FIELD AND MODEL STUDIES

OF THE NEARSHORE CIRCULATION

by

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ABSTRACT

Investigations into the characteristics and underlying mechanism of the circulation of water near the shore are reported. The two main types of circulation, one a cellular system resulting from waves propagated nearly normally to the shore, and the other an essentially alongshore flow associated with oblique waves, are treated separately.

The cellular circulation studies were made in the field at Virginia Beach and more extensively in wave tanks. From the field experiments data were collected about the dimensions of the cells, the way in which the water circulated, the rate of exchance of surf zone water and the extent of recycling. A method for measuring the changes in the mean sea level over intervals of time greater than those of the wave periods, was developed. The model experiments carried out in uniform wave tanks showed that the cellular circulations could be well simulated. Measurements were made of the cell dimensions, the velocity of the longshore and rip currents, and of the recycling regime.

A finding from the wave tank studies was the presence of standing waves formed by transverse edge waves. The interaction of these standing waves with the generated waves normal to the shore could be the initial cause of rip currents and the cellular circulation.

Studies of the alongshore system were made in the field only. A method for measuring the volume of flow of longshore currents was developed, tested, and applied. Calculated volumes of flow using a theory based on continuity and the solitary wave theory (as proposed by Inman and Bagnold) compared tolerably well with the field observations.The calculations of volume of flow required a knowledge of the wave height spectra in the surf. This was established by making wave height recordings in the between-breaker zone. It was found that the characteristics of the spectra compared reasonably well with those predicted by the Longuet-Higgins theory, previously assumed to apply to deep water waves only.

A mechanism for the transition from cellular to alongshore system is proposed.

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LIST OF SYMBOLS

Where the work of other authors has been quoted the original symbols have usually been retained, except in the case of depth which is throughout represented by "d".

Otherwise the following symbols have been used:-

a	wave amplitude
A	asymmetrical cellular circulation
b	offshore gradient of the water surface
$\_b$	pertaining to breakers
B	factor to convert significant wave heights to root mean square wave heights
C	wave phase velocity or concentration of tracer
C_f	Chezy's friction coefficient
d	water depth below still water level
D	mean horizontal orbital displacement of a solitary wave
ε	surf frontage through which water other than directly recycled rip current water, enters the surf
g	the acceleration of gravity
H	wave height from trough to crest
$H_{\frac{1}{3}}$	significant wave height
K	dimensionless coefficient of friction = $\sqrt{g/C_f}$; or = $\frac{2\pi}{L}$
L	**wave length**
L_c	length of crest of solitary wave approaching the shore obliquely and contributing water to the longshore current. Approximately equal to the corresponding length of shoreline when the obliquity is small
L_s	distance of sampling stations down current from the point of insertion of a tracer
n	the mode of edge waves
N	nodality i.e. the number of nodes in a fixed distance
$\_o$	pertaining to deep water

LIST OF SYMBOLS
(continued)

Q the volume per unit crest length of a solitary wave

Q_L the discharge of longshore currents

Q_r the discharge of rip currents

R ratio of cross-section to wetted perimeter

r surf frontage through which rip current water recycles directly

S symmetrical cellular circulation

T wave period

U mass transport velocity

u horizontal component of orbital velocity of waves

V_L mean longshore current velocity

V_r mean rip current velocity

$\bar{V}_c$ net mean horizontal orbital velocity under the wave crest of a solitary wave

V_o rate at which tracer is discharged continuously

w semi-width of a rip current at its exit from the surf

α angle between wave crest and the shoreline

γ ratio wave height to water depth for a solitary wave

Δ- increment of length or volume between sampling stations in alongshore studies

λ the width of a cell of circulation

σ $\frac{2\pi}{T}$

PREFACE

The studies here recorded formed part of a general enquiry in the water movements in the nearshore region off the Natal Coast near Durban in South Africa and cover the period 1960-1966. The enquiry was instigated to evaluate that part of the sea as a receiving water for waste disposal. In the course of the work certain fundamental problems were revealed and the study was therefore extended to include not only qualitative and quantitative aspects of the water movements but also the basic underlying principles.

Certain of the qualitative findings have already been reported in an M.Sc. thesis .

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While detailed acknowledgements are recorded at the end of the report special mention may be made of Dr. G. Stander, Director of the National Institute of Water Research who made the research possible, and Prof. D. Clarence, Head of the Department of Physics of the University of Natal, whose encouragement and guidance were invaluable.

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INTRODUCTION

1. The nearshore region of the sea is that region in which the water movements are chiefly the consequence of wave deformation in shoaling water - especially wave breaking. It commonly extends seawards from the shoreline to a distance of roughly half a mile. The water movements in it are usually described as the nearshore circulation.

2. The purpose of this study has been (a) to determine the qualitative and quantitative nature of the circulation and (b) to enquire into the underlying cause of certain types of circulation especially those which are characterised by a cellular structure.

3. Though the pattern of the circulations may vary greatly with changing conditions, past experience has shown that there are certain essential features which make broad classifications possible. In this report a distinction is drawn between those in which the cellular nature is marked, and those in which the movement of water is almost wholly alongshore to the virtual exclusion of cells. The former have been named the "cellular circulations" and the latter "the alongshore system". These two types are treated seperately in the account which follows - the former being dealt with first.

4. The cellular circulations are a physical phenomenon of great interest but do not appear, from the available literature, to have received much attention. In special circumstances their form has been successfully ascribed to diverse underwater topography. There has also been some speculation on the possible importance of wave resonance. But there is at present no general theory to explain them nor is there much information

5. The alongshore system on the other hand has been more thoroughly investigated. This is primarily because practical problems of sediment transport, such as beach erosion and harbour mouth siltation, often arise from strong longshore currents. The theoretical approach has almost always been concerned with the prediction of longshore current velocities, and the capacity of the current to transport sediments. The quantity of water moving alongshore has not been a primary interest. Studies of coastalengineering problems with models, usually deal with specific situations, and the diverse topography tends to obscure the fundamental considerations.

6. There is however a large body of literature treating subjects which are related to elements of the circulation. Notable are those on wave propagation and deformation in shoaling water, and set-up at the shore caused by wave action. While the literature on these subjects is of great value it is in the main limited to the two dimensional case - in many instances a limitation consequent upon the use of wave flumes.

7. In the studies reported below the aim has been to examine the circulations as a whole. This policy was adopted both because of the original need for answers to practical problems and because this approach has been so seldom tried in the past. Inevitably the consequences of such a policy are that depth of study must sometimes be sacrificed, and order of magnitude measurements sometimes accepted.

The work carried out on the cellular circulation comprised

(a) measurements in the field, made mainly at Virginia Beach, to determine the dimensions of the cells and the circulation, including the order of magnitude of the volume of flow. Some sea level measurements were also made.

(b) experiments in wave tanks where the emphasis was on the underlying principles, especially the part played by wave interaction.

The longshore system study included

(a) wave height spectra determinations in the between breaker zone

(b) the development and testing of a method to measure longshore currents rate of flow

(c) the application of the method on selected occasions off beaches in the vicinity of Durban; and a comparison of the results with those predicted by theory.

The location of the sites of the field tests are shown in Fig. 1-1.

8. The arrangements of the data is as follows.

The first four chapters are concerned with the cellular circulations; Chapter I with a general literature summary; Chapter II with a summary of a previous study on the Natal Beaches; Chapter III with a report of the experimental work - the field work coming first; and Chapter IV with a discussion.

Chapter V deals with the literature, experiments and discussion pertaining to the alongshore system.

Chapter VI has a summary of the findings and certain suggestions for further research.

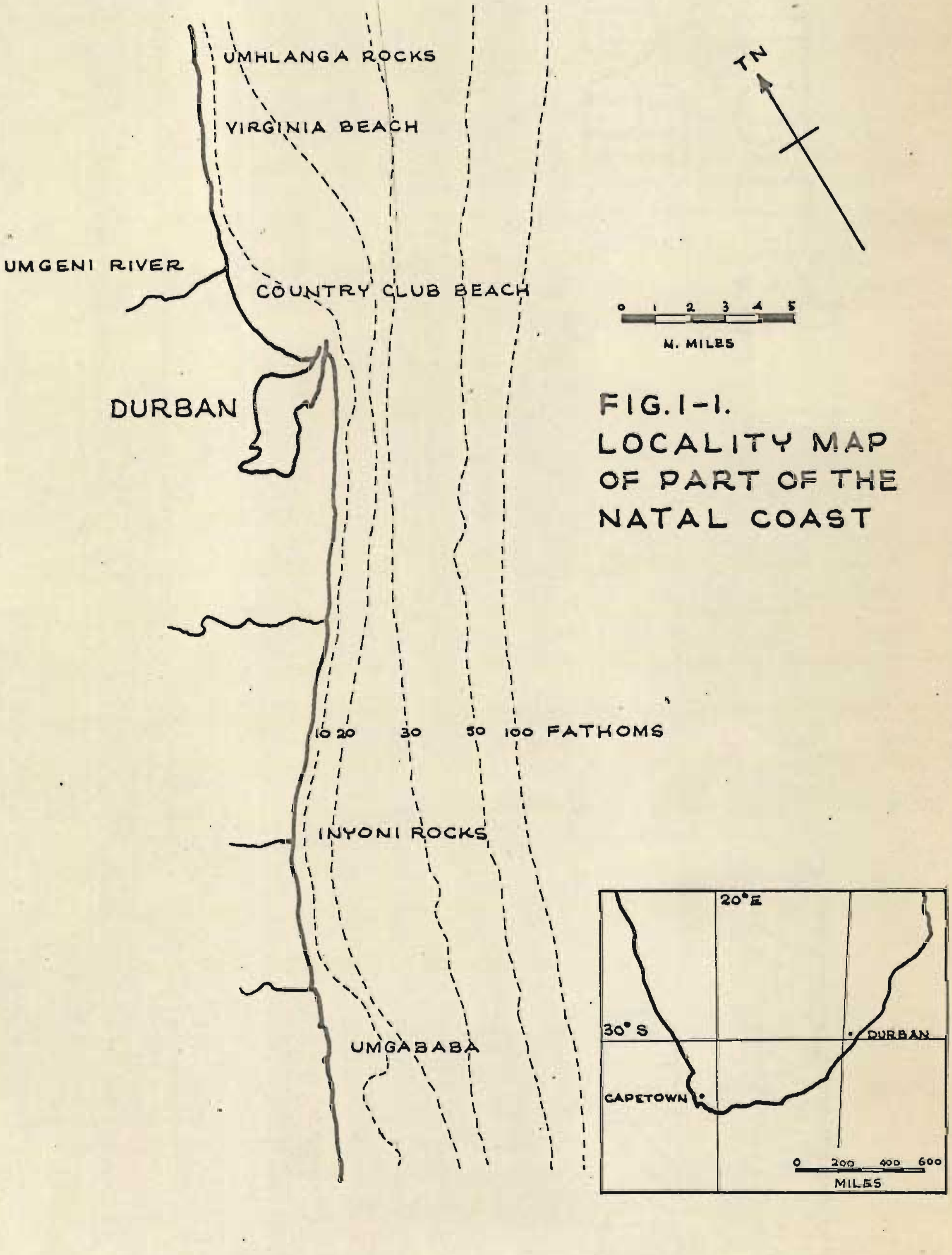


FIG. 1-1.
LOCALITY MAP
OF PART OF THE
NATAL COAST

CHAPTER I

THE LITERATURE

1. THE LITERATURE

1. General

While the literature reviewed in this section has general relevance to water movements near the shore, it will have greater applicibility to the cellular circulations. The literature specific to the alongshore system will be reviewed in the appropriate chapter.

An essential feature of the nearshore circulations is the modification of waves in shoaling water. A result of this modification is a change in wave shape and attendant changes of particle orbits and of mass transport shorewards. The process of wave modification is still very imperfectly understood, but some insight may be obtained by consideration of three types of progressive waves.

(a) the sinusoidal wave

(b) the cnoidal wave

(c) the solitary wave - a wave of translation

Some properties of these waves will therefore be reviewed. Mention will also be made of waves which may travel parallel to the shore - edge waves.

On breaking, the component of wave momentum perpendicular to the shore must cause a set-up of the water level at the shore. The hydrostatic head built up will, in theory, set in motion a uniform return seaward which to a greater or lesser extent will counterbalance the shoreward momentum. The excess of water transported shorewards over that returned seawards is carried (via longshore currents) to rip currents which return it seawards again. Thus a cell of nearshore circulation is created. Literature referring to set up will also be reviewed.

The waves and breakers approaching the surf will encounter the contrary currents of the rips, and will undergo modification - especially refraction. Mention will be made of this aspect.

Finally some of the existing ideas on the mechanism of the nearshore circulation will be examined.

2. Waves propagated over shoaling water

2.1. Sinusoidal.

The profile of the simple sinusoidal progressive wave is given by

$$y_s = a \cos (kx - \sigma t).$$

Where y_s = the surface ordinate (y being positive upwards)

a = the wave amplitude

$k = \frac{2\pi}{L}$ where L is the wave length

$\sigma = \frac{2\pi}{T}$ where T is the period

x = distance in the direction of propagation

For a wave of wave length L in water depth d the velocity C is

$$C^2 = \frac{gL}{2\pi} \tanh \frac{2\pi d}{L} \tag{1.1}$$

Where g is the acceleration of gravity

Where d is "shallow" (at a depth $d/_L < 0.05$ for practical purposes)

$$C^2 = gd. \tag{1.2}$$

Airy (1845) showed that, assuming the amplitude to be small and the fluid inviscid, the horizontal component of orbital velocity of the particles is given by

$$u = \frac{gak}{\sigma} \frac{\text{Cosh } k\,(y+d)}{\text{Cosh } kd} \text{ Cosh }(kx - \sigma t)$$

The orbital paths are closed circles in deep water and ellipses in shallow water. The motion in the direction of wave propagation under the crest is similar to the motion in the reverse direction under the trough. The closed orbits do not admit the possibility of mass transport.

The small amplitude Airy waves do not represent satisfactorily the shallow water waves which are trochoidal in shape having flatter troughs and steeper crests. They also possess mass transport. A better approximation is given by the waves of finite height of Stokes (1847).

The so called Stokes waves have a phase velocity to the second order which is the same as that given by Equation 1.1. The particles within the wave do not describe closed orbits and they have a second order mean velocity in the direction of propagation. This velocity is the mass transport velocity (U) or drift velocity.

The horizontal orbital velocities are given by

$$u = \frac{\pi H}{T} \frac{\text{Cosh } k(y+d)}{\text{Sinh } kd} \text{Cos }(kx-\sigma t) + \frac{3}{4} \frac{\pi^2 H^2}{LT} \frac{\text{Cosh } 2k(y+d)}{(\text{Sinh } kd)^4} \text{ Cos } 2(kx-\sigma t)$$

The equation describing the mass transport is (Longuet-Higgins 1953)

$$U = \frac{a^2 \sigma k \text{ Cosh } 2\,(y-d)}{2 \text{ Sinh}^2 kd} + C$$

Whence C is an arbitrary constant and y is positive downwards.

In this expression the drift velocities with depth are all positive. In closed wave channels often used for wave studies, or off a beach where as a result of wave action there is a set-up of the water surface at the shore, it is necessary to postulate a uniform return flow so that the net mass transport across a section is nil. In this case the arbitrary constant C becomes

$$C = \frac{-a^2\sigma}{2d} \text{ Coth } kd$$

The Stokes expression for the condition of no net flow predicts flow in the direction of wave propagation in the upper portion of the water and a seaward flow in the lower portion.

Model studies such as those carried out by Bagnold (1947) and Russel and Osorio (1958) have shown that the Stokes concept of mass transport does not give a true representation of the transport in shallow water. Mass transports near the bottom in the direction of waves travel were found. Evidently it is necessary to take viscosity into account.

For shallow water Longuet-Higgins (1953) investigated theoretically the nature of the mass transport velocity in a viscous fluid, the motion being assumed rotational. His theory is based on the concept that vorticity generated in a thin layer near the bed may diffuse by conduction or convection into the body of the fluid and produce a drift velocity profile different to that derived by Stokes. Assuming no net flow, a positive drift is found near the bottom and a negative drift near mid-depth, when the wave amplitude is small compared with the boundary layer (a rare condition). The so-called "conduction solution" predicts that the drift velocity will vary with depths according to the following, with y positive downwards

$$U = \frac{a^2\sigma k}{4 \text{ Sinh}^2 kd} \quad 2\text{Cosh}2kd(\mu-1)+3+kd\text{Sinh}2kd(3\mu^2-4\mu+1)$$
$$+3\left(\frac{\text{Sinh}2kd}{2kd} + \frac{3}{2}\right)(\mu^2 - 1)$$

The symbols are as for the Stokes expression and $\mu = Y/d$.

In their drift velocity studies in a closed channel Russell and

Osorio (1958) found that between the values $0.7 < kd > 1.5$ the observed values "at all depths were in agreement with those calculated from Longuet-Higgins' "conduction" solution (in spite of its limitations). A feature of the drift velocity profiles was a strong forward velocity near the bottom.

2.2. Cnoidal Waves

Korteweg and de Vries (1895) in a mathematical study of the change of form of long waves proposed a new type of wave whose form is given by (in the notation of Wiegel 1960)

$$y_s = y_t + H\,cn^2 \left[2K(k) \left(\frac{x}{L} - \frac{t}{T} \right) k \right] \tag{1.7}$$

Where y_s is the depth from surface to bottom

y_t is the depth of trough to bottom

cn is the Jacobian Elliptic function associated with the Cosine

K(k) is the complete Elliptic function of the first kind with modulus (k)

x is the distance from the wave crest

L is the wave length

t is the time elapsed after the passing of the crest

T is the wave period

The wave velocity for a wave of height H, (when the horizontal momentum of the water has been reduced to zero) is

$$C = \sqrt{gd} \left[1 + \frac{H}{d} \frac{1}{k^2} \left(\frac{1}{2} - \frac{E(k)}{K(k)} \right) \right]$$

Where E(k) is the complete Elliptic function of the second kind with modulus (k), and d is the still-water depth.

It can be shown that this equation approximates to the velocity of the Stokes wave (linear) theory,(when $k^2 \rightarrow 0$, $E(k)/K(k) \rightarrow$ unity) and of the solitary wave when $k^2 \rightarrow$ unity,(k) is unity and K(k) is infinity.

No expression for the mass transport of the cnoidal wave has been found in the literature. Wiegel (1960) does however give an expression for the horizontal orbital velocity and this is very cumbersome.

2.3. The Solitary Wave

A notable feature of waves just prior to breaking is the modification which they undergo. The troughs flatten, and the crests peak. The wave profile tends to the form of the solitary wave which was observed by Scott-Russell in 1884 during work on waves in canals. The theory of the solitary wave was developed by Boussinesq (1872) and later by others notably McCowan (1891). Munk (1949a) summarized past work and organised it in a form suitable for application to surf studies.

The solitary wave is in theory a single crest raised above the still water level. It can be propagated without change of shape and has infinite wave length.

Following Munk (1949a) the equation of the wave profile is given by

$$\mu = \gamma \ \operatorname{sech}^2 \left(\sqrt{\frac{3\gamma}{4}} \ x \right)$$

Where μ = the elevation of the free surface divided by the depth d.

γ = relative wave height = H/d.

X = distance horizontally from the wave crest divided by the depth.

H = wave height

d = depth below still water level.

Integration of this expression per unit crest length yields the volume Q.

$$Q = 4d^2 \sqrt{\gamma/3} \tag{1.3a}$$

Note this may be written $Q = 4H^2/\sqrt{3\gamma^3}$ (1.3b)

When for example $\gamma = 0.5$, 90% of the volume is contained between $X = \pm 2.4$ (see Fig. 1-2). The concentration near the crest is, according to Munk, "the basis for applying the expressions for the volume of a solitary wave to oscillatory waves of finite length".

Since the volume Q is transported during time T, (the "effective" wave period) the mean rate of transport per unit time is Q/T and the volume transport velocity averaged from surface to bottom is

$$U = Q/(Td) = 4d \sqrt{\gamma/3}\,/T \tag{1.4}$$

If the waves were breaking normally to the shores this would also be equivalent to the theoretical velocity of the uniform return flow.

In the solitary wave the trajectories of the water particles are shallow parabolas in the direction of propagation. There is no movement in the reverse direction. The horizontal components of particle velocities under the crest have been calculated by Munk (1949a) and are shown in Fig. 1-3. The mean horizontal displacement (D) of the particles is

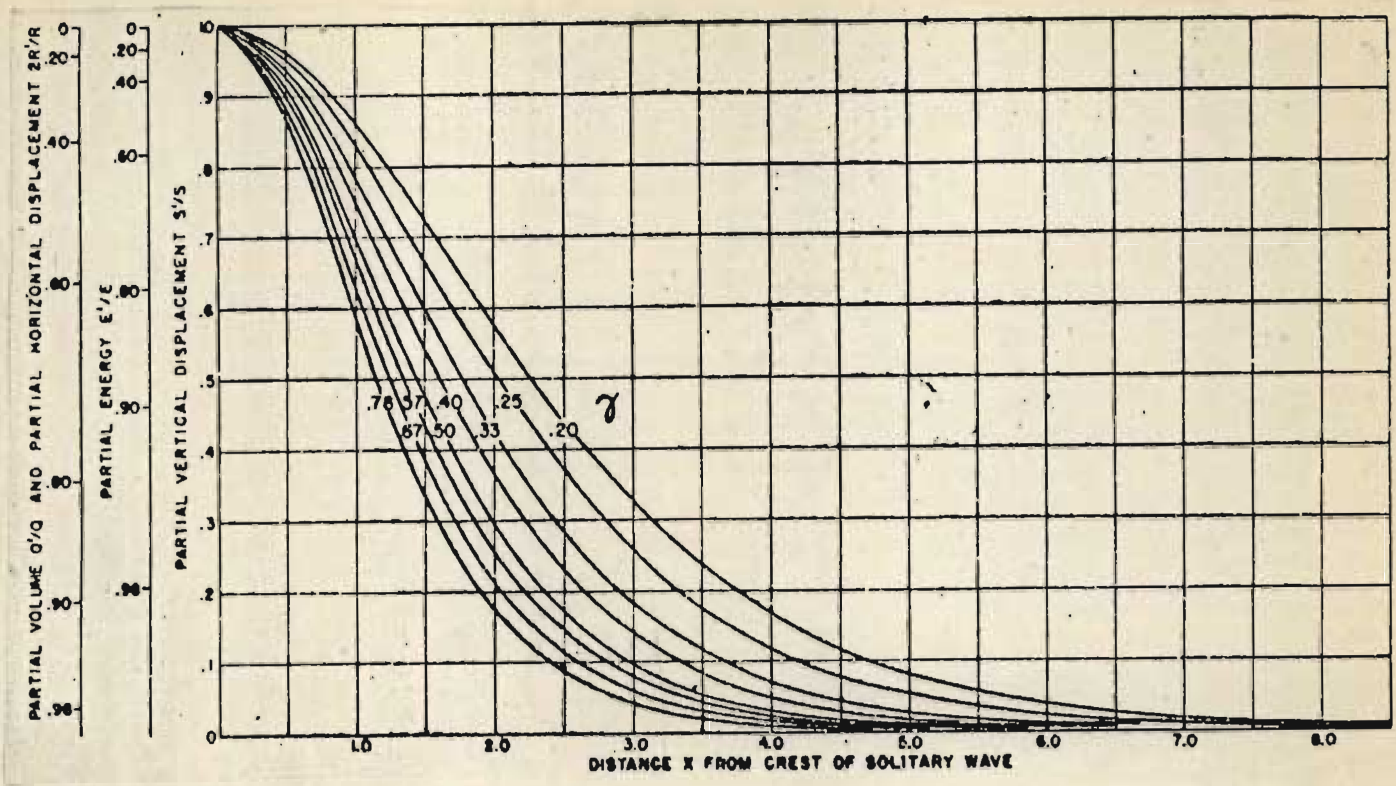


Fig. 1-2. Curves showing partial volume Q'/Q contained between the solitary wave crest and distance X (actual distance divided by the depth). (After Munk 1949a)

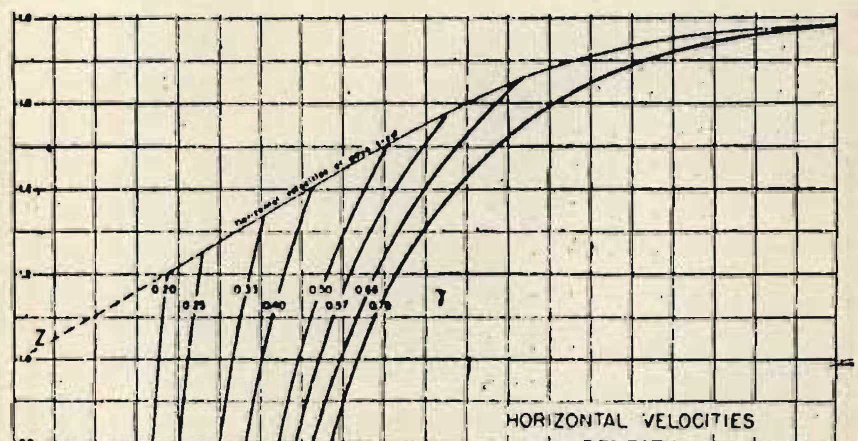


Fig. 1-3. Horizontal particle velocities under the crest. Z is the crest height above the bottom divided by the S.W.L. depth. U is the particle velocity divided by the phase velocity.

$$D = \frac{Q}{d} = 4d\sqrt{\gamma/3} \tag{1.5}$$

For the breaker of height H_b in depth d_b (where $\gamma = \gamma_b$) the rate of volume transport per unit length of wave crest is given by

$$\frac{Q_b}{T} = \frac{4d_b^2\sqrt{\gamma/3}}{T} \tag{1.6}$$

The theory predicts the height at breaking to be 0.78 d_b. The velocity (C) of the wave is given by

$$C = \left[g\,\overline{d + H}\right]^{\frac{1}{2}} \tag{1.7}$$

2.4. Discussion on Waves

The question arises as to which wave form most closely represents the natural wave near its break-point. In this connection there are several field and model studies which adduce comparative evidence.

Miller and Ziegler (1964) made a notable study of the internal velocity field in near-breaking and breaking waves. The study was made at Cape Cod and the breakers "were of the type that break directly on the shore and include within the sequence, the run-up and the backwash from the foreshore". Three types of breakers, plunging (symmetrical); spilling (very asymmetrical) and intermediate types (asymmetrical), were all experienced on the same bottom slope.

The averaged results were reported in the form of wave profiles. That for breaking wave ("symmetrical" or plunging) is shown in Fig. 1-4 and is compared with the analogous near breaking profiles for the cnoidal wave. In the figure

t is time

T is the wave period

Z is the depth

$\bar{B}_d$ is the height of the breaker crest above the bottom

wd is the height of the cnoidal wave crest above the bottom

L is the wave length

x is the distance from the crest horizontally

U is the horizontal orbital velocity with U_{max} the maximum.

The authors conclude that the "near breaking the cnoidal wave most nearly resembles the natural wave". Considerable differences were however apparent.

It may be noted for later reference that the wave height to still water depth ratio is about 0.77.

Miller and Ziegler (1964) suggest that the present consensus of opinion of other workers is that in the very shallow zone just short of the breaking point, Stokes finite amplitude theory holds reasonably up to $^d/L = 0.10$, the cnoidal theory in general in the region $0.10 > {}^d/L > 0.02$ and the solitary wave in the region $^d/L < 0.02$.

Wiegel (1965) has compared the theoretical wave profiles with some experimental tank measurements. His graphical representation is shown in Fig. 1-5. The symbols in this figure are

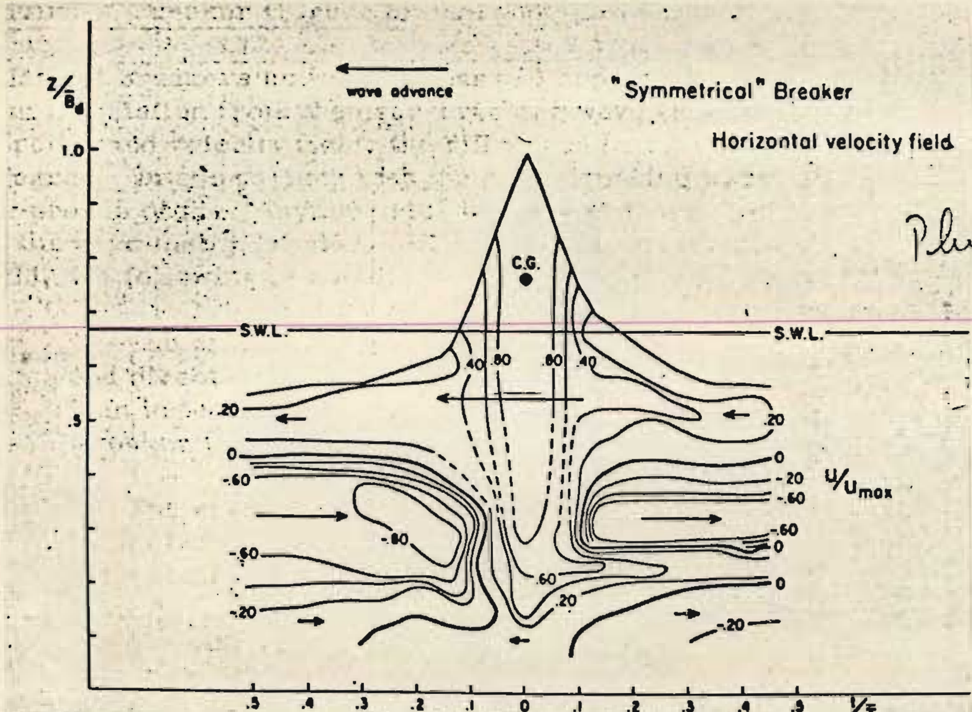


FIG. 7. SYMMETRIC BREAKER: CLASS I

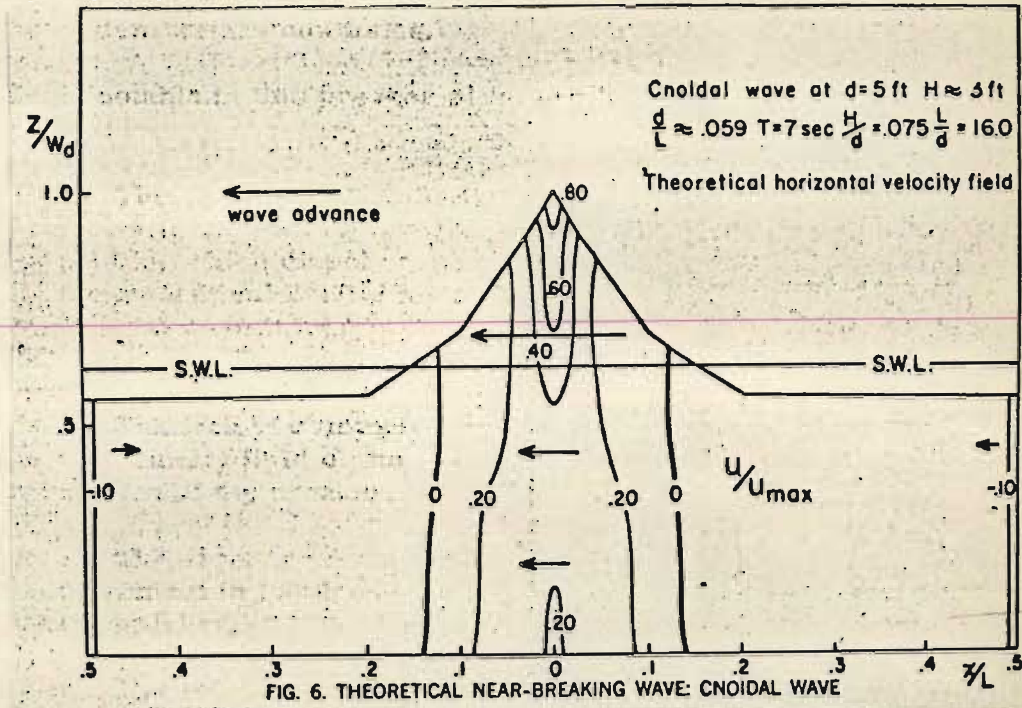


FIG. 6. THEORETICAL NEAR-BREAKING WAVE: CNOIDAL WAVE

Fig. 1-4 (above) Comparison of the measured horizontal velocity profile for a breaker (left) with that predicted by the cnoidal theory for the near breaking wave (right) (After Miller and Zeigler 1964)

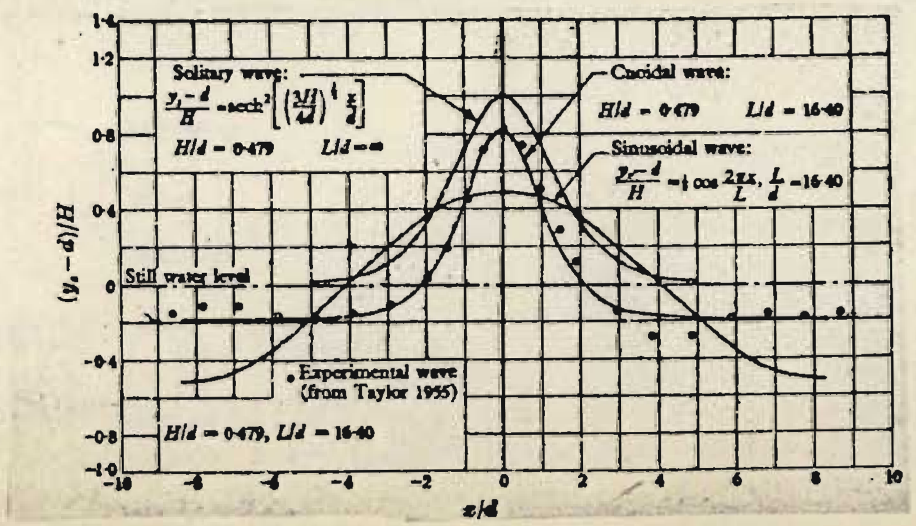


Fig. 1-5 (left) Comparison of solitary, cnoidal and sinusoidal wave profiles with that measured experimentally. (After Wiegel 1965)

H = wave height

d = depth (S W L)

y_s = surface ordinate

L = wave length

x = horizontal distance from the crest

Other evidence has been summarized by Harris (1964) as follows.

"Munk (1949a) has reconstructed the trajectory of a particle under a solitary wave. He superimposed a uniform return velocity such as might be encountered near the shore because of the hydrostatic head which could be set up. The combined movements result in orbital paths similar to those determined in wave channel experiments by the Beach Erosion Board (1933)"

"Inman and Nasu (1956) measured the orbital velocities associated with waves near the bottom in shallow water just outside the surf zone. Their conclusion was '..... the observed maximum horizontal orbital velocities in general compare more favourably with velocities predicted from the solitary wave theory than from equations of Airy and Stokes. The observed velocities were in better agreement with solitary wave theory when

(a) the wave profile was not complex

(b) the effective wave period was larger than about six seconds, and

(c) the relative wave height was greater than about 0.2.

While agreement with theory was somewhat better for the longer waves, in general it was still quite good for most simple waves with relative wave heights greater than about 0.4'."

Munk (1949a) obtained the "criterion for the outer boundary of the solitary wave regime" by calculating the changes of wave height with depth according to the classical and the solitary wave theory and comparing the results with actual observations made off the Scripps Pier. His conclusion was that the transition took place at a depth of about 1.4 times the depth of breaking. This corresponds to a relative wave height of 0.35.

The solitary wave theory predicts, according to Munk (1949a), depth at breaking by equation

$$d_b = 1.28 H_b \tag{1.8}$$

or the height at breaking $H_b = 0.78 d_b$.

Observations made in the surf at Scripps (referred to above) give some practical confirmation of this relationship. Munk (1949a) describes how "precise and simultaneous measurements" were made by holding a hollow tube with its end just in the sand in the region of breaking. The water level in the tube indicated the still water level while the breaker height was measured on the outside of the tube. The results accorded well with the solitary wave theory.

Ippen and Kulin (1955) using a long tank examined in some detail the depth at which solitary waves broke. They showed that the relationship

$$d_b = 0.8 H_b$$

or

$$H_b = 1.2 d_b$$

was true for all initial waves on a slope of 0.023. For steeper slopes the value $\frac{H_B}{d_b}$ was greater than 1.2 and increased sharply with decreasing initial wave height.

Ippen and Kulin conclude that the behaviour of true solitary waves is not well predicted by theory. On this basis they argue that except for the flattest slope the agreement between the values obtained in the Scripps surf and the theoretical values obtained from the solitary wave theory must be judged fortuitous.

Finally Keulegan (1950) has proposed that the dividing line between the applicability of the Stokes waves and the cnoidal waves should be the depth where the wavelength to depth ratio is 10.

When calculations relevant to the nearshore circulation are to be made, an essential condition is good similitude between the mathematical model and the breaking wave. Consideration of the foregoing evidence suggests

(a) that the form of the breaking wave is quite well stimulated by the profile of the cnoidal wave and little less well by that of the solitary wave.

(b) the solitary wave predicts tolerably well the depth at breaking.

(c) field tests have shown that the horizontal orbital velocities near the bottom in shallow water just outside the surf zone compare better with those predicted by the solitary wave theory than the Stokes wave theory.

It seems that the solitary wave theory might be used without involving any great error. The theory has the advantage that it has been well worked out for surf zone application. Furthermore its expression for mass transport is easy to handle, while the corresponding one for the cnoidal theory would be most cumbersome.

With these points in mind it has been decided to use the solitary wave theory in nearshore calculations which follow later. Nevertheless its shortcomings are noted.

3. Waves travelling along the shore

Apart from the ordinary waves approaching the beach there is the possibility of waves travelling parallel to the shore.

3.1. Isaacs, Williams and Eckart (1951) have drawn attention to the possibility of surface gravity waves which have been generated in the surf zone e.g. by reflections of incident waves, being unable to escape from the surf zone because of their being refracted back from deep water. This could occur if they approach deep water at angles greater than the critical angle.

The authors also note that waves from deep water may be "captured" by the surf zone where the configuration of the coast is suitable.

They also speculate on the possibility of resonant effects being produced and that they may be important in explaining shore processes e.g. Surf beats as described by Munk (1949b).

Edge waves, described below, would seem to be a special case of incident waves being reflected (by promontories) alongshore or being captured as suggested above.

Alongshore wind would also be a fruitful source of waves travelling parallel to the shore.

3.2. Edge Waves

When a uniform wave is propagated parallel to a sloping boundary it would appear that it must by refraction turn towards the shore. By equation 1.2 the velocity of the portion in deep water must travel faster than that in shallower water. It was however shown by Airy (1845) that when the wave became sufficiently "bowed" it may be propagated without further change. Consider two such bow shaped waves proceeding parallel to the coast and separated at all points by an equal distance measured parallel to the coast.

The distance between the waves perpendicular to the wave front (i.e. the direction in which the wave must travel) is considerably less at the shallower end than at the deeper. If the curve of the wave front is such that the perpendicular distance is just that which the wave could traverse at that depth in a wave period, the wave form will not alter. This of course explains why a tidal wave propagated parallel to the shore can produce cotidal lines parallel to the shore.

Kelland (1839) made a theoretical study of waves in a canal of triangular section with sides sloping at 45°. He found that a wave without any bow could be propagated without change. The wave was higher at the shore than in the centre of the canal. Stokes (1847) remarked that there was a critical limit to the slope of the canal beyond which it would be impossible to propagate such a wave. He noted that when the wave length was long compared with the depth it seemed that "the circumstances of the motion of any one wave cannot be materially affected by the waves which precede or follow it". He also notes a similarity between the form of the ridge of these waves and that of a solitary wave. This would have mass transport implications.

Stokes derived an expression for these waves for boundary slopes of any angle. The name given to them is edge waves and a brief account is given of them at this stage because references will be made to them below in connection with certain phenomena revealed in wave tank experiments.

Edge waves derive their name from the fact that their amplitudes diminish exponentially as distance from the boundary increases (Lamb 1962). According to the small amplitude theory their heights which are greatest at the shore become insensible at the distance of a wave length measured perpendicular from the shore.

The general form of edge waves was given by Lamb (1962) as:

$$\phi = H\left[\text{Exp} -k\ (y \text{ Cos } i - z \text{ Sin } i)\right] \text{Cos } k\ (x - Ct)$$

when ϕ is the velocity potential

H = wave height

k = 2π/wave length L

i = angle of boundary slope to the horizontal

x, y and z are the axis respectively parallel, perpendicular to the boundary and vertically upwards from the bottom.

C is the velocity and t the time.

Stokes gave the wave velocity as

$$C^2 = \frac{g}{k} \text{ Sin } i \qquad (1.9)$$

where g is the acceleration of gravity.

Ursell (1952a) showed that the Stokes edge wave was the zero mode of a series of possible modes whose velocity was given generally by:

$$C^2 = \frac{g}{k} \text{ Sin } (2n + 1)\ i \qquad (1.10)$$

where n is an integer indicating the mode.

Since CT = L where T is the period, the velocity can be expressed as:

$$C = \frac{gT}{2\pi} \text{ Sin } (2n + 1)\ i \qquad (1.11)$$

and the wave length is given by:

$$L = \frac{gT^2}{2\pi} \text{ Sin } (2n + 1)\, i \qquad (1.12)$$

Ursell also showed that for a semi-infinite canal closed by a sloping boundary (beach) the system (inviscid) might have a natural spectrum containing both discrete and continuous eigen-frequencies i.e. it had a mixed spectrum. When the motions were anti-symmetrical about a midpoint along the sloping boundary i.e. the edge waves originated alternately from either side, the discrete frequencies were given by the expression

$$\sigma^2 = gk\,(2n + 1)\, i \qquad (1.13)$$

where $\sigma = \frac{2\pi}{T}$ for period T and $k = \frac{\pi}{b}$ for a canal of width b.

The continuous spectrum was defined by

$$gk < \sigma^2 < \infty \qquad (1.14)$$

Stokes' edge waves are thus the first of a sequence of discrete modes. For any acute angle i the number of modes is the greatest integer contained in

$$\frac{1}{2} + \frac{\pi}{4i}$$

The discrete modes lie outside the continuous spectrum.

Ursell further demonstrated that resonance may occur when the frequency of an externally applied forcing oscillation coincided with the discrete frequencies or the cut off frequency of the continuous spectrum. His theory indicates that the resonance is confined to the region of the beach except with the cut off frequency. Even in the latter case viscous forces tend to confine the resonance to the neighbourhood of the beach. The resonance frequency can be modified by viscosity..

An interesting feature of these edge waves which appears to be inherent in equation 1.12 is that for a given period of a <u>forcing</u> oscillation, on a beach of given slope, there may occur a range of edge waves with wavelengths corresponding to the different values of "n". Unlike simple harmonic motion the wavelengths are not related directly to the mode number (n), but have a more complicated relationship by virtue of "n" occurring in the equation as part of the angle whose sine is to be taken. It seems that the superposition of the waves from a number of modes could result in a complicated motion.

4. Wave set-up at the shore

The set-up of the sea level at the shore consequent on the breaker momentum has been examined both theoretically and practically.

4.1. Munk (1949a) reports "the elevation of the mean position of the waterline on the beach, above mean sea level in deep water" is given by

$$y = \frac{2V_b^2}{g} \qquad (1.15)$$

Where V_b is the mean velocity of water at breaking and g is the acceleration of gravity. (He does not show how the expression was derived).

For the solitary wave at breaking Munk found $\gamma = 0.78$ (approximately) and in view of equation 1.4 the set-up becomes

$$y = \frac{13.7}{} \left(\frac{H_b}{} \right)^2$$

where H_b is the wave height at breaking. Munk reports that the rise was measured in wave tank experiments with the following results

y computed = 0.05 ft

y observed = 0.052 ft

The wave height was 0.3 ft and the period 0.86 secs.

4.2. Longuet-Higgins and Stewart (1964) have examined theoretically the second order currents and changes in mean surface levels associated with waves. Their theories make use of the concept of radiation stress (analogous to radiation pressure in electromagnetic waves) which they describe as "the excess momentum due to the presence of waves". Of their conculsions the following are relevant to nearshore waters.

(i) A wave of steady amplitude propagated into water of gradually varying depth will produce a negative tilt or depression of the mean surface as the depth diminishes, provided that there is no loss of energy by friction or reflection.
If the wave height is limited by breaking, the tilt will be positive and a set-up will result in the direction of propagation. The significance of this is that it is to be expected that the water level will be setdown in region outside the surf zone where the water depth decreases and set-up inside the surf zone. Comparison of theory with model experiments made by Saville (1961) show good agreement.

(ii) With groups of high and low waves in a wave train, there will be a setdown of the mean surface under the high waves (with relatively negative mass transport) and set-up under the low waves, when the lengths of the groups are large compared with the mean depth, and when the groups are of equal length. When groups of waves are propagated into shallow water, the radiation stresses tend to increase as the depth decreases. The significance of this is that a variation of setdown may be expected in the region outside the surf zone depending on the height of wave groups, and that the effect will be more pronounced as the water shallows.

In the surf region where the wave height is limited by breaking, the authors note that no adequate theory exists, but they are nevertheless able to draw the tentative conclusion that the gradient of the surface is proportional to the bottom gradient. A figure of 0.15 for the proportionately constant is suggested on the basis of experiments performed by Saville (1961). Applying the theories for inside and outside the surf zone we note that as a result of the arrival of a group of higher waves, the mean water level outside would be lowered while inside it would rise. With the arrival of a low group wave the reverse (relatively) would occur. This may be a significant fact when related to the mechanism of rip currents.

4.3. Experiments on Set-up

Saville (1961) measured the set-up in a wave channel. His results (some of which have been referred to above) are illustrated in Fig. 1-6 where the set-up is plotted against the distance from the shoreline under still water conditions for various values of wave height, and for two beach slopes and two cases with submerged barriers or break-waters. Noteworthy are

(a) the setdown just seaward of the break point (as predicted by Longuet-Higgins and Stewart - see above).

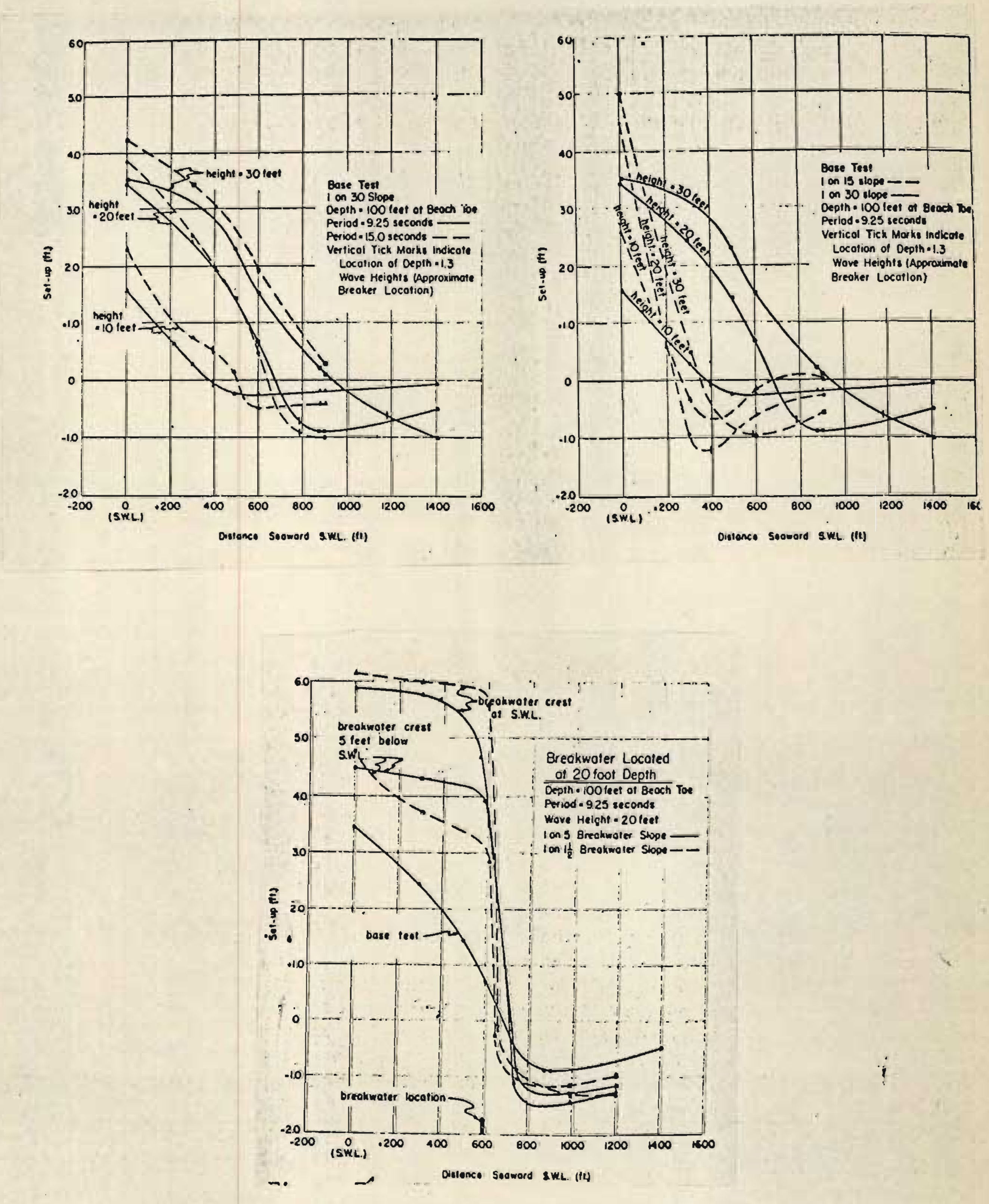


Fig. 1-6. Wave set up in a wave channel - the units have been scaled up to the prototype. (After Saville 1961)

(b) the set-up inshore of the break point

(c) very rapid set-up inshore of a submerged barrier or breakwater.

