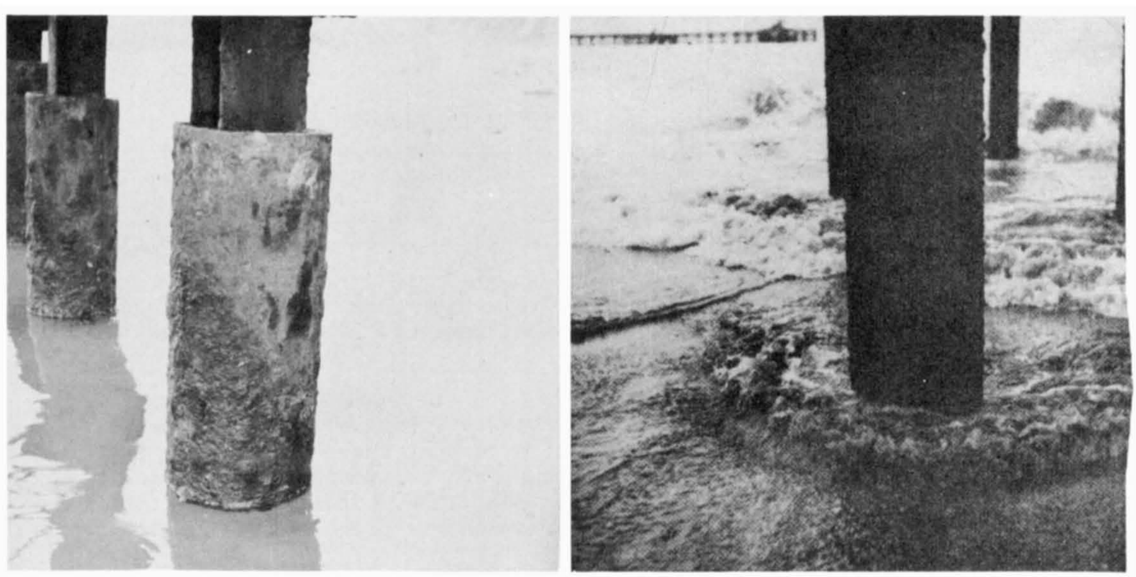
On one pier we acquired by purchase, the "H" sections in the breaker area, at the first sign of scouring, were protected by boxing the exterior of the pile with creosoted two inch thick planks, securing same with 1/2 in. x 2 in. galvanized bands top and bottom. Incidentally, after fifteen years, the wood is intact but the top bands are cut through. Reference to illustrations will show the wisdom of this choice on new work and the boxing of piles already driven (Figs. 1 and 2).

The two basic divisions of the structures, as constructed, were the well or derrick foundations, and the working area or pier approach.

Fig. 1. ABRASION IN BREAKER LINE

\- A -

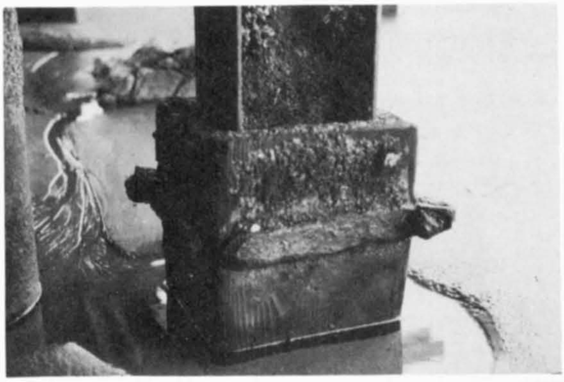
General Effect on Unprotected H Piling.



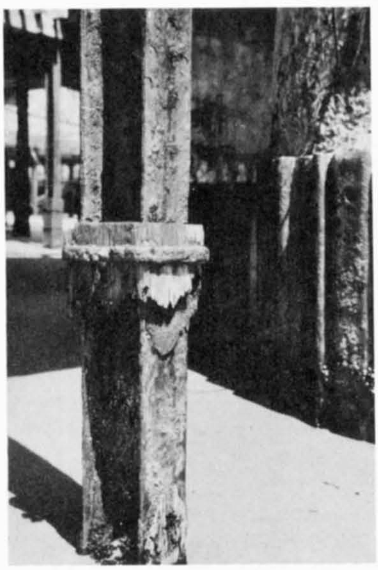
\- B -

Protection Provided in the Planning 8 in. H Piling, 14 in. diameter, 3/16 in. thickness driven cylinders, interstitial space cleaned out and filled with 1:2 cement grout. In service 21 years without maintenance or coating of any type.