(d) the set-up at the original still water level foreshore was 10-20% of the wave height.

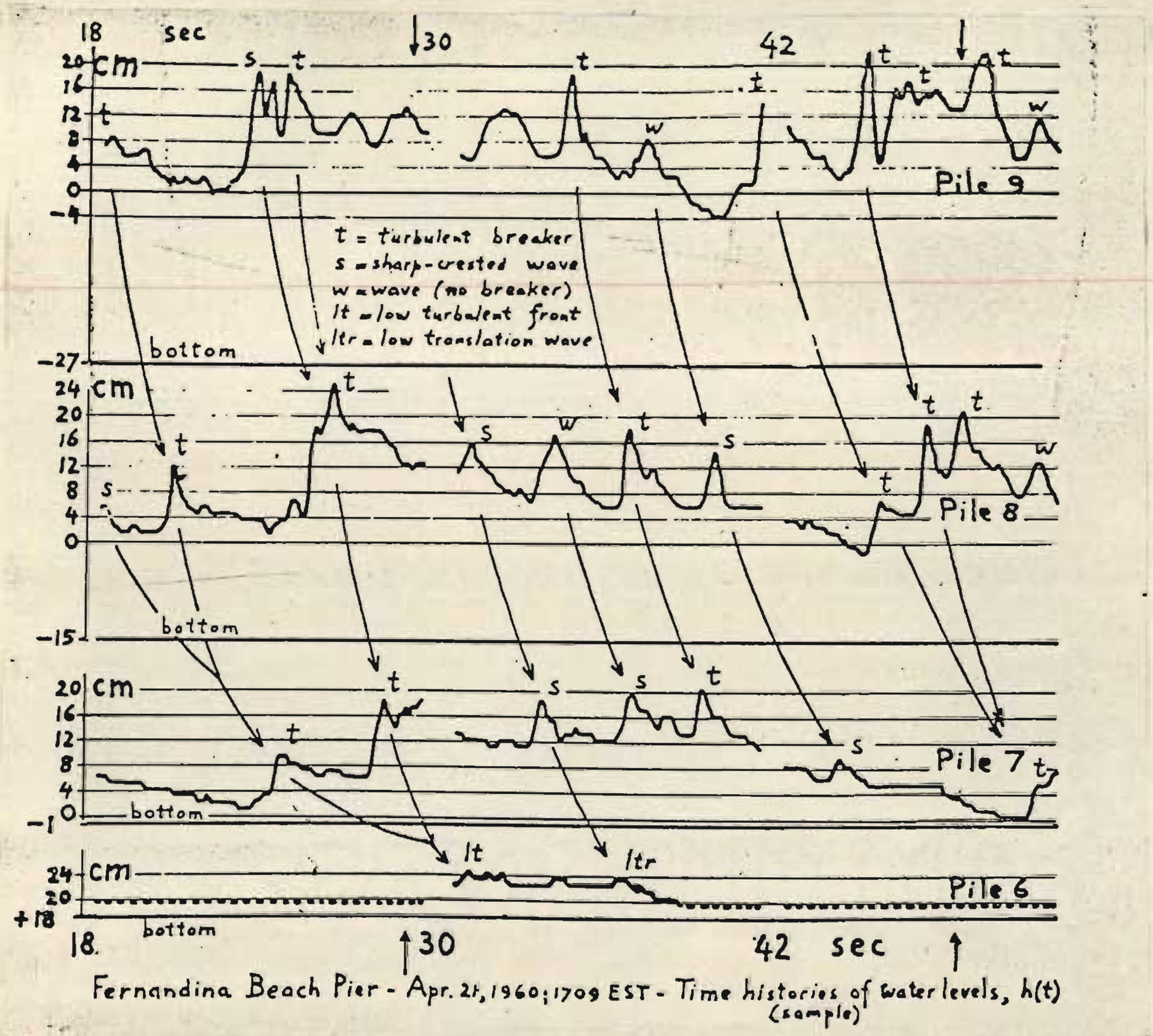
Dorrenstein (1961) made measurements of the wave set-up in the field. The method used by him was to install two tide gauges along a pier. At the beach use was made of perforated plastic pipes pushed into the sand and levelled to a bench-mark. A float on the surface of the water inside the pipe indicated the sea water level - short term changes being damped out. The response of the float is shown in Fig. 1-7. The results were as follows:

Set-up observed (cms)	12	8	10	13	4
Set-up theoretical (cms)	6	6	12	10	3
Wave period (secs)	6.6	6.8	6.8	6.0	2.8

His conclusions from the tests are that the set-up is roughly one tenth of the deep water wave height.

The fact that as a result of the breaking wave the water level at the shore is set-up, seems well established both by theory and experiment. There remains the problem of how the system reacts to this equilibrium. It seems on the face of it unlikely that it will be maintained because of the inequalities occasioned by diverse topography, spatial variation of wave heights, and wave disturbances which must be present travelling across the system alongshore.

It is well known that in practice the equilibrium is not maintained but that there is a return flow seawards in narrow fast flowing rip currents spaced at varying intervals along the shore. These are usually fed by longshore currents flowing in from either direction, the various elements forming a horizontal cell of circulation.



.Fig. 1-7. Samples of the time histories of water elevations in the surf. The zero line corresponds to mean water level outside the surf.
(After Dorrenstein 1961)

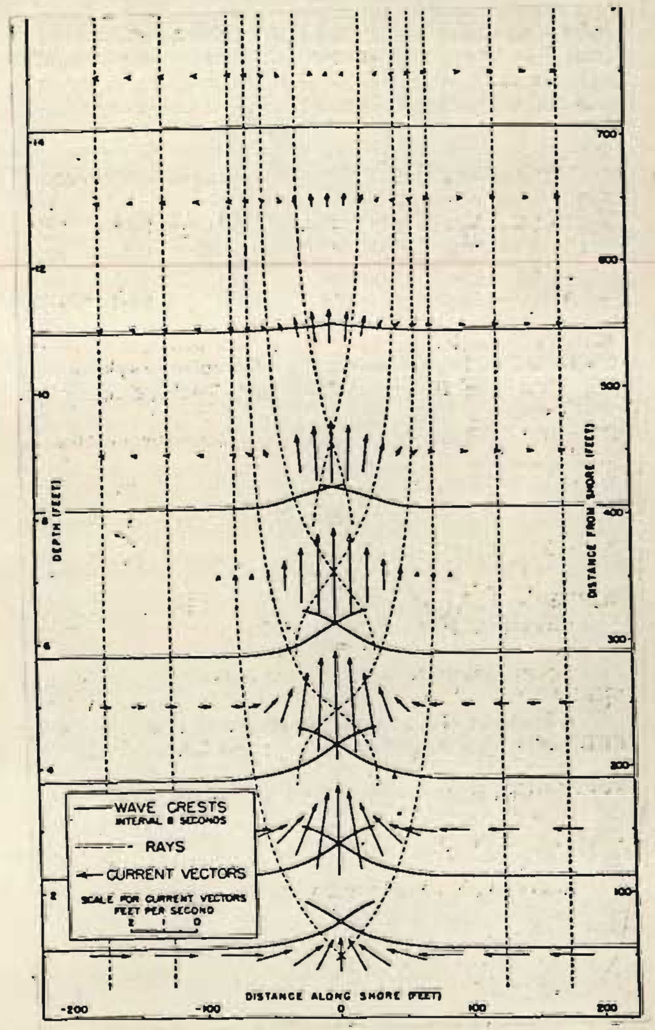


Fig.1-8. Diagram showing computed results of refraction by both rip currents and under-water topography.
(After Arthur 1950)

4.4. Surf Beats

Munk (1949b) reported that a tsunami recorder placed 1000 ft off shore had recorded waves with irregular oscillations with long periods of several minutes which correlated with the fluctuations of height and period of the incoming waves. These oscillations he termed "surf beat" since they appeared to have their origin in the rise and the fall of the water level in the surf zone consequent on the breaking of high and low groups of ordinary waves. The height of the long waves were of the order of one tenth that of the swell and appeared to be proportional to the landward mass transport in the surf (which in turn was proportional to the square of the ordinary wave height). Tucker (1950) measured similar long waves having a 2-3 minute period which (assuming a phase lag of 4-5 minutes) correlated with high wave groups. He put forward a slightly different explanation. He suggested that the increased mass transport from high groups just outside the surf required an acceleration which would give rise to an equal and opposite acceleration seaward, and that the elevation resulting from the high groups would be reflected seawards. The resulting sequence of events would be the movement seaward of an elevation, then a depression and then an elevation. His analyses showed that a depression was a salient feature of the seagoing oscillation. This depression seems to be in accord with that predicted by the theory of radiation stress outlined above. Tucker found that the height of the surf beat was proportional to the heights of the ordinary waves - <u>not</u> their squares. His suggested explanation for this is that the low groups of waves which break in the shallower water, would have more time during which their height would increase during the approach to breaking and would consequently carry more water than would be predicted by the square law. The difference between the mass transport by large breakers and small breakers would be less than theory would suggest and hence the rise of the sea level in the surf would be more linearly related to the wave heights than as their squares. This linear relationship finds some support in the tentative theory of Longuet-Higgins and Stewart. It may not be applicable on steep beaches with offshore sandbanks where waves of a variety of heights tend all to break on the bar.

The occurrence of surf beats provides evidence of the pulsed nature of the sea level changes in the surf and suggests an explanation for the variations in longshore current and rip current flow.

An interesting observation of Tucker (1950) was that the surf beats correlated best with waves from distant storms and worst with waves, generated over a larger area near the coast, and having short irregular groups.

4.5. Dmitriev and Bonchkovskaia (1954) summarized some Russian work on wave currents as they affect sediment transport. A brief account of their ideas taken from a translation (which was not always clear) is as follows. Experiments in a small wave trough (15.2 cm wide and 2 m long) with a moveable sand bed, revealed that the flow rate of sediments oscillated between offshore and onshore with a periodicity of several minutes for slopes of 15° - 30°. The period decreased as the slope of the foreshore increased. The authors suggest that "these results reveal the mechanism of a phenomenon related to rip currents". The work does suggest that with a moveable bed the changes in profile have a certain periodicit which reflects a periodic change in the direction of net mass flow.

The authors suggest that the cause of this periodicity of flow "may be connected with great surges necessary for compensating withdrawals to take place. Incidental factors such as the temporary slackening of wave action may occasion reflux motion away from the slope which sharply lowers the surface incline. Then the monotonous wave action results in gradual shoreward accumulation of water and, so to say a new disruption process takes place". They say that "observation in nature showed the formation of overflow (sic) surrents between every two approaches of a sequence of crests. At abrupt shores such currents cause the tilting and complete breakdown of a wave, and on shoals they are braked by the next oncoming wave. The authors then go on to develop the problem theoretically, making the assumption that "total reflux can only occur when the surge is sufficiently great for the reflux rate to attain during one period the Stokes progressive flow rate".

The mean inclination of the free surface is

$$- b_o = \frac{\partial \varepsilon}{\partial x} \tag{1.16}$$

where x is the horizontal axis perpendicular to the shore. The z axis is downwards. ε is the elevation of the surface. It is shown that the slope angle of the surface is directly proportional to the coefficient of viscosity and the square of the wave height and inversely proportional to the wave length and period.

The authors assume the following equation for the study of a periodic oscillation:

$$\frac{\partial}{\partial x}\left(\frac{\alpha V^2}{2g}\right) - i_{surf} + \frac{\alpha}{g}\frac{\partial V}{\partial t} + i_f = 0 \qquad (1.17)$$

where V is the velocity of horizontal transport

g is the acceleration of gravity

i_{surf} is the inclination of the surface

α is a correction factor for assuming the velocity is uniform in the section

i_f is the friction slope

t is the time.

(Equation 1-17 appears to be the differential equation for unsteady flow in uniform channels.)

Assuming

$$i_f = \frac{V^2}{C^2R}$$ from Chezy's relationship where C is a friction coefficient and R the hydraulic mean depth

$$i_{surf} = b_o = \text{initial incline of the surface}$$

$$\frac{\partial}{\partial x}\left(\frac{\alpha V^2}{2g}\right) \simeq 0$$

and $\alpha \simeq 1$ (current uniform)

then $$\frac{\partial V}{\partial t} = gb_o - \frac{\sigma V^2}{C^2R} \qquad (1.18)$$

Introducing a value for the fictitious velocity which would be attained after an infinite time if slope b_o is maintained

$$V_\infty = C\sqrt{Rb_o} \qquad (1.19)$$

Then to simplify take the upper value $b = b_o$

Solution of equation 1.18 gives

$$\ln \frac{1 + {}^{V}/V_{\infty}}{1 - {}^{V}/V_{\infty}} = \frac{2\, g\, t\, b_o}{V_{\infty}} \qquad (1.20)$$

$$\text{whence } V = V_{\infty} \tanh \left(\frac{g\, t\, b_o}{V_{\infty}} \right) \qquad (1.21)$$

which gives the growth of the outgoing velocity with time.

$$\text{This can be rewritten } b_o = \frac{V_{\infty}}{g\, t} \operatorname{arth} \frac{V}{V_{\infty}} \qquad (1.22)$$

When the outgoing velocity V is equal to the mass transport velocity U (in the original U is the mass transport velocity of the Stokes wave)

$$b_o = \frac{V_{\infty}}{gt} \operatorname{arth} \frac{U}{V_{\infty}} \qquad (1.23)$$

When b_o is small equation 1.23 becomes

$$b_o = \frac{U}{gt}$$

and

$$U = b_o g t \qquad (1.24)$$

In order to establish a slope of b_o, a water volume of the order of $\frac{b_o x_o^2}{2}$ should pass unit width at distance x_o. When total reflux occurs, the following relationship obtains

$$\frac{b_o x_o}{2} = t^{*}\, \bar{U} \qquad (1.25)$$

where t^{*} is the minimum time required for the required volume to pass and is the time of the collapse. The duration of the reflux can be evaluated by integrating the reflux velocity in equation 1.21 and by equating the result with the volume of water flowing out.

The authors state that the duration of the collapse usually proves in calculation to be less than the duration of the surge.

The authors report that the auto-oscillation period shows a rapid increase with decreasing incline and note the similar tendency found in their flow rate experiments referred to above.

The authors suggest that the collapse of the set-up at the shore in inevitable, and that when it occurs it takes place in an "incidental manner, i.e. it sets in at any one point and a lowering of the level ensues. As a result of the lowering, motive forces arise along the shore and produce shore currents which converge at the point of collapse and feed the stream escaping the sea". The level being equalized, the current ceases until a new surge produces another collapse. They note that shoreline promontories promote rip formation.

5. Wave refraction in rip currents

In developing a theory of refraction of shallow water waves under the combined influence of currents and underwater topography, Arthur (1950), takes as an example the interaction of outgoing rip currents and incoming waves. He treats the problem as one analagous to that of determining the minimum flight path of an aircraft flying in a variable wind field, and involving Fermat's principle. The direction of wave propagation corresponds to the aircraft heading and the wave velocity relative to the current, to the air speed. The current corresponds to the wind field and the ray path to the minimum flight path or path of least time.

Let the rectangular coordinates of a moving point on a ray be given as a function of time, t, i.e. $x = x(t)$ and $y = y(t)$. Let the x and y components of the current velocity V_H be V_x and V_y and let θ be the angle between the relative current velocity, C, and the positive x-direction. Then x = x(t) and y = y(t) must satisfy

$$\frac{dx}{dt} = V_x + C \cos\theta$$

$$\frac{dy}{dt} = V_y + C \sin\theta$$

A third equation is deduced. This is based on the aircraft analogy, but make allowance for the fact that the wave velocity (wind field) varies with x and y

$$\frac{d\theta}{dt} = -\left[\partial(V_c + C)/\partial_y\right] \cos\theta + \left[\partial(V_c + C)/\partial_x\right] \sin\theta \quad (1.26)$$

When V_c is the component of V_H in the direction of C.

Using these three equations Arthur constructs a diagram (see Fig. 1-8) showing the wave crest pattern and the rays for a wave front encountering a rip current. The example used is based on data obtained in the field by Shepard, Emery and La Fond (1941) and Shepard and Inman (1950) and it is assumed that the beach slope is 1 in 50, the maximum rip current velocity is 3.5 ft/sec (apparently just clear of the breakers), the rays are initially normal to the coast in a depth of 15 ft, and the width of the rip neck 100 ft. The equations used have been those that satisfy continuity on the assumption that V_x and V_y are uniform with depth. To deal with the problem of velocity distribution Arthur assumes it to be given by

$$F(\alpha) = 1/\sqrt{2\pi}\ \mathrm{Exp} - \alpha^2/2 \quad (1.27)$$

i.e. a Gaussian distribution.

Arthur remarks that "waves in rip currents in some cases show features similar to those illustrated in the figure. The crests generally show a lag in the neck of the current.....". "Many rips seem to show only a gentle retardation of the wave crests.....". And he suggests that this may be due to much lower current velocities and velocity gradients. "Another factor is the channel which results from erosion of the sand bottom below the currents in the feeders and neck. The effect on velocity of the greater depth in the channel tends to offset the effect of current shear in refracting waves".

Although the author did not remark on it, this interaction of rip currents and waves probably plays an important part in the circulation mechanism. It is discussed later in connection with experimental work.

6. The Nearshore Circulation

6.1. The nearshore circulation was first described comprehensively by Shepard and Inman (1950), following investigations carried out on the beach near the Scripps Institution of Oceanography. Subsequently further quantitative measurements were made by the authors (1951) and by Inman and Quinn (1952). Inman and Bagnold (1963) summarized the existing information amongst which was the following.

(a) The circulation for waves incident normally or nearly normally, is cellular - the chief elements being mass transport, shorewards, longshore movement feeding narrow fast rip currents flowing seawards, and expanding outside the surf zone. There is usually inflow to the rips from longshore currents from either direction though the of the circulations that the flow is intermittent. This is ascribed to the variation in height of the incident waves.

If the wave direction is not normal, the "accompanying discharge has in any case a longshore component over and above that due to any local disequilibrium in force components normal to the shore and a definite longshore current must result". This longshore current is confined largely within the surf zone.

(b) The spacing between rip currents i.e. the width of the cell ranges from 30-1000 m. "Measurements following on a period of low waves along a section of a straight beach at Clatsop Spit, Oregon, gave a mean separation between rip currents of 400 m, and a standard deviation of 145 m (Shepard and Inman 1951)".

(c) The spacing between rip currents decreases as the intensity of wave action increases. It is assumed that the term increasing intensity of wave action means increasing wave height and/or increasing frequency.

(d) It is calculated that rip currents discharge between 2-10% of the total volume transported in between rip currents, this being of the same order as the estimated longshore component of shoreward transport. The implication here is that "the normal component of shoreward waves transport has no real existence relative to the shoreline because the general body of water is being displaced outwards at an equal rate" (Inman and Bagnold 1963).

6.2. Not a great deal of attention has been given to the cause and mechanism which underlies the cellular circulation. The task of discerning why a cellular type of circulation occurs is of course made difficult by many factors such as the physical problems arising from the vigour of the surf, the wide range of variables including the movement of the sea bottom, and the irregularity of many coastlines.

The field measurements of Inman and Quinn (1952) show that the longshore current velocity increases progressively from zero immediately downstream from the rip current to some maximum value just preceeding its seaward deflection at the next rip current. Based on this Inman and Bagnold (1963) have suggested that the longshore current flow "has some limiting value above which it breaks seaward into rip currents". This then would provide a mechanism once the longshore current was operating, but it offers no explanation as to how the longshore current starts.

The original research work (Shepard and Inman 1950) which revealed the cellular circulation, was conducted off a beach (the Scripps Beach at La Jolla) off which the topography was characterised by deep canyons. The canyons which ran perpendicular to the coast affected very significantly the height of the breakers. In the deep water over the canyons,the waves diverged, and over the shallower intervening water,the waves converged. Consequently opposite the canyons the breakers were lower. This canyon effect led Shepard and Inman (1950) to the conclusion that the underlying mechanism of circulation was spatial variation in wave heights consequent upon irregular offshore topography.

Several workers, MacKenzie (1958), Bruun (1963),Dmitriev and Bouchkovskàia (1954) noted the coincidence of elements of the circulation with topographical features within the surf zone.

Bruun and MacKenzie remarked on the location of rip currents where the surf bed or submerged sandbar is lowered. Dmitriev and Bonchkovskaia noted that promontories promote rip current formation. The problem nevertheless remained, and was remarked upon, as to whether the circulation formed the topography or the topography the circulation.

6.3. Bruun (1963) has put forward a theory to describe the circulation when rip current activity is prevalent off a beach with an offshore submerged sandbar. He draws attention first to the balance that must exist between the topography i.e. the position of the bar, the depth over the bar and trough and their slopes, and the waves and longshore currents which shape them. He observes that:

(a) Longshore currents flow into the root of the rip currents, and that even if the general direction of the current is dictated by some obliquity in the direction of wave approach, there is "a local reversal of flow at every root of a rip current".

(b) That rip currents occur at intervals determined by wave and offshore bottom characteristics. He quotes Larras (1957) as giving 500 meters for the average spacing.

(c) That rip currents occur at lowerings in the bar.

(d) At the root of the rip the shoreline conforms to local flow conditions and an 'S' shape or slight tomoblo may develop.

In developing the theory, Bruun assumes that the water derived from the breaking wave over a certain length parallel to the shore, flows out again through lowerings in the bar system. Bruun leaves open the question as to whether the more or less regularly spaced lowerings in the offshore bar system are caused by rip currents, or whether the currents seek these incidental openings for discharge. The development of the theory is as follows:

Consider a situation with an offshore sandbar where rip currents are operating at a distance "ℓ" apart. Let the waves approach obliquely and make angle α with the bar. Let the distance from the down wave rip current ($x = \ell$) to the point of origin of the longshore current ($x = 0$) be ℓ', and the area of the cross section of the trough at any point x be A_x. Assume that the quantity of water at x brought in by unit length of breaker crest be Q_{Bx}. The component directly to the shore will be Q_{Bx} Cos α. If T is the wave period then the velocity of the longshore current at x will be

$$V_x = \frac{Q_{Bx} \text{ Cos } \alpha \; x}{A_x T} \tag{1.28}$$

Assuming x and A are related linearly then the maximum velocity of the longshore current will be

$$V = \frac{Q_B \text{ Cos } \alpha \; \ell^1}{AT} \tag{1.29}$$

Where A, α, and Q_B are averages. The mean velocity (V_m) will be related to the frictional forces by Chezy formula and

$$V_m = Cf \sqrt{Ts/\rho g} \tag{1.30}$$

where Cf is the Chezy formula friction coefficient Ts is the determining shear stress for bottom stability (Bruun and Gerritsen 1960) ρ is the density of sea water and g the acceleration of gravity.

The assumption made in this treatment is that the water transported in normally to the shore flows out at the rip currents. This assumption, is not in accord with experience, for (as will be shown later) only a small percentage of the water derived from the breaking wave takes part in the circulation.

CHAPTER II

A SUMMARY OF A PREVIOUS QUALITATIVE STUDY ON THE NATAL COAST

A SUMMARY OF A PREVIOUS QUALITATIVE STUDY ON THE NATAL COAST

A qualitative study by the author of the nearshore circulation off a Natal beach (Virginia Beach) with a submerged offshore sand bar, was recorded in a thesis for a Masters degree (Harris 1964).

Since the present investigations are in some respects a continuation of this study and because the bulk of the field work was done on the same beach, the earlier findings are summarized below.

1. The test beach at Virginia

1.1. Sediments and Profiles

The beach forms part of an almost straight section of the coast between the Umgeni River and Umhlanga Rocks - a distance of about 6 miles (See Figure 1-1). The intertidal zone consists of sediments with a median diameter of about 0.75 mm and has a slope of 10-20% - facts which are in accordance with the work of Bascom (1951). There are no rock outcrops.

Though the offshore profile in the surf was of course variable the presence of a submerged sand-bar 500-700 ft from the shore was a frequent and important feature. Inshore of the bar was a trough. The ratio of the depth of water over the trough to that over the bar was about 1.7 - somewhat larger than the 1.4 that found by Shepard (1950) but comparable with the 1.69 found by Keulegan (1949). An example of a commonly occuring profile is shown in Figure 2-1 (in determining profiles, the depth in the surf was measured on a measuring staff, by a swimmer whose position was fixed from the

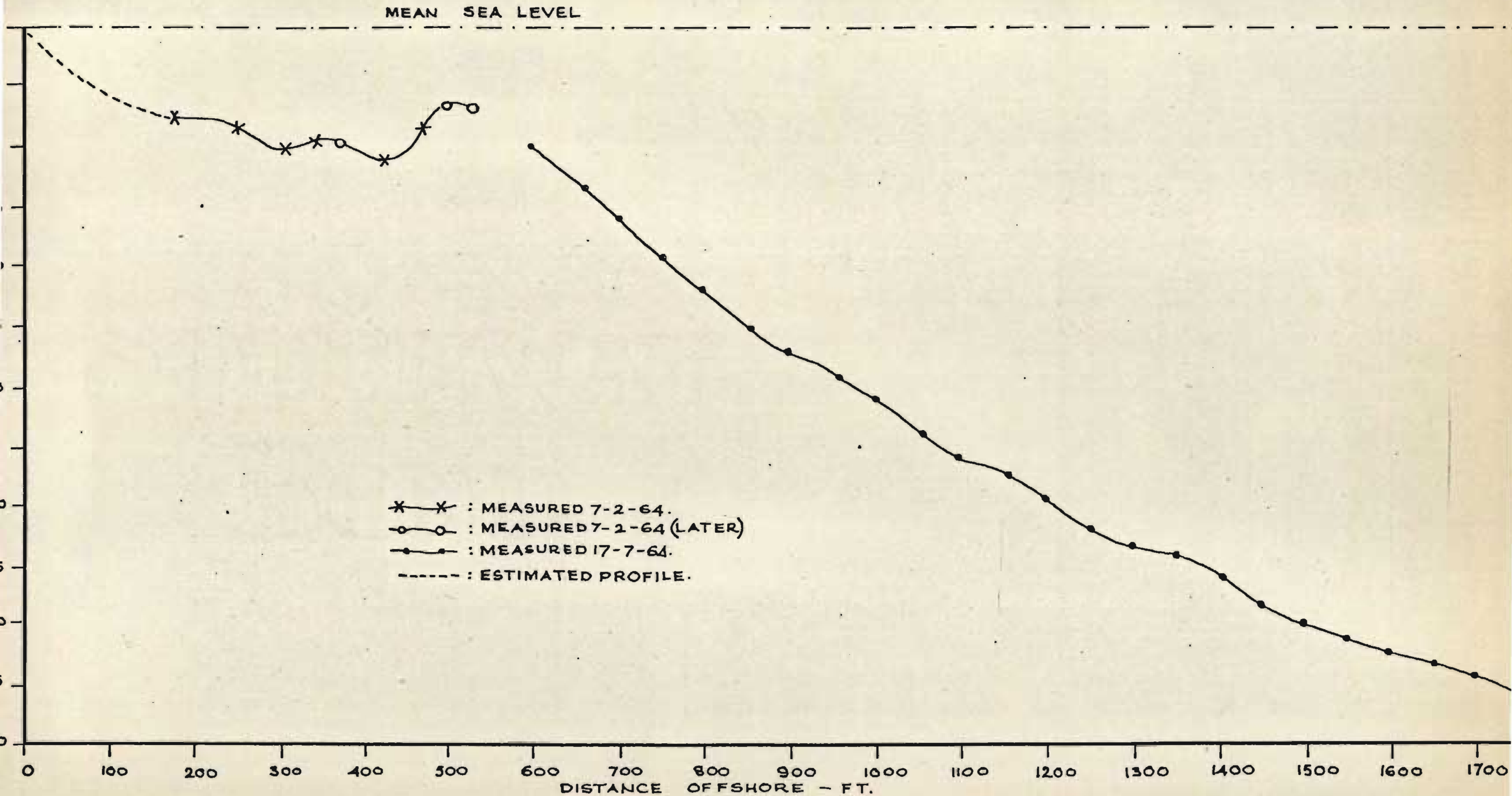


FIG. 2-1. SEABED PROFILE OFF VIRGINIA.
AFTER HARRIS (1964)

shore. Seaward of the surf depths were determined by echo sounding).

Fig. 2-2 shows the profile at another Natal Beach (Invoni Rocks).

1.2. Waves and Breakers

Waves on this stretch of coast are characterized by mean significant heights of 5-6 ft and mean significant periods of 7 seconds. During the tests they were respectively 4 ft and 8 seconds. Maximum significant wave heights were 12 ft and the maximum significant period was 12 seconds.

In spite of refraction, breakers were sometimes found to make large angles with the shores; 10° being common and up to 30° possible.

The usual regime was for waves breaking over the bar to form plunging breakers which generated waves which travelled unbroken over the trough before finally breaking on the foreshore. Frequently the relative height of the regenerated waves was too small for solitary waves to form, so they were usually of the oscillatory type having low mass transport. Four zones were identified. (See Figure 2-3) - the offshore zone, the outer breaker zone (over the sand bar and some distance inshore of it), the between-breaker zone (a zone of regenerated oscillatory waves) and the inner breaker zone with breakers on the foreshore.

1.3. Coastal and Tidal Currents

Coastal currents were measured $\frac{1}{2}$ mile offshore and were usually

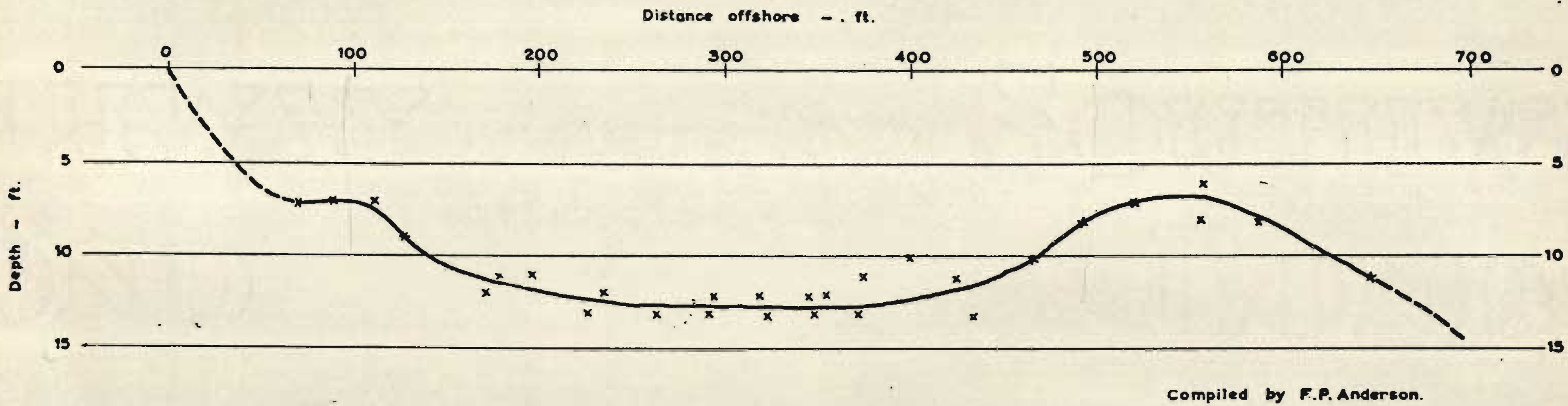


FIG. 2-2 UNDERWATER PROFILE IN THE SURF ZONE AT INYONI ROCKS 1·8·1960

(AFTER HARRIS ET AL 1962)

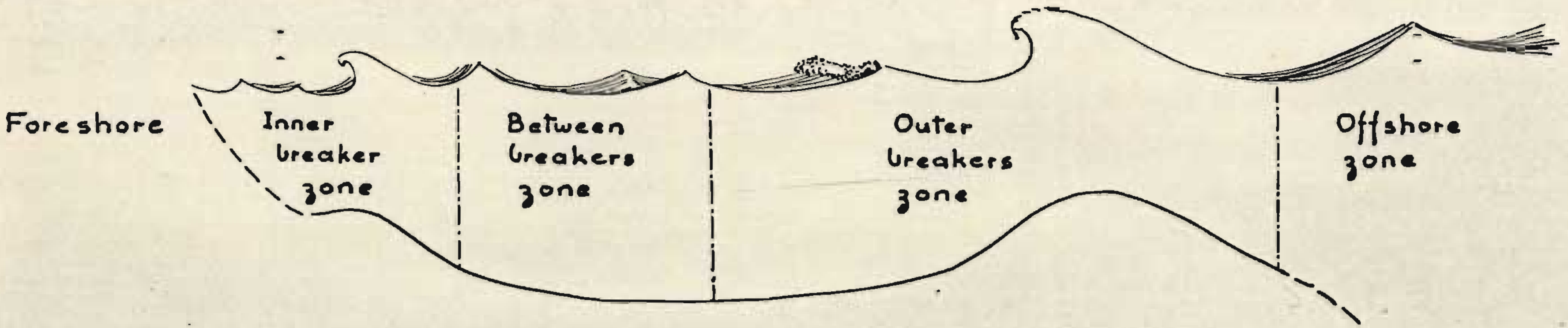


Fig. 2-3. Definition Sketch.— zones of nearshore region (after Harris 1964)

up to $\frac{1}{4}$ or $\frac{1}{2}$ knot rarely exceeding 1 knot. The percentage frequency distribution of direction was, 76% approximately parallel to the shore, 12% within 45° of the parallel, and 12% within 45° of the perpendicular.

Tides, which were diurnal, had a maximum range of 6 ft. Tidal currents were negligible - the cotidal lines being roughly parallel to the coast.

2. Nearshore Circulations

The studies at Virginia disclosed nearshore circulations essentially similar to those described by Shepard and Inman (1950). The main elements were, mass transport shorewards by waves and breakers, lateral transport within the surf by longshore currents and a return flow seawards in narrow swift-flowing rip currents which "mushroomed" out in the offshore zone. The whole circulation formed a cell with movement about a vertical axis. Flow was intermittent. Order of magnitude dimensions were, cell width (spacing of rips) 500 yds., rip widths 100-200 ft, and periodicity of pulses 10 minutes.

The studies at Virginia revealed three types of circulation, largely dependent on wave obliquity. There were the symmetrical and asymmetrical cellular circulations, and the alongshore system. (Note: In the original thesis these types were designated A B and C respectively. It was thought best to alter them so as to bring them into line with the nomenclature now proposed).

Of these there were two extreme cases. The symmetrical cellular circulation, a result of normal wave approach, with rip currents flowing perpendicularly out from the shore, being fed by longshore currents flowing in equally from either direction. With this type the cellular patterns were marked. With the other type, the alongshore system commonly associated with low oblique wind waves, the longshore current was unidirectional and was the dominant element. Rip currents were widely spaced (up to a mile) and usually occurred at discontinuities in the coast. Between these two types there were a variety of patterns all of which were associated with some degree of wave obliquity and were characterized by a nearly unidirectional longshore current feeding a well defined rip current. They were all placed in a single category called the asymmetrical cellular circulation. Schematic diagrams in Fig. 2-4 illustrate the main features.

During 53 observations the three types of circulation occurred with the following percentage frequency.

Type	% Frequency
Symmetrical cellular	38
Asymmetrical cellular	52
Alongshore system	10

The submerged sand bar and trough were found to exert an important influence on the circulation - causing seperate circulations in the outer and inner breaker zone with varying degrees of coupling, depending on the relative height of the wave over the trough and hence the degree of approximation to the solitary wave shape. Under commonly occurring conditions, the large rip currents (spaced of the order of 500 yds apart) were common to the circulations in both zones.

It was noted that when a coastal current overlapped the nearshore circulation the rip currents outside the surf were slewed round into the direction of current travel.

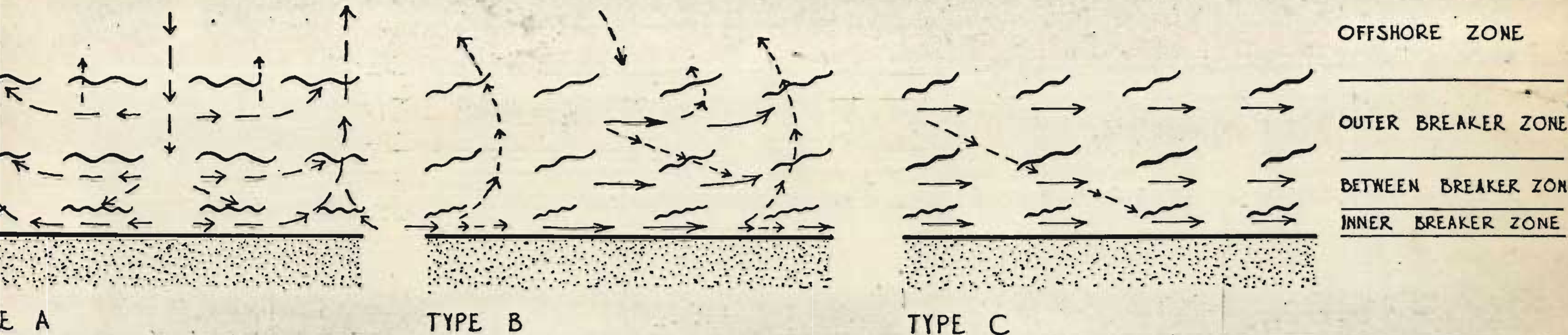


FIG 2-4 SCHEMATIC DIAGRAMS OF THE MAIN TYPES OF CIRCULATION

3. The sand bar and trough

The presence of the marked sand bar and trough, are according to Shepard (1950) associated with plunging breakers and longshore scour. The cause of plunging breakers is still rather obscure though the work of Longinov was thought to be relevant. Longinov as reported by Zenkovitch (1962) considers that the criterion determining whether a wave will develop into a plunging or a spilling breaker depends upon the gradient of the approach to surf. If it is greater than 0.03 plunging breakers are formed. The approach to the surf at Virginia was clearly in this category. It should be noted then that the experience of Miller and Zeigler (1964) quoted above is at variance with Longinov's views.

4. Exchange

It was suggested that the exchange of offshore water with that in the surf zone would be greatest when symmetrical cellular circulation prevailed, and that the exchange would be least (and perhaps negligible) with the alongshore system. It was furthermore noted that the effect of a coastal current overlapping the circulation would be a decrease in the exchange, since rip currents would be slewed parallel to the shore and mainly rip current water would be cycled shorewards.

CHAPTER III

EXPERIMENTS WITH THE CELLULAR CIRCULATIONS

EXPERIMENTS WITH THE CELLULAR CIRCULATIONS

The primary purpose of the field experiments was to gain some understanding of the quantitative aspects of the cellular circulations and to throw some light on their cause and mechanism.

Initially it was decided to extend the qualitative work undertaken in the past, to fill in some of the gaps, especially to find out more about the way in which the water entered the surf zone. Three types of preliminary experiments were undertaken.

(a) Experiments in which the behaviour was observed of a line of dye established parallel to the coast, well seaward of the surf zone, and long enoughto cover at least one cell width. These were known as "line source" tests.

(b) Experiments in which a point source of dye was established just seaward of the surf zone and the rate of build up of dye concentration in the surf measured.

(c) Experiments in which dye was discharged continuously at the waters edge.

Ideally these experiments should have been combined in one large experiment, but the expense and difficulty of maintaining sufficiently large concentrations of dye necessitated their being treated separately.

Later a series of tests were put in hand to assess the quantity of water circulating in and out of the surf zone.

Besides producing qualitative information these tests yielded data about the dimensions of the cells. In particular it emerged that there appeared to be some constancy in the ratio of the cell width to the width of the breaker zone. This fact recalled an analagous ratio which resulted from a mathematical analysis by

Rayleigh (1916) of the falling away from unstable equilibrium of a thin horizontal layer of liquid when heated uniformly from below. It is thought that if this analogy was valid then the cellular circulation should be basically a hydrodynamic effect - not a topographical one - and that being the case it should be a quite general phenomenon. It was therefore decided to see if it could be reproduced in a uniform model. In fact it was found to be reproduced.

Parallel with the exploratory field work, some work on sea level changes in the surf was initiated. It was argued that since the circulation must be motivated by hydrostatic head differences, measurement of these would be a starting point in the investigation of cause and mechanism. Equipment was designed and made operational, and some results obtained. It was at this stage that the possibility of studying the cellular circulations much more easily in a model was discovered, so the sea level studies were curtailed.

The experimental work is reported in two parts.

(a) Field Experiments - Section I.

(b) Model Studies - Section II.

SECTION I

EXPERIMENTS IN THE FIELD

1. Line Source of Dye Tracer

Purpose - To examine qualitatively the water movements near the outer boundary of the cellular circulations.

Method and Procedure - A line source of sodium fluorescein dye was established, by trailing behind a boat, a porous container with 5-10 lbs dye, or discharging continuously a solution of the dye. As the boat moved along the coast the dye diffused out slowly and formed a line whose subsequent movement could be observed or photographed for about one hour before it faded. By laying the dye at a suitable distance offshore, the onshore (and offshore) movements and entry into the surf zone could be easily observed.

This line source technique was applied on seven occasions when the circulation was of a symmetrical or asymmetrical type. The line was established at distances ranging from 100-800 ft seaward of the outer breaker zone. The dye was dispensed from a surf ski propelled by a swimmer or a boat operating close inshore. The difficulty of getting the surf ski through the surf and the hazards to the boat, limited the tests to occasions when the breaker heights did not exceed about 5 ft. Photos 3.1 and 3.2 illustrate the method.

Results

A consistent feature of the tests was the distortion of the initial straight line of dye by offshore movements in the vicinity of rip currents and onshore travel between them. Examples of the distortions are shown in Fig. 3-1.

The velocities of onshore travel between the rips was extracted from the photographic record, and are shown in Table 3.1. They
4cm 10.7 cm
varied between 0.14 and 0.35 ft/sec.

Also extracted from the photographs was the frontage through which the "new" offshore water (as opposed to directly recycled

Photo 3.1. Line of dye laid parallel to the cadst - Virginia Beach.

Photo 3.2. Distortion of dye line by a rip current. Note small rips in inner-breaker zone marked by foam (23-6.65)

Photo 3.3. Example of small rip currents in inner breaker zone marked by silt - Virginia Beach.

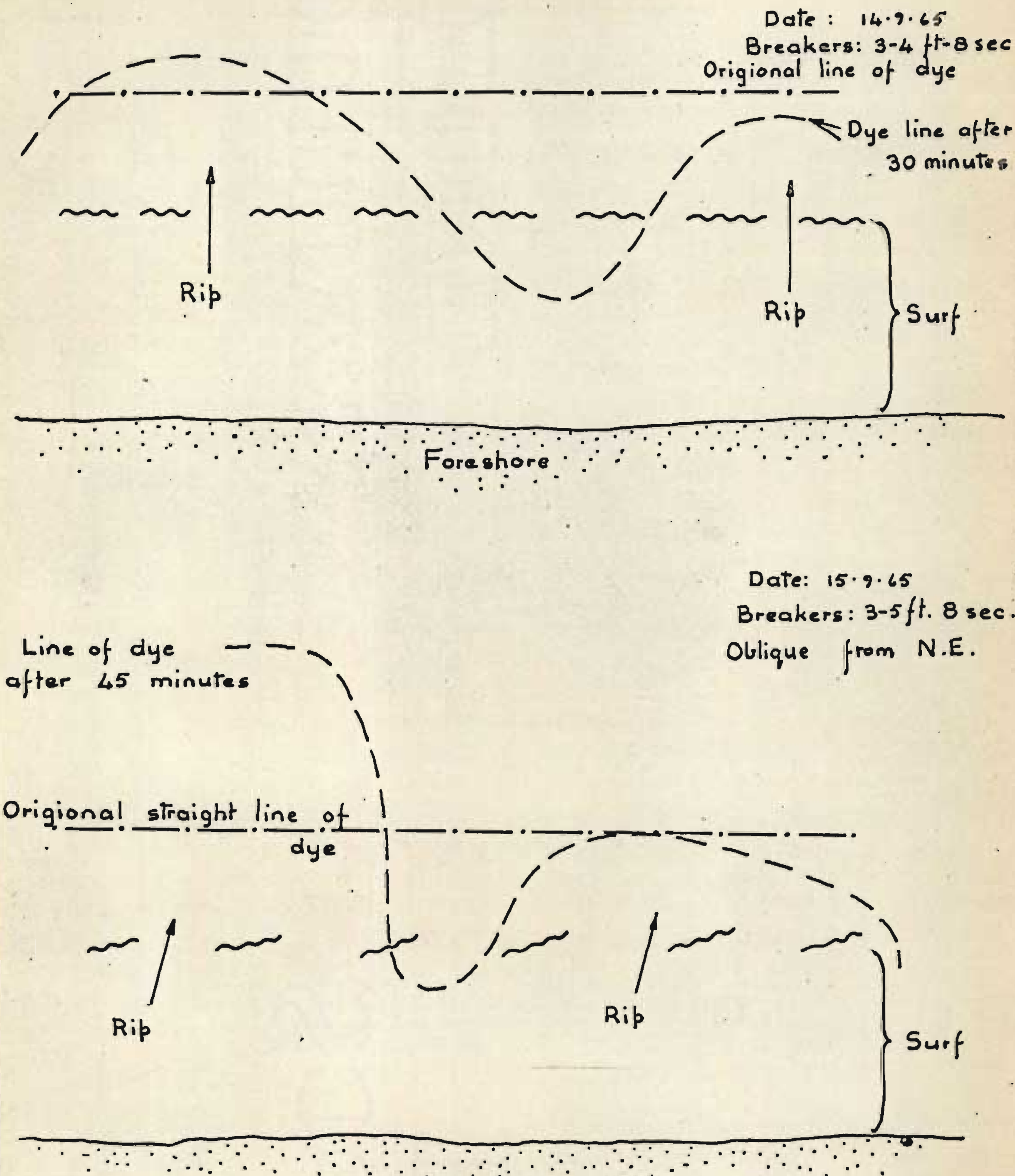


Fig.3-1. Tracings from aerial photographs showing the behaviour of a line of dye at Virginia Beach. Scale approx 1" = 300 ft.

TABLE 3.1

DATA FROM LINE SOURCE TESTS

Date of Test	Breaker Height Ft.	Period Secs.	Distance of initial dye line from surf Ft.	Time to reach bar Min.	Velocity of Onshore drift Ft/Sec.	Approx. Rip spacing Yds. (b)	Approx. frontage of "New" water entry Yds. (a)	$\frac{a}{b}$	Type of circulation.
13.9.65.	4.5	8	500	60+	0.14-	-	-	-	S
13.9.65	4.5	8	100	7	0.24	300	100	0.33	S
14.9.65	3.5	8	200	15	0.23	300	100	0.33	S
15.9.65	3.5	8	200	22	0.14	500	100	0.2	A
23.9.65	4	7	300	-	-	550	140	0.25	A
28.9.65	-	-	300	14	0.35	400	150	0.37	S
29.9.65	5	7	600	45	0.22	400	150	0.37	A

S = Symmetrical

A = Asymmetrical

the table and expressed as a fraction of the distance between the axis of the cell's rip currents. The ratios varied from 0.2 to 0.37. This means that on the occasions of the tests, and assuming that surface observations reflect the behaviour of the whole cross section, the percentage of recycled water was 63-80%.

Much of the surf frontage was therfore taken up by the outflowing water from rips or recycled water from rip currents.

Experiments with models (reported in Section II) have indicated that cell widths tend to increase with wave height. The above ratio will therefore perhaps increase with high long waves. In the field tests the waves were 3.5-5 ft with periods of 7-8 seconds.

2. Point Source of Dye Tracer

Although the experiments using a line of dye were useful in throwing some light on the way the offshore water masses behaved whilst approaching the surf zone, the dye was of insufficient concentration to show up movements within the surf zone, once it had entered. To remedy this another series of experiments were carried out using point sources of dye released just outside the surf zone. In addition to making qualitative observations samples for analysis were taken on most occasions.

In all, ten tests were made. The usual procedure was to release 10 lbs of dye from a ski boat 100-300 ft outside the surf zone. Samples were usually taken at known intervals of time in the initial dye patch, at midsurf nearly opposite the point of entry, and at the foreshore. These were analysed with a fluorimeter.

Results - The following are the main results:

(a) The test confirmed that the entry into the surf zone is restricted to certain rather narrow sectors.

(b) The inflow showed marked signs of being pulsed.

(c) Having entered the surf zone the dye flowed in a stream along the surf zone and out at the rip. Though diffusion did of course occur, the main action was one of displacement of the resident water. This is evidenced by the fact that the build up of dye in midsurf was very rapid. See Fig. 3-2. It was clear that the through-flow was quickest in the outer breaker zone and near the rip. There was evidence of short-circuiting and therefore some regions mainly about the mid point of the cell tended to be stagnant.

(d) The recycled water from rip currents formed a large fraction of the water entering a cell of circulation.

(e) Though the dye entered and circulated in the outer breaker zone it evidently took a good deal longer to reach the inner breaker zone. In fact it frequently happened that no dye at all could be discerned in the inner breaker zone during a two hour experiment.

3. Continuous discharge of dye in the inner breaker zone.

In order to study recycling, arrangements were made for a continuous discharge of a large quantity of dye at the water's edge.

Procedure - A 600 gallon tank was placed high up on the beach about 20 ft above sea level at Virginia. It was filled with fresh water in which about 30 lbs of dye were dissolved. The dye concentration was 8000 ppm. This dye was allowed to discharge to

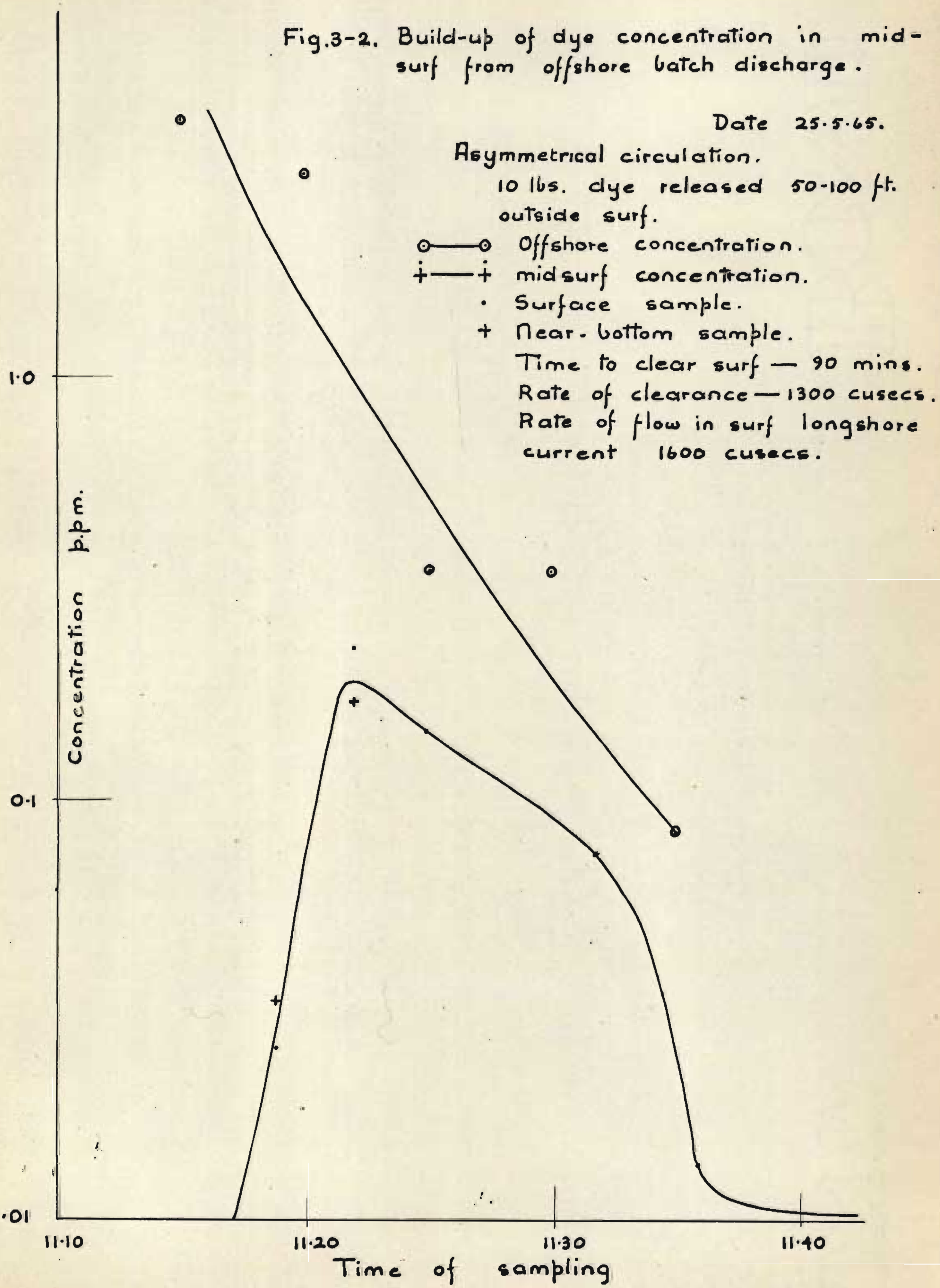


Fig.3-2. Build-up of dye concentration in mid-surf from offshore batch discharge.

the sea by way of a 200 ft plastic hose, at the rate of 1.5 gallons per minute. Some difficulty was experienced in anchoring the hose in the breakers. This was overcome by arranging for the hose to terminate in a few feet of iron pipe welded to a plate (2 ft diameter), to which was fixed four 6" iron spikes on its under side. These spikes were pushed into the sand on the sea floor. In addition, the hose was pinned onto the sand in the inter-tidal zone by "U" shaped stakes made from reinforcing rods.

<u>The point of discharge was 300 ft from a rip current.</u>

Samples were taken close to the base of the rip at the water's edge, in the rip head just outside the surf zone, and at mid-surf when recycling occurred.

<u>Conditions</u> - The circulation was a symmetrical one which resulted from 4-5 ft breakers with periods of 8 secs. In the inner breaker zone the waves were 1-2 ft high.

At first there was no wind. Two hours after the start of the test a 10-15 mph wind got up from the North East and the circulation changed quickly to asymmetrical. Small rip currents spaced about 50 ft apart were in evidence in the inner zone.

<u>Results</u> - The test produced a number of interesting qualitative results.

(a) When the large rip current was flowing there was a marked inflow of water from the between breaker zone into the inner breaker zone <u>near</u> the rip.

(b) Having penetrated the outer-breaker zone, the pulses of dye from the rip current did not behave uniformly. One moved seawards, one recycled to the North of the rip and one moved to the South. This erratic behaviour appeared to be associated with the shifting of the position of the rip axis several hundred feet to either side.

(c) During the two hours of the test only one pulse recycled to the shore.

The quantitative results are as follows:

(a) The rip current was 100-150 ft wide.

(b) It operated with a periodicity of about 25 minutes.

(c) The seaward velocity of rip within the surf zone was 1.6 - 2.2 ft/sec.

(d) The average concentrations at the three stations were

(i) at the rip base 0.14 ppm

(ii) at the rip head during rip pulsing 0.01 ppm

(iii) at the mid surf during recycling 0.008 ppm.

Fig. 3-3 shows the change of concentration with time, near the rip base, (the vertical wave line marks a break in the record). The marked troughs in the curve at 15 and 40 minutes are due to the inflow of undyed water from the between breaker zone, which entered the longshore current between the point of insertion of dye and the rip when the rip was operating.

The mean longshore current discharge in the inner breaker zone as calculated from the dilution of the influent was 285 cusecs.

A notable feature was the operation of small rip currents origniating in the inner breaker zone and penetrating about 100 ft into the surf. Penetration into the surf increased with decreasing distance from the large rip current which of course traversed the whole of the width of the surf zone into the offshore zone.

The small rip currents were spaced irregularly - the mean distance apart being about 100 ft. The width of the inner breaker zone could only be determined very approximately. It was between 20 and 50 ft.

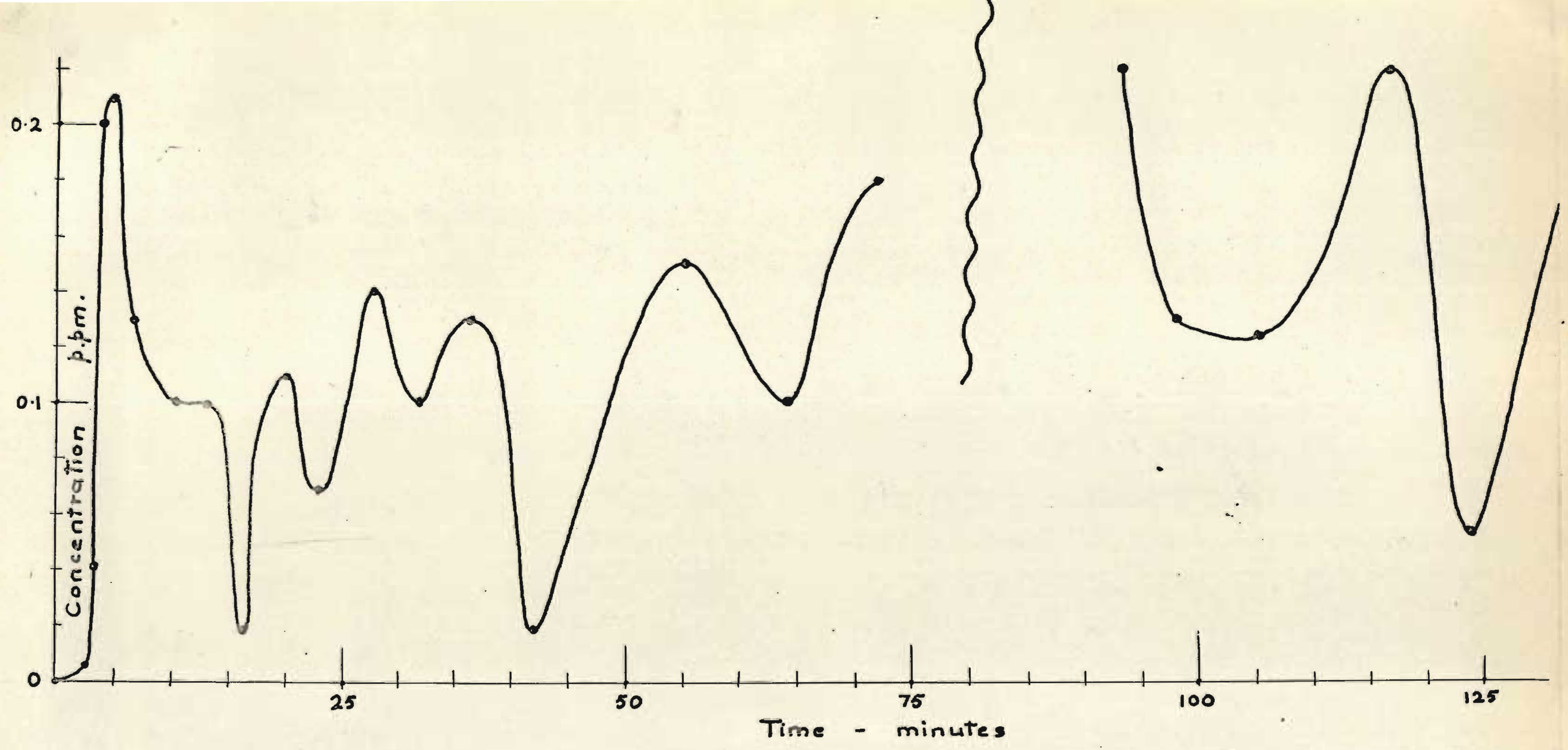


Fig. 3-3 Graph of concentration at rip base V.S. time, for continuous discharge at Virginia. 17·3·66

It is noteworthy that the spacing of beach cusps along the intertidal zone was the same order of magnitude, as that of the small rip currents. The rips and the cusps were not always in phase. This is perhaps understandable because the cusps were probably formed under conditions of the previous tide.

The operation of the small rip currents was intermittent, their periodicity being less than that of the large rip. No accurate measure was made of this periodicity, but it was of the order of 5-10 minutes.

No aerial photographs were taken of these small rip currents, but re-examination of photographs taken on other occasions revealed examples of similar occurrences. See photographs 3.2 and 3.3. Measurements of the spacing of small rips which were made on several subsequent occasions are recorded later.

CONCLUSIONS which have a bearing on waste disposal are:

(a) The mean dilution which a discharge of 1 gallon per minute of effluent at the water's edge will undergo is about 8×10^4.

(b) In its passage through the breaker zone a further dilution of the order of 10 can be expected.

(c) The recycled pulse will undergo little further dilution.

(d) The region of the rip head just outside the breakers is a region of large scale eddy diffusion.

(e) The first recycled dye arrived at mid surf 82 minutes after release started.

(f) Symmetrical circulations change rapidly to asymmetrical under the influence of even a light to moderate wind.

4. Exchange of surf zone water

Purpose - To establish the order of magnitude of the rate of exchange of the surf zone water with that seaward of it.

Procedure - The water in a section of the surf (usually about one cell of circulation was selected) was marked with dye dropped from an aircraft. The time for the dyed water to be cycled out of the surf was measured. From a knowledge of the dimensions of the marked mass of water its volume was calculated, and hence the rate of exchange estimated.

Methods

(a) The water mass was marked by sodium fluorescein dye dropped in 2 lb packets from an aircraft. It was usual to drop 6-8 packets spread over the region of interest.

(b) The time taken for the surf zone to clear was measured from the time of the first dye drop to the time when the bulk of the dyed water had been cycled offshore by rip currents. On some occasions small pockets of dye remained in relatively stagnant areas - these were neglected. It is estimated that the clearance time was measured to an accuracy of ±15%.

(c) Obtaining accurate dimensions of the marked mass of water was of course a difficult task. Aerial photographs taken with a 35 mm camera assisted in estimating the length and width of the dye patch. (See Photo 3.4) The average depth was calculated from bottom profiles. These were constructed from depths measured by a skindiver using a sounding rod or lead-and-line. His position was determined by triangulation from the shore.

The methods used were necessarily rather crude. It is estimated

Photograph 3.4. Taken during exchange test of 31.3.65 showing surf zone water, marked with sodium fluorescein, being transported offshore in a large rip current. Recycling had just begun. Note characteristic inner- and outer-breaker zone. Cusps may be seen in the sand along the foreshore.

that the length and width could be measured to an accuracy of ±10%. Depth measurements were assessed to be accurate to ±5%, and the average depth ±10%.

The compound error in the calculated volumes would be ±40%.

The rate of exchange was taken as the volume divided by the time needed for the exchange.

In view of the substantial errors which might occur in both the volume and time measurements, the calculated values may under the worst conditions be in error by about 50%.

Results - Seven tests under conditions of normal or near normal wave approach were carried out. The wave heights and periods on these occasions were roughly comparable.

The results and the conditions under which the tests were carried out are set out in Table 3.2.

The discharge rates fall into three classes:

(a) Those between $28.31\ m^3sec^{-1}$ 1000-2000 cusecs from tests 1 and 2 when the surf was shallow, the trough being narrow. Waves were breaking across the wide bar.

(b) Those between 500 and 1000 cusecs from Tests 3, 4 and 5 which were associated with what was found to be the normal profile.

(c) Those less than 500 cusecs from Tests 6 and 7 where some factor, such as deep water over the bar which resulted in few waves breaking there, or adverse winds, inhibited the circulation. The importance of water depth over the bar underlines the potential influence of tide on the rate of exchange.

Examples of two extreme profiles, one shallow and one deep, are shown in Fig. 3-4(a) and (b).

In all the above tests there was some obliquity of the waves. Though usually small it was nevertheless sufficient to impart a

TABLE 3.2 - DATA FOR SURF WATER EXCHANGE TESTS

Test No.	Date 1965	Breakers: Height (Bar) Ft.	Breakers: Height Fore-shore Ft.	Breakers: Angle at Shore o(1)	Breakers: Period Secs.	Dimensions of Dyed Region: Length Ft.	Dimensions of Dyed Region: Width Ft.	Dimensions of Dyed Region: Average Depth Ft.	Dimensions of Dyed Region: Calculated Volume Cu.Ft. × 10^6	Time to clear surf Mins.	Rate of Clearance Cusecs (2)	
1	1.2	5.5	2.5	5	6	2000	500	5	5.0	45	1800	Narrow trough and shallow surf
2	3.2	5	1	5	6	2100	380	3.5	2.79	40	1200	Narrow trough and shallow surf
3	23.2	4.5	2	12	8	780	475	5	1.89	30	1000	- -
4	8.4	5	2.5	n	8.5	650	430	7	2.14	35	900	- -
5	7.4	5	1.5	5	9	900	475	6	2.56	60	700	- -
6	31.3	5	2	3	8	600	450	9	2.43	120+	300-	Wind opposed circulation
7	9.8	4.5	1.5	n	9	500	350	9	1.57	120+	200-	Few breakers on the bar

(1) n = negligible (2) rounded to nearest hundred

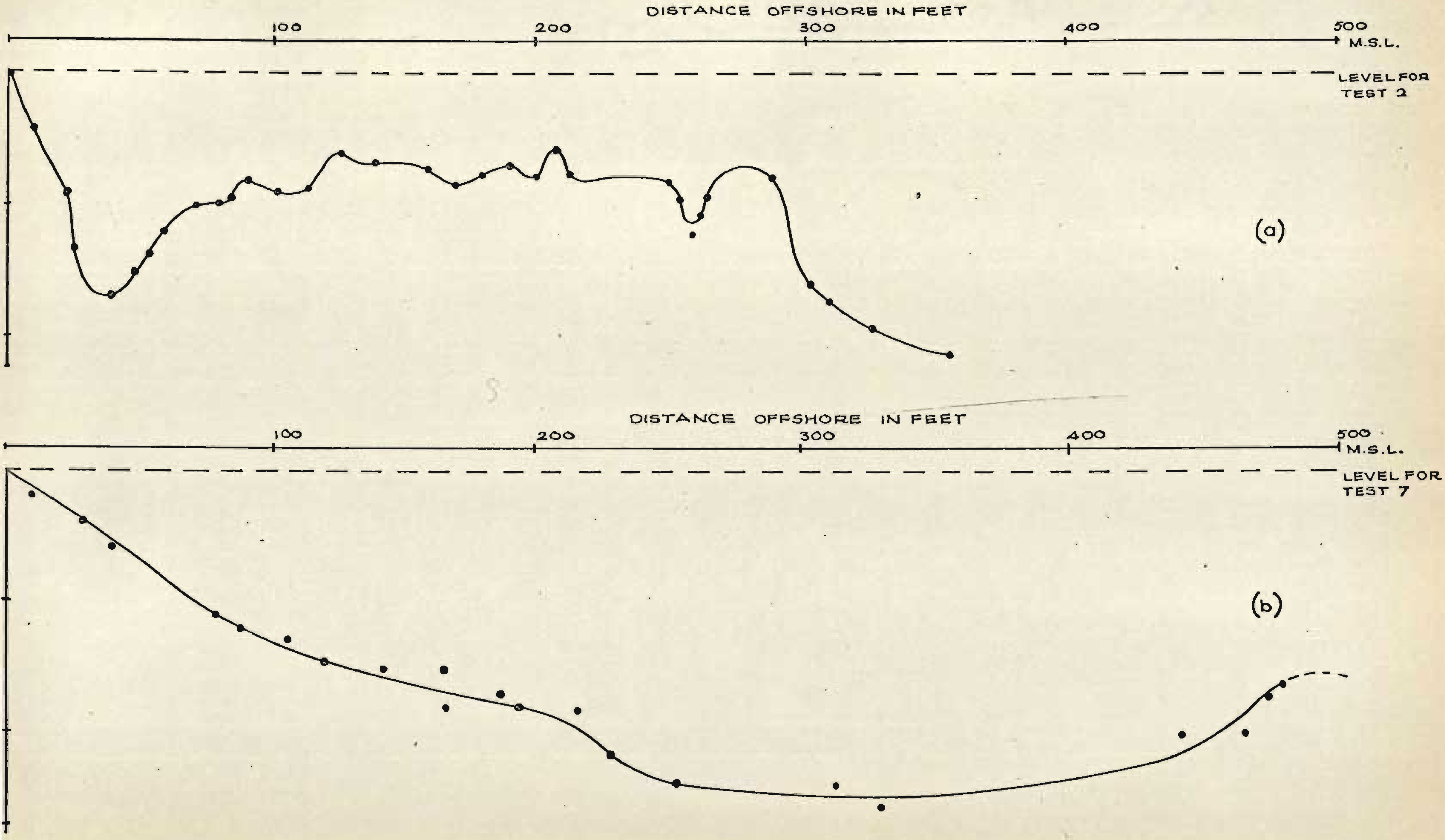


FIG. 3-4. EXAMPLES OF BEACH PROFILES (a) SHALLOW (b) DEEP, AT VIRGINIA.

asymmetry to the cell. Since in all tests the offshore exchange was effected through the agency of one rip, the exchange rates recorded must reflect a large proportion of the net rip discharge. Having regard to the errors in measurements discussed above, the discharges are probably within a factor of two of the true values.

5. Other results from the field work

During the tests described above some additional information was collected. It was chiefly concerned with the general qualitative circulation of water and with the dimensions of cells. It is set out below.

(a) General Circulation. Though a wide variety of circulation types were experienced the previous idea that they could be broadly classified into three classes was confirmed. As stated above these were

Cellular circulation - symmetrical and asymmetrical

Alongshore System.

Schematic diagrams illustrating these are shown in Fig. 3-5.

(b) Cell dimensions. Measurements of the cell dimensions i.e. the spacing of adjacent rip currents and the associated width of the surf, are set out in Appendix I. They are summarized below. The width of the inner breaker zone was especially difficult to measure

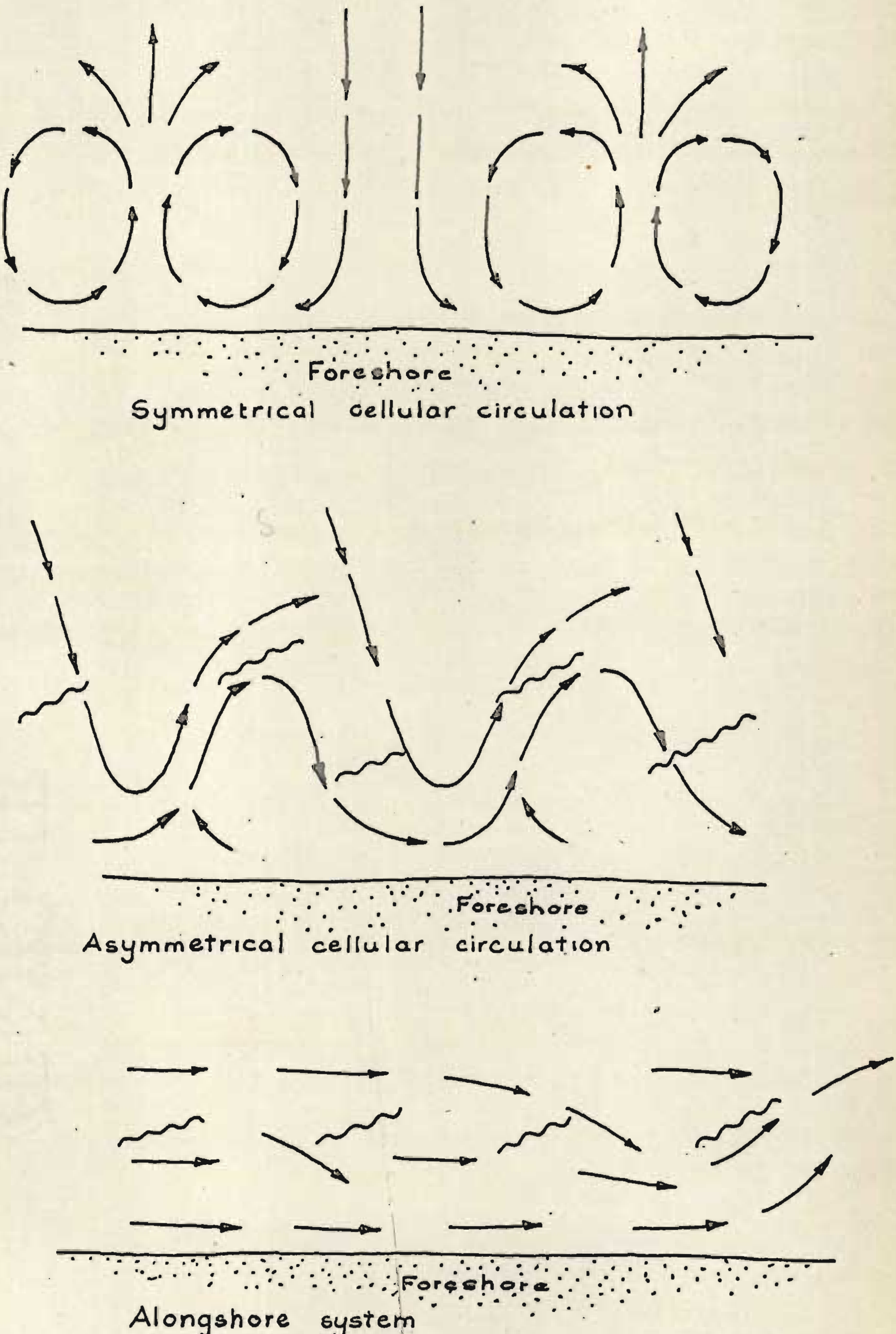


Fig.3-5. Nearshore circulation types.

Table 3.3 - Cell dimensions

Large rips penetrating the whole surf			
	No. of observations	Mean	Standard deviation
Spacing of rips (ft)	32*	1830 ft	450 ft
Surf width (ft)	28	470 ft	65 ft
Ratio spacing / surf width	16	3.7	1.0
Small rips penetrating only the inner-breaker zone			
Spacing (ft)	35	127 ft	47 ft
Surf width (ft)**	34	47 ft	15 ft
Ratio spacing / surf width	34	2.8	0.8

** Measured from mean uprush to point of incipient breaking

* includes the average of 11 rips observed from an aircraft on a single occasion.

(c) Periodicity of large rip currents

Based mainly on dye and tethered float observations the following are some measurements of the periodicity of the pulsing of large rip currents in their passage through the surf.

No. of observations	Mean Period	Standard Deviation
30	8 mins	3.6 mins

6. Studies of sea level changes in the surf

Introduction

Both the theory and the field and model studies which have been summarized in Chapter I, make it clear that the momentum associated with the breaking wave causes a set-up of the still water level at the shore. This gradient is almost certainly the cause of the greater part of the water returning seawards.

In searching for a cause and mechanism of the cellular circulations, it seemed that some investigation of it would make a start in the right direction. As mentioned above, the transfer of effort and resources to model studies resulted in this work being curtailed before finality had been reached. Because some interesting data resulted from the study and for completeness, the details of the experiments are reported below.

Equipment and Methods

The measurement of sea level changes over periods of time greater than several wave periods and less than the tidal period calls for equipment which will damp out ordinary wave motion. In addition since sea level changes are not likely to be large, some degree of amplification is desirable.

Munk (1949a) used a tube placed with one end embedded a short distance in the sand of the surf bottom. The sand provided a satisfactory degree of damping. Dorrenstein (1961) also used a tube into which the water could leak through holes punched in the side of the tube. Both these systems require water level changes to be read in situ - a requirement militating against continuous measurement over protracted periods.

It was decided to use a system which is illustrated in Fig. 3-6.

It consisted of a manometer which was placed high upon the beach clear of the water, connected by a 70 ft plastic hose to 70 ft of iron pipe ($\frac{3}{4}$" diameter), which terminated in a short section bent vertical. To facilitate launching and anchoring the seaward end of the pipe it was fixed to a sledge. A damping orifice plate with $\frac{1}{8}$" hole was inserted in the pipe section which was below sea level.

The manometer arms were made of two 1" plastic pipes whose lower ends were joined by a short length of plastic tube, the flow of water in which could be controlled by a tap. The manometer was filled with tap water. Floating on the water surface in the open arm of the manometer was a small square-section float. Fluctuations in the level of the water surface in the open arm were transferred to a clockwork recorder by a thread, connected to the float and to the arm of the recorder pen. (See Fig. 3-7.)

A certain amount of trial and error was necessary to find a suitable design for the diameter of the arms, the shape and nature of the float and the size of the damping orifice plate. The operation of the apparatus was as follows:

The pipes having been connected up to the sledge, the sledge was pushed into water of about 6-8 ft depth (just seaward of the inner breakers). Care was taken to see that the landward end of the iron pipe extended up the beach to a distance clear of the uprush. The plastic pipe was covered with sand to minimize heating by the sun. The manometer was then connected up and adjusted to restrict the maximum fluctuations of the recorder pen within the paper width of the recorder.

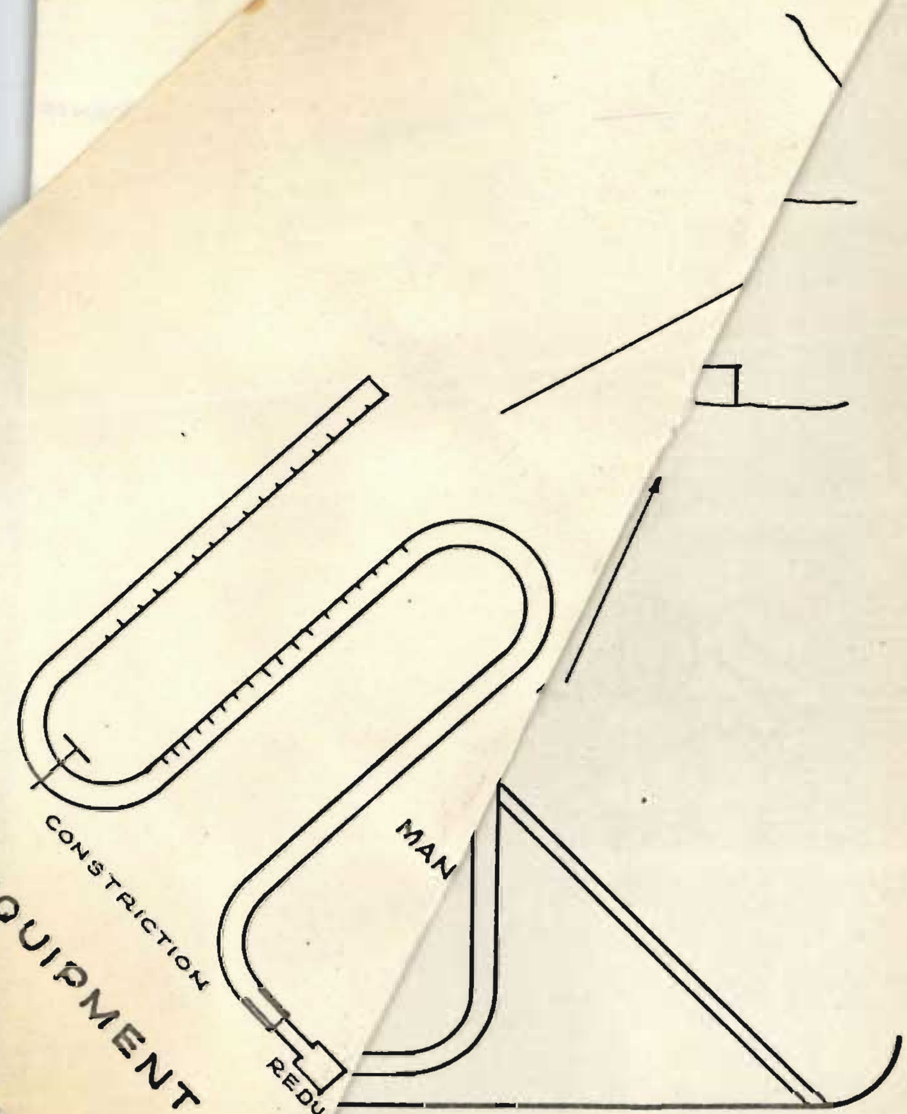


FIG. 3-6. EQUIPMENT FOR A-LEVEL CHANGES.

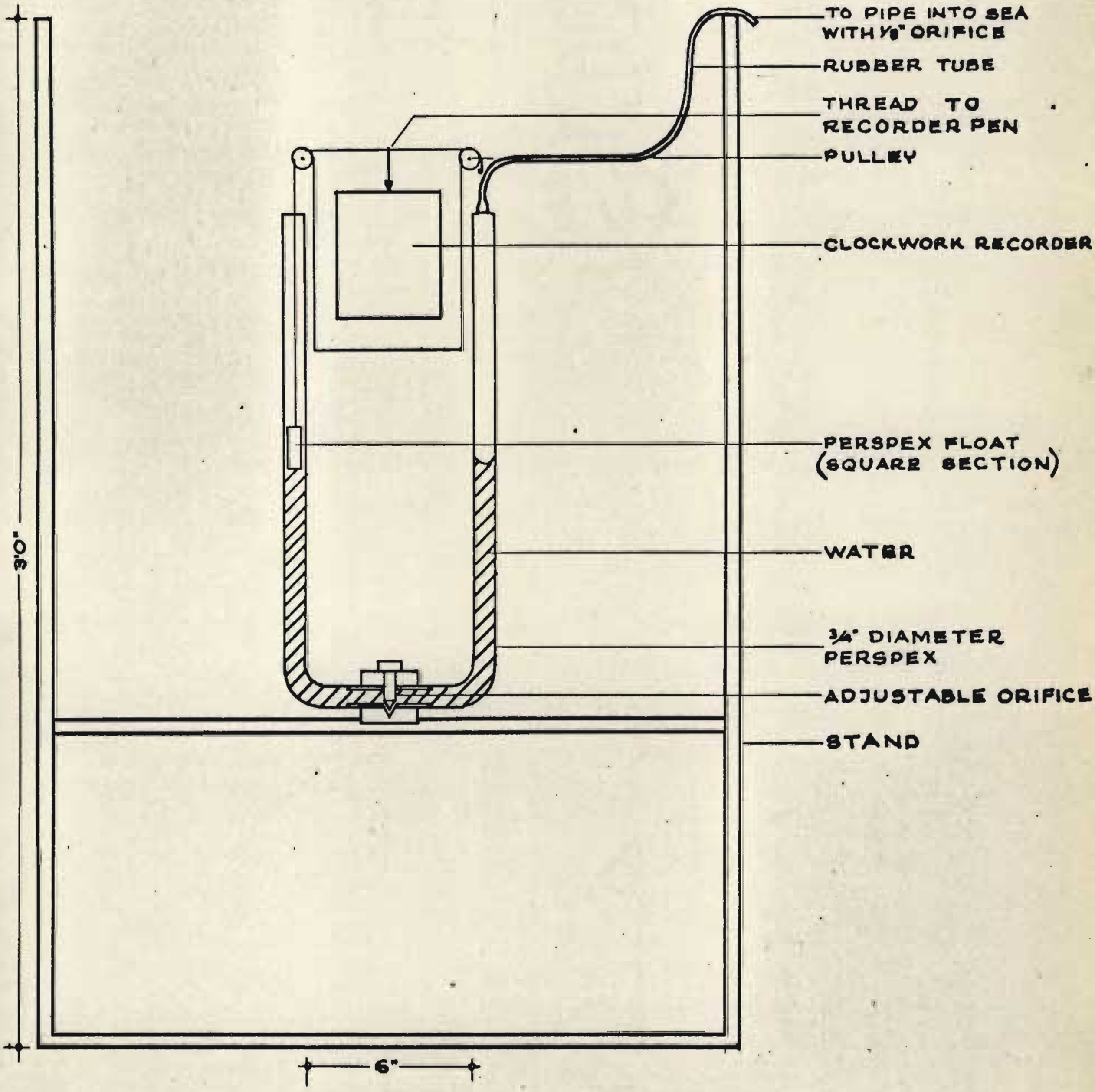


FIG. 3-7. SKETCH OF MANOMETER AND RECORDING SYSTEM FOR MEASURING SEA-LEVEL CHANGE

Fluctuation of the sea level was transmitted to the manometer by way of water in the iron pipe and air in the plastic hose.

For calibration the sledge was pulled out of the water and set up on the beach so that the pipe length which had been at the water-air interface was lying at the same slope as it had been when recordings were made. A graduated perspex tube extension was then secured to the vertical section of pipe (which during operations had been submerged). Water was added to this tube to produce a variety of hydrostatic heads, and the corresponding pressure changes were recorded. A specimen calibration curve is shown in Appendix 2.

In order to determine the response time of the apparatus static leads of 4, 6 and 8 inches were applied and the response on the manometer read at intervals. The results of this test are shown in Fig. 3-8 where y is the response at time t for an applied head (A). Using the electrical analogy of the charging of a condenser (an idea suggested by Dr. Schoute-Vanneck), the response time is given by the expression

$$1 - \frac{y}{A} = \varepsilon^{-t/T}$$

where T is the time constant (T gives the time to reach 63% of the maximum). The curve of the above expression is also plotted in Fig. 3-8 where a time constant of 17 sec (found by curve fitting) was used. The condenser charging equation seems to predict the response of the manometer system reasonably satisfactorily.

A time constant of 17 seconds would seem to be acceptable since it ensures the damping out of the waves of period usually 8 seconds while allowing freedom for response to changes of the order of a minute.

GRAPH OF $1-\frac{y}{A} = \varepsilon^{-\frac{t}{T}}$

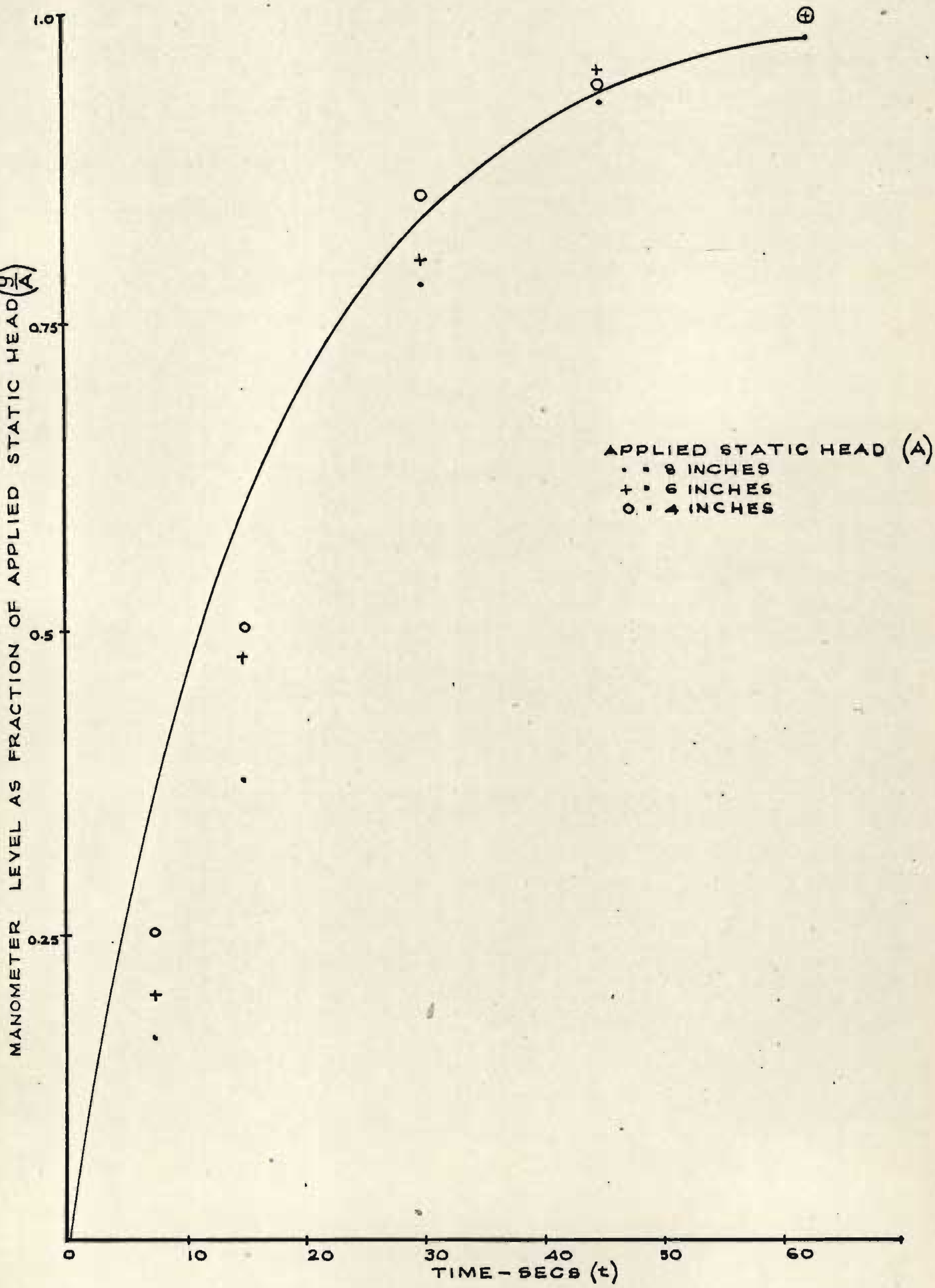


FIG.3-8. GRAPH SHOWING RATE OF RISE OF LIQUID LEVEL IN A DAMPED MANOMETER COMPARED WITH THE THEORETICAL CURVES USING A TIME CONSTANT OF 17 SECS

It was obviously desirable to attempt a correlation of water movements with the field measurements of pressure changes. On some occasions therefore, buoys with tethered floats were moored in the vicinity of the nearest rip current, or dye was released from time to time. Later it was clear that it would be most desirable to have in addition a simultaneous record of the wave heights. To achieve this a wave recorder based on the principle of one developed by the National Institute for Oceanography in the United Kingdom was constructed. Essentially it consisted of a rubber ring secured to a heavy iron plate (50 lbs) which was placed on the sea bottom. A rubber hose ($\frac{1}{2}$" diameter) connected the ring to a mercury manometer, in the open arm of which was a float which transmitted changes of level to a clockwork recorder.

Results

In all twelve recordings of sea level changes were made. Five of the most satisfactory were selected and parts of the records are reproduced in Figs. 3-9a and 3-9b.

The conditions under which these tests were made are set out in the Table 3.4.

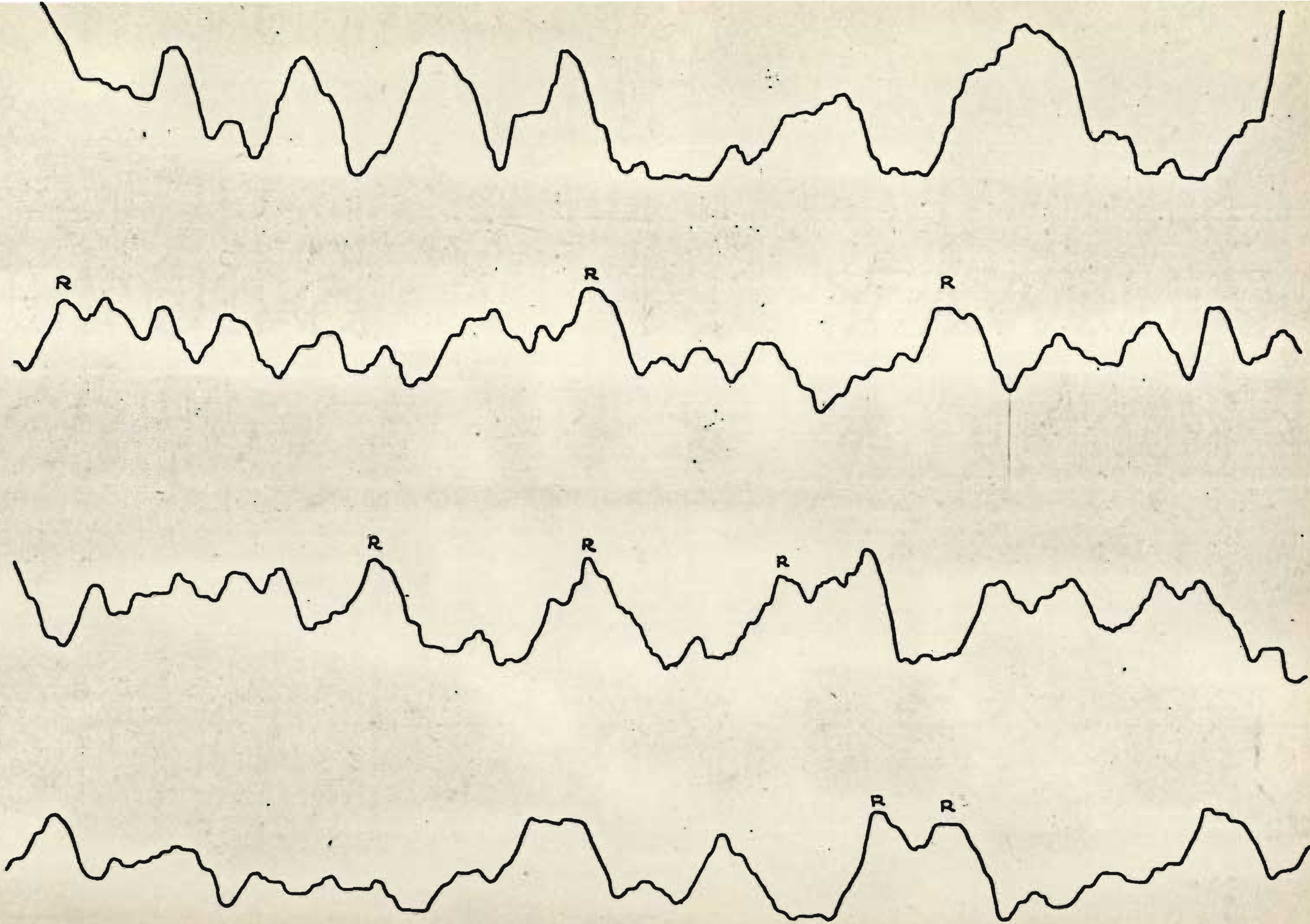
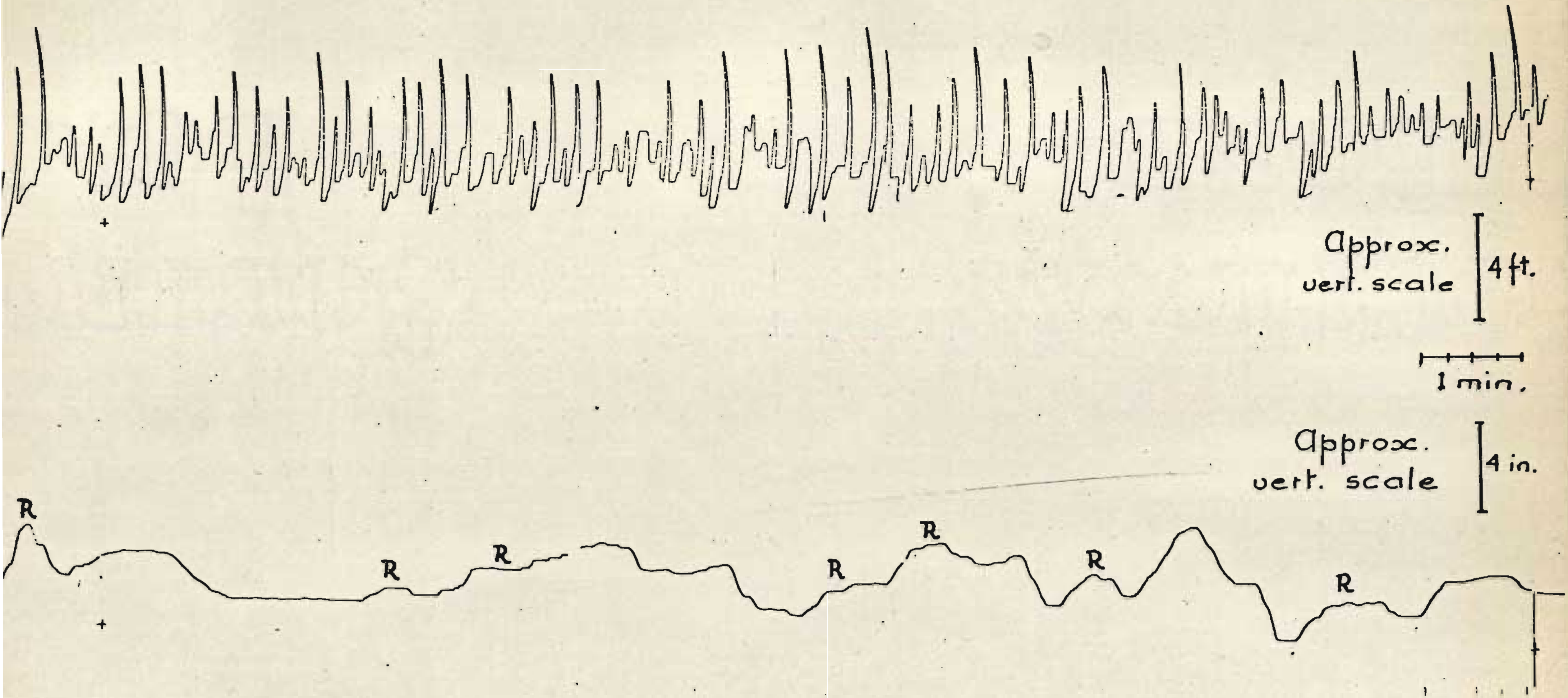


Fig.3-9(a) Sea level changes in the surf. Scales: 1 min 5 inches



3-9(b). Simultaneous recordings made 50-100 ft. offshore at Virginia (21-7-66) of waves (above) and mean-sea-level (below). "R" indicates when small rip discharges were observed.