Fig. 2. ABRASION IN BREAKER LINE



- A -



- B -

- C -

- A -

Protection Provided after Construction 10 in. H Piling, 2 in. x 12 in. creosoted boxing secured in position by 1/2 in. x 2 in. galvanized clamps. In service 16 years. Condition: Timber - good; clamps - cut through at corners.

- B -

Attempts to Protect after Construction. Same as cut A except untreated timber.

- C -

Attempt to Protect after Construction. Spiral-wrapped reinforced concrete. Failed when sheet iron form failed. Still protected, however. Limited to Tidal Range.

The various interests through their respective engineering departments set varying load conditions and it is worthy of note that there has been no failure through loading up to now on any of the structures -- twenty-one years in service-- that were originally designed for a fifteen year life, notwithstanding the fact that today's transportation and drilling equipment is twenty-five to fifty percent heavier than that of twenty years ago.

The premises of design and protection adopted by the Signal Oil and Gas Company on their structures will be discussed in detail from here on, giving the thinking behind each deviation from precedent, the results of the choice after a twenty-year service, and the changes that suggest themselves if we were to start off anew.

CHOICE OF MATERIALS

Inasmuch as the ocean bottom is shale, wood piling was out -- except in a few cases where piers traversed aforementioned silt-filled shale depressions on the ocean bottom.

Little was actually known about the effect of pile cross-section pattern on the drag coefficient in 1929. Steel piling seemed to be the only solution, and because of the immediate availability and economics, structural sections were used. Our experience to date indicates the choice of steel was right, but the cross-section or pattern of the pile was wrong. The superstructure, consisting of caps, joists, and deck, was built of untreated structural grade Oregon pine.

Progressively, the next item we considered was the location of the welded transverse and longitudinal angle bracing. When you consider the long unsupported length of the piles or the high slenderness ratio, it is apparent that bracing of some kind is required. It is impractical to do this below the water line and too close to sea level. The braces gather floating kelp and debris. This becomes a hazzard because of the additional weight, and especially because of the increased area exposed to wave action. Hence, by observation of an existing pier in the area, we decided our first bracing should be 8 ft. above the high tide line. We have never lost a rod or brace in twenty years and have never had to remove kelp suspended from the bracing.

We next chose the most suitable deck elevation above mean high water. Precedent indicated 20 ft.; however, we added 2 ft. to this and used 22 ft. as our deck elevation. The reasoning we used was based on observation that waves build up just before breaking. So far we have had no trouble from waves lifting the bottom of our deck system; however, a few high waves have reached the under side of the superstructure, on other piers in the vicinity.

In 1935, six years after the construction of the original 1650 ft. pier, it was deemed advisable to add approximately 800 ft. to the original pier to provide drill sites as close as possible to the axis of the anticline (or top of oil structure). The reason for bringing this information into the paper is to present the variation in design employed and results obtained therefrom.

On advice of the consultant, protection (coating) on piling was deemed of little value and he recommended the price of coating had better be spent in providing thicker steel. Hence, in this section of pier a four-pile bent of 10 in. x 10 in. -- 54 lb. "H" Section, with no coating of any kind was used. Now, bear in mind the original pier built in 1929 consisted of a three-pile bent of 8 in. x 8 in. -- 32 lb. piles coated, while this new section built in 1935 consisted of four 10 in. x 10 in. -- 54 lb. piles uncoated. The bracing and superstructure are similar.

The wisdom in coating the piling as a protection against oxidation above the tide range is again illustrated by the pictures (Fig. 3). All hardware throughout, such as nails, drifts, bolts, nuts and rods were hot-dipped galvanized. The concrete mix for well foundations such as derrick legs, cellars and scupper decks was scientifically worked out. San Gabriel sand and gravel were used and the mix called for seven sacks of high silica cement per cubic yard with a maximum slump of six inches. The concrete work was coated with bitumastic immediately after removing the forms.

In resume, the only protective measures taken during construction were coating the piling, jacketing the piling in the breaker line, ventilating all lumber to the fullest, creosoting all lumber bearings and laps, using galvanized hardware throughout and coating all freshly stripped concrete. It is worthy of note here that after twenty-one years' exposure, the piers are in continuous service today with a minimum of replacement and maintenance.

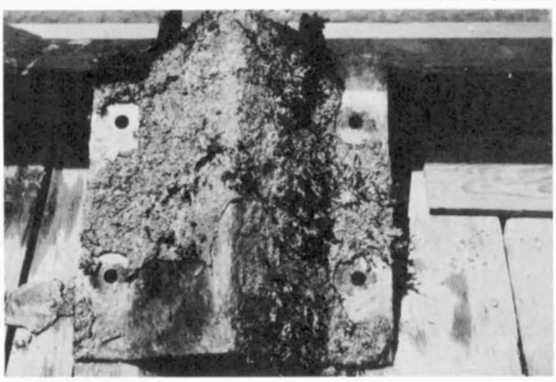
Regular visual inspections of exposed works above the sea level are made by coring lumber and calipering steel. Periodic subsurface inspections are made by divers, again calipering the steel after removal of the organic growth. The Signal Oil and Gas Company have taken the caliper readings as furnished by the diver and recalculated areas and section moduli and from this data we are able to have a close check on our original premises of design.

Fig. 3. ATMOSPHERIC CORROSION

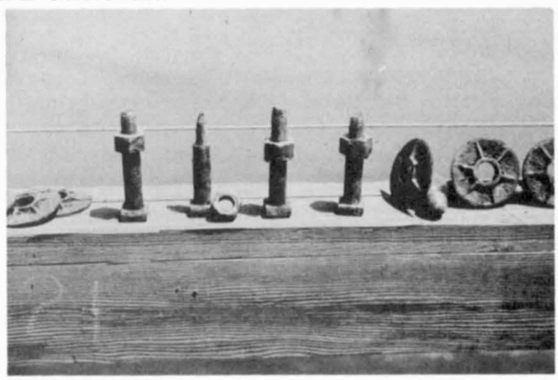
- A -

- B -

ELECTROCHEMICAL CORROSION



- C -



- D -

-A- Very severe case of oxidation in atmosphere above tide range. 10 in. H Pile, 0.564 in. web thickness; no protective coating. In service 16 years.

-B- 8 in. H pile, 0.300 in. web thickness, coated with 1/16 in. of Asphalt Chromate Emulsion. In service 21 years. Never retouched.

-C- At Base of Piling beyond the sand line in deep water where the bottom is mud and decaying kelp. This is a highly anaerobic area.

-D- Action on Auxiliary Non-structural Sleeves placed by diver at base of piling.

Twenty-one years of observation on all piers in the vicinity has brought forth the following recommendations:

Piling, breaker line or sand bottom area. All piling should be cylindrical or circular in cross section to provide as near as possible a streamline flow past the pile for any direction of wave impingement. An auxiliary sleeve over the pile of sufficient length to cover same between the limits of sand depth, is desirable. This choice of section provides an equal section modulus for any axis; also, if paints, wrapping, etc., are used, the circular cross-section provides less comparable surface area to cover and no angles, corners or sharp edges where it is difficult to build up coatings.

Piling should be spaced a minimum of five diameters for a dampening of eddies set up by adjacent piling. The piling should be filled with concrete for additional strength and stiffness to resist any distortion, or the pile should be sealed off or neutralizing agent should be added to prevent interior corrosion. Structural sections in the breaker range can be protected by boxing, preferably with creosoted plank secured with heavy metal straps.

Coatings above the low tide range, certainly in our case, have proved to be better than the additional uncoated steel.

The above recommendations are based on the following observations:

(Breaker line) In about three years, noticeable wear and sharpening of flanges was noted where structural shapes were used. In five years the cross-section was considerably reduced and lenticular holes appeared in the webs.

("H" pilings) Those oriented so the web was normal to the wave direction were especially susceptible to abrasion. Many piers have had their piling replaced or reinforced in the breaker line within five years.

The condition of the piling given a protective coating twenty years ago is a convincing argument in favor of a protective coating at least for oxidation in the zone above the water line.

(Deep water piling inspection) Cleaning a selected number of piling from the underside of the cap to the ocean floor and observing and calipering same disclosed the following, proceeding up the pile:

No sand cut or abrasion was noticeable.