Table 3-4 Conditions during sea-level measurement tests

Test Number	Date	Significant wave height (ft) at the bar	Wave period (secs)	Circulation type	Distance from nearest rip (ft)
1	27.9.65	6	8	S	0
2	1.9.65	7	8	A	300
3	16.9.65	5.5	8	S	600
4	2.9.65	12	10	A	300
5	21.7.65	4.5*	11	S	50

S = Symmetrical

A = Asymmetrical

* = No sand bar

Though the arrangements made for observing the water movements were rather unsophisticated, they did show that the movement of water offshore was usually associated with a fall away from a peak of the sea level. (Offshore movements in rips when recorded have been indicated with an "R" in Figures).

Furthermore there was sometimes a correlation between the arrival of high groups of waves following a low group, and the action of rip currents.

In tests 1 - 4 a sand bar and the normally associated circulations at the bar and the foreshore were present.

The sea level changes resulting from the nearshore circulation are no doubt complicated, and interpretation of the record was correspondingly difficult. For example, in some instances a rise in sea level was clearly due to a longshore pulse of water. In test No. 5, however, there was no bar so the sensing heads were effectively outside the surf. The circulation appeared to consist only of small rip currents near the shore. On this occasion, a simultaneous wave record was available. The sea level record and the wave record are shown in Fig. 3-8b. The time of operation of rip currents were in some cases noted and they do seem to be correlated with the arrival of a group of high waves. Furthermore, the sea level outside the surf seems to fall with the arrival of the high wave groups. This lowering of the sea level is in accord with the concepts of radiation stress but it may well be associated with rip current activity.

Table 3.5 shows the average of the ten larger changes in sea level.

Table 3.5 Sea level changes

Test Number	Average of larger sea level changes - inches
1	5.6
2	6.8
3	5.3
4	8.2
5	4

The changes of sea level recorded in Table 3.5 are roughly in accord with the field observations of Dorrenstein (1961) who found the deep water wave height to set-up ratio to be about 10, and the wave channel results of Saville (1961) who found the set-up to be 10-20% of the wave height.

SECTION II

WAVE TANK STUDIES WITH THE CELLULAR CIRCULATION

1. Introduction

Models have been widely used for the study of processes near the shore. Mostly the studies have been concerned with such subjects as wave characteristics in shoaling water, wave currents, sediment transport, and wave set-up at the shore. Commonly two dimensional wave channels have been used, but in the study of coastal engineering problems such as beach erosion or harbour silting, where local topography has had to be simulated, three dimensional models are needed. Few studies seem to have been devoted to uniform beaches. Notable amoungst these are the longshore current studies of Putnam et al (1949) and Brebner and Kamphuis (1963). Models to investigate mixing along a uniform beach was used by Jordaan (1964). He reported the occurrence of rip currents, but the beach he used was of moveable sand. No paper reporting the study of the nearshore circulation by means of normal waves in a uniform three-dimensional fixed beach model has been found.

Experiments in three dimensional wave tanks were undertaken in the first place to see if the nearshore circulation could be simulated on a small scale and if so, to see if the dimensions of the cells formed were in proportion to those found in the prototype. These experiments were carried out in part of an existing wave basin (designated Wave Tank Mark 1).

The study showed that the cellular circulation could indeed be simulated. A good deal of time was spent in trying to establish the cell dimensions under varying wave conditions. The opportunity was also taken to make some measurements on the longshore currents and rip currents and the dilution of inserted tracers. The work with this wave tank was therefore a fairly generalized study aimed primarily at obtaining some quantitative data about the circulation. In the course of the work, however, some phenomena (standing wave oscillations) were observed and they seemed to warrant further investigation. It was not possible to do this on the Mark 1 wave tank because the building which housed it reverted to its original use. It was decided to build a second wave tank - Mark 11.

Financial considerations limited the second tank to modest proportions, However : the wave generating equipment was redesigned to produce a more uniform wave train with a greater range of periods. The studies in this model were primarily to try to find the underlying cause of the circulations and to seek an understanding of the standing waves. The latter study revealed the importance of edge waves.

At this stage it became clear that the problem called for an intensive work on wave interaction, requiring rather more refined equipment than was available, and a more accurately uniform model - especially the beach. It was therefore decided that although the enquiry was far from complete, it had progressed to a point where it might be recorded.

It should be noted that when the model work started there was little in the literature on which to base a programme. The programme developed as the work progressed.

2. Design considerations

It is obviously desirable to avoid distortion of the horizontal and vertical scales when dealing with wave phenomena involving orbital velocities.

The choice of scales is dependent upon the nature of the processes being investigated. Complete similitude between prototype and model requires that they be geometrically, kinematically and dynamically similar. This means similarity, respectively, of the ratios of dimensions, velocities and forces. In practice a compromise is usually necessary, especially from the dynamical aspect.

In problems with turbulent flow with a free water surface, it is usual to arrange for equality of Froude number for model and prototype. The assumption here is that the gravitational forces predominate over the viscous forces. The Froude number F is given by:

$$F = \frac{V}{\sqrt{gL}}$$

where V is the velocity, g is the gravitational acceleration, and L is some length. Using subscript "p" for the prototype and "m" for the model.

$$\frac{V_p}{\sqrt{gL_p}} = \frac{V_m}{\sqrt{gL_m}}$$

Since L = CT where L is the wave length, C the phase velocity, and T the period.

$$\frac{L_m}{L_p} = \left[\frac{T_m}{T_p}\right]^{\frac{1}{2}}$$

i.e. if the length scale is fixed at μ then the time scale is $\mu^{\frac{1}{2}}$.

Le Mehauté and Collins (1961) have examined the choice of scales for nearshore studies. They stress that similitude here implies similar wave patterns and similar break points. Since the wave velocity is given by

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

Similar wave patterns are produced if

$$\left(\frac{d}{L}\right) \text{field} = \left(\frac{d}{L}\right) \text{model}$$

or

$$\frac{L_m}{L_p} = \frac{d_m}{d_p} = \mu$$

They show that

$$\frac{T_m}{T_p} = \mu^{\frac{1}{2}}$$

depth at breaking ratio $\dfrac{d_{Bm}}{d_{Bp}} = \mu$

and deduce that longshore and rip currents are related by $\mu^{\frac{1}{2}}$ and that mass transports are also related by $\mu^{\frac{1}{2}}$.

It appears that if the wave period ratio is fixed the above phenomena are produced to scale.

Mechanically it is convenient to have wave periods in the range 0.5-1 sec. In nature the range 5-10 sec. is common. A convenient time scale ratio is therefore 10. The length ratio thus becomes 100.

3. Studies with wave tank Mark 1

The Wave Tank

Within a covered model wave basin (about 130' × 100') which had been used for other studies, a smaller rectangular section was partitioned off. This smaller basin consisted of a beach 36 ft. (later 24 ft.) long parallel to, and 40 ft. from, a bank of pneumatic wave generators. The sides were formed by moveable wave boards. To form a backing for the beach asbestos sheets 2 ft. wide were laid at 22.5°. Against these sheets, beaches with slopes of 6° and 12° were constructed of sand or tin plate. (4 ft. wide). See Fig. 3.10.

From the beach to the wave generators the bottom of the basin was sand whose contours were uniform and parallel to the beach. The depth of water over the sand bottom was 6'" from still water level.

The pneumatic wave generators produced waves with heights varying from ¼" to 3" and a small range of periods 0.635 to 0.82 secs. Unfortunately, being old the wave generators produced waves whose heights were spatially somewhat variable. The direction of wave propagation was normal to the beach.

Methods of Measurements

Wave Periods - The periodicity was determined by timing 30 waves.

Wave Heights - Heights of waves were measured at a position some 10-15 ft from the beach by recording the vertical oscillation of a cork float on a clockwork recorder. Difficulties were encountered probably on account of seiching.

Beach Widths - The position of maximum and minimum uprush, mean "sea" level, the point of breaking of the wave, and the point at which the wave face became vertical were measured against a scale

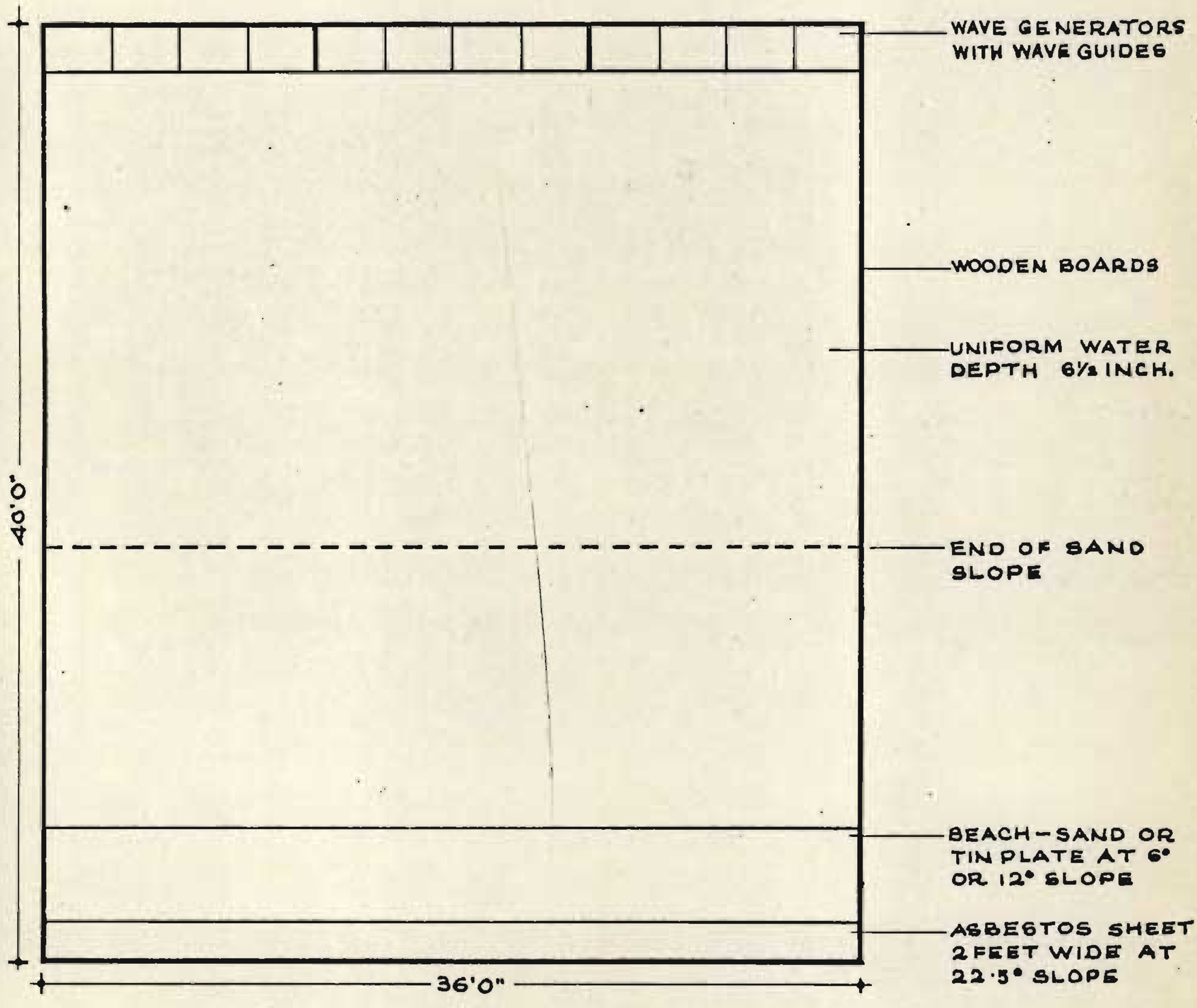


FIG. 3-10. PLAN VIEW OF WAVE BASIN MODEL MARK I

by observation through a telescope with crosswires.

Rip and Oscillation Spacing - The distance apart of rips and the peaks of oscillations was measured against a scale painted on the asbestos boards.

Longshore Currents and Rip Current Velocities - Velocities were determined by timing the head of a dye patch over 12-20 inches.

Dye Concentrations - Dye samples (sodium fluorescein) were taken batchwise and analysed in a fluorimeter.

Photography - Photographic records were made using a 16 mm camera with Kodak Tri-X (black and white) with aperture f 8-11, and Type A colour film with f 4-5.6 both at 16 and 24 frames/sec.

Still photographs were taken with Agfa CT18 with aperture f 5.6 at 1/30th sec. A few were taken with Kodak Tri-X pan fast black and white.

For lightning four spot lights of 500 watts and two "sun guns" of 1000 watts each were used. Later the sun guns alone were found to be sufficient. [Note: In the photographs which follow straight black lines are to be seen. These were wires stretched over the tank well above the water].

TEST M.1.

Purpose - to see if the breaking of a uniform wave train approaching normally to a uniform beach would produce a cellular type circulation.

Procedure - A solution of sodium fluorescein and/or rhodamine B dye (suitably diluted with ethyl alcohol to adjust its density to that of fresh water in the tank) was dispensed along the waters edge, or continuously at a fixed point, when the waves were breaking.

Results - It was at once apparent that cells of circulation as marked by the dye occured all along the beach. These cells are illustrated in photograph 3.5b when sodium fluorescein (yellow) and rhodamine B (red) were released continuously at the water's edge at a point mid-way between the rips and were carried out by rip currents. The different dye colours can be seen in each half of the common rip current. At the time the photograph was taken recirculation back towards the beach had not yet commenced. In photograph 3.5a the dye was dispersed in a batch along the waters edge and was photographed when the dyed longshore current had cleared into the five rip currents. The start of recycling is evident. The white circles at the base of each rip are foam. Photograph 3.6 shows some detail of a large rip current. For comparison photograph 3.7 of a large rip current occurring in the field is included. The similarity in shape is remarkable. In photograph 3.8a and b smaller rips between the large ones can be seen.

Other points of interest are:

(a) The water in the rip appeared to move offshore in the interval between the arrival of the breakers. Visual observation suggested that with the breaking of the wave, the outflow was halted. (Use is made of this fact in later calculations). There was no long period pulsing as observed in nature - but unlike those in the prototype the waves were of course of uniform height.

(b) The second notable feature was that the outflow of water in the rip currents produced an apparent retardation of the incident wave front. Looked at from the beach the wave front was concave in the region of the rips. This concave portion of the front resulted in a component directed along shore towards each rip. Examples are clearly to be seen in Photographs 3.6 and 3.8.

(a)

(b)

Photographs 3.5a & b. Cellular circulation from waves 0.5" high period 0.75 sec on a tin plate beach at 6° slope.

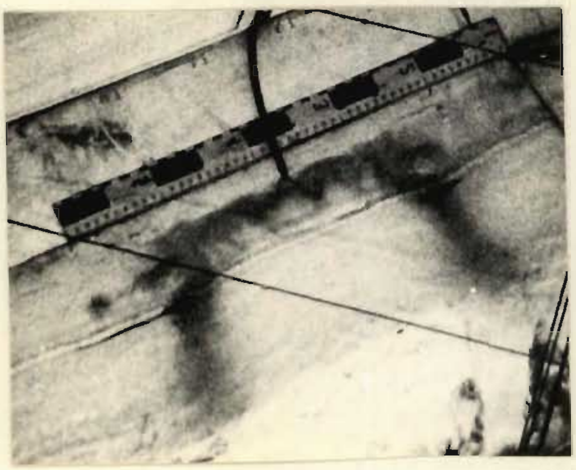
(a) five rip currents marked with red dye - recycling has just begun. White circles at the rip base are foam.

(b) two circulations cells marked with red and yellow dye applied continuously.

Photograph 3.6 detail of a large rip current in the model with small rips in the red dye.

Photograph 3.7. A large rip current in the field. Note wave refraction.

Note: In the three wave tank photographs the generated waves approach from the bottom and break on the beach just above which is a measuring scale. The black lines across the photographs are overhead wires.



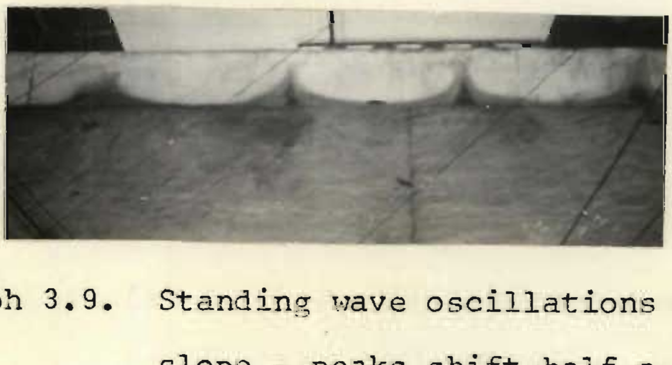
(a)

(b)

Photograph 3.8 (a) Cell of circulation showing small rips between the dominant ones, and wave retardation by dominant rips - continuous application of dye

(b) Detail of a half cell showing rip feeder and sideways motion near rip base - the wave had just broken

(Waves of height 0.5" and period 0.75 secs were propagated from the bottom of the photographs).



Photograph 3.9. Standing wave oscillations on 22.5° asbestos slope - peaks shift half a wave length with each successive generated wave. Generated waves of period 0.75 sec approaching from the bottom of the photograph.

The wave obliquity was greatest near the rips and negligible mid-way between them. This latter fact was particularly apparent when rip spacing was large. Dye travelled away from the midpoint towards each rip - as in nature. The average angle of obliquity of the wave front at breaking was about 8° for ½" waves. It extended for not more than a foot in each side of the rip.

(c) The recycling action was well simulated in the model - a portion of the water in the rip head recycling into one or each of the adjacent cells. Recycling was most vigorous near the rips.

The above refers to a tin plate beach at 6° with ½" waves of 0.75 secs

With a moveable sand beach the rips were not so clearly defined and tended to shift position. It seemed that the beach did not settle down to an equilibrium state. A submerged sand bar and trough inshore of it formed as in nature. At first a sand-bar built out seawards at the rips, and later (after 10-20 minutes) the beach contour at the waters edge built seawards at the rip base forming a small promontory.

As with the tin plate base small rips not penetrating the breaker line occured between the large rips. Gradually, beach cusps developed in between these small rips which had a seperation of about 9". (wave period 0.75 secs)

Using the tin plate beach a phenomana of some interest occurred when the waves were first started. Under the still water condition dye had been inserted along the shore. With the arrival of the first waves the dye behaved in the following way:

(a) It was compressed into a fairly uniform band by the first few waves.

(b) With arrival of more waves small rip currents formed but they did not penetrate the surf.

(c) Later some of the small rip currents grew, penetrated the surf and established the circulation.

Selected ciné pictures record this process in Photograph 3.10. The upper picture shows the organising of the dyed water into a band. Prior to the arrival of the first waves it was irregularly dispersed across the surf zone. The centre picture shows the incipient rips. Note the transverse oscillations occurring along the incoming wave front just before breaking. The lower picture shows the two rips which had fully developed. One undeveloped rip still marked with dye can be seen within the surf zone. The apparent retardation of the wave front by the rip currents can be seen in the breaker surging up the beach.

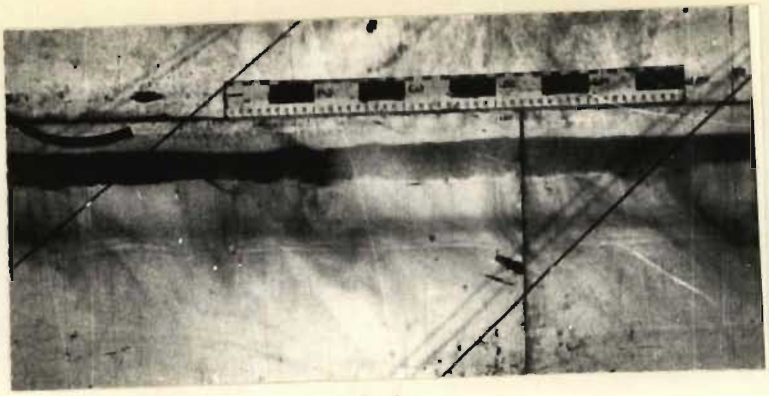
TEST M.2.

Purpose - To measure the spacing of rip current with waves of various heights, (period fixed) and the accompanying surf width.

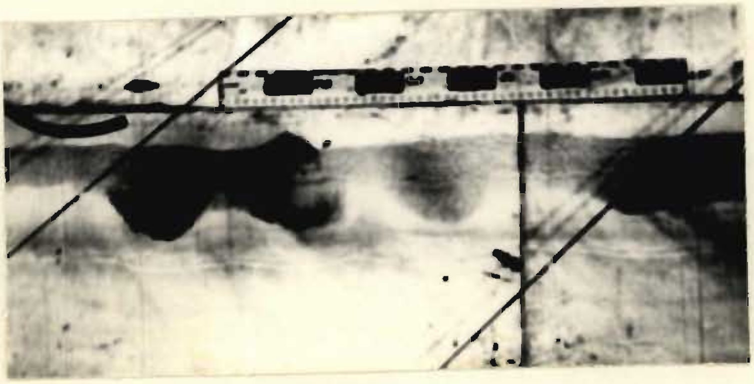
Conditions - Tin plate beach at 6°. Wave period 0.75 secs.

Spacing of Rip Currents - The spacing of dominant rip currents as marked by dye was measured for waves of various heights. Spacing were not absolutely regular and furthermore, the presence of small rips (hardly penetrating the break point) and evanescent rips sometimes complicated the picture. A further complication was the fusing of two rips before or just after passing out of the surf zone in a way similar to "feeders" in the prototype. In general the small rips were disregarded and the fused rips treated as one. Fig. 3-11b based on data in Appendix 5 shows the change of average spacing. The trend is an irregular increase of spacing with wave height.

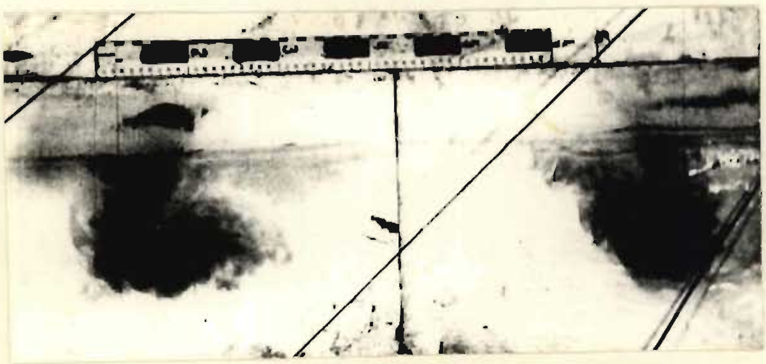
Surf Width - Wave Deight Relationship - When a wave enters shoaling water its steepness increases until a point is



(a)



(b)



(c)

Photograph 3.10. Enlarged frames from a 16 mm. ciné film of stages in the development of rip currents starting from the still water condition. The direction of wave propagation is from the bottom in each photograph.

(a) virtually undisturbed band of dye at the water's edge - just below the measuring scale

(b) potential rips developing at 4 points - note undulations in the wave front

(c) only two rips have fully developed.

Total elapsed time - 30 secs.

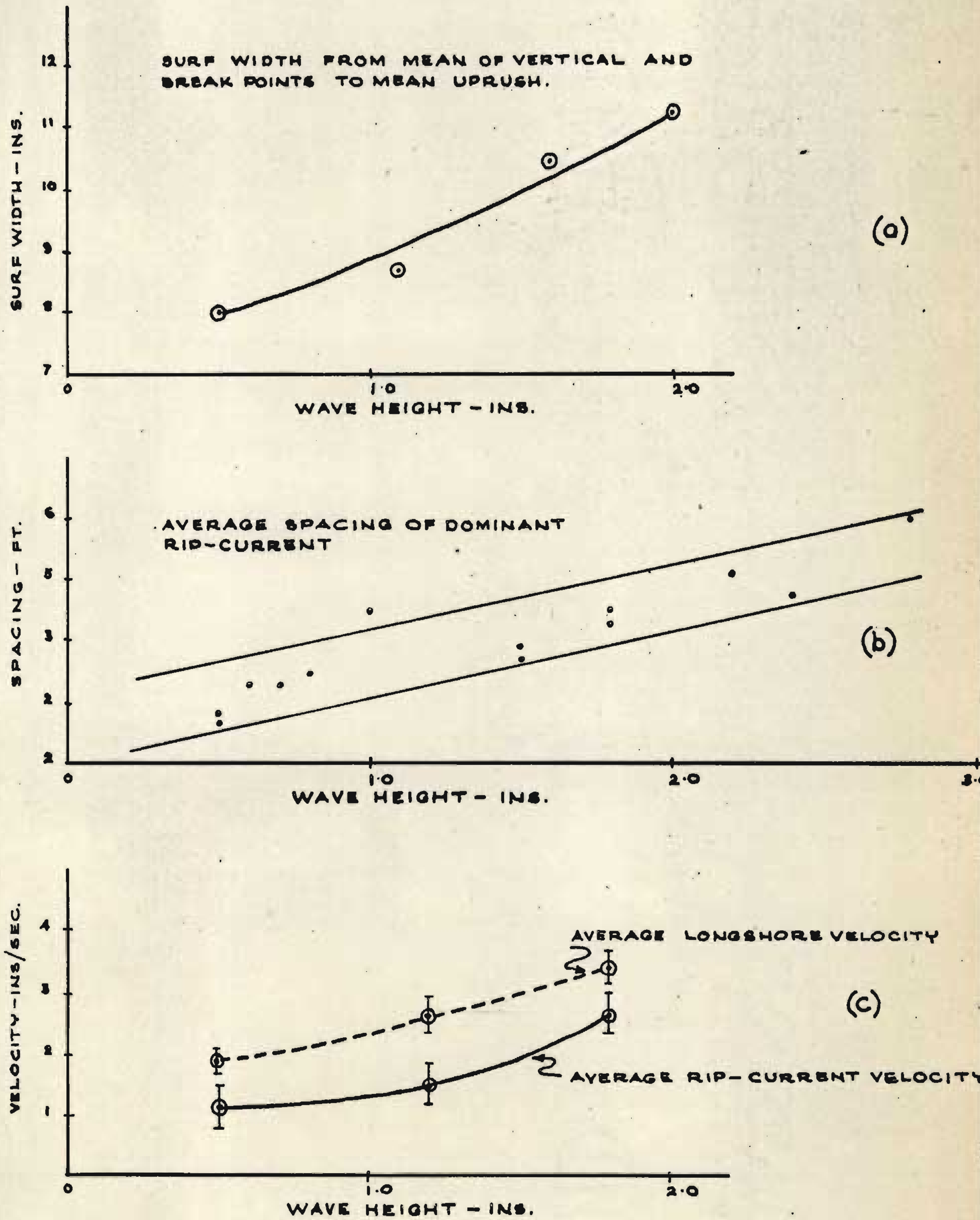


FIG. 3-11 EXPERIMENTAL DATA FOR A TINPLATE BEACH AT 6°
WAVE PERIOD = 0.75 SEC.

reached where the wave face is vertical. The wave then breaks. However, since breaking is a dynamic process there is a significant distance between the point where the face is vertical (afterwards referred to as the vertical point) and the point where the wave collapses (the break point). The breaking wave sets up the water-line above still water level. The level reached is the point of maximum or minimum uprush. In determining the surf width to wave height relationship all five points were recorded, i.e. the vertical point, the break point, the still water sea level, and the points of minimum and maximum uprush. The observations are recorded in Appendix 4.

The surf width of course increases with wave height. The distance from the mean of the vertical and break points to the mean uprush is plotted against wave height in Fig. 3-11a.

TEST M. 3.

Purpose - To measure the mean velocity of the longshore current between rips.

Conditions - Tin plate beach at 6° slope; wave period 0.75 sec.

Method - The longshore currents flow from some point approximately mid way between adjacent rips to the rip base. Over a measured distance 12-20 inches long, the passage of the head of a small patch of sodium fluorescein dye was timed, for various wave heights. Care was taken to start each run in the same wave phase.

Results - The velocity measured was an average velocity and ranged from about 2-3.5 inches per second with waves of height 0.5 - 1.8 inches. The trend was one of increasing longshore velocity with increasing wave height.

The results have been recorded in Appendix 6 and summarized in Fig. 3-11c).

TEST M. 4.

Purpose - To measure the mean velocity of the rip currents.

Conditions - Tin plate beach at 6° slope. Wave period 0.75 secs.

Method - A method similar to that for Test M.3. was used. The velocity measured was an average velocity over an arbitrary measured distance from the break point. Measurements were started at the same wave phase.

Results - The results are recorded in Appendix 6 and summarized in Fig. 3-11c. The velocities were less than for the corresponding longshore current and ranged between 1.2 to 2.6 inches per second over a wave height range of 0.5-1.8 ins.

TEST M.5 CONCENTRATION CHANGES OF INSERTED DYE.

Purpose - Two experiments were carried out to measure the build up of dye concentration at the shore resulting from continuous releases of dye.

Conditions - Tin plate beach at 12° slope. Wave period 0.75 sec. Wave height 2.5 inches.

Method - The dye sodium fluorescein solution mixed with alcohol to render it neutrally buoyant was released through a fine nozzle from a constant head apparatus. Samples for analysis in a fluorimeter were taken batchwise at the shore in small test tubes. The raw dye solution had a dye concentration of 1,480 ppm.

Experiment 1

The dye was released about 4 ft offshore on the sea bottom near one of the rip heads, at the rate of 2.44 ml/sec. It was virtually in the rip head. The results of the fluorescence analysis of samples taken at the shore are recorded in Appendix 8. Fig. 3-12 shows the build up of concentration. The time taken for the first dye to arrive at the shore was 75 seconds. A cycle took 120 seconds.

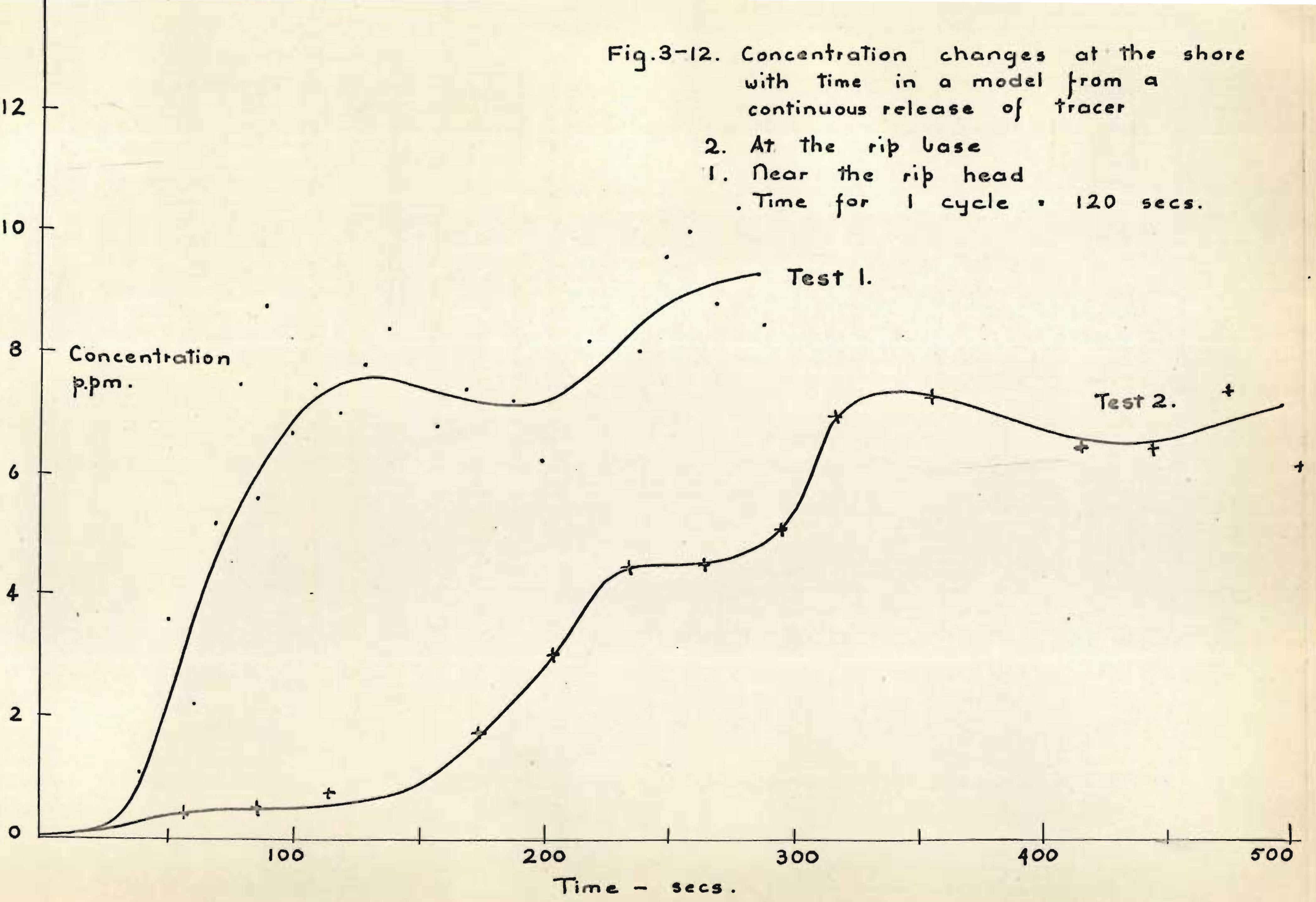


Fig. 3-12. Concentration changes at the shore with time in a model from a continuous release of tracer
2. At the rip base
1. Near the rip head
. Time for 1 cycle = 120 secs.
Test 1.
Test 2.
Concentration p.p.m.
0
2
4
6
8
10
12
100
200
300
400
500
Time - secs.

Experiment 2

In experiment 2 dye was released at the rate of 1.64 ccs/sec at the base of the rip. The results of the analysis of samples taken at the shore are recorded in Appendix 9. Fig. 3-12 shows the build-up curve of the recycled dye at the shore. In the rip itself the concentrations of dye just before the end of the test was 23.3 ppm.

It may be noted that the experiment was of insufficient duration to allow a full equilibrium situation to become established. In all recycling about five cycles were observed. The time for one cycle was approximately 120 seconds.

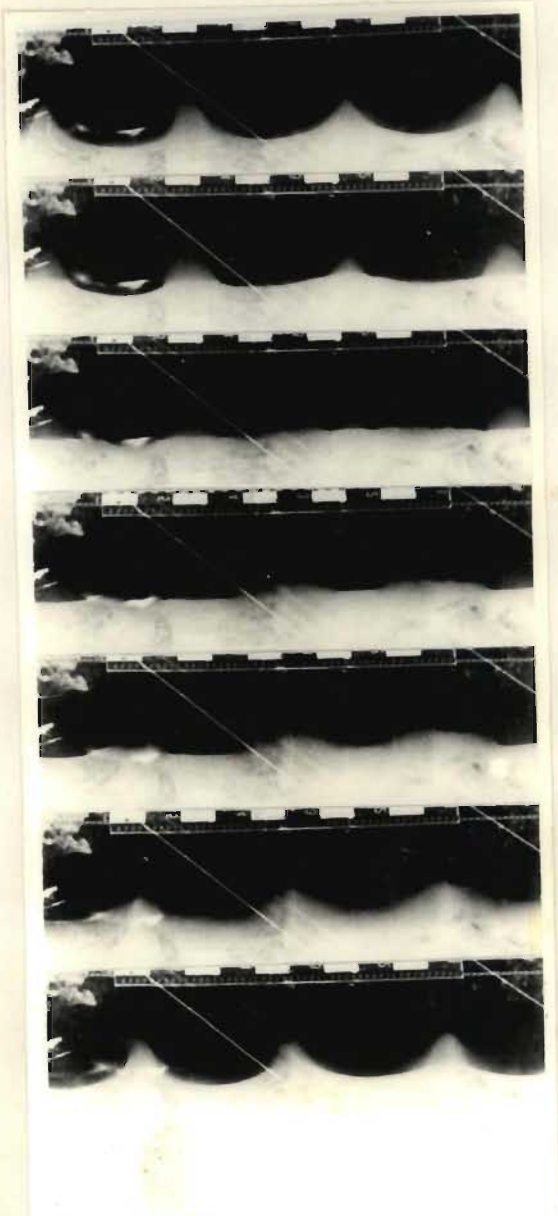
TEST M.6

It was found that an interesting phenomenon occurred with small waves breaking on a 12° slope or when the water level was raised such that the waves surged up the 22.5° asbestos boards backing the beach. In a matter of minutes oscillations built up transversely along the waters edge. See Photograph 3.9.

These oscillations were of three main types each of which could occur under the same wave conditions.

(a) The most common type is illustrated on Photograph 3.11 which is composed of selected ciné photographs. The measuring scale is set along the top of the 12° tin plate beach and the generated waves approach from the bottom of each photograph.

It may be noted that the waves look like standing waves whose crests or peaks shift a half wave length with each successive generated wave. The three dimensional shape of the wave may be seen in some of the photographs. These oscillations suggested that the



Photograph 3.11. Selected 16 mm. ciné frames showing the half wave length shift of position of crests of a standing oscillation when two successive waves (0.65 sec) surge up a 12° slope towards the measuring scale.

components of the standing wave had periods twice that of the generated wave.

(b) The second type is illustrated in the accompanying sketch Fig. 3-13. Here the peaks occurred in the same place rising with each incident wave and falling between the waves. The amplitude increased with each wave until it reached a maximum (about 9") after which it might transform into type (a) above. A fairly vigorous outflow seaward was associated with the recession of each peak. With this type it would seem that the period of the components of the standing wave were originally equal to that of the generated wave.

(c) A variant of type (b) was where the crest rose with the arrival of every alternate generated wave. This did not transform into the other type. This kind of standing wave might be formed if one component had a period equal to that of the generated wave and the other twice it. Type (a) was subsequently studied in more detail in the Mark 11 model.

4. Studies with Wave Tank Mark 11

The second model which was built at Kloof was 19 ft. long and 20 ft. wide. It was constructed by building a 2 ft. wall around a cement slab. The walls were plastered on the inside. 9 ft. from the end with the wave generators a 6° cement beach sloped up to the other end of the tank. The surface of a band across the beach which included the breaking and run up zone, was rendered smoother and more uniform by the application of a fine grained

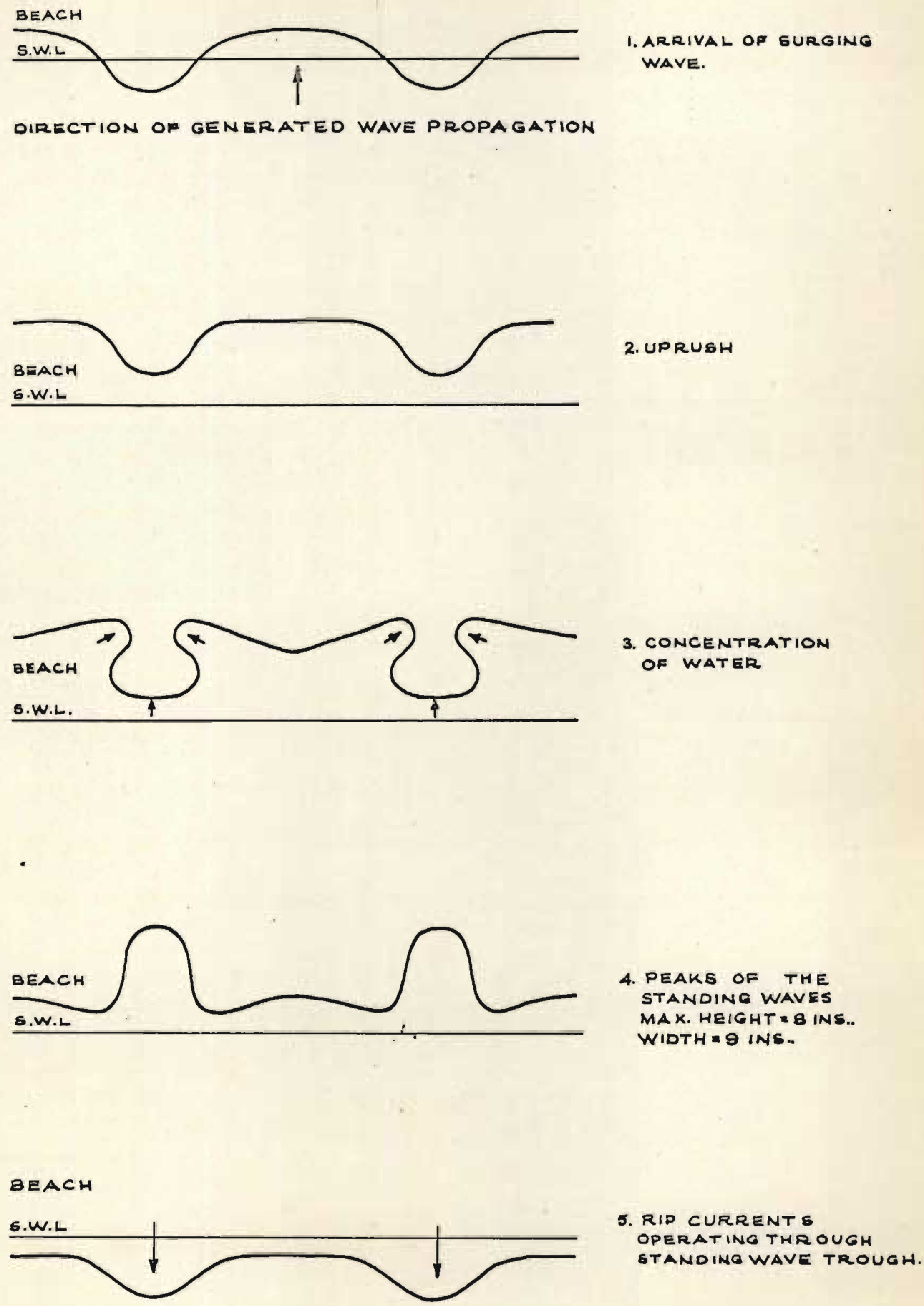


FIG. 3-13. SCHEMATIC DIAGRAMS SHOWING PHASES OF A FIXED POSITION STANDING WAVE, DUE TO WAVES SURGING UP A 22·5° SLOPE (WAVE PERIOD 0·75 SEC). PEAKS OCCURED IN THE SAME PLACE WITH THE ARRIVAL OF EACH GENERATED WAVE.

"polyfiller" which was painted white. Photograph 3.12 gives a general view of the tank.

The wave generator was of the paddle type, hinged at the bottom. It consisted of a 20' × 12" × $1\frac{1}{2}$" hardwood board (coated with epoxy resin) which could be moved to and fro by a reciprocating motion provided by a $1\frac{1}{2}$ h.p. motor acting through a gearbox. The wave period could be varied by altering the tension on a variable speed pulley. With suitable ratios of pulleys, periods of 0.6 - 1.65 sec. could be obtained. Alteration of the length of stroke of the reciprocating mechanism allowed a range of wave heights of 0.2-1.5 inches.

The uniformity of the wave front obtained by this generator was superior to that in the original model. The waves produced were, however, not unique but contained harmonics.

The initial purpose of the continued wave tank experiments was:

(a) to investigate further the effect of wave height and period on the disposition of rip currents.

(b) to study the marked standing wave oscillations found previously.

TEST M.7.

With regard to the study of the disposition of rip currents, the procedure adopted was to record the places (as indicated by dye released along the waters edge) at which rip currents formed. Combinations of the following wave heights and periods were used.

Wave heights (inches)	0.25	0.55	0.75	1.0	
Periods (Secs.)	0.65	0.75	0.85	1.1	1.3

Wave periods were measured by timing thirty waves.

Wave heights were measured by means of a pair of external curved calipers. Measurements were made in the portion of the tank at two points one half wave length apart. The error in wave heights was estimated at ±10%.

Results

The wave front was satisfactorily uniform and in this respect a great improvement on that from the pneumatic generators. The rip current pattern was in general very similar to that obtained previously. Rips appeared - so far as the size was concerned - to fall into four classes.

(a) very small ones several inches apart which did not penetrate the breaker line

(b) small ones, some of the order of one foot apart, which in general did not penetrate the surf

(c) moderate rips which penetrated the surf, but travelled little beyond it

(d) dominant rip currents which penetrated well beyond the surf zone.

The first two types were transitory and the dye carried out by them soon drifted alongshore to join up with the larger rips. It was not possible to decide with certainty just where they were situated. Discrimination between the moderate and dominant rips was usually fairly clear cut, but there were occasions when the decision was necessarily subjective.

Fig. 3-14a shows a plot of the change of spacing of the dominant rip currents with various wave height for fixed wave periods. Fig. 3-14b is a plot of the changes of spacings with various periods for fixed wave heights.

There seems to be a trend of increasing spacing with increasing wave height but it was not constant or regular. On the other hand the spacing wave period graphs show no clear tendency.

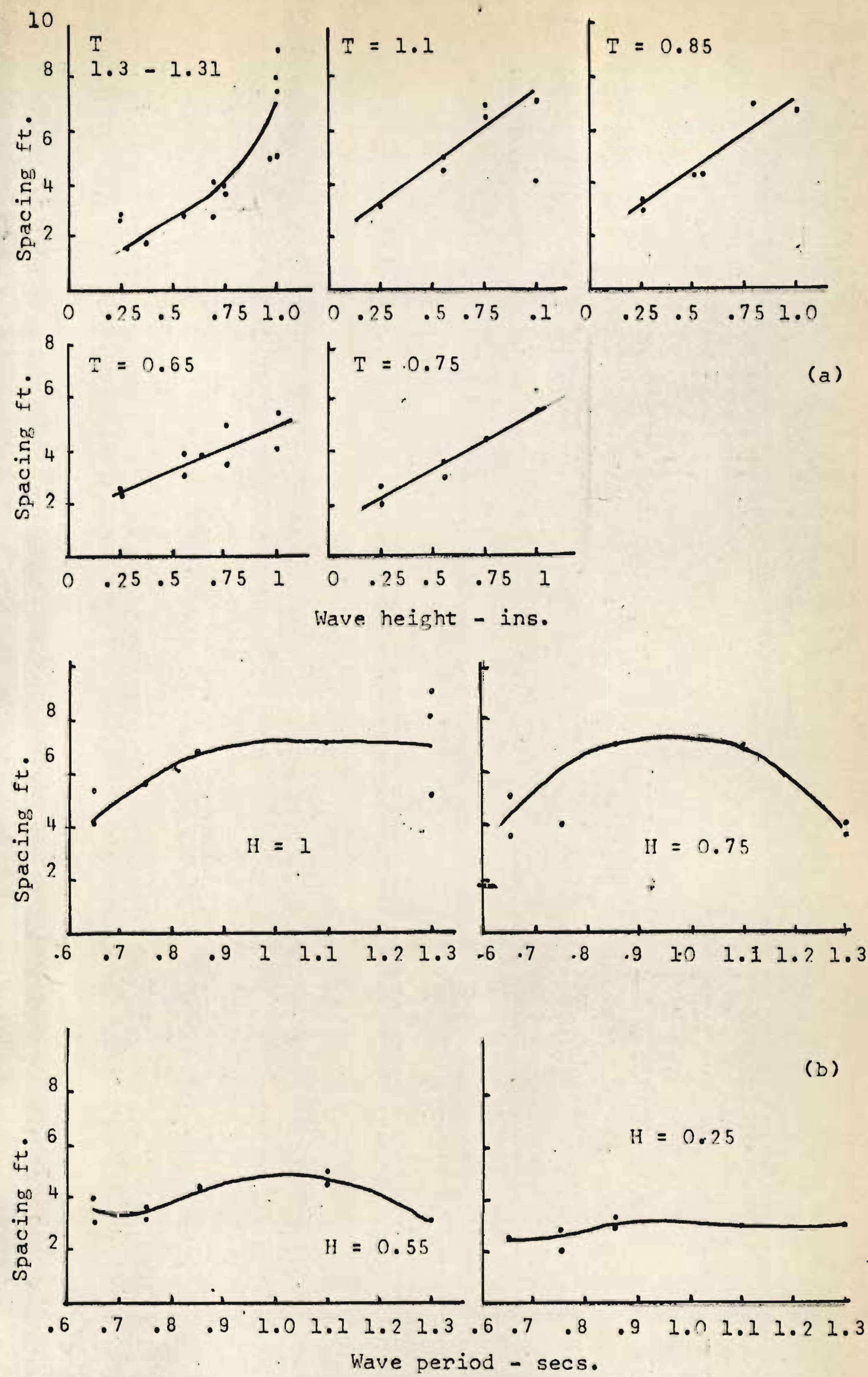


Fig. 3-14. Graphs of spacing of dominant rip currents with change of
(a) wave height (b) wave period
in Wave Tank Mk. II (T = period H = wave height)

There was a remarkable occurence when the wave period was about 0.65 seconds and the wave height 1.2 to 1.5 inches. Three dimensional standing waves were very clearly observed just "sea ward" of the break point. The wave crest at breaking was corrugated in the vertical plane and scalloped in the horizontal plane. At fairly regular intervals of about 12" a short section of wave crest (2"-3") broke in advance of adjacent portions. Associated with each section there was a small rip current - about 20 over the width of the tank.