At approximately 2 ft. above the bottom, a highly anodic area was found with a noticeable loss of section. This may be related to the point of flexure; however, the environment is highly anaerobic, common to harbors and areas of decaying kelp. We corrected this by having a diver add metal sleeves to the piling and subsequent inspections have shown this has corrected the condition. The metal loss, whether it be chemical or electrolytic, is now from the auxiliary metal jacket rather than the pile and we believe that no further loss of metal will take place at this critical point.

When you consider that on an average of 6 sec. wave periods in the space of one year you have 5,350,000 reversals of stress, multiplied by twenty years, flexure might well be responsible for at least a part of the condition explained above (Fig. 4).

Proceeding up the pile, anodic pits were found at various points. They followed no specific pattern or location and while not deemed hazardous, locations and lead impressions were carefully made for future comparison.

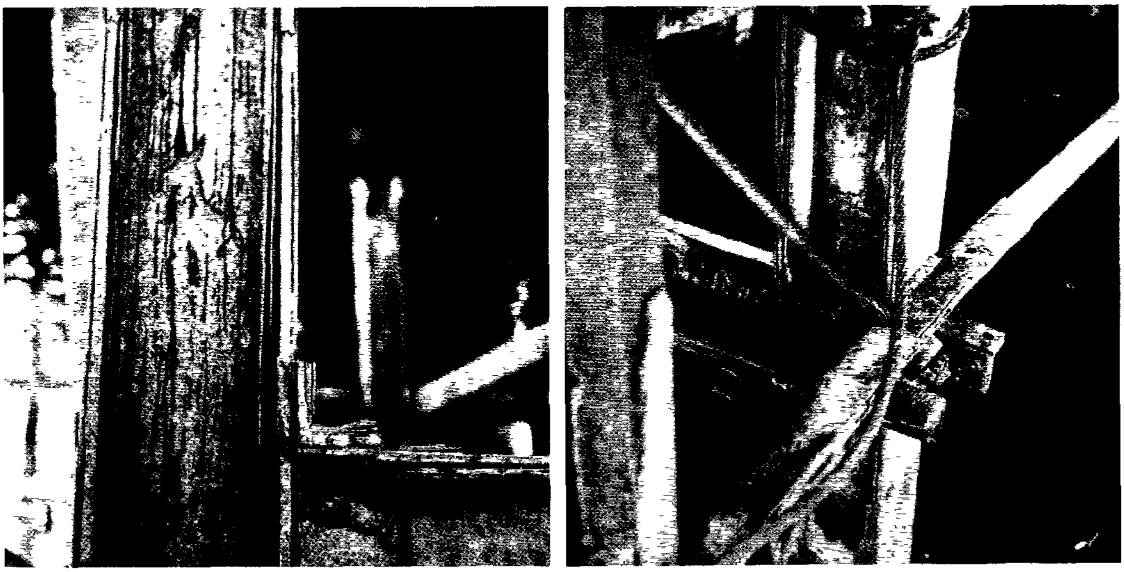
Our next perceptible loss of section on uncoated piling was above the water in the vicinity of the horizontal bracing welded to the piling. The loss of pile cross-section can be seen by eye on removal of the laminations of rust. The flange thickness is reduced to about sixty percent of its original thickness and, strangely, in a few cases, the webs are badly corroded. This condition does not exist to such a marked degree, if at all, where the piling and joints were coated twenty years ago. This again is a point of restraint and flexure. The only cure so far is removal of the section and welding in a "dutchman".

The writer recently attended the annual Sea Horse Convention at Kure Beach, North Carolina, where salt water corrosion was the sole topic. The general observation brought forth paralleled what we have here on the West Coast, with recommendations for prolonging the life of steel piling similar to those we are taking or have taken.

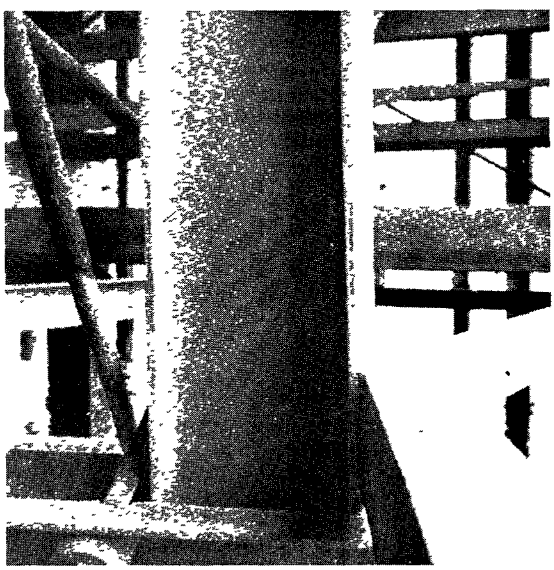
From the study of the subject discussed at the convention, it certainly convinces one that each environment has its own specific problems.