Photograph 3.13 shows the standing waves, Photograph 3.14a and b some of the surface patterns, Photograph 3.15 the scalloped breaker front and Photograph 3.16 the corrugations along the crest.

Examination of the ciné film from which photograph 3.16 was taken showed that there were differential times of breaking along the wave crest. The troughs or shallower portions of the corrugations broke first.

TEST M.8.

Study of Standing Wave Oscillations

Another interesting feature was the occurrence of regular standing waves (similar to those reported for Model Mark I) formed in the plane of the beach along the waters edge. These developed about 90-120 seconds after the wave generator had been started. The oscillations are illustrated in Photographs 3.17 and 3.18. They consist of something like parabolic shaped crests separated by shallow troughs. Their position shifted one half wave length at the arrival of successive generated waves. They gradually grew in amplitude until they reached a maximum. Beating was sometimes observed. The oscillations were only observed with small generated waves (less than 0.5 inches) and periods between the maximum

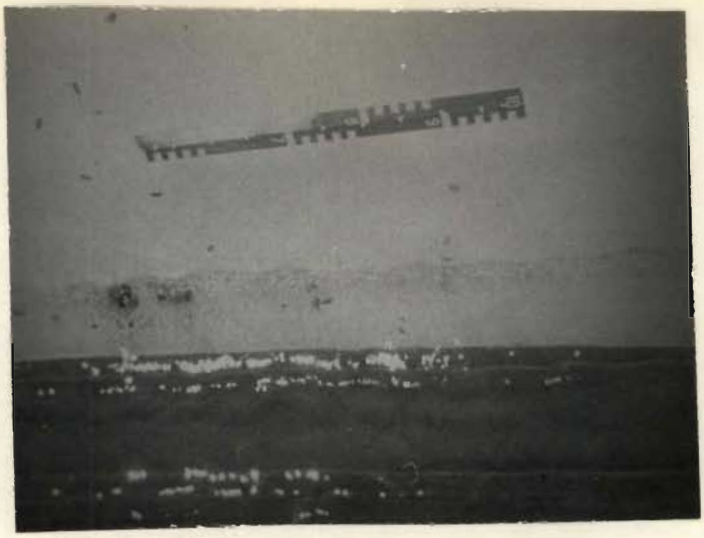
Photograph 3.12. Wave tank Mk.II built at Kloof. Wave generating board is top right. 6° concrete sloping beach has been painted white. Width of tank 20 ft.

Photograph 3.13 showing standing waves across the tank (Mk.II) just before the wave break point. Note patterns over white portion of slope. Wave height 1.5" wave period 0.65 secs wave direction left to right. (Dark line is ½" expansion joint at bottom of 6° slope).

(a)

(b)

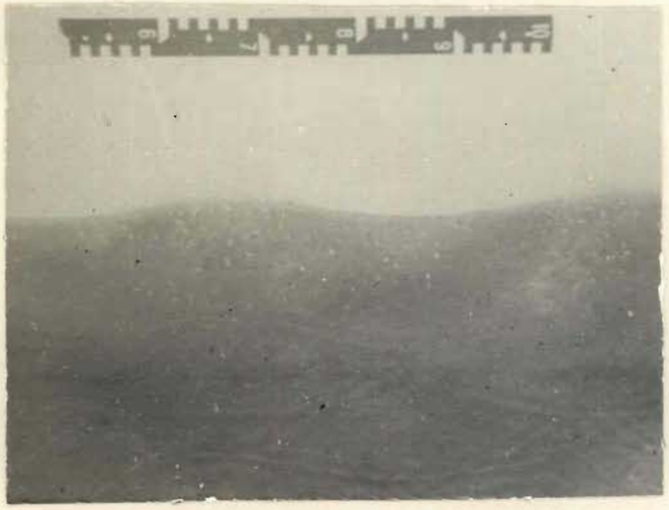
Photograph 3.14. Standing waves - probably resonance condition - caused by 1.5" waves of period 0.65 sec propagated from left to right. Associated with the generated wave just on the white portion of the 6° slope, the components of the standing wave can be seen - especially at the top of Photograph 3.14b. They are almost certainly edge waves.



Photograph 3.15. Vertical photograph of waves 1.2" high and of period 0.63 sec propagated from the bottom of the photograph, showing scalloped pattern of wave at breaking just prior to rushing up the beach (6°) towards the scale. Scallops are probably caused by a standing wave (near resonance ?) which can be seen traversing the tank just below the white part of the slope.

Photograph 3.16. Very oblique view of a wave at breaking showing marked transverse corrugation along the crest. The photograph was taken from near still water level at the beach. The waves (height 1.2" period 0.65 sec) are approaching the camera. From a 16 mm ciné film taken at 24 frames per sec on the same occasion as photographs 3.14 a & b.

Photograph 3.17. Oblique view along the 6° slope with waves (height 0.3" period 1.31 sec) propagated from right to left and interacting with a transverse standing wave along the water's edge (running up the photograph) to produce weak rip currents marked with dye. The generated wave has just spent itself and produced large amplitude surges opposite what had been a down slope trough of the standing wave.



(a)

(b)

Photograph 3.18 a & b. Vertical view of the beach (white) with generated waves (height 0.3" period 1.295 sec) propagated from the bottom of the photograph towards the scale, interacting with transverse standing waves along the water's edge. The water was dyed dark red. Note patterns on the water surface.

(a) just prior to the arrival of the generated wave

(b) just after the wave had spent itself on the 6° slope - water from the crests or surges is returning down slope.

possible period, 1.65 seconds, and about 0.95 seconds. There appeared to be a tendency for the oscillations to disappear or be masked with higher wave heights. Amplitudes were not measured but they were clearly greater with the longer periods.

When dye was dispersed along the waters edge it was observed that slow moving rip currents formed opposite the retreating crests. See Photograph 3.17.

The number of nodes along the 20 ft. beach and the wave length - distance between simultaneous peaks - are given in the Table 3.6. With some periods the nodality was not unique but varied between values differing by one. During the decay of the oscillations after the generator had been switched off, what appeared to be the component waves of the standing wave could be observed. There seemed to be a possibility that the standing wave was due to "edge" waves. To verify this the following experiments were performed.

EXPERIMENT WITH TRANSVERSE WAVES

TEST M.9.

Purpose - The main purpose of the experiment was to generate waves whose direction of propagation was parallel to the beach.

Equipment - The generating gear was disconnected from the main wave generating board, and connected by means of cords running over pulleys to two short wave boards set along either side of the tank. The boards were 6 ft. long and 6 inches deep and were secured by hinges attached to the side walls near the bottom, about halfway down the slope. The tops of the boards were level, and a few inches above the still water level.

When the generating gear was set going the boards oscillated to and fro producing waves progagated simultaneously, and initially

TABLE 3.6

NODALITY AND WAVE LENGTHS OF THE STANDING WAVE FOR VARIOUS PERIODS OF THE GENERATED WAVE-SLOPE OF BEACH 6°

Period - secs	Nodality (N)	Wave length ft. = $\frac{2 \times 20^{*}}{N}$
1.62	7	5.71
1.60	7	5.71
1.56	8	5.00
1.54	8	5.0
1.50	9	4.45
1.45	9	4.45
1.42	9	4.42
1.40	10	4.00
1.32	10	4.00
1.30	11	3.65
1.29	11	3.65
1.25	12	3.33
1.22	12	3.33
1.20	13	3.08
1.185	13	3.08
1.17	14	2.85
1.135	15	2.66
1.125	15	2.66
1.11	16	2.50
1.0	18	2.22
0.95	21 (approx.)	1.92

parallel to the beach.

Results - The initial wave fronts were soon distorted by refraction, the portion nearest the waters edge lagging. Small edge wqves were seen to move rapidly along the waters edge.

By superposition the transverse waves (generated from either side) produced a standing wave qualitatively similar to that reported above. Because of the time taken for the wave expansion to occur the oscillations near the sides of the tank were not clearly defined. See Photograph 3.20. When only one one board was oscillated the edge waves as shown in Photo. 3.19 were produced.

TEST M.10.

Purpose - To examine the interaction of waves generated normally to the beach and from both sides.

Procedure - The wave generating mechanism was reconnected to the main wave board so that it simultaneously generated normal and transverse waves.

Results - The standing wave at the waters edge was different in this case. Antinodes now occur at fixed positions with each wave. There was not the half wavelength shift of antinode positions with the arrival of successive normal waves that there was in Test M.8.

TEST M.11.

Purpose - To examine the interaction of edge waves generated from only one side, with normally incident waves.

Procedure - One of the side flaps only and the main wave board were connected to the generator. There was therefore interaction between only these two wave trains.

Results - At breaking the main wave train was distorted where it was superimposed on the transverse wave. At the distortion a rip

Photograph 3.19. Vertical photograph (an enlargement from a 16 mm ciné film), of two edge waves of period 1 sec propagated from right to left along a 6° beach slope. The waves which are probably the 3rd mode are outlined by dye. The measuring scale marks the water's edge.

Photograph 3.20. Oblique photograph (looking along the water's edge) of two edge waves travelling in opposing directions and forming a standing wave - which is marked with dark dye.

current was formed. Its direction of travel was slightly oblique in the direction of the transverse wave propagation.

CHAPTER IV

DISCUSSION ON THE FIELD AND WAVE TANK STUDIES

1. The similitude of model and prototype

One of the most interesting findings was that the symmetrical cellular circulation of the prototype could be reproduced in a uniform wave tank. Most of the salient features of the former were well reproduced. The longshore currents flowed away from mid-cell in opposite directions, rip currents flowed seawards and in the process retarded the oncoming wave. Feeders supplying the rips were also reproduced, and the similitude between the spreading of the rip head and recycling was good. A notable difference was that the wave tank rips exhibited no long period pulsing. However since pulsing is thought to be associated in nature with the arrival of successions of high and low groups of waves, the uniformity of wave heights in the model may well explain the difference.

Concerning the dimensions of the cells the following is a comparison of the average cell width to surf width ratios.

			Average Ratio
Field	(a)	Cells involving the whole surf	3.7
	(b)	Cells involving inner breaker zone	2.8
Wave tank			4.75

The wave tank ratio was derived from Figures 3-11a and b. The differences may perhaps be ascribed to

(a) the difficulty of deciding on what exactly are the boundaries of the inner breaker zone - or of any breaker zone.

(b) a difference in depth at breaking which might result from the moveable sand bed in nature and the smooth tin plate in the tank; and also the effects of the bar and beach step in nature.

The mean spacing of the large rips were 1830 ft (565 m) compared with 500 m (Larras 1957 as quoted by Bruun 1963) and 400 m measured at Clatsop by Shepard and Inman (1951).

2. The causes of cellular circulations

The reproduction of the nearshore circulation in wave tanks provides a useful tool for studying the underlying principles.

So far as is known the only previous theory with experimental backing which identifies a cause of the circulations is that of Shepard and Inman's (1950). In this instance the theory explained satisfactorily the field observations. As noted above it was based on the spatial variations in wave heights resulting from divergence and convergence in the region of the very striking submarine canyons which were situated off the test beach.

The field work at Virginia Beach where the offshore topography is relatively uniform and the production of the circulations in the wave tank where the topography and generated wave heights were uniform, leads to the conclusion that topographical irregularity is not a primary prerequisite for cell formation. Though there is of course no doubt that where topographical features are present they have a most potent influence. The wave tank data suggests rather that a mechanism for cell formation might lie elsewhere and that it might be found in factors which tend to distort the incident waves or breaker fronts - spatial variations in hydrostatic pressure along the shore being a consequence.

3. Systems in Unstable Equilibrium

3.1. It has been noted that as the waves progress into shallowing water the mean horizontal displacement of particles beneath the wave increases. At breaking the momentum sets up the water level at the shore, which in turn induces a return flow. There is also an accelerated imparted to water returning seaward as the backwash flows down the beach slope. Intuitively it would seem that these opposed accelerating systems must be unstable and sensitive to small transverse disturbances.

3.2. Similar systems in unstable equilibrium have been the subject of study in other branches of physics. One of the first unstable systems to be analysed was that of liquids heated from below.

Bernard (1900) as reported by Rayleigh (1916) found that a thin layer of liquid when heated uniformly from below "rapidly resolves itself into a number of cells, the motion being an ascension in the middle of the cell and a descension at the common boundary between a cell and its neighbours". Using mathematical methods suitable for systems falling away from an unstable equilibrium Rayleigh was able to predict the mode of greatest instability of a disturbance (cell width) in terms of the depth of liquid, viscosity and heat conductivity. He found a constant ratio between the wave length of the mode and depth of liquid. The seemingly analogous ratio between the rip spacing and surf width observed in the field prompted the wave tank experiments reported above.

3.3 Lewis (1950) described experiments in which a layer of liquid whose surface was disturbed by applied oscillations, was accelerated downwards by the release of high pressure. The outcome of the experiment was that "fingers" of air penetrated into the liquid. See Fig. 4-1. These "fingers" are somewhat reminiscent of rip currents. Taylor (1950) has developed the theory of the rate of penetration. Brooke Benjamin (1957 and 1961) studied theoretically and experimentally the stability of a thin film of liquid flowing down a uniform slope. He found that the system was unstable and developed waves travelling both in the direction of the fluid flow and transversely. Fig. 4-2 taken from his paper (1961) shows this. Though he was mainly concerned with the development of instability in response to a perturbation (in this case the pressure of air from a jet - see upper patch of waves in the figure) his experiments also showed transverse waves developing from what he suggests was an oscillation in the trough supplying the fluid. These are the waves in the lower portion.

Though the possible applicability of this system to the surf zone was arrived at independently from the field work, the idea was first speculated upon by Isaacs (1964).

3.4. The essence of the Rayleigh-Taylor instability (as it is named) is that the way in which an unstable system falls away from an unstable equilibrium, is influenced by small oscillations impinging on the system. One mode ultimately dominates the system and this is the one which grows most rapidly with time.

There is some evidence that such a process is at work in the very first stages of the development of the cellular circulation. It will be recalled that Photograph 3.10. revealed the formation of a number of closely spaced embryo rips with the arrival of the first few waves. But only some of these developed to set the dominant pattern. It may be supposed that these are the ones

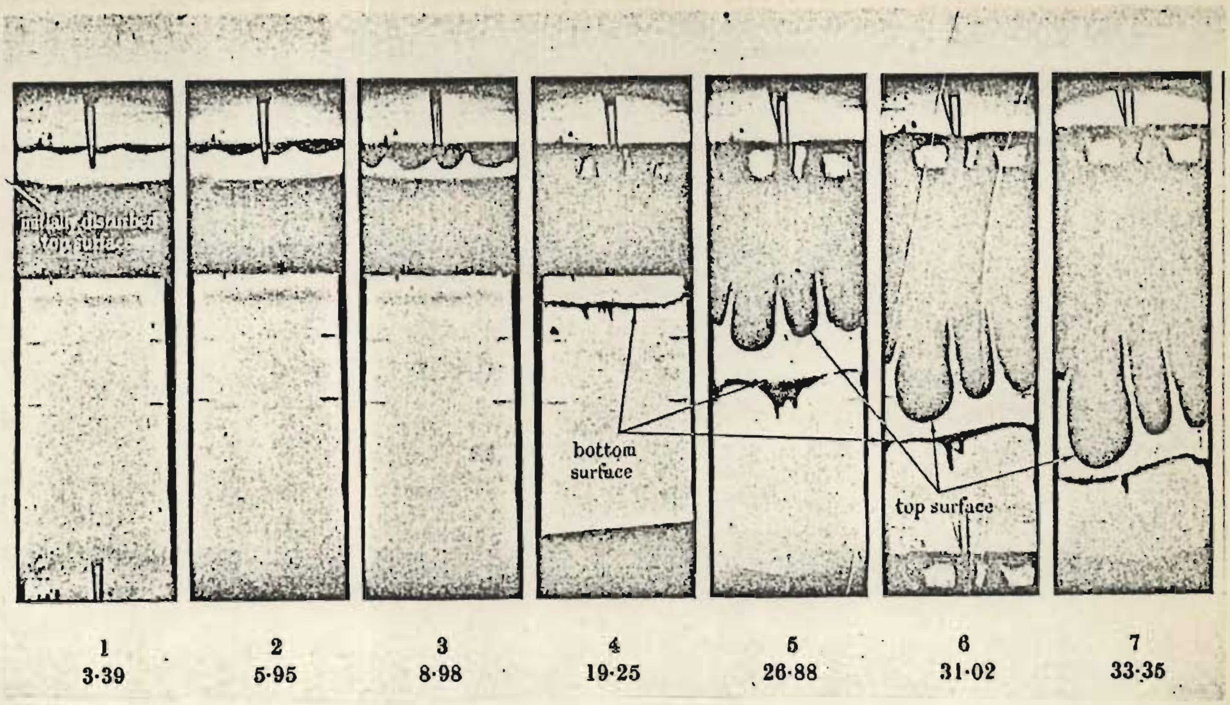


Fig. 4-1. Successive stages (micro seconds) of the acceleration of a water film showing penetration of air. (After Lewis 1950)

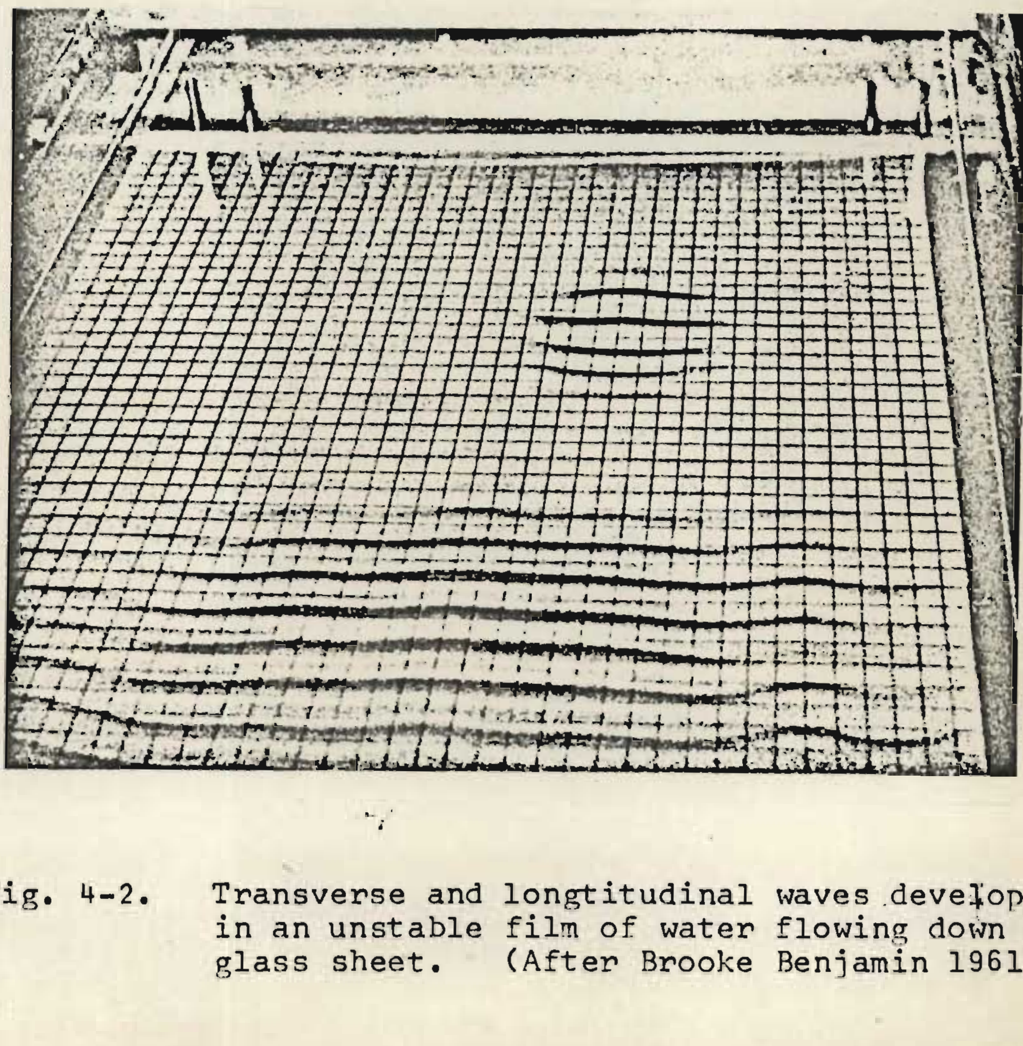


Fig. 4-2. Transverse and longtitudinal waves developed in an unstable film of water flowing down a glass sheet. (After Brooke Benjamin 1961)

which correspond to the mode which developes most rapidly with time. As to what determines the growth with time, such factors as the mechanics of the process and the need to satisfy continuity will play an important part. When an equilibrium circulation is reached however the spacing of the rips should bear some direct relationship to the wavelength of the dominant mode.

It is proposed to examine some of the wave tank observations to see whether there is some characteristic transverse oscillation which might inpress itself on the falling away from instability. From the evidence already available edge waves suggest themselves. Having looked at the problem from this aspect, attention will then be turned to the mechanics of the process.

Three cases will be examined

(a) that where waves of low amplitude were associated with large transverse standing waves along the waters edge

(b) that when rather large waves (1.2 - 1.5 inches) of period in the region of 0.65 secs were associated with notable transverse standing waves near the region of breaking

(c) those of a more general nature not associated with any particular generated wave characteristics, where the cellular circulation was present.

4.1. Standing Wave Oscillations at the Waters Edge with Low Amplitude Waves

Very large amplitude oscillations of the standing wave type, along the waters edge and extending well up the beach from S.W.L. (e.g. Photograph 3.17,) occurred in wave tank Mark II when the waves were small (deep water height less than about 0.25 inches) and when their periodicity was in the range of 0.95 - maximum possible (1.65 sec).

In appearance the oscillations were standing waves with a half wave length shift in the position of the crests between each succeeding generated wave. The motion of dust particles within the oscillations was typically rectilinear.

The occurrence of transverse oscillations is common in wave flumes and wave tanks, and they are assumed to have their origin in reflections off the side walls.

4.1.1. The water in a rectangular wave basin may oscillate in directions at right angles to either pair of opposite sides. The problem for uniform depth has been tackled by inter alia Rayleigh (1874) and Lamb (1945). The period of the oscillations will be a natural one based on the dimensions of the basin and may occur as a number of modes. Antinodes are set up at the positions where the amplitudes of the incident and reflected waves reinforce each other and troughs where they cancel or are opposed. Since the wave lengths of the incident and reflected waves are the same, the points of reinforcement and opposition are theoretically fixed in space.

When the basin is subjected to an impressed vibration it may respond by oscillating with one of its natural modes or be forced to oscillate at the period of impressed force. When the period of the forcing vibration is the same as one of the natural modes (the fundamental or a harmonic) the amplitude of the resulting oscillation is a maximum.

A wave tank such as was used is an example of a system being subjected to a forced oscillation. Such systems occur in harbours and have been studied for vertical boundaries by a number of workers notable McNown (1952), McNown and Danel (1952), Wilson (1953) and Carr (1953).

In the system used in the experiments reported above the generated waves constitute the forcing mechanism and the resulting oscillations will occur with the periodicity of generated oscillations or of their harmonics. Maximum oscillation will occur when the periodicity of the generator and the natural periodicity of the tank are the same.

4.1.2. In the wave tank experiments the depth was not uniform but part of the tank sloped up to still water level. The velocity of a transverse wave must therefore vary with the depth.

Waves which can be propagated without change under such circumstances are called edge waves and their properties have been outlined in the literature study above. Their velocity is given by Equation 1.10.

$$C^2 = \frac{gL}{2\pi} \text{ Sin } (2n + 1) i$$

Where L is the wave length i is the angle of the slope to the horizontal and n is an integer denoting the mode. Since CT = L where T is the period and recalling Equation 1.11.

$$C = \frac{gT}{2\pi} \text{ Sin } (2n + 1) i$$

and the wave length L is given by Equation 1.12

$$L = \frac{gT^2}{2\pi} \text{ Sin } (2n + 1) i$$

This last equation is interpreted to mean, that for a given forcing period T and a given slope i, there will be a number of possible edge waves whose wave lengths depend upon "n".

In the wave tank we may expect edge waves to be generated from the opposite sides of the tank and to form standing waves. At the shore the edge wave will cause a wave in the plane of the beach such that the waters edge will take on a wave form. The combination of two edge waves of the same period travelling from either side will set up a standing wave along the waters edge. The nodes will be separated by a half wave length.

It is clear from the nature of the standing wave most commonly observed that its components have a period twice that of the main generated waves. It should therefore be possible to calculate the wave lengths.

As an example consider the case when the generated wave period was 1.3 secs. In the equation expressing the wave length

$$L = \frac{gT^2}{2\pi} \sin (2n + 1) i$$

set $T = 2.6$ secs $g = 32.2$ ft/sec^2 $n = 0$(zero mode) $i = 6^\circ$ (slope of beach in model Mark 11).

$$L = \frac{32.2 \times 2.6^2}{2 \times \pi} \sin 6^\circ = 3.65 \text{ ft.}$$

This compares with 3.76 measured in the tests.

The theoretical nodality (N) across a tank 20 ft. wide for a wave of length 3.65 ft would be

$$N = \frac{2 \times 20}{3.65} = 10.9$$

compared to 10 - 11 as measured.

In Table 4.1 the calculated and observed nodalities for various generated wave periods and for two slopes have been set out.

TABLE 4.1

CALCULATED AND OBSERVED NODALITY OR WAVE LENGTH OF STANDING WAVES

Slope of beach	Incident wave period secs.	Period of standing wave secs	Nodality in 20' calculated	Nodality in 20' observed	Wave Tank
6°	1.25	2.5	12	12	Mk 11
6°	1.20	2.4	13	13	Mk 11
6°	1.17	2.34	13.7	14	Mk 11
6°	1.135	2.27	14.53	15	Mk 11
6°	1.11	2.22	15.4	16	Mk 11
			Calculated wave length ft.	Observed wave length ft.	
12°	0.66	1.32*	1.85	1.93	Mk 1
12°	0.86	1.72*	3.13	3.34	Mk 1

* one measurement only

In Fig. 4-3 the wave lengths calculated from Equation 1.12 for various periods (of the standing waves = 2 x period of generated wave) have been plotted against the square of the period of the generated wave and compared with the observed values listed in Table 3.6. The agreement is good and indicates that it was indeed edge waves causing the standing waves.

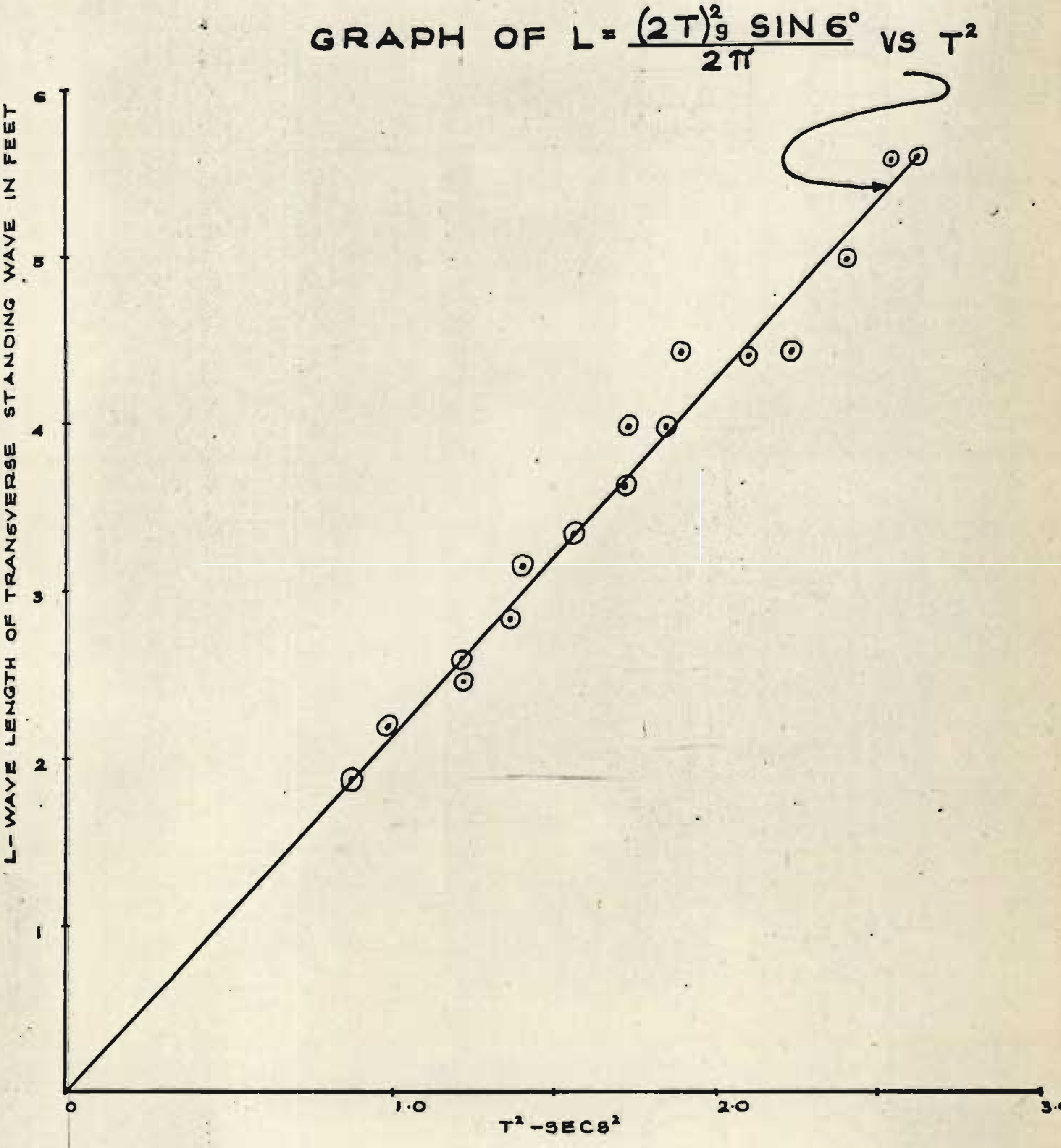


FIG. 4-3. GRAPH OF OBSERVED ⊙ AND CALCULATED (UNBROKEN LINE) WAVE LENGTHS, OF TRANSVERSE STANDING WAVE PLOTTED AGAINST THE SQUARE OF THE PERIOD OF THE GENERATED WAVE IN A MODEL.

The conclusion here is that theory predicts the phenomenon tolerably well if the periodicity is assumed to be twice that of the forcing periodicity. It must be noted that theory is based on the assumption of small amplitude which would only be applicable in the initial stages.

4.1.3. It is difficult to explain why the period of the edge waves was twice that of the generated wave. Kravtchenko and Santon (1954) have reported on what they call parasitic waves, produced from a wave generator and having twice the period of the generator. There is little doubt that the wave generator in these experiments did not produce monochromatic waves. Possibly, the explanation lies in the way the edge waves are produced. When the generated wave breaks on the wavy waters edge it may be that edge waves are produced more easily when the phase of the standing wave at the side boundary of the tank is such that a section of the generated wave breaks outwards towards the side, rather than inwards.

4.2. Examination of the phenomenon when T = 0.65 (approx.) and wave height is >1.2".

4.2.1. The marked corrugation of the surface in the region of the breaking wave and the scalloped nature of the breaker front under these conditions calls for some explanation. The corrugations have the appearance of a standing wave. (Photographs 3.13 to 3.16)

Due perhaps to inaccuracies in the construction of the model the scalloped wave front was not absolutely regular all the way across the width of the tank, and there was some difficulty in counting the exact number of concavities. There appeared to be about 20 in the 20 ft and this compares reasonably with the measured width of each concavity of 12". Their position appeared to be the same for each wave.

4.2.2. A study of photographs shows the presence of standing waves just before the break point and strongly suggest that they are caused by the superposition of edge waves. If the constituent edge waves had a period equal to that of the generated wave, antinodes of the standing wave spaced one wave length apart could coincide with a wave crest while the antinodes formed one half wave length away after a half period (of time), would then coincide with a trough of the generated wave. The next set of antinodes would occur with the next crest of the generated wave in positions the same as those for the preceeding crest. This then could account for the constant position of the cusps in the generated wave.

However, with a wave period of 0.65 secs the wave length for zero mode is:

$$L = \frac{0.65^2 g}{2\pi} \text{ Sin } 6^\circ = 0.216 \text{ feet} \qquad (4.1)$$

The measured wave length is about 1 foot. Consider the second mode (when n = 2) the wave length becomes:

$$L = \frac{g \times 0.65^2}{2\pi} \text{ Sin } 30^\circ = 1.08 \text{ feet} \qquad (4.2)$$

which is more nearly in agreement with observation.

4.2.3. It is thought that this condition is one of resonance. According to Ursell's theory resonance would occur at a discrete natural mode for the tank or at a cut-off mode. It is predicted by the inviscid theory that the resonance would tend to extend down the tank away from the beach but that in practice viscous forces would confine the resonance to the vicinity of the beach. In Photograph 3.13 it will be seen that the resonance affects the surface of the whole of the sloping portion but diminishes considerably at the foot of the slope - marked by a dark line (in the photograph) which is a joint in the concrete.

Figure 4-4a is an attempt to reconstruct the kind of pattern which might result from the two trains of similar edge waves propagated in opposing directions. The treatment is schematic and the slope of the waves is somewhat arbitary.

Such a figure, it is suggested, might explain the following aspects of the photographs:

(a) Photograph 3.13: the three dimensional humps on the water surface are probably the antinodes. These would occur at the intersections of the edge wave ridges. The figure indicates that any three adjacent antinodes would form the apices of an approximate triangle. The humps in the photograph, do suggest this formation.

(b) Photograph 3.14a: the wavy white ridge just left of centre could be represented by the dashed line in the figure.

(c) Photograph 3.14b: the intersecting wave ridges in the upper left hand quarter form a pattern of approximate parallelograms like those in the figure.

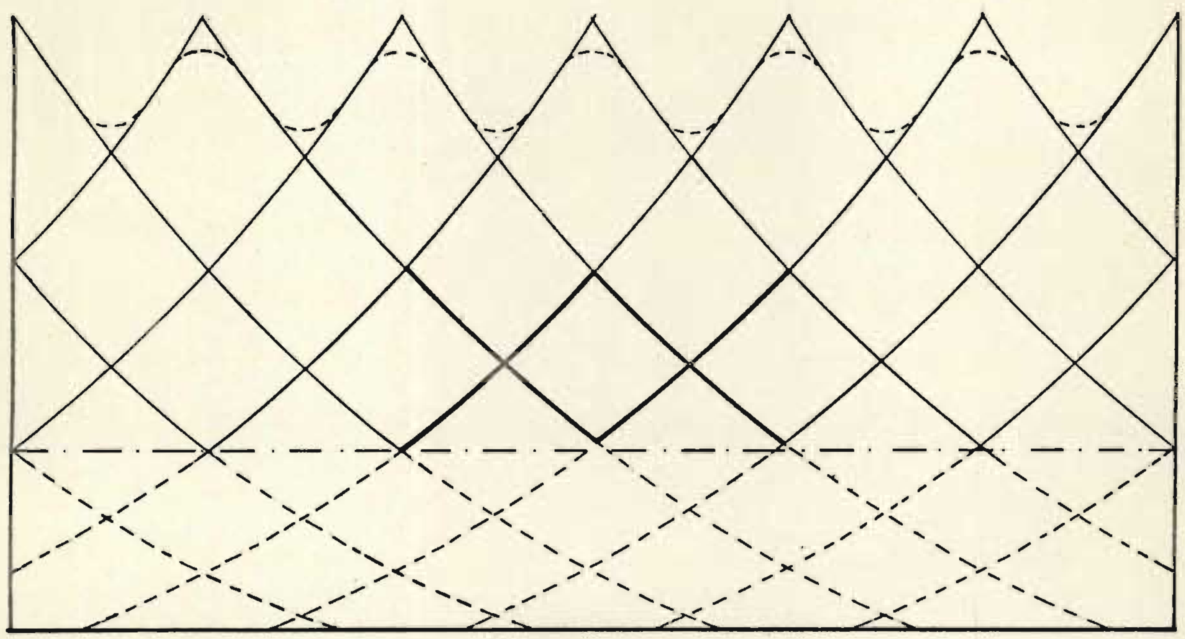


FIG. 4 – 4(a) SCHEMATIC DIAGRAM ILLUSTRATING THEORETICAL STANDING WAVE PATTERN FROM EDGE WAVES. ANTINODES WOULD OCCUR AT THE INTERSECTION OF EDGE WAVE RIDGES.

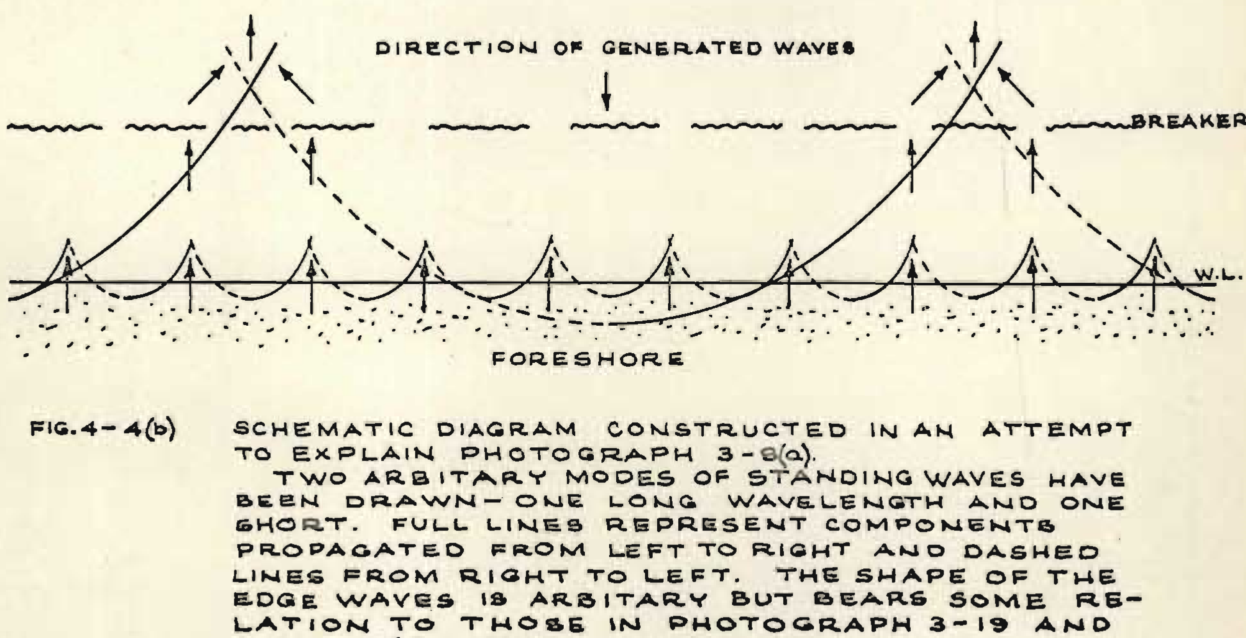


FIG. 4 – 4(b) SCHEMATIC DIAGRAM CONSTRUCTED IN AN ATTEMPT TO EXPLAIN PHOTOGRAPH 3-8(a).

TWO ARBITARY MODES OF STANDING WAVES HAVE BEEN DRAWN – ONE LONG WAVELENGTH AND ONE SHORT. FULL LINES REPRESENT COMPONENTS PROPAGATED FROM LEFT TO RIGHT AND DASHED LINES FROM RIGHT TO LEFT. THE SHAPE OF THE EDGE WAVES IS ARBITARY BUT BEARS SOME RELATION TO THOSE IN PHOTOGRAPH 3-19 AND FIG. 3-14(a)

ARROWS SHOW POSSIBLE STREAM LINES OF RIP FEEDERS AND RIPS.

After drawing Figure 4-4a, a re-examination was made of the ciné film which was taken vertically downwards. It was viewed frame - by frame - under a microscope and clear evidence of the approximate parallelogram patterns (mentioned in paragraph (c) above) was found. Photograph 4.1 is an enlargement of one of the frames. The presence of these patterns just behind the breaker establishes the presence of edge waves. It may be seen from the photograph that the ridge of the edge wave is not a single ridge but a parcel of smaller ones.

It is certain therefore that the generated wave just before breaking had to pass over a transverse system of standing waves. From the observations made in the wave tank it was clear that distortions of the wave front occurred. The wavelength of the distortions is close to that of an edge wave mode. The conclusion is that the phenomenon is one of wave interaction.

The importance of the wave height in this particular case would probably be that it positions the break point so that it coincides with one of the lines of standing waves. Since the positions of the standing wave is a dependent of the edge wave shape which is in turn dependent on the period, it seems that both the wave height of the generated wave and the period of the edge waves have to be of the correct value for this very well tuned interaction to occur.

4.3. Circulations with other than low wave amplitude

4.3.1. The standing wave phenomenon recorded in Wave Tank Tests M 6 and M 8 ceased or was not apparent when the wave heights exceeded about 0.5 inches and the period was other than about 0.65

Photograph 4.1. Enlargement of a frame from a ciné film taken almost vertically of 1.5" waves of 0.65 secs propagated from the top of the photograph and breaking near the bottom.

Notable features are (a) the three dimensional standing wave caused by the superposition of 3 edge waves, 3 travelling from left to right and 3 from right to left.

(b) the near-parallelogram patterns and the scalloped nature of the water topography over which the generated wave must pass (Enlargement made by Mr. P. Zoutendyk).

seconds. It was succeeded by a regime with much more vigorous rip current flow. Notable in this connection is, the modes of oscillation (with small amplitude waves) during the experiments with Wave Tank Mark I, which occurred at a fixed point with the arrival of every generated wave, and which were accompanied by quite vigorous rip currents. See Fig. 3-13.

4.3.2. Examination of the spacing of these rip currents when wave height and period were independently varied revealed the following:

(a) with increasing wave height there was tendency for the mean spacing to increase but the increase was not regular.

(b) with increasing period there was no definite tendency.

(c) under any particular set of circumstances the spacing between rip currents along the beach was not necessarily uniform though some times (see Photograph 3.5a) they were.

4.3.3. Of interest are the small embryo rips which were noticed on many occasions and which are clearly shown in Photograph 3.8.

4.3.4. Explanation of these phenomena is difficult and it is doubtful whether the quality of the data which have an ingredient of subjectivity justifies close and detailed analysis. It is proposed to examine only one case and to see if the edge wave concept can throw any light on it.

4.3.5. Consider the cell of circulation in Photograph 3.8a. It is bounded by two large rip currents which penetrate the surf. Only the half-rips are marked by dye. Considering the outer boundary of these half-rips, they are separated by 3.2 ft outside the surf and 2.6 ft at their roots (inside the surf). Between the two roots are 4 small embryo rips (one not very clearly defined). The mean separation of these small rips is about 0.45 ft. It appears possible that the root of the main semi-rips coincides with two more embryo rips, one on either side of the four noted above. See Photograph 3.8b. Where the generated wave encounters the main rips the wave front is significantly retarded.

Now assume that under these conditions each generated wave produces an edge wave by reflection from the sides. Using equation 1.12 and substituting the wave period 0.75 sec and the beach slope (6°), the following are the wave lengths of the possible modes:

Mode (n)	Wavelength (ft)
0	0.305
1	0.89
2	1.44
3	1.92
4	2.32
5	2.62
6	2.8
7	2.87

By combining the edge waves of zero mode and say the 7th mode it is possible to construct a pattern made up of two standing waves. This has been done schematically in Fig. 4-4b. The short wavelength edge waves would produce peaks seperated by 0.3 ft and the long edge waves peaks seperated by 2.87 ft. These compare with 0.45 ft and 2.9 ft respectively as measured from Photograph 3-8(a). The agreement for the short edge wave pattern is not very good, but propagation along a turbulent surf would produce distortions.

4.3.6. Qualitatively such a system would contribute to an understanding of the following features observed in the model experiments and in the field.

(a) the "feeders" which contribute to the rips from adjacent cells - each feeder originating at a small rip

(b) the presence of smaller cells within larger ones

(c) the limited penetration of the smaller rips because edge waves become of insignificant amplitude at a distance of a wavelength (of the edge waves) from the waters edge

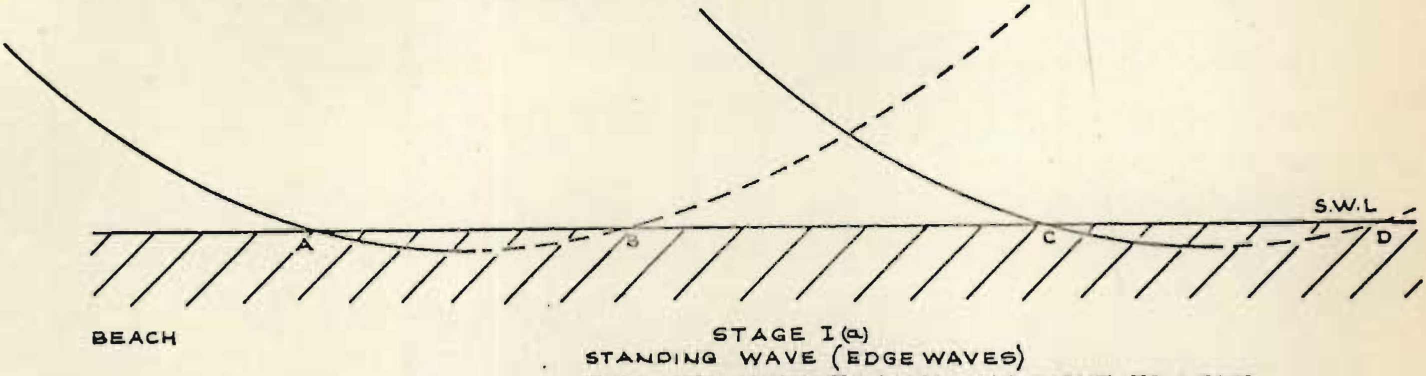
(d) the tendency sometimes for rips to start at one point and subsequently flow sideways to join an adjacent rip.

4.4. Consideration of the evidence from the three cases examined suggests that, certainly in two cases a transverse oscillation was present in the wave tank and that the mode of motion was that of edge waves. There was therefore present a characteristic oscillation which could set the pattern for the falling away from unstable equilibrium in the manner of the Rayleigh-Taylor instability. It remains to consider possible mechanisms.