As of this instant, we are testing various types of pile protection such as paints, somastic coating, metal spray, galvanizing and also cathodic protection, which seems to offer a solution at least to our under-water problems.

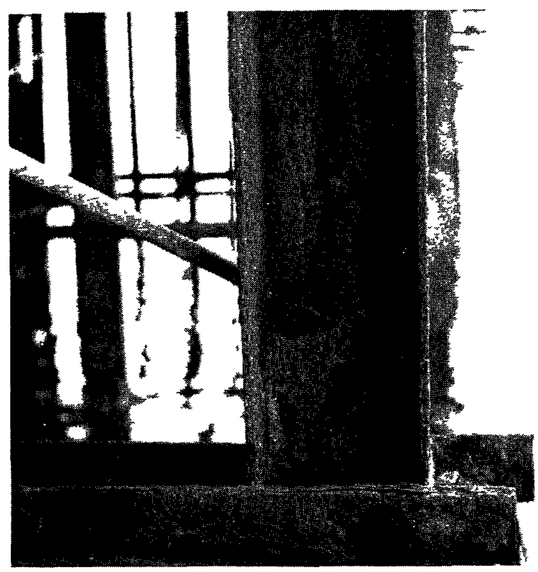
Fig. 4. STRESS CORROSION



- A -



- B -



- C -

-A- Typical Corrosion in vicinity of restraint showing laminations of rust. In service 16 years. No coating protection.

-B- Typical Corrosion in vicinity of restraint with rust removed. Note loss of section on the flange. In service 16 years.

-C- Note absence of above conditions where steel was coated with Asphalt Chromate Emulsion. In service 21 years.

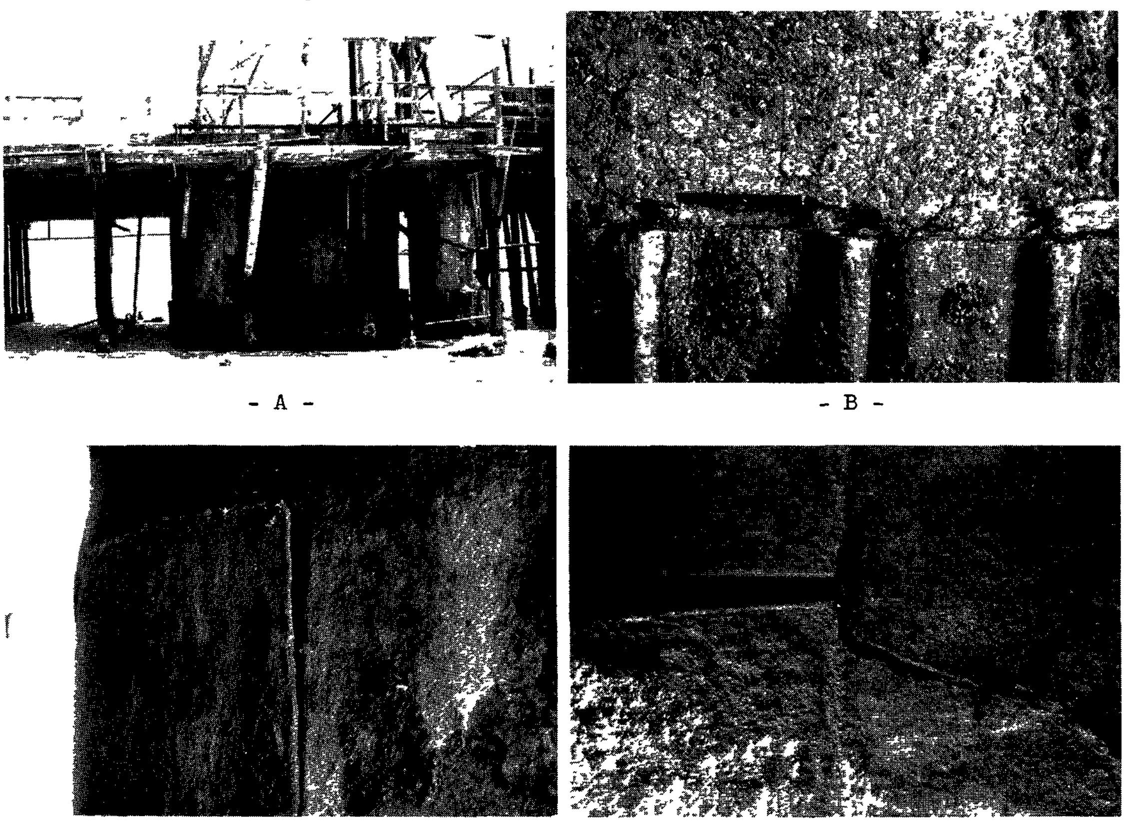
Bracing. Bracing should be of a minimum, located above normal wave heights and in areas where kelp is prevalent, consideration should be given to kelp loading. The section, where possible, should be round to reduce the drag coefficient of waves and the ease of re-coating. The principal objection to round section is fabrication.

Timber. The subject of wood preservation is a science by itself and all I can relate here are the precautions taken on our works to try to insure our superstructure and the results obtained. Again, the choice of untreated timber over treated lumber, even for fifteen years, is a matter of environment. All lumber was rough structural grade. All bearings were mopped with hot creosote. All laps were mopped with hot creosote. All lumber was spaced or gauged insofar as possible to provide free air circulation. Deck planks were gauged 3/4 in. All nails, drifts, bolts, etc., were galvanized.

After twenty years, aside from a few planks, the original deck and the original joists remain; some few caps, possibly sixteen percent, have been reinforced due to heart rot from water entry through checks.

By all means, provide as much air circulation as possible; keep the top surfaces of caps cleaned of all earth or wood fibre that might sift through the deck spacing and form a damp mat and induce decay from the top. When fungi or dry rot have taken over and auxiliary members are required, remove the ailing timber. The presence of such timber is a source of fungi spore and it is no longer a structural component of the work.

Fig. 5. CONCRETE IN SALT WATER ENVIRONMENT



- A - - B -

- C - - D -

-A- Mass Concrete in Wave Area.

-B- Vertical Rise from Sheet Pile Forms.

-C- Mass Concrete in 1/4 in. Steel Shell above high tide line. Note weld failures.

-D- Mass Concrete in 1/4 in. Steel Shell above high tide line. Note shell failure.

Concrete. Concrete offers the same picture as anywhere else along the Southern California coast. It "grows," whether reinforced or mass concrete. We have had success with high-silica cements and a scientifically graded aggregate for density, stripping and coating as soon as possible to exclude moisture. This also has protected ordinary Portland cement concrete.

There are instances along the coast where concrete cylinders 13 ft. in diameter have increased their diameter approximately 6 in. and have grown approximately 4 in. in height. Other cases have occurred in which cylinders of 1/4 in. plate and 6 ft. diameter, filled with concrete, have either ruptured the weld or the steel failed.

Reference to accompanying photographs should emphasize the necessity of investigation of the chemical reaction between cements and aggregate in a damp environment (Fig. 5).

In summation, there is no panacea for the caprices of nature in its never-ending task of destruction and building. The insatiable appetite of nature must have sustenance, preferably coatings and/or anodes rather than structural members.

In designing, the economic comparison between long life with costly preparation to receive and ultimate application of coatings, must be made with shorter life using more frequent applications of a less expensive coating. In fact, in the original design provision might well be made for the ease of ultimate replacement of certain structural members.

The recent convention at Kure Beach, North Carolina, more than ever convinced the writer that each installation is a separate problem. Figures on the average i.p.y. (inches per year) losses by oxidation, abrasion, etc., of timber, steel, galvanizing, etc., certainly were not applicable to Elwood structures. To keep various construction members, -- timber, piling, etc., -- in balance, it is necessary to inspect, repair or replace at regular time intervals.

In a study of this report and accompanying pictures (Figs. 1 - 5), the reader should bear in mind that many of the illustrations are of structures before replacement of members and others are of structures already razed or in the process of being razed. Where structures are in service, every possible known means of maintenance is being used to preserve them.