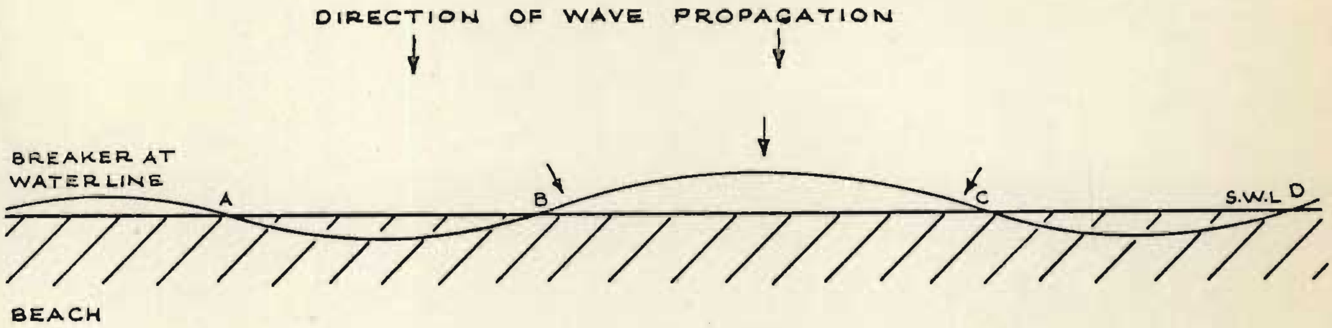
5. Mechanisms

In general the interaction between waves travelling in different directions is small. Because of this it has been possible to gain such a good understanding of wave phenomena in the sea with quite simple mathematical models. There is however some slight second order interaction which Phillips (1960) has predicted and Longuet-Higgins and Smith (1966) have confirmed experimentally. It seems likely however that if the interactions were to occur in the region of incipient breaking where mass transports are relatively great the interaction could be appreciable and significant.

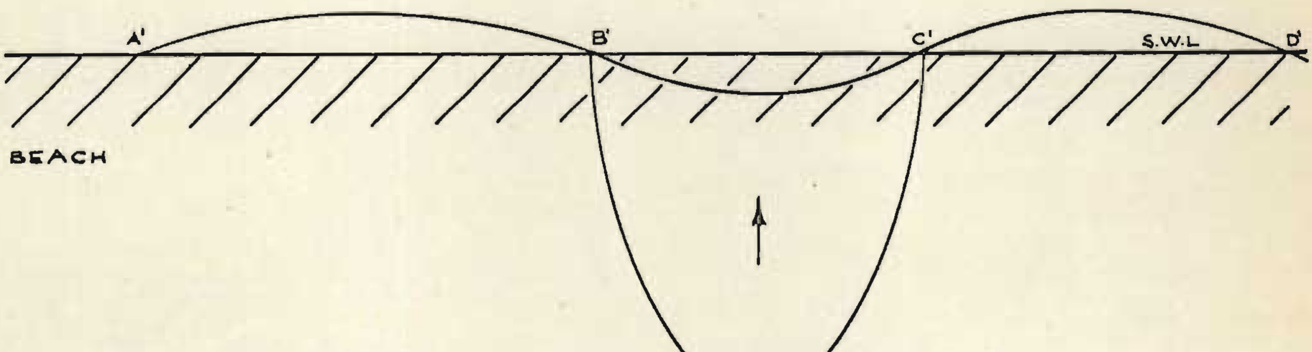
5.1. Consider first the case where small amplitude generated waves were associated with very large amplitude oscillations along the waters edge. If these are edge waves travelling transversely in either direction, they will form a standing wave. The standing wave will cause or impart a wave shape to the waters line, the water level being higher up the beach at the crests (viewed from the "sea") and below still water level at the troughs. This is illustrated in Fig. 4-5 stage 1a. A generated wave imposed on this will, if it is of small amplitude, break close to the still water line. The position of breaking will vary along the wave according to the depth of the water encountered. Breaking will occur first into the trough (viewed from the "sea") of the standing wave. In other adjacent portions of the generated wave crest, breaking will be delayed. In this way the water is focussed by convergent mass transport (Stage Ib of Fig. 4-5) and surges up the slope. The back wash (Stage II Fig. 4-5) flows down the slope to meet the succeeding generated wave which has now broken on the standing wave crest - a system of diverging mass transport.



STAGE I (a)
STANDING WAVE (EDGE WAVES)
FULL LINE = WAVE TRAVELLING RIGHT TO LEFT.
DASHED LINE = WAVE TRAVELLING LEFT TO RIGHT.



STAGE I (b)
WAVE BREAKING ON WAVY WATER LINE



STAGE II
BACKWASH RETURNING DOWN THE BEACH SLOPE TO MEET NEXT ON-COMING WAVE.

FIG. 4-5. SKETCH SHOWING SUCCESSIVE STAGES OF THE OSCILLATIONS, WITH SMALL WAVES ON A SLOPING BEACH IN A MODEL

The effect of the back wash is to flatten the standing wave crest. Back wash water originally from a breaker crest length BC returns over a reduced front B'C' because it has been subject to convergence. It has more momentum than the oncoming incident wave and some of it penetrates seaward of the surf zone as a weak rip current.

In the case where the standing wave has a period equal to that of the generated wave (Fig. 3-13), there is a contribution to the rip current every wave instead of every odd wave, and as a consequence the rip current is stronger.

5.2. Another example is that where the waves were large (1.2" - 1.5") and the period about 0.65 secs. Here the waves broke at an appreciable distance away from the waters edge. (See Photograph 3.13 to 3.15).

As recorded for the wave tank experiment the effect on the breaker is twofold. At breaking the wave front is scalloped in the horizontal plane (Photograph 3.15) and corrugated along the crest in the vertical plane. (Photograph3.16). A possible explanation of effects observed is as follows. When the generated wave has to pass over a transverse standing wave, portions of the wave front will be super-elevated over the antinodes and adjacent portions over the nodes will be lowered. The crest of the wave will be corrugated. If the standing wave is situated at such a position down the slope that it is in a depth corresponding to the point of incipient breaking of the wave, the portions of the crest over the nodes will break first. In doing so they will impart a scalloped appearance to the wave front.

Differential breaking time will lead to a convergence of mass transport (as in the previous case) and spatial variations of head at the shore will result. Outgoing currents will be strongest opposite the points of greater head. The outgoing currents will cause increased refraction of the incident wave front and increased convergence of mass transport. The conditions for cellular circulations are then established.

5.3 In both the cases considered wave interaction resulted in differences in water depth in the region of breaking. This resulted in differential times of breaking along the breaker front, and as a consequence there was convergence of mass transport and spatial differences in hydrostatic head along the surf.

5.4. A corollary to the above conclusion, if it is correct, is that the mode of edge waves which causes standing waves in the region in which the processes of wave deformation and breaking occur, is the one which will most influence the system. It will be recalled that the edge wave amplitude is greatest at the shore and falls off exponentially with distance offshore, and is insignificant at about one wavelength (of edge waves). If the generated waves are high enough and the surf therefore wide enough the short wave length edge waves may not be effective in the zone of breaking. For this reason it may be expected that there will be a tendency for the spacing of dominant rip currents to increase with increasing wave height (if the period is constant). Because edge wave wavelengths are not harmonically related, and because of the possible interaction of different edge wave modes it is not likely that the tendency will be regular. These factors may be a reason for the irregularities encountered in the experimental work.

5.5. There is another aspect which could conceivably influence which edge wave mode dominates. Consider two similar edge waves approaching each other. They will meet first at their seaward end. At their intersection there will be an antinode and the sea level will be raised. As the edge waves progress the locus of the point of intersection will be a straight line perpendicular to the shore. If the velocity with which the point of intersection moves along the locus coincides with the velocity of the generated wave, the influence of the edge wave mode should be at its greatest. There may also be an additional effect due to the differential velocities of the generated wave over the different water depths of node and antinode. In considering the application of the system of wave interaction to the field, it is of interest that for an 8 second forcing period on a 6^0 slope, the second mode edge wave would have a wavelength of 162 ft compared with the mean observed value of 127 ft. However, though there is a report by Munk et al. (1956) of edge waves (from storms) in deep water over the continental shelf there is no evidence of edge waves occurring near the surf. On the other hand it is not known that they have ever been sought.

There are several possible sources of waves travelling parallel to the shore. Reflections from headlands and from the heads of rip currents have been observed. Wind waves too have been observed to influence the breaking of waves and to cause a pattern rather similar to that in Photograph 3.16. It cannot however be stated that there is at present any justification other than analogy for using the wave interaction concept to explain cellular circulations in the field.

5.7. Though it may be possible to establish that the wave interaction mechanism applies in the field, it would be necessary to deal with the additional factor of the moveable sand bottom. The model experiments with a sand base showed that the pattern of the circulation becomes imprinted in the bottom topography. Sand promontories built out at the rips, just outside the surf and at the waters edge. Scouring has been reported by several workers e.g. Shepard Emery et al. (1941). Bruun (1963) has reported lowering of the submerged sand bars at rips. It is supposed that where the topography has been tailored to the circulation, it will tend to resist changes in the circulation which would be occasioned by changed wave conditions.

Offshore topography too might well have an over-riding influence.

5.8. The formation of regularly spaced beach cusps in the sand of the intertidal zone has long been a matter of study. The formation of such cusps in association with the small rips 9" apart in the wave tank experiments with a moveable sand base is therefore of interest. The wave period was 0.75 sec. It is quite likely that they were caused by the first mode of edge waves of period 0.75 secs.

6. Continuity and the spacing of rip currents

When a wave front is distorted by an oscillating disturbance with a number of modes, the dominant mode will be that which grows most rapidly in amplitude. In the case of nearshore cells it is supposed that in addition to the factors already mentioned the growth rate will be greatest where the rip currents penetrate furthest, because the deeper the penetration the greater will be the retardation or refraction of the generated wave front.

For the dominant rips the hydrostatic head at the shore must be large enough for the horizontal orbital velocity of the oncoming wave to be exceeded. In addition continuity must be satisfied.

The rip current is fed by the longshore current which in turn is supplied by the alongshore component of breakers. This supply depends on wave height and period and the angle of obliquity of approach. The dominant rips must be spaced sufficiently widely so that the essential supply of water can be maintained. There seems to be no reason why the spacing should not be greater than that prescribed by the conditions of continuity. Indeed examples when this is so are available in Photographs 3.6 and 3.8, if it is assumed that the continuity is satisfied by inflow where the wave front is refracted.

Once having formed, the cell system in equilibrium would have the following mechanism. Considering first the rip, the outflowing water spreads out outside the surf. Depending on the velocity profile, the oncoming wave is refracted - decreasingly with distance away from the rip axis - in a manner indicated by Arthur (1950). This would cause wave distortion in addition to that caused by a transverse oscillation.

Portions of the wave front now approach the shore obliquely. At breaking a component is directed alongshore and forms a current. At the rip base the current is opposed by an equal and opposite one from the next cell and sets up a head which directs water offshore. It is not clear what determines the rip width.

With increase of wave height the wave velocity, orbital velocity, and depth at breaking all increase. Wave tank experiments show that both the rip current velocity and the longshore velocity also increase. Fig. 3-11(a, b, & c). A larger hydrostatic head will be required for the rip current. It appears that this will be met

(a) by the increased inflow with each breaker (porportional to the wave height squares)

(b) a consequent increase in the longshore current - offset somewhat by the wider surf (proportional to the wave height)

(c) an increase in the length of breaker approaching obliquely - a consequence of the increased rip velocity and increased spread parallel to the shore of the offshore component of velocity. This will be offset by the increased wave velocity which reduces the time during which refraction of the wave around the rip can occur.

In spite of the complexity of the system it is of interest to essay at least a partial analysis for a particular case. Appeal will be made to the wave tank experiments on three points, the rip width, the angle of wave obliquity, and the maximum head required for the rip current.

It will be assumed that the circulation is in a state of equilibrium and that the solitary wave theory as outlined by Munk (1949a) is applicable. An approximate expression for the mean rip current velocity and discharge will be derived. The length of breaker crest needed to supply the water for the rip discharge will then be calculated.

6.1. Calculation of rip current velocity

As a starting point in the calculation use is made of a finding from the model experiments which strongly suggests that the outflow of the rip is just halted by the arrival of the elevated portion of the solitary wave. This implies that there is an offshore slope of the sea surface just sufficient to produce an offshore flow which will just equal the mean horizontal orbital velocity of the particles under the wave crest of the oncoming wave just prior to breaking (or perhaps the mean horizontal orbital velocity under the elevated portion of the wave).

Horizontal orbital velocities under the crest near breaking have been calculated by Munk (1949a) and inspection of the graph (reproduced in Fig. 1-3) shows that the mean value from wave crest to the bottom is close to 0.45 C where C is the velocity of the solitary wave. This velocity will be diminished by the uniform return flow. Consequent upon set-up generally at the shore Munk has shown this to be equal to the mean horizontal displacement(D) divided by the effective period (T) i.e. D/T. Let the net mean horizontal velocity under the wave crest be $\overline{V}_c$ then

$$\overline{V}_c = 0.45\ C - D/T \qquad (4.3)$$

As shown by Dmitriev and Bonchkovakaia (1954) (Equation 1.19) the gradient b directed offshore which will produce a velocity of $\overline{V}_c$ is

$$b = \frac{\overline{V}_c^2}{Cf^2R} \qquad (4,4)$$

R is the ratio of the area of cross-section to the wetted perimeter and in the case of a rip will be equal to d_b the depth at breaking.

Cf is Chezy coefficient for friction. Bruun (1963) indicates that $Cf = (g/K)^{\frac{1}{2}}$. The value of K for longshore currents has been the subject of study by Inman and Quinn (1952) who recommend a value

$$K = 0.024V^{-3/2} \tag{4.5}$$

where V is the longshore current velocity (K is dimensionless). This study is dealt with in greater detail under Chapter V on alongshore currents. It will be assumed that this relationship applies to flows across the surf zone.

Substituting $R = d_b$ and $Cf^{2} = g/0.024\bar{V}_c^{-3/2}$ in equation 4.4.

$$b = \frac{\bar{V}_c^{1/2} \times 0.024}{g\ d_b} \tag{4.6}$$

When the wave is expended the rip current will again begin to flow under this gradient b. Its velocity at any time t thereafter is given according to Dmitriev and Bonchkovskaia (1954) (Equation 1.24).

$$V_t = g\ b\ t \tag{4.7}$$

The assumption here is that b is small.

The period during which acceleration may take place before the arrival of the next wave is the wave period **T** (actually slightly less). The rip current velocity (V_{max}) at the end of this time is therefore given approximately by

$$V_{max} = g\ b\ T \tag{4.8a}$$

and the mean rip current velocity over the period

$$V_r = g\ b\ T/2 \tag{4.8b}$$

6.2. Continuity principle applied to rip-longshore current system

The mean longshore current rate of flow per unit time is given by Inman and Bagnold (1963) - see fuller reference in Chapter V on the alongshore system - as

$$Q_L = Q\ L_c \text{ Sin } \alpha \text{ Cos } \alpha/T \tag{4.9}$$

where Q is the quantity of water contributed by unit length of wave crest (See equation 1.3), L_c is the length of wave crest, α is the mean angle the wave crest makes with the shore. Without significant error cos α may be neglected when α is small and L_c becomes the corresponding length of shore line.

The quantity Q_L must be equal to the rate of flow of the semi-rip current (Q_r) across a cross section near the break point of the waves.

$$Q_r = V_r \times w \times d_b \tag{4.10}$$

whence w is the semi-width of the rip current and d_b the depth at breaking.

Equating Q_r and Q_L , substituting in terms of equations 4.9, 4.8b and 4.10 and rearranging makes

$$L_c = \frac{g\ b\ T^2\ w\ d_b}{2\ Q \text{ Sin } \alpha} \tag{4.11}$$

where $b = 0.24\, \overline{V}_c^{\,1/2} / (gd_b)$ from Equation 4.6

$\overline{V}_c = 0.45\, c - D/T$ from Equation 4.3

$D = Q/d_b$ from Equation 1.5

$Q = 4d_b^2 (\gamma/3)^{\frac{1}{2}}$ from Equation 1.3

$C = \left[gd_b(1 + \gamma) \right]^{\frac{1}{2}}$ from Equation 1.7.

6.3. L_c is then the minimum length of crest having the height and obliquity necessary to supply the rip current. It seems that it may represent the minimum semi-spacing of rip currents. The spacing may be larger than L_c. During the model tests when rip spacings were large the incident wave crest while being markedly curved close to the rip was evidently straight over a region midway between the rips.

It would be desirable to express the semi-rip width (w) and α in terms of the wave characteristics. This would seem to be possible at least for α by employing the approach adopted by Arthur (1950), to compute refraction in a rip current. But certain assumptions would have to be made about the velocity distribution. Since w and α can be got from photographs in the model it was decided to substitute model data in the equation predicting L_c.

6.4. The following data from a test in Wave Tank Mark I. (See Photograph 3.8a) was substituted in Equation 4.11.

d_b = 0.073 feet calculated from surf width and beach slope (See Fig. 3-11) for H = 0.5"

T = 0.75 secs

α = 8° a mean value from photographs

w = 0.33 ft from photograph.

The calculated value for L_c = 1.64 ft.

It was noted in the wave tank tests that although the longshore current drew water from the midpoint between the rips to the rips, a distance of about 1.5 ft, the distance from the rip to the position where the wave obliquity became insignificant was less - not more than 1 ft. The predicted value for L_c is therefore nearly twice the observed value. It is not thought that either the approximates in the theory or the experimental data could be out by so large a factor. It may be that the error lies in the assumption that the value of k obtained for longshore current flow applies to athwart-surf flow. In any case the fact that longshore movement is along the whole of the semi-rip spacing suggests some additional mechanism. Possibly it is associated with the fact that where there is no wave obliquity there should be a greater set-up than where there is. On this account there might be a gradient from between rips to rip base. Additionally or alternatively the water in the region of no obliquity is set in motion alongshore by entrainment by the water that is moved alongshore by wave obliquity.

6.5. Unfortunately there is no comprehensive field date to substitute in Equation 4.11. However for interest the following data for the inner breaker zone has been substituted

d_b = 2 ft - corresponding to a typical wave height

T = 8 sec - a typical period

α = 8^o assumed comparable with that found in the model

γ = 0.78

w = 5 ft from field data.

The calculated value of L_c is 25 ft compared with a mean semi-rip spacing of 63.5 ft for cells in the inner breaker zone.

7. Transition from cellular to alongshore

The mechanism for the cellular circulation suggested by the model studies leads to a possible unifying theory for all three types, the symmetrical.- asymmetrical - cellular types and the alongshore system.

As has been stated above, with the normal wave approach the concavity of the wave front (viewed from the shore) causes longshore currents in opposing directions each flowing towards the same rip. In the model, under uniform conditions the currents in each direction acted over about equal distances. When, however, the incident wave approach is slightly oblique some of the obliquity just down wave of the rip is opposed by the wave obliquity due to retardation and the latter's influence on the circulation is diminished. While in the next half cell down wave the obliquities are compounded and the down wave influence increased. The cell is now asymmetrical with only a short up-wave longshore current. See Figure 4-6.

If now the incident wave obliquity is further increased there comes a point when the wave obliquity reduces to nil the up-wave longshore current. Longshore currents may now flow down wave past the base of the rip. In the extreme case there are no rips except at beach and coastal discontinuities or after long distances of uninterrupted longshore travel. In nature distances up to at least a mile have been measured. At coastal discontinuities where the angle of wave obliquity may be reduced, conditions favourable for rip currents may obtain.

The circulations therefore fall into two distinct classes. Those where cell is a feature and at least some element of longshore current in each direction is evident, and those where the cellular pattern is suppressed and the longshore current under oblique wave action dominates.

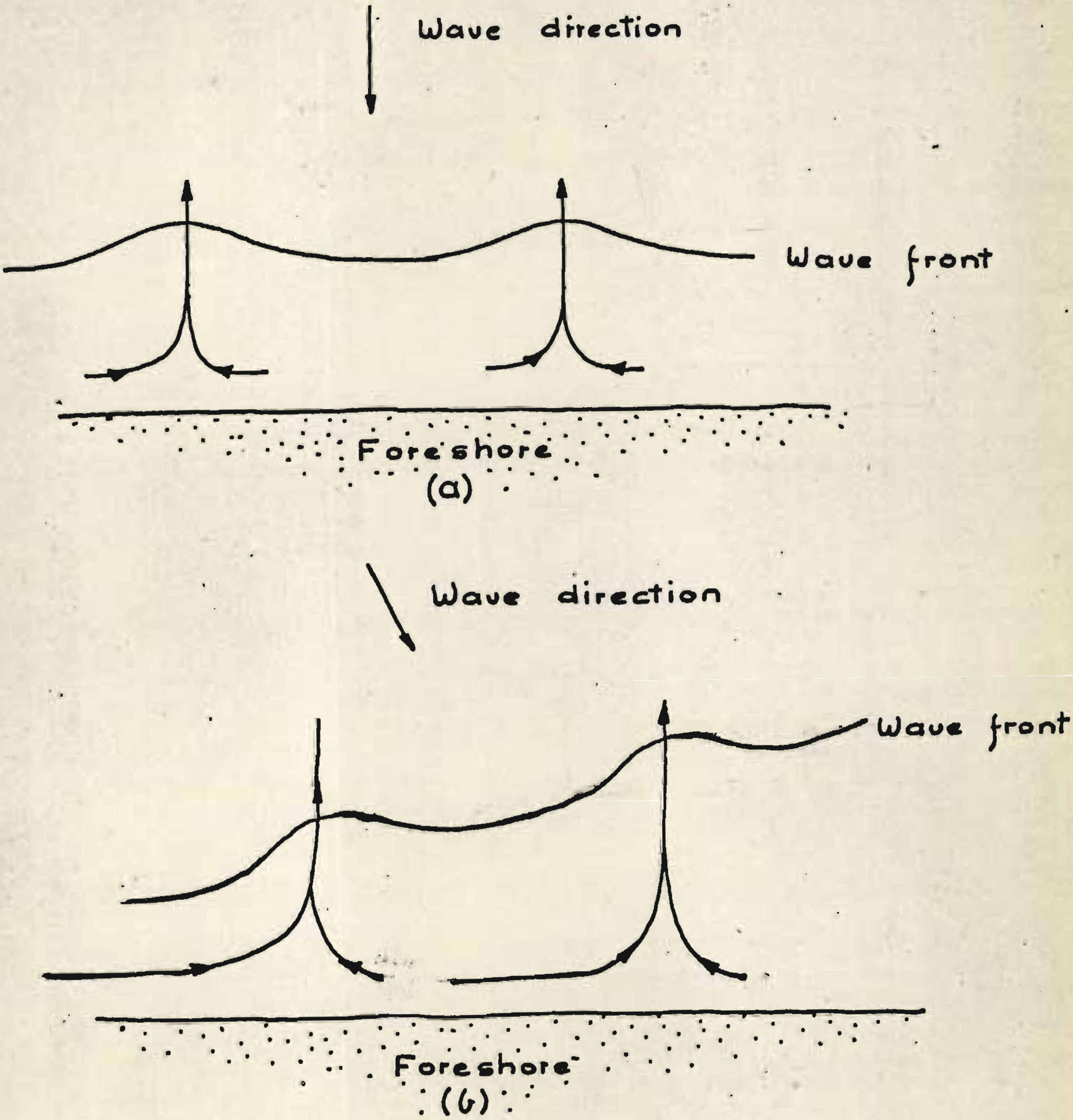


Fig. 4-6. Schematic diagram of wave front distortion by rip currents.
(a) With normal wave approach (symmetrical)
(b) With oblique wave approach (asymmetrical)

8. Exchange of Surf Zone Water

The exchange of the water in the surf zone is a matter of some interest for it controls, for example the return to the shore of effluents released offshore. It is proposed to examine this process in the light of the experimental data outlined above.

Under the test conditions the volume of water in the surf zone of a cell of circulation was ordinarily of the order of 10^6 cu.ft. The experiment to determine the exchange rate showed that of the order of 1 hour was required for this water to be replaced by water entering from outside the surf under mass transport. This process has been shown to be largely one of displacement. The outgoing water is transported in rip currents which have a net volume flow of the order of 10^3 cusecs. Once outside the surf zone the rip current spreads laterally and in due course much of the rip current water recycles into the surf zone. Only a small portion of the breaker frontage is transporting in water which has not been directly recycled. This proportion is perhaps $^1/_5$ to $^2/_5$ under condition of symmetrical circulation. Assuming that this frontage is proportional to the net volume of water entering through it, it is possible to conceive the buildup of the concentration in the surf of a waste discharged offshore in the following simplified terms. Let λ be the width of a cell, i.e. the frontage of which water enters. Let "r" be the frontage through which the recycled water enters and "e" the frontage over which effluent or deep sea water enters.

$$\lambda = r + e \qquad (4.12)$$

Suppose r and e are proportional to the quantity of water entering in their respective frontages. Let C_o be the concentration of effluent just outside the surf zone. Suppose the effluent and recycled water entering the surf zone mix completely in the long-shore current to give concentration C and after the first cycle C_1 and the second C_2 and so on. Assume also that in that part of the cycle outside the surf zone the mixture undergoes no dilution.

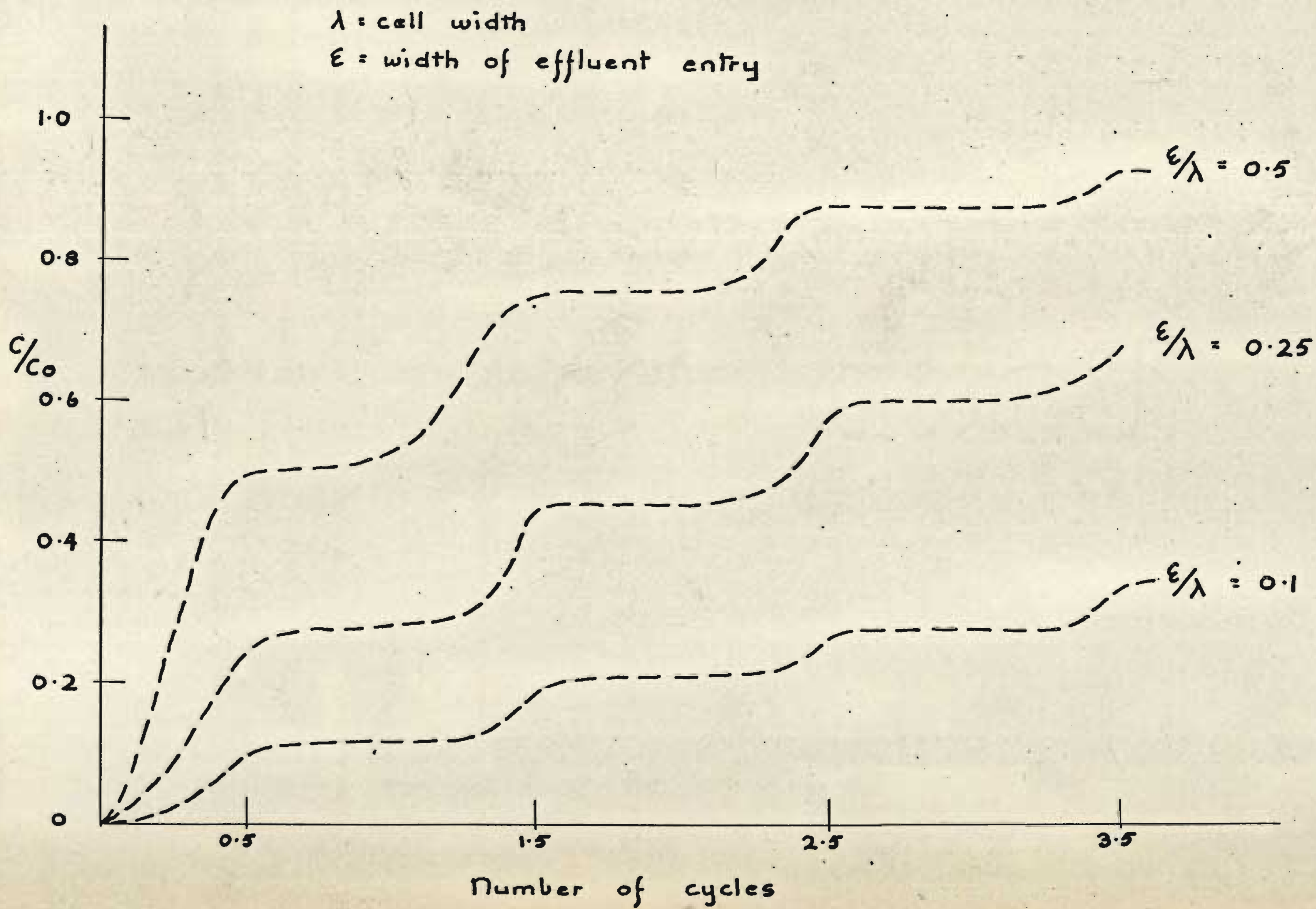
It can be shown that the concentration after "n" cycles is in the form of a progression

$$C_n = \frac{C_o}{\lambda}\left(1 + \frac{e}{\lambda} + \frac{e2}{\lambda^2} + \ldots\ldots \frac{e^n}{\lambda n}\right) \qquad (4.13)$$

Figure 4-7 shows the way in which the relative concentration C/C_o increases with each cycle. Curves for three values of e/λ have been drawn. The shape of the curve between the values of the concentration after each cycle should be a stepped one. There is no field data to indicate what it is in the prototype because of the difficulty of sustaining a sufficient supply of high concentration outside the surf for the long period required. Recourse has therefore been made to the results of the model experiments (Fig. 3-12) which indicated that the steps should be rounded. Since however the dynamic similitude between model and prototype has not been established, this source of information must be treated as qualitative only. It does seem however that the build up of concentration is stepwise.

Taking a very conservative standpoint it may be noted that when the recycled water represents half of the total quantity of water entering a cell i.e. larger than the maximum observed, at least 1½ cycles are required for the concentration to reach 80% of the effluent concentration outside the surf. In nature this would represent at least two hours. For the same concentration to be built-up in the inner breaker, an additional hour should be added.

Fig. 4-7. Idealized graphs of rise of concentration (C) in the surf from offshore discharge of concentration (Co) just outside the surf.



making a total of 3 hours from the time the effluent entered the nearshore region.

This figure is of some practical interest in the coast region south of Durban where so far as present knowledge goes the duration of the shoreward currents i.e. those which would be required to bring an effluent into the sphere of influence of the nearshore circulation, have an average duration of about 2 hours. The conclusion therefore is that in the region in question the probability of an effluent building up at the shore, by cellular circulation, to an equilibrium concentration, is rather unlikely.

It may reasonably be supposed that the symmetrical circulation represents the condition of greatest exchange. As the circulation tends to the alongshore system exchange diminishes until in the limiting case exchanges may only be expected at headlands or shore-line discontinuities.

It is of interest to calculate what percentage of the water which potentially could be transported in by the breakers, is in fact exchanged.

The solitary wave theory gives the volume associated with unit length of each wave in Equation 1.3(b)

$$Q = \frac{4H^2}{\sqrt{3\gamma}^3}$$

The rate of inflow over period T = Q/T.

Taking wave characteristics commonly encountered at Virginia, H = 5 ft T = 8 sec, assuming $\gamma = 0.78$ and considering a cell of width 1800 ft (about the mean value measured), the potential mean inflow is

$$Q/_T = \frac{4 \times 5^2 \times 1800}{8\sqrt{3 \times .78}^3} \quad \text{cusecs}$$

= approximately 1.5×10^4 cusecs.

The magnitude of the actual rate of water exchange during the tests ar Virginia was 700-1800 cusecs, (See Table 3.2) which represent 4.5-12% of the potential inflow. This compares with a value of 2-10% calculated by Inman and Bagnold (1963). It is evidently quite a small proportion which circulates, confirming the view that inward transport is almost balanced by the uniform sea return.

CHAPTER V

THE ALONGSHORE SYSTEM

SECTION I

1. Introduction

The Alongshore System is essentially a unidirectional current flowing along the shore, there being negligible tendency for water to flow offshore except at coastal discontinuities, such as head lands. On the test beaches which had submerged sand bars the current's velocity was greatest in the outer breaker zone where the waves broke on the sand bar, and in the inner breaker zone. In the between-breaker zone the water often moved in sympathy but at a much lower velocity.

It seems that the conditions suitable for this type of system are oblique wave approach and the absence of any basic swell which would be refracted so as to approach the shore at small angles of obliquity. On the Natal Beaches examined such conditions occur rather rarely, (about 10% of the time) probably because the basic swell is so seldom absent.

The longshore current takes the form of a "river of water" flowing down wave - with apparently negligible exchange offshore for long distances. On the test beaches the current of the inner breaker zone was seldom more than 100 feet wide, extending out to just beyond the breakers in the inner-breaker zone.

Such a system which is without appreciable influence by rip currents and which is within easy reach of the shore, provides an opportunity for the study of quantitative aspects of the system. In the experimental work described below the main object was to find out the rates of flow or dischages of the current, and to compare them with those predicted by theory. As a preliminary it was necessary to develop a method for measuring discharges and to determine wave height spectra so that the theory could be applied. The preliminary work is described in Section II and the main study in Section III.

2. The Literature

Because the alongshore system can be a potent one for moving sediments parallel to ths shore, it has been the subject of study by a number of workers. Their objective has usually been to elucidate the velocity of the current since this is an important factor in sediment transport.

Theories of longshore transport of water have mostly been based on the assumption that the incident waves are of the solitary form and approach with an appreciable angle to the shore.

Longshore current mechanisms have been postulated using three different approaches, based respectively on considerations of conservation of energy, momentum and continuity.

Putman, Munk and Traylor (1949) derived theoretical expressions for the velocity of the longshore current, in terms of wave height, period and angle, and beach slope, based on the energy and momentum predicted by the solitary wave theory.

2.1. Energy Approach

The first theoretical approach of Putman et al. is as follows:

Let a wave approach the breaker zone at angle α. "Consider a volume of water ABCDE (see Fig. 5-2) extending between the shore and the breaking wave, from top to bottom. Over a width of beach

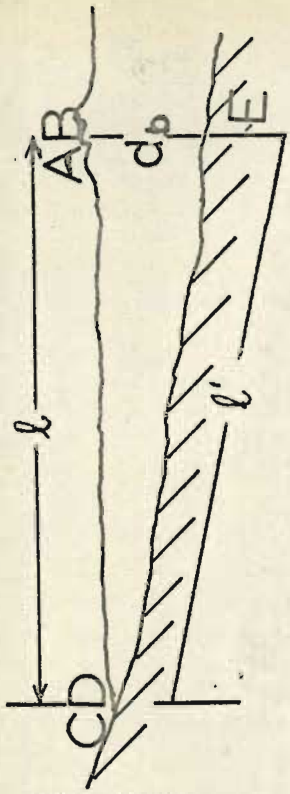


FIG. 5-1.

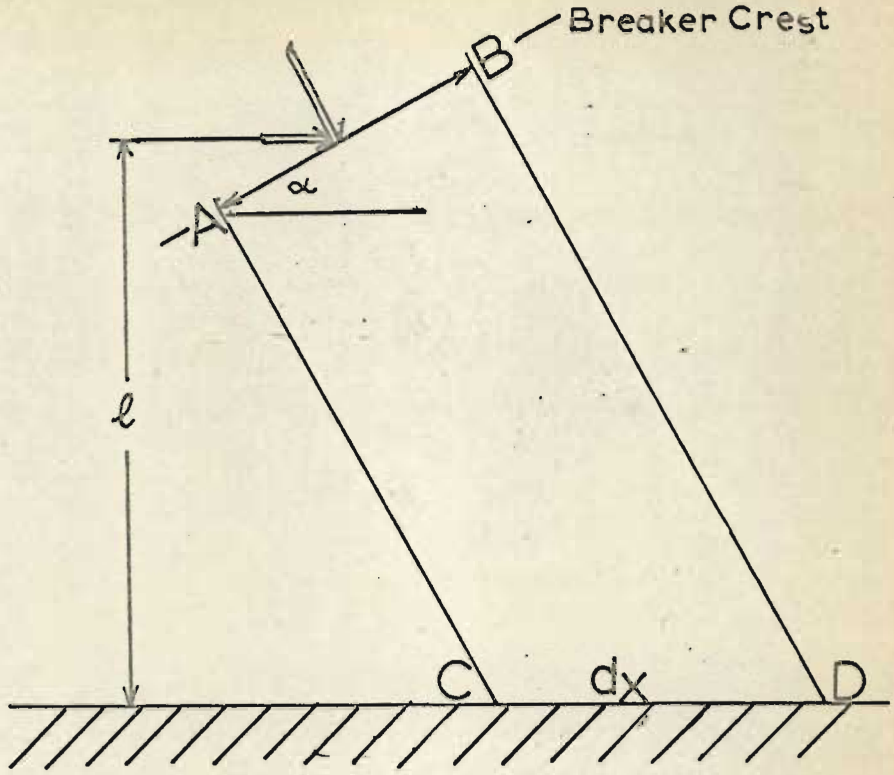


FIG. 5-2.

Definition sketch for the theories developed by Putman et al (1949)

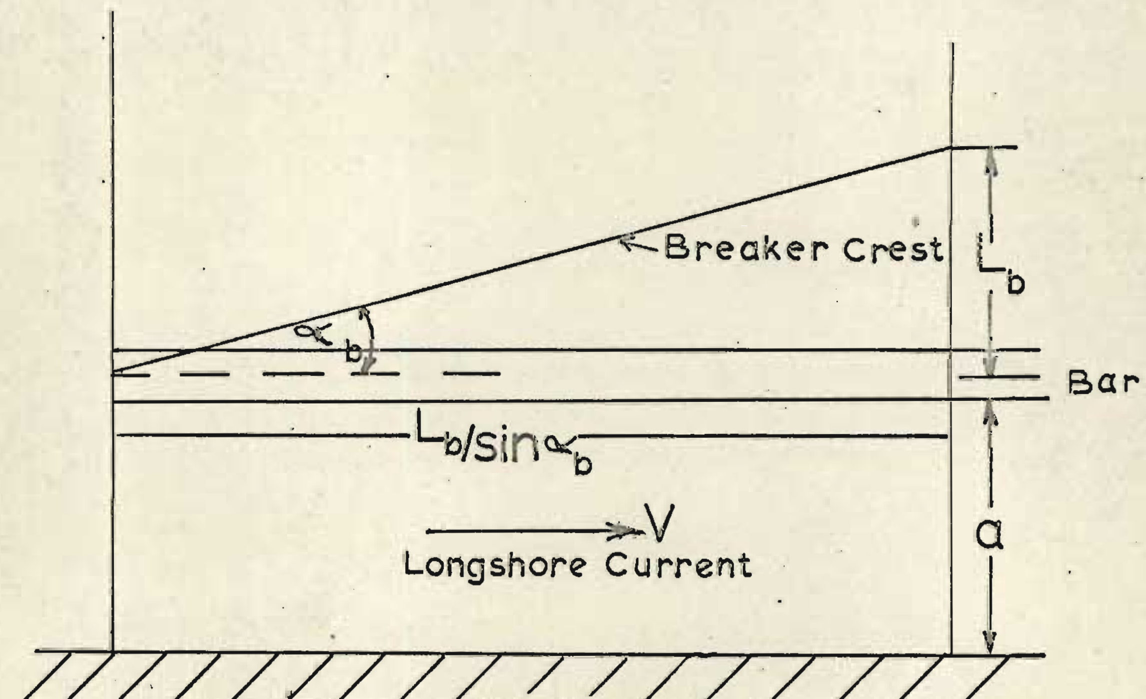


FIG.5-3.

Definition sketch for the theory developed by Per Bruun (1963) dx the total wave energy entering the volume ABCDE equals C E Cos α dx where C is the velocity (almost equal to the group velocity) for breaking waves, E is the mean energy per unit surface area of the breaking waves. Of the energy that advances parallel to the shore $\left[(CE \cos \alpha\, dx) \sin \alpha\right]$, a small fraction S is responsible for setting up longshore currents. The greater part of the wave energy entering this volume is destroyed in breaking and turning into heat, or is responsible for piling up water against the shore and setting up rip currents". "Energy is dissipated in a frictional layer along the bottom. The frictional force per unit width of beach equals"

$$k\ \rho \cdot V^2\ \ell'\ dx \qquad (5.1)$$

and the rate at which energy is dissipated in the volume ABCDE equals

$$k\ \rho\ V^3\ \ell'\ dx \qquad (5.2)$$

where k is the friction parameter depending on the hydraulic roughness of the bottom, V = is the velocity of longshore current, ρ is the density of seawater and ℓ' is the distance along the bottom from shore to breaker line. With good approximation $\ell' = \ell$ (see Fig. 5-1). If m denotes the average beach slope, and d the depth at which the waves break, then

$$m = \frac{d}{\ell} \qquad (5.3)$$

"Assuming a steady state, all the energy available for longshore currents entering volume ABCDE due to wave action must be equal to the loss of energy due to friction". Hence,

$$S\ CE \cos \alpha \sin \alpha\ dx = k\ \rho\ V^3\ dx\ d/m \qquad (5.4a)$$

From the solitary wave theory Munk (1949a)

$$C = \sqrt{g(H + d)} = \frac{L}{T} \qquad (5.4b)$$

$$E = \frac{8}{3} \frac{\rho g d^3}{L} \gamma \sqrt{\gamma/3}$$

$$d = 1.28 H$$

where H is the wave height, d the water depth, T is the period, g the acceleration of gravity, $\gamma = \frac{H}{d}$, and L = wave length. Substituting for these in Equation 5.4a and rearranging

$$V = K \left[(mH^2/T) \text{ Sin } 2 \alpha \right]^{\frac{1}{3}} \qquad (5.5)$$

where

$$K^3 = 0.871 \text{ g } (S/k) \qquad (5.6)$$

2.2. Momentum Approach

The other approach adopted by Putman et al. is based on the consideration of momentum. "With the breaking of every wave a certain mass of water is thrown into motion in the direction of wave propagation. The longshore component of this motion provides the driving force for the longshore current. Consider again the volume ABCDE (see Fig. 5-1). Let Q represent the cross-sectional area of a breaking wave crest moving with velocity C, L the length of the breaking wave, α the angle formed by the breaking crest with the shoreline and ρ the density of water. Then ρ Q C/L is the average momentum per unit area and C (ρ QC/L) Cos α dx is the mean flux of momentum into ABCDE. The component of momentum flux parallel to the shore equals"

$$C \sin\alpha \, (\rho\, Q\, C/L) \cos\alpha \, dx \qquad (5.7)$$

"The broken water is quickly slowed down by the turbulent friction to the mean velocity V of the longshore current and eventually expelled from the surf zone giving rise to momentum flux outward of"

$$V \, (\rho\, Q\, C/L) \cos\alpha \, dx \qquad (5.8)$$

The difference between the momentum flux in Equations 5.7 and 5.8 is the net momentum flux applied by the breaker to the surf zone water mass and equals

$$(C \sin\alpha - V) \, (\rho\, Q\, C/L) \cos\alpha \, dx \qquad (5.9)$$

This force must be balanced by a frictional force as in Equation 5.1. Equations 5.1, 5.3, 5.4b and 5.9 give

$$V^2 = a \, (C \sin\alpha - V) \qquad (5.10)$$

where

$$a = m\, Q \cos\alpha / kd \qquad (5.11)$$

From Equation 1.7 and because

$$d = 1.28\, H \quad \text{(Munk 1949a)} \qquad (5.12a)$$

C can be replaced by $C = (2.28\, gH)^{\frac{1}{2}}$ (5.12b)

Substituting for Q from Equation 5.11 leads to

$$a = (2.61\, mH \cos\alpha)/kT \qquad (5.13)$$

where according to 5.10

$$k = (2.61\, m\, H \cos\alpha)\,(C \sin\alpha - V)/TV^2 \qquad (5.14)$$

Both approaches assume that a steady state exists and that the region under consideration is far enough from obstructions for a full strength current to be generated.

Field observations were made at Oceanside Beach, California. Longshore currents were measured by timing the travel of a current-cross over a known distance, or photographing the movement of fluorescein dye, from a low flying airship. Observations were also made in a wave basin (58 ft long and 39 ft wide) where the velocity of travel of dye in the longshore current was measured. Comparisons of the field and model measurements with the predictions of the theory was made. For the energy approach observed velocities were plotted against $\left[(m\ H^2/T) \sin 2\alpha\right]^{\frac{1}{3}}$ (see Equation 5.5) since a value for K (i.e. k and S) was not known. Average values of the friction parameter were determined by substitution of the observed data in Equation 5.14 for the various types of beach surface investigated.

"Except for the roughest beach it was possible to draw a straight line through each set of data the slope of which is the proportionably constant K", of Equation 5.5. The following table gives the values obtained for k, K and s.

TABLE 5.1

Values of Parameters for Field and Laboratory Data

(After Putnam et al. 1949)

Beach	k	K	s
Field	0.0078	8.2	0.15
Lab. 1	0.0397	5.12	0.19
Lab. 2	0.0070	11.02	0.33
Lab. 3	0.385		

It is pointed out by the authors that the five-fold difference in k between the field and Lab. 1 measurements may be due to the fact that the former had bars whereas the latter was a plane beach. However, "k depends upon hydraulic roughness of the beach rather than absolute roughness" and in the experiments with Lab. 1 beach the current velocities were relatively small (so the viscous forces were relatively large). It was suggested that the "scatter of the data resulted primarily from the difficulty in measuring the breaker angle α, both in the laboratory and in the field".

It is clear that in the application of an energy-or momentum-based theory the selection of a value for this frictional coefficient k is of great importance. Relevant to the problem is the field study by Inman and Quinn (1952) in which measurements of longshore current velocity were made at 15 stations along the beach. The results showed that longshore current velocity was very variable - the standard deviation being equal to or greater than the mean velocity. The friction coefficients calculated from the data by the momentum method revealed considerable variation. The authors showed however that if k is permitted to vary with the velocity, useful predictions are possible. The variation with velocity is given by $k = 0.020\ V^{-1.51}$ for field data and $k = 0.029^{-1.54}$ for model studies i.e. the friction coefficient decreases with increasing velocity. Since the data was obtained over a wide variety of bottom material it seems as Bruun (1963) remarks, that the "hydrodynamic elements in k may have a predominant influence on k".

Inman and Quinn substituted the value $k = 0.024\ V^{-3/2}$ in Equation 5.10 obtaining

$$V = \left[\left(\frac{1}{4x^2} + y\right)^{\frac{1}{2}} - \frac{1}{2x}\right]^2 \qquad (5.15)$$

where $x = 108.3$ Hm Cos α/T

$y = C \text{ Sin } \alpha$

Bruun (1963) refers to coefficient (Cf) measured by Bruun and Gerritsen (1960) for tidal inlets and says that they are comparable in the range 2-4 ft/sec to those found by Inman and Quinn (k), which are listed in Table 5.2. (Note : $Cf = \sqrt{\frac{g}{k}}$ where g is the acceleration of gravity).

TABLE 5.2

Friction Coefficient k or Cf

(After Bruun 1963)

V ft/sec.	1	2	3	4	5
V m/sec.	0.3	0.6	0.9	1.2	1.5
k	0.020	0.0071	0.0038	0.0025	0.0018
Cf. ft $^{\frac{1}{2}}$/sec.	40	67	91	112	134
Cf. $m^{\frac{1}{2}}$/sec.	22	39	50	62	74

For troughs behind bars Bruun (1963) recommends the use of the following "experience" formulae for the velocity range 2-4 ft/sec.

$$Cf = 30 + 5 \log A \qquad (5.16a)$$

where A is the cross section of the trough in metres squared and $Cf = m^{\frac{1}{2}}$/sec or

$$Cf = 45 + \log A \tag{5.16b}$$

where A is in ft and Cf in $ft^{\frac{1}{2}}$/sec.

Values of k for V above 5 ft/sec are very small compared with Cf for normal water courses.

2.3 Continuity of Volume Approach

A third approach to the quantitative prediction of longshore currents was suggested by Inman and Bagnold (1963). It is based on the concept of volume continuity. Let the volume of water brought in by the breaker per unit length be given by Q for a wave period of T. If the angle of wave approach is α, then the component parallel to the shore is Q Sin α Cos α per unit length of beach and it will increase with distance slong the beach.

Assume the distance between rip currents is L_s then the discharge at this distance downstream will be

$$Q_L = QL_s \sin\alpha \cos\alpha/T \tag{5.17a}$$

since the water is brought in over one period i.e. T, and in view of the solitary wave theory, (see Equation 1.3b)

$$Q = \frac{4H^2}{\sqrt{3\gamma^3}} \quad \text{or} \quad 4d^2 \sqrt{\gamma/3} \tag{5.17b}$$

where H is the wave height and d the depth.

The average discharge will be at $L_s/2$. This discharge will be spread across the cross section (A). Hence the mean longshore velocity $\overline{V}$ becomes

$$\overline{V} = L_s \; Q \sin\alpha \cos\alpha/(2\;AT) \tag{5.18}$$

Bruun (1963) developed this continuity approach for the case of a beach with an offshore submerged sand bar. He takes the depth (d_{bp}) at breaking to be equal to the breaker height (H_{bp}) and assumes that breakers of height $H_b(\frac{1}{3})$ determine the depth over the bar crest (where $H_b(\frac{1}{3})$ is the wave height whose height is surpassed by one third of all waves). He assumes that the depth over the bar crest (Dcr) is $0.8\ H_b(\frac{1}{3})$.

Assuming 10% for reflection the depth

$$d_{bp} = 1.12\ Dcr\ H_{bp}/H_b(\tfrac{1}{3})$$

where p is the probability of the waves being equalled or exceeded in height at breaking.

Using data from a study of Longuet-Higgins (1952) which predicts wave spectra in deep water, and applying Equation 5.17b, Bruun derives an expression for the water brought in over the bar (Q_b) as follows

$$Q_b = 2.92 \int_{1.416}^{\infty} X^3\ \varepsilon^{-X^2}\ dx \qquad (5.19)$$

$$\text{where } X = \frac{H_{bp}}{\bar{a}} \text{ and } \bar{a}^2 = \frac{1}{N}\ (H_1^2 + H_2^2 + \ldots H_N^2) \qquad (5.20)$$

if N waves break over the bar. By integration

$$Q_b = 0.587\ Dcr^2 \qquad (5.21)$$

Bruun then develops a method for predicting the longshore current velocity for oblique wave approach. He considers the wave when the shore-most end has just broken. At the point of breaking the sea level must rise. Further down stream the wave has not yet broken. Hence there will be a gradient. He then uses Chezy's formula to calculate the longshore velocity.

Referring to Fig. 5-3 where L_b is the wave length and T is the wave period, the inflow of water in T seconds over $L_b \sin \alpha_b$ length of shore, is $Q_b L_b \cos \alpha_b$. A unit length of shore receives an amount of water equal to $Q_b \cos \alpha_b/T$ per second. Where the wave breaks the sea level will theoretically rise by an amount $Q_b \sin \alpha_b \cos \alpha_b/a L_b$ where a is the distance from bar to shoreline. At one unit length down stream the level will be unchanged. The slope of the sea water will therefore be

$$I = Q_b \cos \alpha_b \sin \alpha_b/(a L_b) \tag{5.22}$$

$$= Q_b \sin 2\alpha_b/(2a L_b) \tag{5.23}$$

Using Chezy's formula the mean velocity of the longshore current will be

$$\bar{V} = Cf \sqrt{R I} \tag{5.24}$$

where R is the hydraulic radius of the trough and I is the slope. It is assumed

(i) that the waters flow out again uniformly across the bar;

(ii) that Chezy's formula is valid for the longshore wave current.

Bruun calculated the mean longshore velocities from field data for two wave periods (8 and 10 secs) and two values of Cf (35 and 45 $m^{\frac{1}{2}}$/sec).

Other relevant field data in the problem were, the trough cross sectional area = 1030 m , breaker angle 25° and $H_{b(1/3)}$ = 5.5 ms. Comparison was made with the velocities obtained when the momentum approach of Putman et al. was used (see Table 5.3). It was noted by Bruun that the latter gave very high readings, not confirmed by experience. It was suggested that the momentum method may not be suitable for application in the case of bar and trough, because some of the momentum is taken up by the waves which regenerate after breaking.

TABLE 5.3

CALCULATED LONGSHORE CURRENT VELOCITIES BASED ON FLOW OF WATER UNDER AN ANGLE WITH THE SHORELINE (COLUMNS 5 AND 6) COMPARED WITH THOSE PREDICTED BY MOMENTUM APPROACH OF PUTMAN ET AL. (COLUMN 7)

1 α deg	2 L_B metres	3 T sec. average	4 $H_{b(\frac{1}{3})}$ metres	5 V m/sec. $Cf=35\ m^{\frac{1}{2}}/sec$	6 V m/sec. $Cf=45\ m^{\frac{1}{2}}/sec$	7 V m/sec
25	55	8	5.5	1.20	1.55	2.6
25	72	10	5.5	1.12	1.45	2.6

(After Bruun 1963)

2.4. Brebner and Kamphuis (1963) also carried out model experiments to measure the longshore current velocity with changing deep wave height, period and obliquity. The velocity profile for a 1.20 sloping beach is shown in Figure 5-4. After an initial increase with distance down stream, the velocity tends to a constant value.

The authors deduced an empirical relationship between velocity (V_L) and deep water wave characteristics. (Height Ho, period T angle of wave approach α_o beach slope θ)

$$V_L = 8 \sin^{\frac{1}{3}} \theta \frac{H^{2/3}}{T^{\frac{1}{3}}} \left[\sin (1.65\ \alpha_o) + 0.1 \sin (3.30\ \alpha_o) \right] \tag{5.25}$$

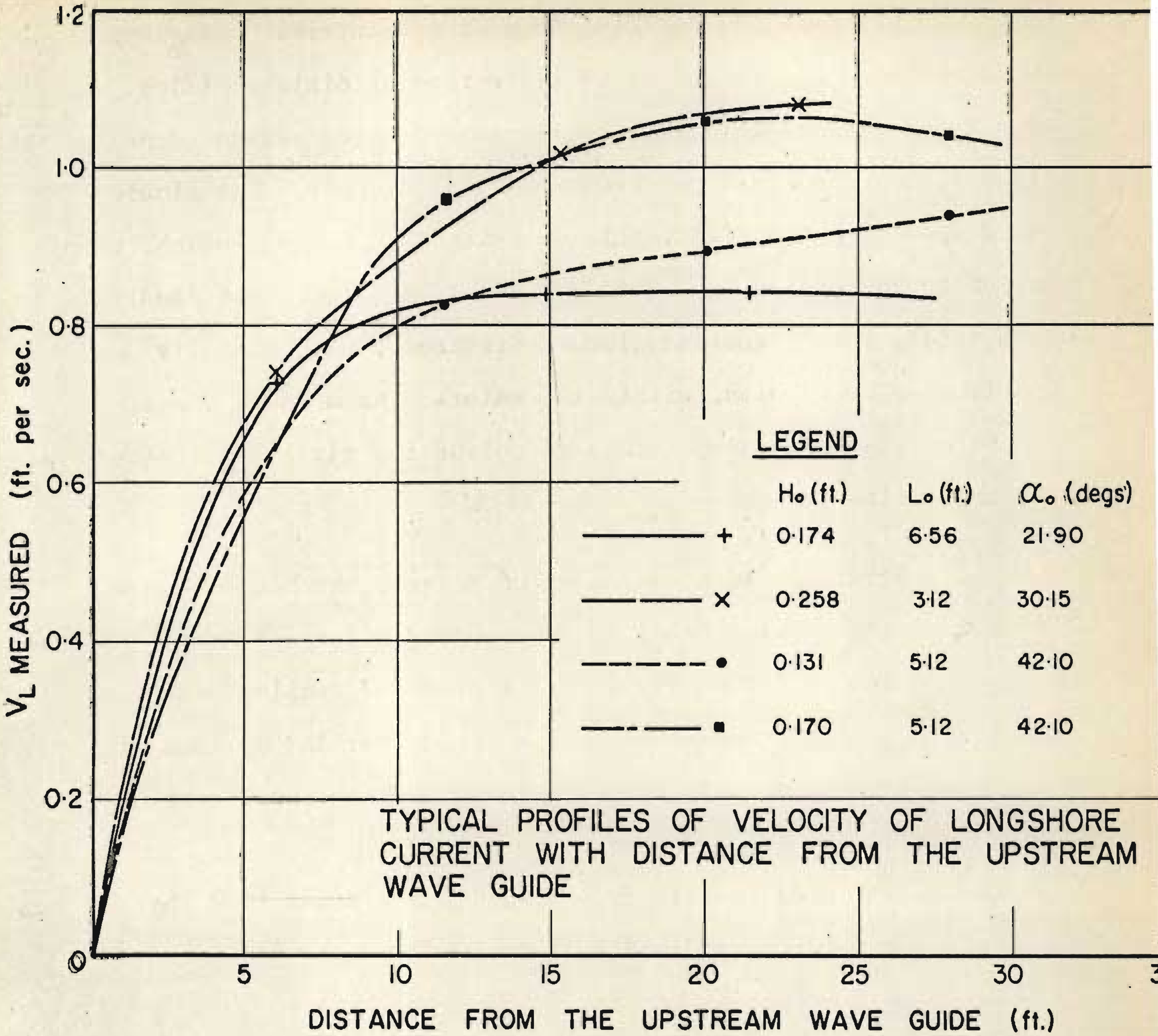


Fig. 5-4. Change of longshore current velocity (V_L) with distance down current in a wave tank. H_o, L_o and α_o are respectively the deep water wave height; wave length, and obliquity. (After Brebner and Kamphuis 1963).

or

$$V_L = 14 \operatorname{Sin}^{\frac{1}{2}} \theta \frac{H^{\frac{3}{4}}}{T^{\frac{1}{2}}} \left[\operatorname{Sin} (1.65\ \alpha_o) + 0.1 \operatorname{Sin} (3.30\ \alpha_o) \right] \qquad (5.26)$$

2.5 In selecting a theory to test against field data the following points are relevant.

(a) the energy approach requires a knowledge of the proportion of wave energy dissipated when the wave breaks. Comprehensive data on this are not available.

(b) both the energy and the momentum approach require a knowledge of the frictional resistance. Whereas some estimates of this force exists, it is difficult to use them because the frictional resistance coefficient depends upon velocity which may be variable.

(c) the empirical equations of Brebner and Kamphuis (1963) depend upon a knowledge of the deep water characteristics. In the absence of a permanent wave measuring installation or the cooperation of a ship these are difficult to ascertain.

2.6. The remaining theory is that based on continuity. In Bruun's development of it the assumption is made that the distribution of breaker heights in the surf is similar to that in deep water, and that consequently the theoretical distribution of Longuet-Higgins can be applied. Since however, some preliminary studies which will be described below, indicated this assumption to be a reasonable one, the continuity theory as modified by Bruun was selected as the one against which field date should be compared.

Combining equations 5.17a and 5.17b the expression for the discharge of a longshore current at a distance L_s from its start may therefore be written.

$$Q = \frac{4\beta\, H_b^2\, L_s \,\mathrm{Sin}\,\alpha\,\mathrm{Cos}\,\alpha}{T\,\sqrt{3\gamma^3}} \qquad (3.27)$$

When H_b is the significant wave height at breaking B is the factor necessary to convert H_b to a root mean square height. α is the angle the breaker front makes with the shore. γ is the ratio of breaker height to depth at breaking. T is the wave period.

2.7. The question that remains before this equation can be used is what value to use for γ. Munk (1949a) summarizes the results of 746 observations made in various field and laboratory studies, and finds a value of $\gamma = 0.75$. He quotes some precise measurements made in the surf at Scripps Beach which gave a value of 0.78.

According to Ippen and Kulin (1955) values obtained by other workers are Boussinesq 0.73, McCowan 0.78, Davies 0.83, Packham 1.03 and Gwyther 0.83. Ippen and Kulin from laboratory tests with solitary waves found the value to be considerably higher. On a 0.023 slope for all initial waves it was 1.2. Brebner and Kamphuis (1963) prefer the value 1 as does Bruun (1963) for a beach with submerged longshore..

From field studies by Miller and Ziegler (1964), a value of 0.77 for a plunging breaker is obtained. McCowan found that theoretically the limiting relative wave height for a solitary wave was 0.78. In the calculations which follow the value of γ for the breaking wave has been taken to be 0.78 following the field work described by Munk (1949a) and Miller and Ziegler (1964).

2.8. The underlying assumption of Equation 5.27 is, that starting at some point (possibly a headland or major discontinuity) where the longshore current discharge is zero, the current gains in discharge down stream by the continuous acquisition of water from breakers entering it. The equation assumes that the gain is proportional to the distance down stream so that velocity and/or current width must increase as well. Clearly there must be some limiting distance down stream where there can be no increment of volume because the surf zone has reached its maximum width and is moving with a maximum velocity determined by the available momentum and the frictional resistance to flow. In this respect the equation is inadequate for it satisfies continuity only.

In work reported later, where the longshore discharge as measured in the field is compared with the predicted discharge from Equation 5.27, the problem arises as to what is the starting point of the current and at what part of the current in relation to this point the discharge was sampled. To avoid this difficulty the observed and calculated <u>increments</u> of discharge between two points a known distance apart are compared.

SECTION II

PRELIMINARY INVESTIGATIONS

1. Measurement of Wave Height Distribution in the Surf Zone

1.1. Introduction

Ordinarily the waves arriving in the surf are the product of wind fields often remote in distance. In the wave generating area a spectrum of wave frequencies is produced. As they travel out of the generating area the longer and faster waves outstrip the shorter slower ones. By the time the waves reach the surf there has been some degree of sorting as to wave length. Small differences do however remain and these can give rise to "beats" resulting in successive groups of high and low waves.

The quantitative problem is to find the distribution of wave heights in the spectrum so that they can be related to some single representative measure. It has become customary to use the mean of the upper one third of all wave heights as this measure. (Sometimes the mean of the upper 30% is used). The reason for choosing this measure, described as the "significant" wave height, is apparently that it is the one which would commonly be measured by a careful observer.

In applying the theory to predict longshore currents in the inner breaker zone, a representative wave height is needed to insert in the equation. This height appears in the equation as its square. The root mean square height of all waves breaking into the inner breaker zone would seem to be the most representative height to use. In practice therefore a relationship between the significant wave heights and the root mean square wave heights is required.

1.2. Longuet-Higgins (1952) has made a theoretical study of the statistical distribution of wave heights in deep water. He assumes that

(a) the spectrum contains a narrow band of frequencies

(b) that the wave energy has been received from a large number of sources whose phase is random.

He starts with the expression

$$p(H)dH = \left(\exp \frac{-H^2}{\bar{a}^2}\right) \frac{2H}{\bar{a}^2} dH$$

which shows the probability (p) that a wave of height H should lie between (H) and (H + dH), where $\bar{a}$ is the root mean square of all occurring wave heights.

If all (N) wave heights seperated by equal intervals "are arranged in descending order of magnitude", the mean value of the first p N of these, where p is a fraction between 0 and 1, is denoted by $a^{(p)}$. Thus the mean of the upper one third is denoted by $a^{(\frac{1}{3})}$ and the mean of all waves by a'. Longuet-Higgins goes on to derive an expression for the ratio $a^p/\bar{a}$ and finds that $a^{(\frac{1}{3})}/\bar{a}$ = 1.416 and $a'/\bar{a}$ = 0.886. This enables an evaluation of the root mean square of all waves to be made from field measurements of say the upper one third. Longuet-Higgins makes a comparison with field observations and shows that the results obtained from actual observations in deep water are in good agreement with those calculated on the basis of his theory. He cautions against the use of the theory in the surf zone where waves are filtered, because the higher waves are reduced in height during breaking. It is necessary to see if this theory holds good in the surf, before it can be used, and this was the purpose of the investigation described below.

1.3. Continuous records of wave height measurements may be made by a variety of methods. Commonly used methods are, to measure the movement of a float, or the pressure changes under the wave, or changes in the electrical resistance of a vertical wire partly submerged. For measurements in the surf there appeared to be some advantages in using the pressure method, since the pressure sensing device could be located in the quieter water near the sea bottom.

Changes in pressure are transmitted pneumatically or electrically. In these tests electrical transmission was used.

The pressure changes (Δp) due to the passage of an oscillatory wave of height H and wave length L over an instrument at depth y below the still water surface in water depth d is given according to Le Mehauté (1960) by

$$\frac{\Delta p}{\rho' g} = \frac{H \operatorname{Cosh} \frac{2\pi(d+y)}{L}}{\operatorname{Cosh} \frac{2\pi d}{L}}$$

The amplitude of the fluctuations decreases with depth. If the instrument is situated on the sea bottom, d = -y and

$$\frac{\Delta p}{\rho' g} = \frac{H}{\operatorname{Cosh} \frac{2\pi d}{L}}$$

In order to compensate for the pressure attenuation it is necessary to multiply the readings of the pressure recorder by a response factor $\left(\frac{1}{k}\right)$ where

$$\frac{1}{k} = \operatorname{Cosh} \frac{2\pi d}{L}$$

If the pressure sensing device is used in the surf zone where the water depth seldom exceeds 8 feet, the response factor for a common periodicity of 8 sec would be 1.077.

Neglecting the response factor would introduce an error of about 7.7% in the absolute value of the wave heights.

1.2 Experimental

Recordings were made of the height of the waves in the region between the outer and inner zone of breakers. The waves measured were therefore those regenerated by the breakers in the outer breaker zone.

The equipment used consisted of a Bourn's pressure head with a bellows which actuated a sliding contact on a resistance that formed part of a potentiometer system. The signals from the potentiometer were carried to the shore by way of a three-core rubber covered cable. The recording was made on a clockwork recorder with a paper speed of 2" per min. The pressure head was housed in a brass casing mounted on a lead plate. It was found that an lead plate 1 ft in diameter and ½" thick was sufficient to keep the instrument stable on the sea bottom. In practice the instrument was carried out of the surf by a swimmer and placed on the sea bottom about 100 ft offshore. To mark its position a small float was secured to the brass container by a polypropylene cord. This cord facilitated recovery. Fig. 5-5 shows the essential features of the pressure head and its housing.

The instrument was calibrated by lowering it to known depths in a quiet part of the harbour at Durban. The calibration test is described in the Appendix 10. Over the range used this instrument

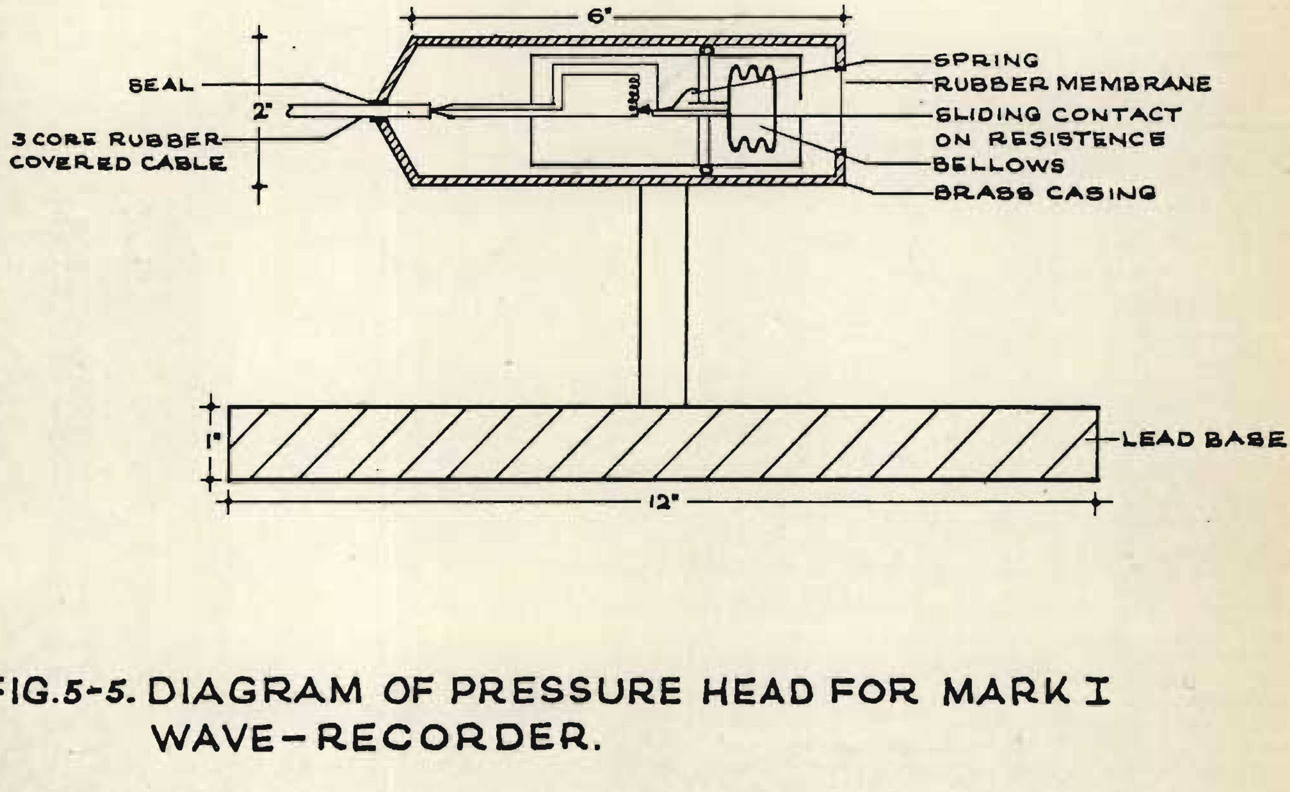


FIG.5-5. DIAGRAM OF PRESSURE HEAD FOR MARK I WAVE-RECORDER.

would have a maximum error of 5%.

13. Results

Specimens of the recordings are shown in Fig. 5-6. The recordings for two days were subjected to detailed analysis. The wave trace was divided into sections of twenty minutes. The wave heights (distance from crest to trough)were measured on a travelling microscope with an accuracy of 10^{-2} mm. The results of the analysis of waves measured are shown in the accompanying Table 5.4.

For comparison the theoretical results for deep water waves as calculated by Longuet-Higgins are included. The value of $\frac{a^{\frac{1}{3}}}{a'}$ from the experimental work is 1.517 which compares with the theoretical value of 1.416. It must be noted that the experimental results apply to a beach having a submerged offshore sandbar, and therefore are relevant to the regenerated waves over the trough.

Based on the results in Table 5.4 the following relationships apply,

(a) to convert significant wave heights to mean wave heights multiply by 0.66.

(b) to convert mean wave heights to root mean square wave heights multiply by 1.11.

In calculations of longshore currents which follow, these relationships have been used to obtain root mean square wave heights from significant wave heights - the significant wave heights being multiplied by 0.73.

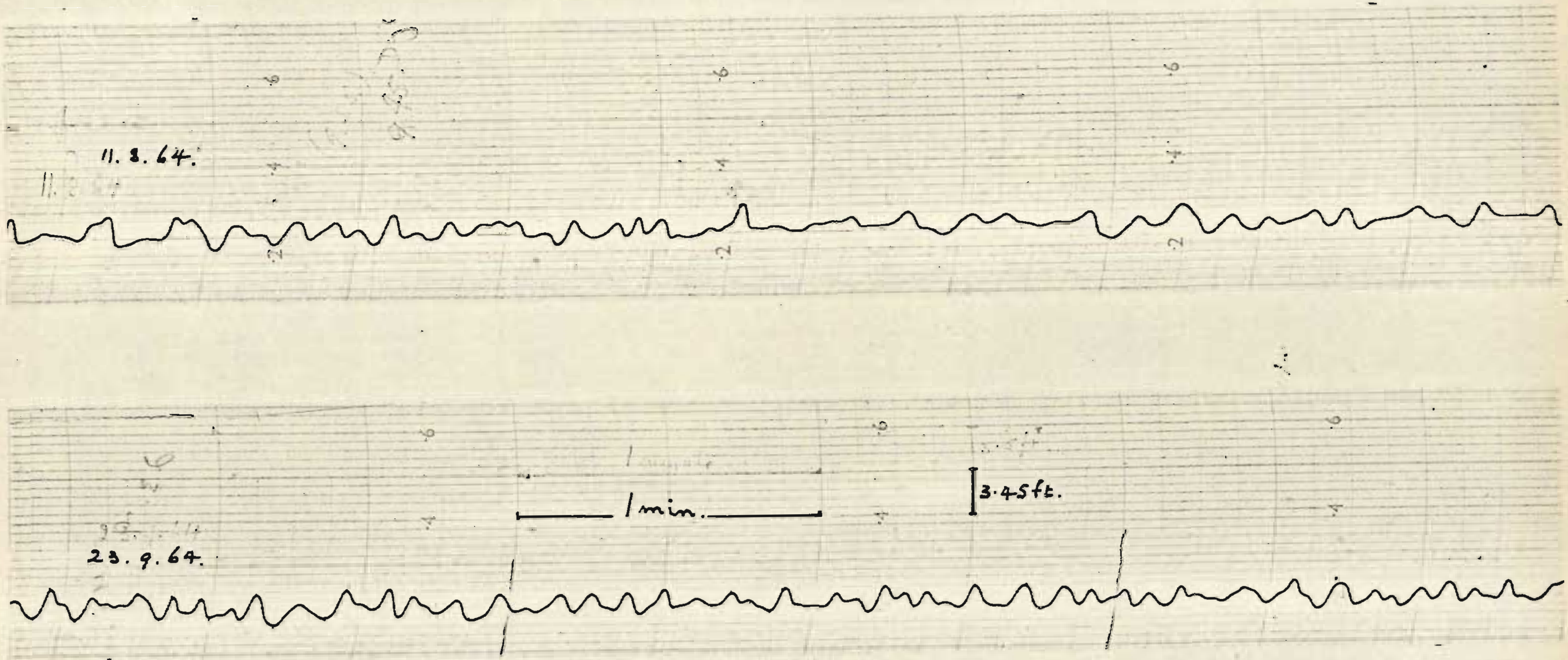


Fig. 5-6. Wave records made with recorder Mk.I in the between-breaker zone.

TABLE 5.4. ANALYSIS OF WAVE HEIGHTS IN THE BETWEEN-BREAKER ZONE

Date	1 No.of waves	2 a'	3 No.of waves in upper $\frac{1}{3}$	4 $a^{\frac{1}{3}}$	5 Ratio $\frac{a^{\frac{1}{3}}}{a'}$	6 No.of waves in upper 30%	7 a^{30}	8 Ratio $\frac{a^{30}}{a'}$	9 Ratio $\frac{a'}{\bar{a}}$
23.9.64.	195	0.2377	65	0.3642	1.532	58	0.372	1.567	0.9057
23.9.64.	180	0.2443	60	0.3691	1.510	54	0.377	1.545	0.9003
23.9.64.	197	0.2404	66	0.3658	1.523	59	0.373	1.552	0.9043
23.9.64.	185	0.2468	61	0.3695	1.497	56	0.378	1.533	0.9133
11.8.64.	147	0.2446	49	0.3729	1.524	44	0.382	1.561	0.9025
Longuet-Higgins data.	-	-	-	-	1.416	-	-	1.454	0.886

Column 2 : a' is the mean wave-height in cms on the wave record

Column 4 : $a^{\frac{1}{3}}$ is the mean "of the" upper one third in cms on the wave record.

Column 7 : a^{30} is the mean of the upper 30% in cms on the wave record.

Column 8 : $\frac{a'}{\bar{a}}$ is the ratio of the mean to the root mean square.

2. Method for measuring Rates of Flow

A method hereafter referred to as the "tracer dilution method" sometimes used for determining the flow in open channels involves the release of a batch of tracer material and the sampling of it at a station sufficiently far downstream that homogeneity has been established. From a knowledge of the mass of the initial tracer (M) and the change of concentration (C) with time (t) the discharge (Q_o) can be calculated.

$$Q_o = \frac{M}{\int_0^t C dt} \tag{5.28}$$

If the concentration is plotted against time, the area under the curve (A) is a measure of the integral. Thus

$$Q_o = \frac{M}{A} \tag{5.29}$$

The river-like flow of the longshore currents prompted the application of this technique in the surf zone. Sodium fluorescein was the tracer material generally used. When a longshore current in the inner breaker zone was being examined, the dye (mass M) was introduced as a concentrated solution or powder at the water's edge. The sampling point was established several hundred feet downstream. Batch samples were taken near the surface in knee depth water. The fluorescence of the samples was measured in a fluorimeter. To evaluate the integral in Equation 5.28 a graph of the concentration in parts per million (ppm) was plotted against the time (in seconds) which elapsed between the first arrival of the dye patch at the sampling station and the time of sampling. The area (A) under the curve was found. To calculate the discharge in cusecs, A in appropriate units (p.p.m. × seconds) was inserted in equation with suitable additions to convert ppm into lbs/cub. ft. Assuming the density of sea water is 64 lbs/cub.ft the equation thus becomes

$$Q_o = \frac{M \times 10^6}{A \times 64} \qquad (5.30)$$

where M is in lbs and A in ppm-seconds.

2.1. Evaluation

Before the tracer dilution method which is designed for open channel flow, could be applied the longshore current, its suitability had first to be tested. Ideally the results obtained by the method should be compared with measurements made by some well established technique. This could perhaps best be done by determining the cross section of the current and measuring by means of a current meter the distribution of velocity across it. Besides being very difficult physically, the measurement of velocities in the surf is complicated by the orbital velocity of the waves and breakers. Furthermore an accurate measure of the cross-sectional area is also difficult because of the variation of sea level changes whichoccur, and the problem of defining the shoreward and seaward boundary. The seaward boundary is in fact a good deal more clearly defined than the shoreward one. It occurs just outside the line of breakers in the inner breaker zone and the sharp discontinuity which marks it is readily shown up when dye is introduced into the current. There being no absolute test of the method available, it was decided to examine the following questions.

(a) How homogeneous is the distribution of the dye tracer concentration over a cross section.

(b) How does the discharge indicated by the tracer dilution method compare with that calculated from a knowledge of the area of the cross section of the current and the current velocity as indicated by the time of travel of the head of a patch of dye (which would give the maximum discharge).

(c) What kind of agreement is there between successive measurements of discharge. i.e. what is the reproducibility.

Since the alongshore system only occurs about 10% of the time it was decided that some of the above tests would have to be done in the current between the rip currents when the asymmetrical circulation was operating.

2.2. Homogenity in the Cross Section

Method - Sodium fluorescein dye was released into the long-current and a sampling station set up at some distance down-stream. A swimmer collected samples as quickly as possible from five points in the cross section, both from the surface and near the bottom.

Results - Analysis was made of the samples taken on three occasions. The results are shown on Table 5.5.

TABLE 5.5

VARIATION OF DYE CONCENTRATION WITH DISTANCE AND DEPTH IN THE CROSS SECTION OF LONGSHORE CURRENTS IN THE INNER BREAKER ZONE

Date of Test	Distance from point of dye release	Circulation type	Approx. width of current (ft)	Approx. Distance from shore (ft)	Approx. Depth (ft)	Concentration ppm
3.4.65	120 ft	Alongshore	75	10	0.5	0.30
				50	0.5	0.24
				50	3	0.42
				70	0.5	0.21
				70	5	0.11
21.7.60	750 ft	Alongshore	85	10	0.5	1.08
				40	1	1.08
				40	3	1.02
				80	1	1.00
				80	4	0.96
11.8.65	375 ft	Alongshore with small rips	85	10	0.5	1.60
				50	0.5	1.30
				50	3	1.00
				80	0.5	0.50
				80	10	0.15

The conclusion from these tests are that if sufficient distance intervenes between the sampling station and the release point, the homogeneity is good. Clearly the 120 ft on 3.4.65 was insufficient. It is also clear that, as may be expected, the intervention of small rip currents cause a falling off of concentration near the outer boundary.

2.3 Comparison with Discharges measured Physically

Method - The cross section at each sampling station was found by measuring the depth below mean sea level of the sea bottom at various distances offshore measured from the mean uprush point of the breakers. The outer boundary of the current was clearly marked by discontinuity of dye. The velocity of the current was measured by timing the travel of the head of a dye patch over a measured distance. It would of course have been preferable to measure the velocity at several fixed points in the cross section (the Euler concept of velocity) but as stated above the orbital velocity of the waves and breakers introduces impossible complications. Simultaneously with the above measurements, the time rate of change of concentration of a tracer released upstream was obtained from samples collected at the station. The results are set out in Table 5.6.

TABLE 5.6

OBSERVED DISCHARGES COMPARED WITH THOSE CALCULATED FROM PHYSICAL MEASUREMENTS OF THE LONGSHORE CURRENT IN THE INNER BREAKER ZONE

Date	Circulation	Velocity ft/sec.	Cross Section sq.ft.	Calculated discharge cusecs.	Observed Discharge cusecs.
30.11.64.	Alongshore	2.2	340	750	680
2. 4.65	Symmetrical	0.24[3]	300	72	150
3. 4.65.	"	0.25	228	57	63
5. 6.65	"	1.0	100	100	63
23. 9.64	Asymmetrical	2.5	150	375[2]	240[1]
10. 8.65	Alongshore	1.86	465	365	690
11. 8.65	"	2.0	450	900	320 (960)[4]

Note: 1. Continuous discharge

2. Mean value

3. Very slow current, difficult to measure

4. Correction for non-homogeneity in the cross section due to small rips was applied.

It should be noted that the discharge obtained from cross section and current velocity give the maximum velocity while the results from the concentration changes yield average velocities.

The results from the tracer dilution method are in fair agreement with the measured values. Either the physical measurements or the tracer dilution method may be the cause of discrepancies

2.4. Reproducibility

In order to test the reproducibility of the method, four tests were carried out successively. 1 lb of sodium fluorescein was released into the current, and the concentration changes with time obtained from samples collected 375 ft downstream. Details of the tests and the concentration data are set out in Appendix 11. The concentration time curves are shown in Fig. 5-7. The discharges calculated by using these curves are listed in Table 5.7.

TABLE 5.7

REPRODUCIBILITY OF THE TRACER DILUTION METHOD

Tests	Discharge in Cusecs.	Deviation from the mean
1	300	6
2	330	34
3	250	44
4	295	1

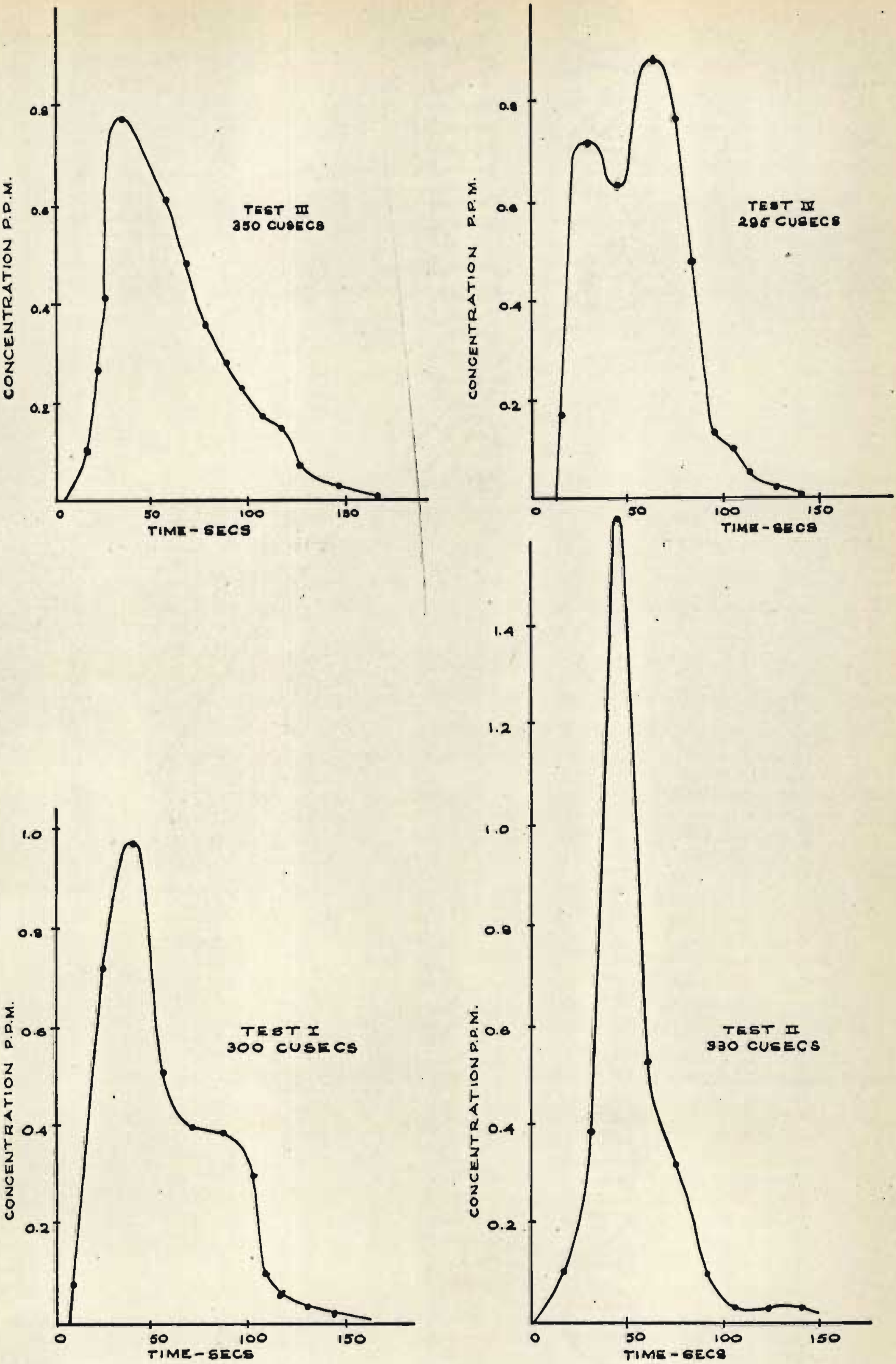


FIG. 5-7. CONCENTRATION - TIME CURVES OF A TEST ON 11.8.65 DONE IN QUADRUPLICATE TO INVESTIGATE THE REPRODUCIBILITY OF THE TRACER DILUTION METHOD.

SECTION III

3. Tests to determine the Discharge of the Alongshore System

3.1. General - Details are given below of tests carried out to measure the discharge of the longshore current when the alongshore system prevailed. It was not always possible beforehand to be certain that the rip currents were negligible. Where it was subsequently found that rip currents had interfered with the uninterrupted flow of the longshore current, the test was rejected.

3.2. Methods and Procedure

In all the tests the "tracer dilution method" was used. Tracer was introduced either batchwise or as a continuous stream at the water's edge. Sampling stations were set up at various distances downstream. Samples were taken by a sampler standing knee deep in the surf. Each sampler was equipped with a set of numbered sampling bottles which varied in capacity from 50-200 ccs. The rate of sampling was greater nearer the tracer release point. After the field test the samples were analysed in the laboratory. When the tracer used had been sodium fluoroscein, the fluorescence was measured on a Turner fluorimeter using a pass filter of 450 μ. This method is sensitive to .0002 parts per million and is accurate to 1%. Care was taken to store samples in the dark because it was found that decay of the fluorescence occurred in daylight. On the occasion when the tracer was suspended clay particles, the turbidity was measured in a spectrophotometer.

Having analysed the samples, graphs were plotted of the concentration against the time elapsed from the time of the first sampling at each station. The area under the curve was determined graphically and the discharge calculated from Equation 5.30.

$$Q_o = \frac{M \times 10^6}{A \times 64}$$

where Q_o is the discharge in cu.ft/sec., M is the mass of the tracer introduced in lbs. A is the area of the concentration time curve in ppm-sec units, and 64 is the density of sea water near the shore, in lbs/cu.ft.

For the test where the tracer was released continuously the discharge (cusecs) was calculated directly from

$$Q_o = \frac{CoVo}{C} \qquad (3.31)$$

where Co is the initial concentration of the tracer in ppm, C is the mean concentration at the sampling station and Vo is the rate at which the tracer was discharged in cusecs.

In order that the observed discharge could be compared with that predicted by Equation 5.27, data as to the conditions prevailing during each experiment were recorded. Observations were therefore made of the following:

(a) Wave height at breaking. The heights of the waves breaking in the inner breaker zone were measured directly with a graduated pole. The accuracy is estimated to be ±10%.

(b) The angle that the wave made with the shore was measured by sighting along the waves at breaking and recording the angle made with some distant point along the shoreline. The accuracy is estimated at ±15%.

(c) The wave period was determined by timing the arrival of a known number of waves.

(d) Beach slopes were measured in the intertidal zone by means of à level and graduated rod. T (this variable is not required in Equation 5.27).

3.3 Description of Alongshore Current Tests

Conditions suitable for tests occurred only rarely. There were several occasions when the system appeared to be suitable but which were marred by small rips which were seen to operate during the course of a test. As the preliminary work, reported above had shown that these small rips militate against homogeneity of the tracer distributions these had to be discarded. In all there were five occasions when satisfactory conditions prevailed. These five tests were carried out on four different beaches.

TEST A.1. (2.6.61)

Description of Beach - The Country Club Beach in Durban stretches south from the Umgeni River. In the northern section the sand had a median diameter of 0.5-1 mm and beach slopes were about 7°. Towards the southern end the sand was found to be finer about 0.25-0.5 mm median diameter and the slope in the intertidal zone correspondingly gentler, about 4°. A submerged sandbar was in evidence along the whole stretch of beach but was more pronounced in the northern section. The beach is nearly straight and had no rock outcrops except at the north end where a short stone groin projects about a hundred feet into the sea at the river mouth.

Wind - During the test the wind was fresh from the North East.

Procedure - 10 lbs of sodium fluorescein were inserted about 200 feet south of the groin. Sampling was carried out at six stations spread out over 6000 feet downstream. Except between stations 1 and 2 where a small rip current occurred the longshore flow was uninter-rupted.

Conditions - The experimental conditions are contained in Table 5.8.

Results - The results of the analysis of the samples are set out in Appendix 12. The concentration-time curves for each station constructed from the data are shown in Fig. 5-8.

TEST A.2. (11.8.62)

Description of Beach - The Umgababa test beach was part of a relatively straight beach which stretched 10,000 ft between two minor headlands which project a few hundred feet seaward of the main coastline. The beach had a gentle slope (2°) consequent on the sand having a rather small median diameter of about 0.2 mm. There was no obvious sandbar. These two characteristics differentiated it from the other test beaches.

Procedure - Approximately halfway between the headlands a continuous supply of a fine clay material (-10 microns) was pumped onto the beach (the clay being a by-product of a mineral processing plant). The rate of the discharge of the clay effluent was 6 mg. pd. The concentration of the material was 11.4 ppm. Laboratory tests showed that when the clay was suspended in sea water and allowed to stand there was no evidence of settling out over a period of twenty four hours. It was therefore assumed that "a fortiori" it would not settle when subjected to turbulence of breaking waves. Advantage was taken of this situation since it was the only available source of large quantities of a tracer material whose characteristics were reasonably well known. Seven sampling stations were set up downstream (to the south) as far as the headland.

On one occasion when the longshore system had been in operation for twenty four hours it was assumed that equilibrium conditions had been established, and samples were collected from the stations. These

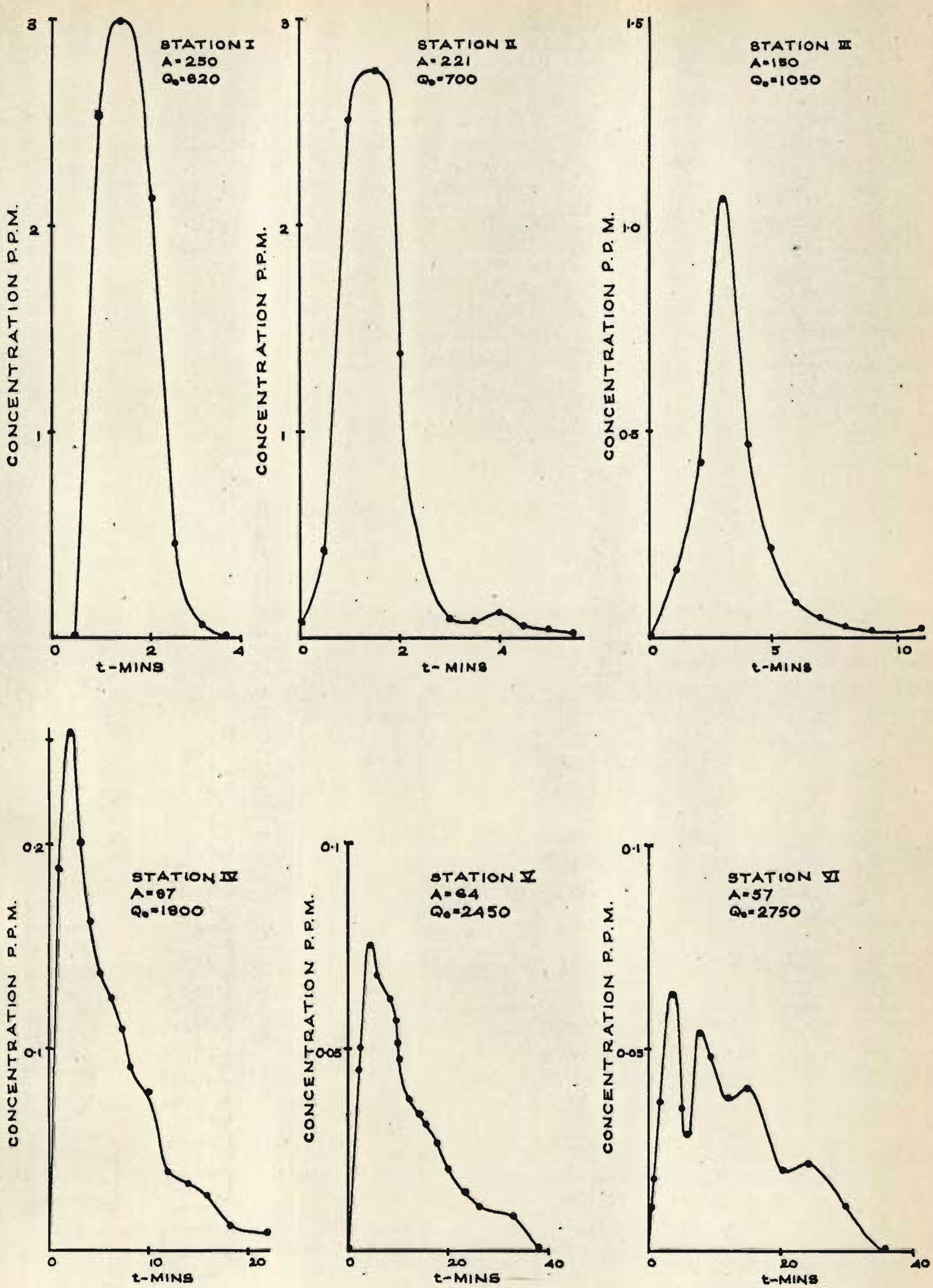


FIG.5-8. TIME-CONCENTRATION CURVES OF TRACER FOR TEST A1 (2·6·61)

A= AREA OF CURVE IN TIME CONCENTRATION UNITS

Q_0=ESTIMATED DISCHARGE IN CUSECS

were analysed for turbidity in a spectrophotometer and the concentration of the fine material determined. (Note the analyses were carried out by Mr. Dowie-Dunn).

Conditions - During the test there was no wind. Other data are set out in Table 5.8.

Results - The results of the analysis are set out in Appendix 13.

TEST A. 3. (27.8.64)

Test A.3 was carried out on Virginia Beach which has already been described. On the occasion of the test the beach slope was 8^{o}.

Procedure - 1 lb of sodium fluorescein was released into the surf. Two sampling stations were set up.

Conditions - The wind was light North East. Wave data are contained in Table 5.8.

Results - The results of the fluorescence analysis on a Turner fluorimeter are set out in Appendix 14, and the concentration time curves in Fig. 5-9.

TEST A. 4. (14.7.65)

Virginia Beach was the site of the test. The beach slope was 6^{o}.

Procedure - 2 lbs of sodium fluorescein was released into the longshore current and sampled at five stations.

Conditions - The wind was light Southerly. The wave conditions are reported in Table 5.8.

Results - The results are listed in Appendix 15, and the concentration curves drawn in Fig. 5-10.

TEST A.5. (2.3.60)

Description of Beach - The test was carried out at Inyoni Rocks Beach which is similar in composition and slope to that at Virginia. A submerged sandbar was in evidence. The test site was the middle

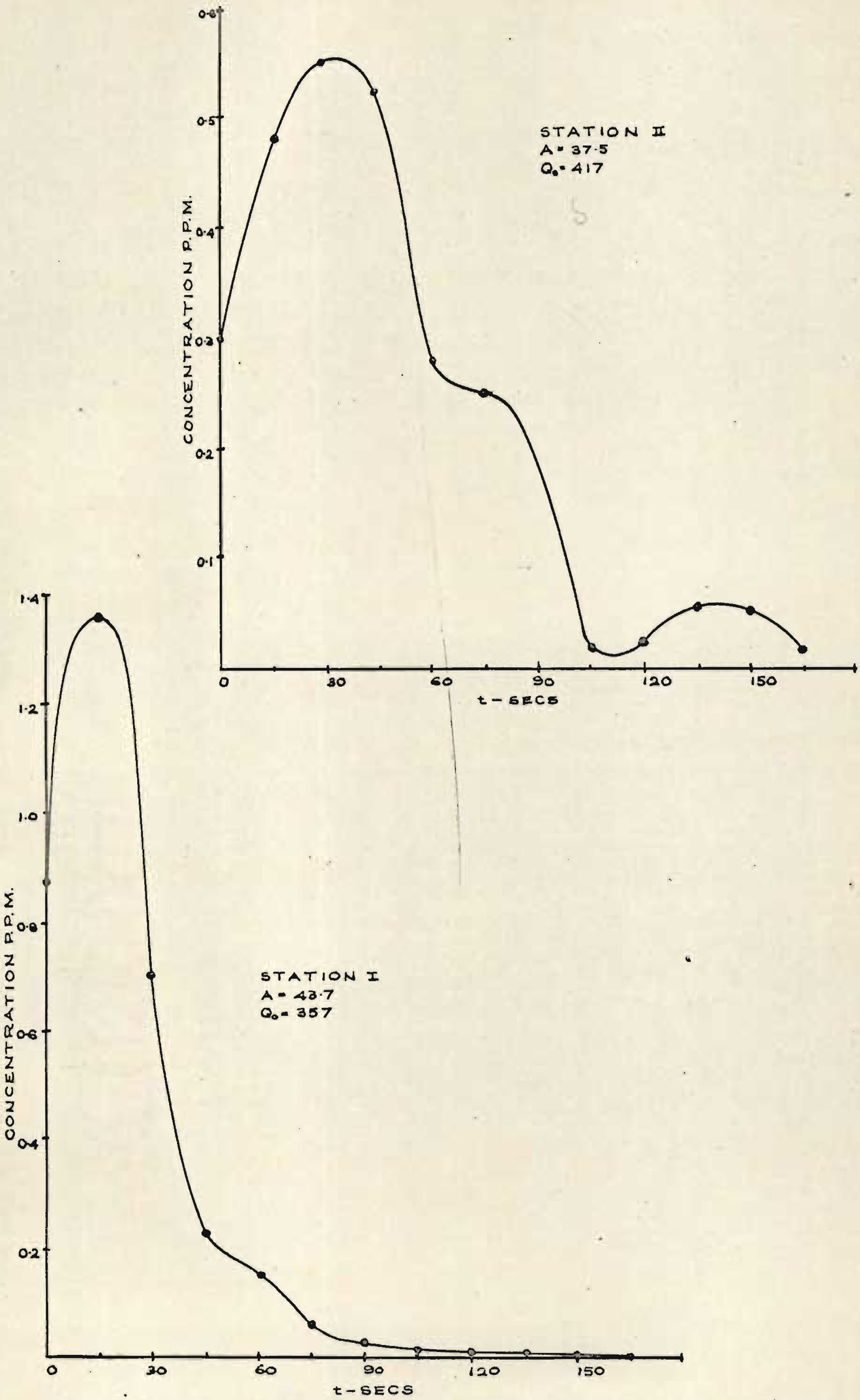


FIG. 5-9. TIME-CONCENTRATION CURVES OF TRACER FOR TEST A3. (27. 8. 64.)

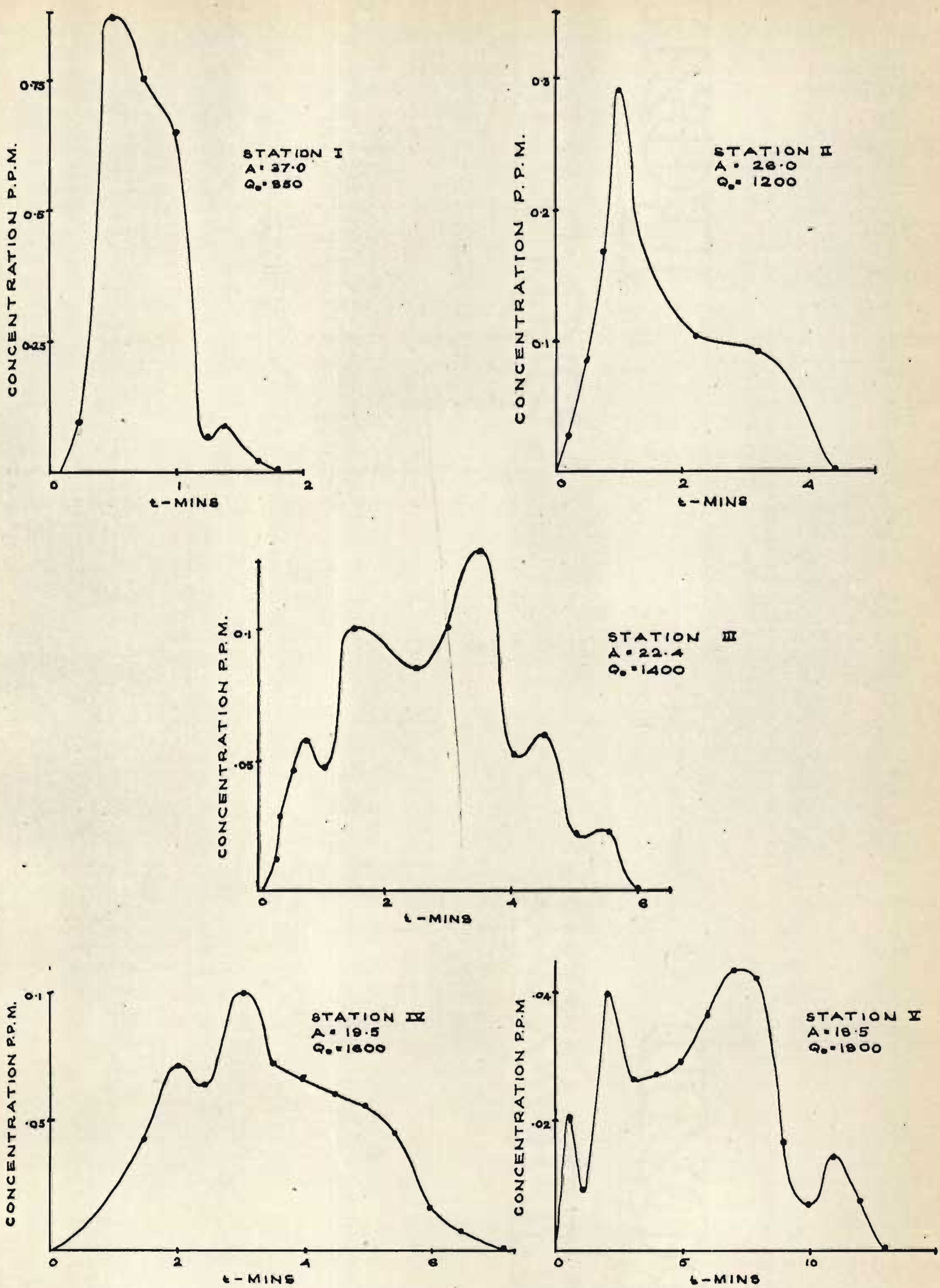


FIG.5-10. TIME-CONCENTRATION CURVES OF TRACER FOR TEST A4 (14.7.65.)
A=AREA IN TIME CONCENTRATION UNITS

portion of the almost straight beach which stretches 7,600 ft. from Inyoni Rocks northward to another rock outcrip.

Conditions - The wind was light. The wave data are set out in Table 5.8.

Procedure - 5 lbs of sodium fluorescein was released into the surf and sampled at three stations downstream.

Results - The results of the analysis of the samples by means of a Farrant fluorimeter are set out in Appendix 16, and plotted on the concentration time curves in Fig. 5-11.

The discharges (Q_o) deduced from tracer dilution method of the above tests have been assembled in Table 5.8. The increments of discharge (ΔQ_o) between stations have been calculated, and are referred to as the observed discharges.

The wave data and distance between stations for each test have been substituted in Equation 5.27 and calculated values of the discharges (Q_c) and the increments (ΔQ_c) between stations found. These have been entered in Table 5.8 and are referred to as calculated values.

Observed and calculated increments are compared in Fig. 5-12. The observed discharges with increasing distance are plotted in Fig. 5-13.

TABLE 5.8

SCHEDULE OF DATA FOR ALONGSHORE SYSTEM TESTS

NOTE: THE SYMBOLS USED HAVE THE FOLLOWING MEANING:

S = station, L_s = distance in feet of station from point of insertion of tracer, ΔL_s = distance in feet between stations, H = significant wave height at breaking in the inner breaker zone, T = wave period secs, α = mean angle the breaker makes with the shore, Qo = the observed discharge in cusecs at the station, ΔQo = increment of discharge between stations, Qc = calculated discharge in cusecs, ΔQc = calculated increment of discharge between stations, θ = angle of slope of beach in intertidal zone, γ = ratio of wave height at breaking to depth at breaking (taken to be 0.78), B is factor for converting significant breaker height to root mean square height = 0.73.

$$\Delta Qc = \frac{4(BH)^2 \text{ Sin } \alpha \text{ Cos } \alpha \;\; L_s}{T \sqrt{3\gamma^3}}$$

TEST A.1. 2.6.61 COUNTRY CLUB BEACH

$H = 2$ $T = 5$ $\alpha = 20^\circ$ $\theta = 4^\circ - 7^\circ$ Tracer 10 lbs sodium fluorescein

S	L_s	Qo	ΔQo	ΔL_s	ΔQc	
1	500	620	-	-	-	
2	1000	700	80	500	240	(discard - rip influence)
3	2000	1050	350	1000	420	
4	3500	1800	750	1500	720	
5	5000	2450	600	1500	720	
6	6000	2750	300	1000	420	

TEST A.2 11.8.62 UMGABABA BEACH

$H = 2$ $T = 7$ $\alpha = 25^\circ$ $\theta = 2^\circ$ Tracer = continuous discharges of suspended clay

S	L_s	Qo	ΔQo	ΔL_s	ΔQc	
1	300	100	-	-	-	
2	900	200	100	600	230	
3	1500	395	195	600	230	
4	2100	723	328	600	230	
5	2700	990	267	600	230	
6	3450	1560	570	750	288	discard -
7	3900	1800	240	450	168	near rocks

TEST A.3 27.8.64. VIRGINIA BEACH

H = 1.5 T = 6 α = 15° θ = 8° Tracer = 1 lb sodium fluorescein

S	L_s	Qo	ΔQo	ΔL_s	ΔQc
1	360	357	-	-	-
2	780	420	63	420	57

TEST A.4 14.7.65 VIRGINIA BEACH

H = 3.25 T = 8.5 α = 15° θ = 6° Tracer = 2 lbs sodium fluorescein

S	L_s	Qo	ΔQo	ΔL_s	ΔQc
1	240	850	-	-	-
2	660	1200	350	420	230
3	960	1400	200	300	162
4	1272	1600	200	312	162
5	1852	1800	200	580	306

TEST A.5 2.3.60. INYONI ROCKS BEACH

H = 3.0 T = 8 α = 20 θ = 6° Tracer = 5 lbs sodium fluorescein

S	L_s	Qo	ΔQo	ΔL_s	ΔQc
1	250	270	-	-	-
2	1000	1300	1030	750	574

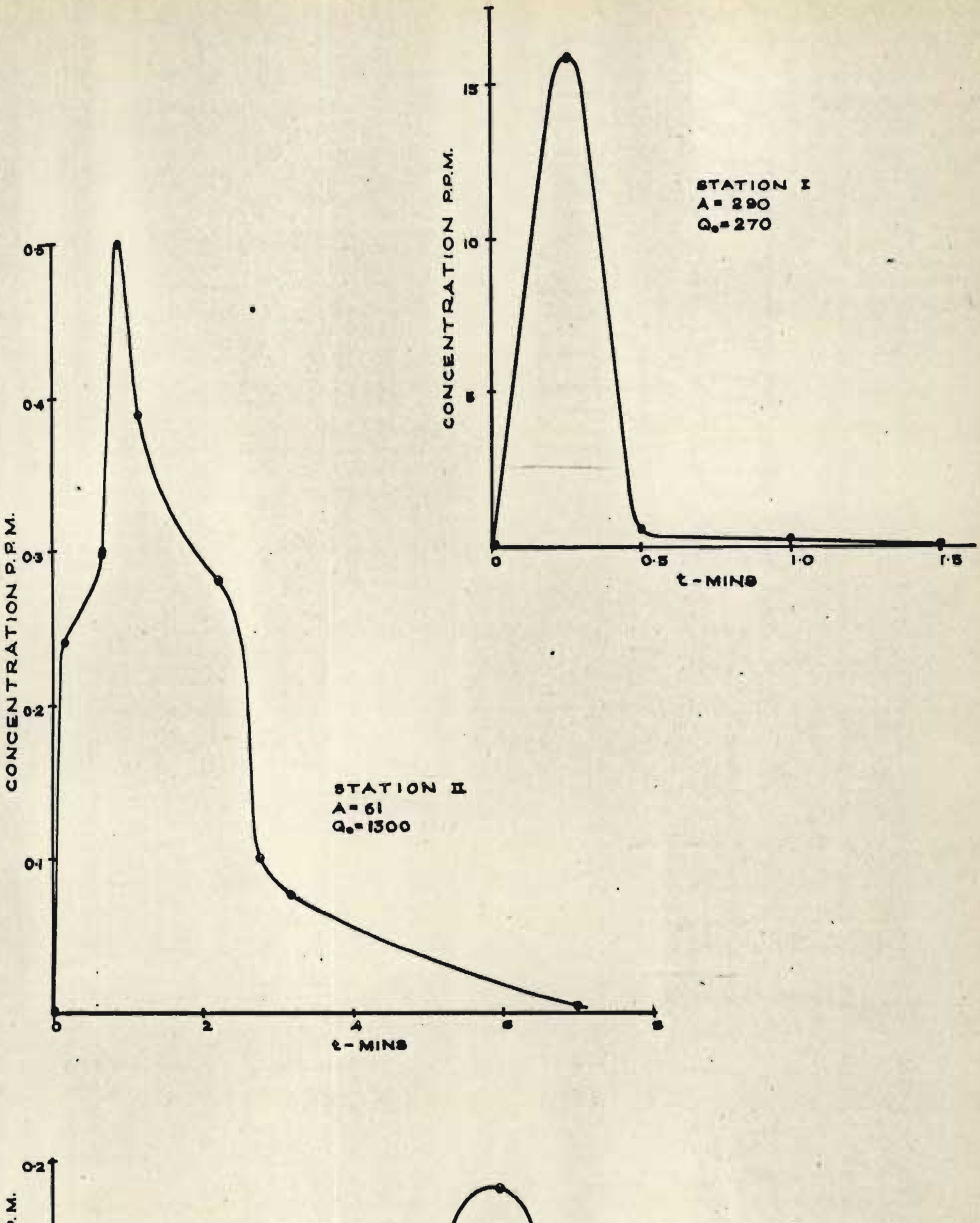


FIG.5-11. TIME-CONCENTRATION CURVES OF TRACER FOR TEST A5 (2·3·60)

A = AREA OF CURVE IN TIME-CONCENTRATION UNITS

4. Discussion on the Alongshore Tests

In Fig. 5-12 increments of discharge as determined by the tracer dilution method and those calculated from the application of the solitary wave theory are compared. The mean ratio of the calculated to observed increments is 0.92 with a standard deviation of 0.22. The agreement is tolerably good, especially in view of the difficulties of making representative measurements of wave characteristics, and of sampling errors.

It must however be noted that both the tracer dilution method and the solitary wave theory have their shortcomings, and might be in error in the same direction. The most likely error in the former is inhomogeneity of the tracer in the cross section which would result in discharges which were on the low side. The latter may be in error because the mathematical model (as has been remarked on) only approximately simulates the waves in the field and because of the inaccuracy of the data describing the wave characteristics during the test. Figure 5-12 lends some support to the assumption that the increase in discharge is proportional to distance, at least over the distance used.

The least satisfactory agreement with theory comes from the observations of Test A.2. It is noted that this beach was different from the others in that it had a gentler slope. Furthermore, the tracer material was a continuous supply suspended clay, the concentration of which may have varied somewhat.

Figure 5-13 depicting the growth of discharge with distance downstream is of some interest. Naturally the slopes differ widely between some tests because the wave conditions were different. Curves A.1 and A.2 show some similarity initially with the discharge growing at an increasing rate for the first 1500-2000 ft. Curves

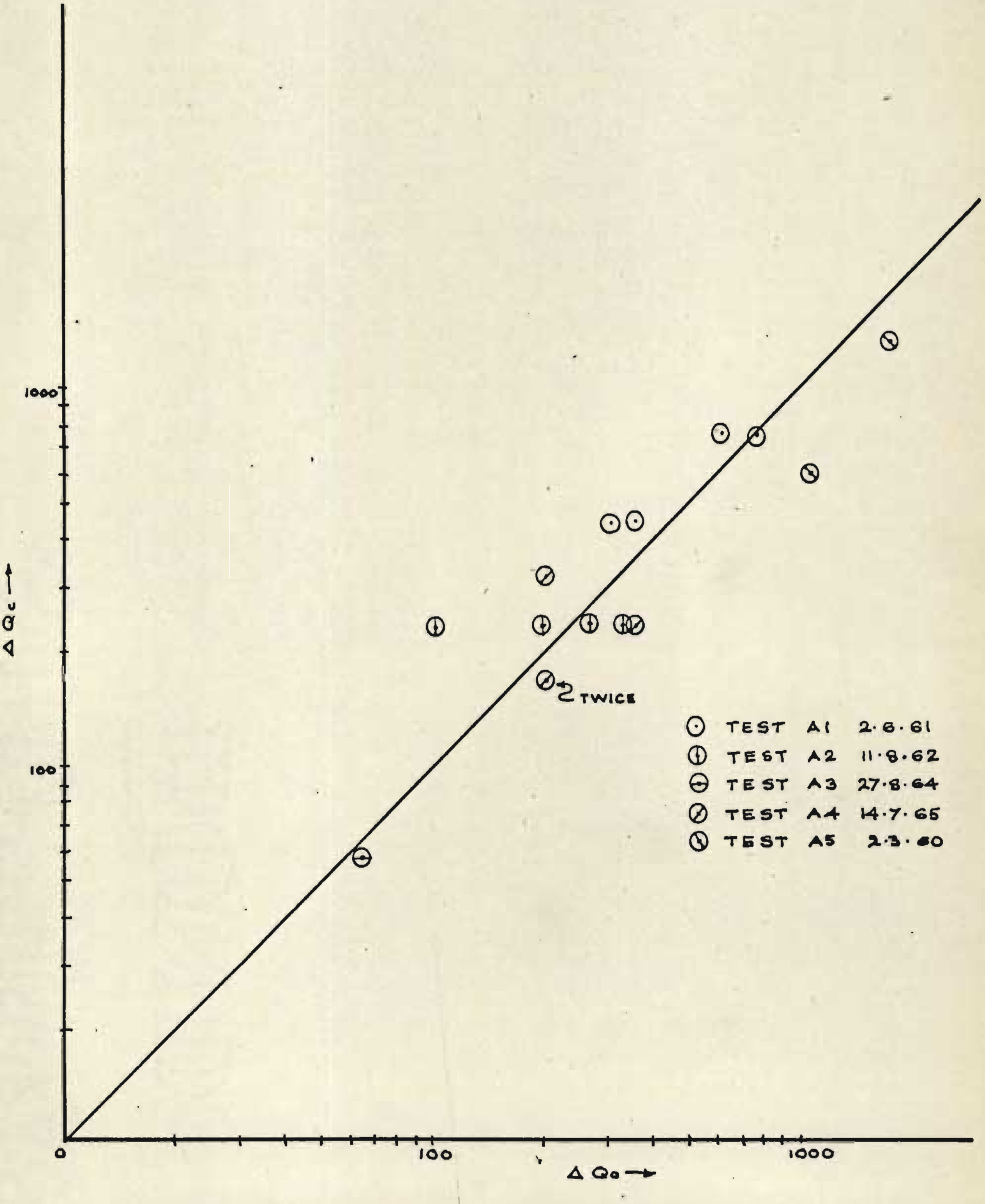


FIG.5-12. COMPARISON OF OBSERVED (ΔQ_o) AND CALCULATED (ΔQ_c) LONGSHORE DISCHARGES

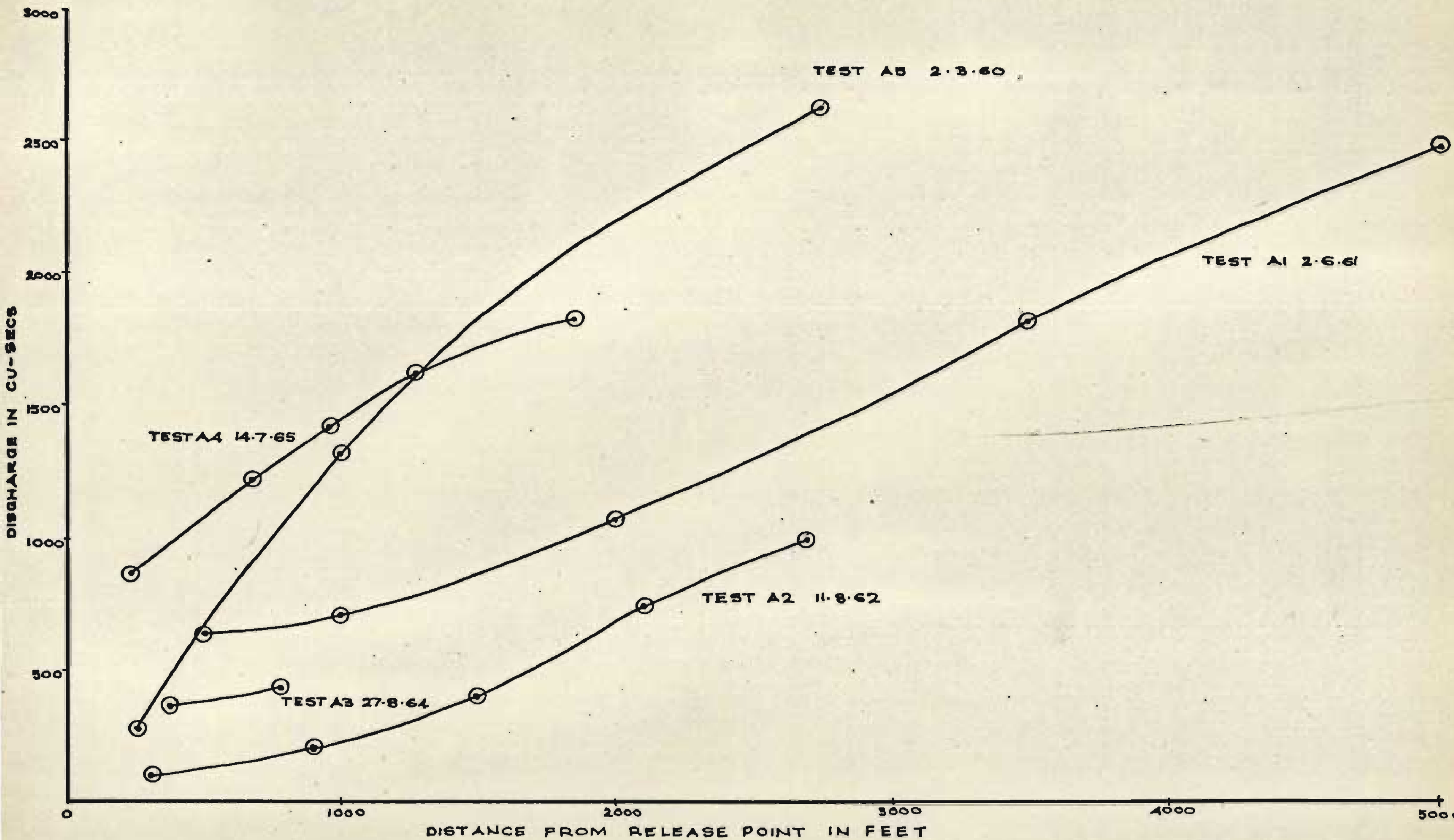


FIG. 5-13. GRAPH OF LONG-SHORE CURRENT OBSERVED DISCHARGES AT VARIOUS DISTANCES FROM THE RELEASE TRACER - RELEASE POINT.

towards the ends of the curves. The fact that the discharges are initially different in the different tests is understandable because the sampling stations were set up somewhat arbitarily at unknown stages in the currents development. Test A.1 probably gives the fullest picture of the growth of the current. The first station was within 300-400 ft. of a coastal discontinuity caused by a short stone groin at the mouth of the Umgeni River - a river which was discharging a few hundred cusess. (This latter fact may have caused the initial discharge to be set rather high (630 cusecs)). The curve as a whole suggests a typical growth curve with accelerated growth initially and decelerated growth far downstream. The first station of Test A.2 was probably also placed at an early stage in the currents development, for 1000 ft upstream marked the beginning of a rock outcrop. The same may be said of Test A.4, while for Test A.5 the current would have been well developed where it was sampled.

CHAPTER VI

SUMMARY AND CONCLUSIONS

FIELD WORK CELLULAR CIRCULATIONS

1. Off the test beach at Virginia the cellular circulation consists of large cells, the mean spacing of whose component rip currents is 1830 ft. Between the rips of the large cells smaller cells originating in the inner breaker-zone, may often be observed. Their mean spacing is 127 ft.

2. The mean width of the whole surf was 470 ft, and of the inner breaker zone 47 ft.

3. The mean ratio of cell width to surf width was 3.7 for the large cells and 2.8 for the small ones.

4. The volume of water bounded by the outer breakers and the rips of the large cells is of the order of 10^6 cu ft. This is exchanged by cycling in the circulation roughly once per hour. The order of magnitude of the rate of exchange is 10^3 cusecs. The actual values were 700-1800 cusecs and these represent 4.5 - 12% of the potential inflow under ordinary wave conditions. Inman and Bagnold (1963) calculate figures of 2-10%.

5. The exchange process is apparently achieved largely by displacement. A rough estimate suggests that the displacing water is made up of 60-80% water recycled directly from the rip currents, the balance being possibly water from outside the circulation. This estimate is based on the frontage over which each type of displacing water enters the surf zone. It does not take into account possible differences of rates of flow.

6. During the field work equipment for measuring sea level changes was developed. Maximum changes were of the same order as

those found by Dorrenstein (1961)-about 10% of the wave heights.

WAVE TANK WORK

6. The symmetrical cellular circulations found in the field are qualitatively well simulated in a wave tank. Essential elements of longshore currents feeding rip currents and recycling are all present. In addition small cells within larger ones were seen. On the other hand, the rip spacing to surf width ratio for the cells in the wave tank is higher being about 5. Also no long term pulsing of rips was observed. However the rip out flow in the wave tank did appear to be interrupted by the arrival of each breaker.

7. A notable feature was the refraction of the incident wave front as it encountered the out flowing rip. This refraction resulted in wave obliquity (with respect to the beach), and is thought to be an important mechanism underlying the circulation. The obliquity is limited to a region adjacent to the rips. As the wave height was increased longshore current and rip current velocity increased. Rip spacing also tends to increase (though irregularly) with wave height. The whole system shows properties of self consistency.

8. Based on the solitary wave theory and on certain experimental observations, an equation predicting the length of breaker crest required to satisfy (by continuity) the volume of flow requirements of a rip current, has been derived. The crest length appears to be greater than the length of crest approaching obliquely (and therefore having a longshore component).

THE CAUSE OF THE CIRCULATIONS IN THE WAVE TANKS

9. With small waves less than about ½", very large amplitude oscillations - in the form of a transverse standing wave - developed along the water line in the wave tank. These have been shown to be caused by the interaction of the generated wave and edge waves travelling across the tank, from either side. This interaction of waves produced rip currents - weak, when the period of the standing wave was twice that of the generated wave, and stronger, when the periods were the same.

10. In circumstances when the waves in the wave tank were about 1.2" - 1.5" and the period in the region of 0.65 secs a prominent three dimensional standing wave was formed just outside the breaker zone. Edge waves were identified in the system and the case is thought to be one of resonance between the generated wave period and possibly that of a discrete or cut off mode of the edge waves. The standing waves appeared to modify the breaker locally producing regular spatial variations of height and causing scalloped configurations along the crest. After the waves had broken small cells of circulation were formed. Their spacing corresponded to the wavelength of the edge wave's second mode.

11. Analysis of a general case (other than the special ones above) pointed to - but not conclusively - the interaction of the generated wave and edge waves as an explanation of the circulation observed.

12. When the wave generators were first started up, the arrival of the first few waves at the shore was accompanied by the development of closely spaced small rip currents - none at

first penetrating the surf. As later waves arrived some of the (more widely spaced) rip currents developed and penetrated the surf while others remained rudimentary or were evanescent.

13. The sum of the evidence is that, certainly for certain special cases and possibly generally the cellular circulations in the wave tanks were caused by interaction between the generated waves and edge waves, resulting in distortion of the wave breaker front, with consequent convergence and divergence of mass transport.

The system seems to represent a type of Rayleigh-Taylor instability, with modes of edge waves determining the falling away from unstable equilibrium.

Alongshore System

1. A tracer dilution method to measure discharge or rate of flow of alongshore currents (similar to that used in rivers) was tested. It was found that provided the alongshore current dominated the system to the virtual exclusion of rip currents, the method gave reproducible results with about 10% standard deviation.
2. The method was applied to measure longshore current discharges on 5 selected occasions.
3. The discharges were also calculated from wave characteristics using the solitary wave theory and applying the approach based on continuity.
4. Taking into account the difficulties of arriving at representative values for the wave characteristics, the calculated

and observed increments of discharge between sampling stations agreed tolerably well. The mean ratio of the former to the latter was 0.92 whith a standard deviation of 0.22.

5. In the course of the above study a test was made on the applicability of Longuet-Higgins' wave height spectra theory for deep water, to the between breaker zone. The relationship between significant, mean, and root mean square wave heights for the experimental data were from 2-10% higher than those of Longuet-Higgins.

Transition from Cellular to Alongshore

It is suggested that the process of transition from the cellular to the alongshore system is as follows.

When the waves are normal they are refracted, in the region of the rip, towards the axis of the rip and an obliquity to the shore is imparted to the wave direction.

When the waves approach is at an angle to the shore, its obliquity will to a greater or lesser extent oppose that due to refraction on the down wave side of the rip. As the angle increases there will be an increasing tendency for longshore current water to flow past the rip base. In the extreme case none will flow out of the rip.

It is thought that the following developments and findings of these studies have not been previously reported

(a) the method used for measuring sea level changes

(b) the application of the tracer dilution to the measurement of longshore current discharge.

(c) the simulation of cellular circulations in a uniform wave tank.

(d) the demonstration that wave interaction can be a cause of cellular circulation under the special conditions of a wave tank.

Further Research

There are a number of problems arising out of the work reported, which might well be profitably pursued.

Theoretical development is required on the following points

(a) the extension of Ursell's work on edge waves, to include the case of symmetrical motion about the midpoint of the "beach"

(b) the theory underlying the falling away from instability of a surge of water returning down a uniform slope, having been initially caused by wave action

(c) a mathematical model is needed for the whole of the cellular circulation leading to an expression for the cell proportions in terms of the wave characteristics and the beach slope. A prerequisite would be an understanding of the velocity field of a rip current.

The great difficulties under which field work is carried out leads to the conclusion that experimental work might best be continued in a uniform wave basin. The basin should be accurately constructed and special attention given to the control and measurement of wave frequencies. Accurate measurements of the changes of water level just outside the breakers are required to test whether the transverse spatial differences so evident in the case of resonance, are more generally present.

In addition such a wave tank could be used for much more accurate work on the velocity fields of the elements of the cellular circulation, with changing wave heights and periods and beach slopes. It would also enable the apparently irregular changes of rip spacing with wave height to be studied. With good data it should be possible to see whether the spacing could be explained by the superposition of several modes of edge waves.

The physical difficulties and indeed danger of work away from the shore in the surf will be a deterrent to field work and comprehensive information does call for sophisticated and semi-permanent installations. As an interim measure field work might be done in the quieter water of harbours, sheltered embayments or lakes.

There are no aspects of the nearshore circulation on which our information is anything like adequate. Measurements are needed on all aspects, and because of the pulsed nature of the system time series measurements are essential. There is scope here for the devlopment of the instrumentation. Many of the necessary variables, such as velocity and sea level changes must be measured against the back ground of the wind generated waves, whose effects are usually of much greater magnitude than that which is to be measured. This will be a primary difficulty in determining what transverse wave motions exist in and near the surf.

For longshore current studies a first requirement is for continuous and simultaneous measurements of breaker height period and direction.

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APPENDICES - 1-16

APPENDIX 1

Field data for the spacing of rip currents in

the inner-breaker zone - Virginia Beach

Date	Wave height at breaking ft.	Wave period secs.	Rip seperation ft.	Inner breaker zone width ft.	Ratio Rip sepera- tion to surf width
6.7.65	1.5 - 3	8.5	100	42	2.4
			81	42	2.0
			75	42	1.8
			240	70	3.4
			330	70	4.7
			120	40	3.0
			60	33	1.9
			78	43	1.6
			210	70	3.0
			110	70	1.6
			120	50	2.4
8.7.65	1 -2.5	7	150		3.75
			90		2.25
			100		2.5
			135		3.4
			87		2.2
			75	40	1.9
			87		2.2
			140		3.5
			72		1.8
			96		2.4
9.7.65	1 -2.5	8	72	40	1.8
			87	40	2.2
3.8.65	3	8	130	46	2.8
			192	46	4.1
			164		3.5
			220		4.7
			152		3.3
			104	46	2.3
			96	approx	2.1
			128		2.8
			198		4.3
			150		3.3
			210		4.6
15.9.65	1 - 1.5	8	75		

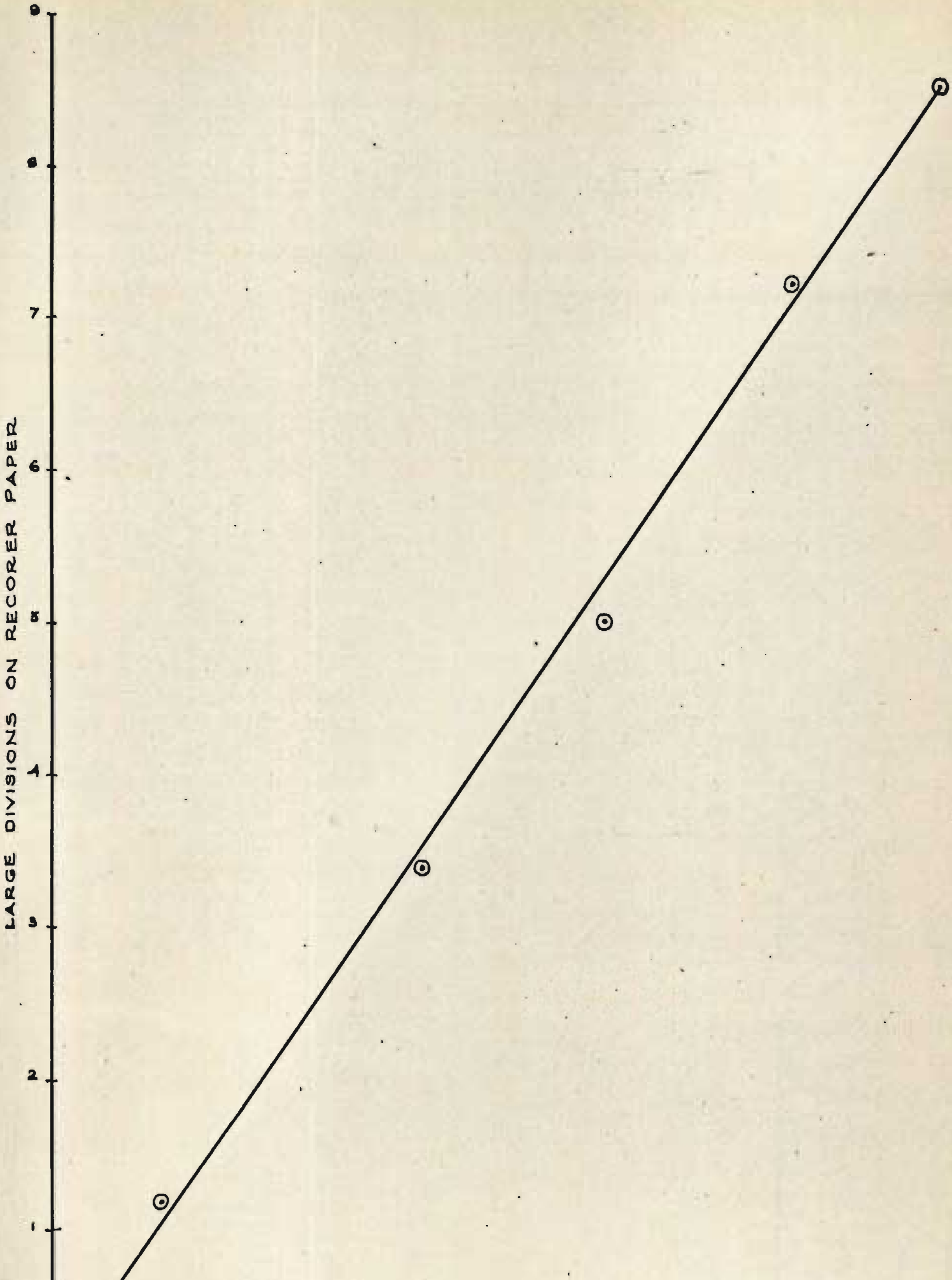
APPENDIX 1

(continued)

Field data for the spacing of rip currents

penetrating the whole surf - Virginia Beach

Date	Wave height at breaking ft.	Wave period secs.	Rip seperation ft.	Inner breaker zone width ft.	Ratio Rip seperation to surf width
23.2.65	4.5	8.0	1000 750	500 500	2.0 1.25
15.2.65	4.5	7.5	1100	450	2.4
3.2.65	5.0	6.0	2400	400	6.0
1.2.65	5.5	6.0	2400	550	4.3
27.7.64	5.5	8	2250	500	4.6
17.7.64	3.5	6.0	1800	550	3.2
15.7.64	4.0	8.5	1800	550	3.2
3.9.65	4.0	6.5	1600	400	4.0
27.7.64	5.5	8.0	2200	500	4.4
30.3.65	4.5	10.0	1660	500	3.3
29.9.65	5.5	7.5	1600	300	5.3
16.7.65	5.0	8.0	1830	650	2.8
23.9.65	4.0	6.5	1600	400	4.0
7.4.65	5.0	9.0	2250	470	4.7



APPENDIX 2. CALIBRATION CURVE FOR SEA-LEVEL RECORDER (24·8·64)

APPENDIX 3

CALIBRATION OF MARK 1 WAVE RECORDER

18.4.64.

Procedure

The wave meter was tested in Durban harbour. The pressure sensing head was lowered to a known depth - at intervals of 1 ft. and the corresponding displacement of the recording pen read off to the nearest 0.1 of a small division on the recording paper.

Results

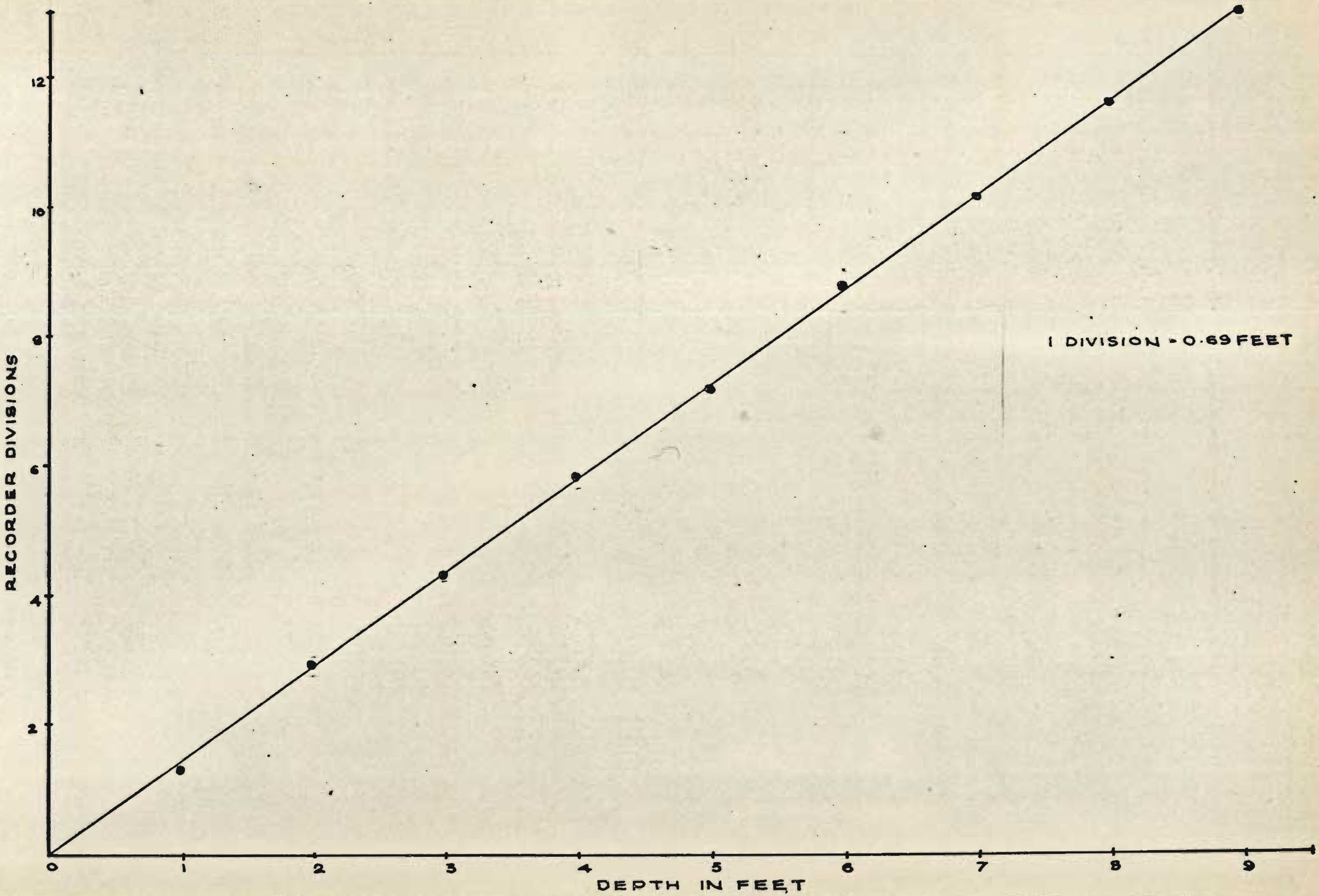
The following were the results. (The reading at zero feet was 1.8 divisions).

Depth in Feet	Number of small divisions on recorder				Mean
1	3.2	3.2	3.1	3.3	3.2
2	4.6	4.9	4.8	4.9	4.8
3	6.1	6.2	6.2	6.2	6.2
4	7.5	7.7	7.8	7.8	7.7
5	9.0	9.0	9.1	9.1	9.05
6	10.2	10.8	10.7	10.9	10.65
7	12.0	12.0	12.1	12.1	12.05
8	13.4	13.4	13.4	13.8	13.5
9	14.9	14.9	15.0	15.0	14.95
10	16.3	16.3	16.3	16.3	16.3

The results are plotted on the accompanying graph.

Conclusions

1 Scale division is equivalent to 0.69 ft.



APPENDIX 3. CALIBRATION CURVE FOR MARK I WAVE RECORDER

APPENDIX 4

WAVE TANK EXPERIMENTS

CHANGE OF SURF WIDTH WITH WAVE HEIGHT

Date : 3.1.1966.

Conditions : Tin plate at 6° Wave period 0.75 sec.

Scale Reading - ins					
Wave Height	Max. run-up	Min. run-up	S.W.L.	Break Pt.	Vert. Pt.
0.5	4.75 4.5 4.25	6.5 6 6.75	8.5	10. 10.5 11	15 15.5 16
1.05	4.5	6.5	8.5	11	16
1.2	4.5	6.5		12	17
1.05	4.5	6.5		11.5	17.5
1.6	4	6	8.5	12	19
	4	6		12.5	18.5
1.7	4	6		12	19
2	3.75	5.75	8.5	12.5	20
1.8	4.5	6		12.5	20
2.1	4.75 5 4	5.75 5.5 6	8.5	13.25 13.5 13.5	21 22 22
2.65	4	5.5	8.5	14.25	23
	3.5 4.5	5.5 6.5		14 14.5	23 24

APPENDIX 5

WAVE TANK EXPERIMENT

SPACING OF RIP CURRENTS WITH VARIOUS WAVE HEIGHTS

Tin plate surface beach 35 ft long sloping at 6°

Wave period : 0.75 sec.

Wavw height ins.	Spacing of Rips (ft)	Mean (ft)
0.5	1.5, 3.5, 3, 3, 3, 2.75, 2.25	2.7
0.5	1.25, 3, 3.25, 2.25, 3, 4, 2, 3, 4.	2.8
0.6	2, 4, 2, 2, 2, 5, 5, 5, 3.	3.3
0.7	2, 3.5, 4, 3, 4, 2.5, 4, 3.5, 2.5.	3.3
0.8	4, 3.5, 3, 4, 3.5, 3.	3.5
0.9	2.75, 4.25, 7.75, 2, 2, 6, 2.	3.8
1.0	5, 3, 4, 5, 5, 4.	4.5
1.5	3, 3.5, 4, 3.5, 4, 3.5, 3.5, 5.	3.9
1.5	2.5, 3.5, 4, 5.	3.7
1.8	2.5, 3.5, 5, 3, 4, 5, 2, 5.	4.3
1.8	4, 4, 1.5, 5.5, 5, 5.	4.5
2.1	3.75, 4, 8, 4, 2.75, 4.25.	4.5
2.2	5, 6, 6, 5, 5, 3.5.	5.2
2.4	3.75, 4, 6.5.	4.75
2.8	6, 5, 6, 5, 8.	6

APPENDIX 6

WAVE TANK EXPERIMENT

THE CHANGE OF LONGSHORE CURRENT (L.S.C.) AND RIP VELOCITIES WITH WAVE HEIGHT

Date: 31.12.65

Conditions : Tin plate base at 6°. Period 0.75 sec.

Wave height ins.	1.8	1.2	0.5
L.S.C. Velocity		2.5	
Ins/Sec	3.75	2.4	1.85
	3.35	3.0	1.61
	3.2	2.8	2.0
	3.7	2.85	1.77
	3.75	2.65	2.1
	3.35		
Av. L.S.C. Vel. ins/sec	3.5	2.65	1.85
Rip Velocity		1.43	1.03
Ins/sec	2.4	1.20	1.5
	2.25	1.6	1
	3.1	1.92	0.8
	2.35	1.47	1.4
	2.5		1.0
	2.7		1.16
			1.35
Av. Rip Velocity	2.55	1.5	1.15

APPENDIX 7

WAVE TANK EXPERIMENT

CHANGES OF RIP- & LONGSHORE CURRENT VELOCITIES WITH WAVE HEIGHT

Date : 6.1.66.

Conditions: Moveable sand base. Initial slope 6°.

Period 0.75 sec.

Wave height, ins	0.4	0.65	1	1.6	2.8
Maximum uprush*	5	5	5	4	2
Minimum uprush*	7	7	6.5	6.5	5
S.W.L.*	8.5	8.5	8.5	8.5	8.5
Break point*	11.5	12	12.25	14	15
Vert. point*	17.5	18	19.25	20	24
L.S.C. vel.	1.9	1.7	1.9	1.9	2.8
Ins/Sec	1.8	1.7	1.9		1.9
	2.1	1.8	2.0	2	2
	2.1			2.1	2
				3.5	2
Aver. L.S.C. vel.	2	1.7	1.9	2.4	2.1
Rip vel.	1.3	1.3	0.9	1.9	2.5
Ins/Sec	1.2	1.6	0.5	1.4	1.0
	1.1	1.3			2.5
	1.4	.14			1.7
		1.3			1.3
Aver. rip vel.	1.25	1.4	0.7	1.7	1.8

Notes: (i) Cusps form and cause small rips (9" spacing).

(ii) Beach takes a considerable time to reach

APPENDIX 8

WAVE TANK EXPERIMENTS

Test No. 1 Raw dye concentration 1480 ppm.

(a) Concentration of dye near continuous release.

Time after release in seconds	Concentration ppm	Time after release in seconds	Concentration ppm
0	0.6	210	6.9
30	9.4	240	5.2
60	15.4	270	8.0
90	28.0	300	4.4
120	18.5	330	15.0
150	14.6		
180	5.2		

(b) Concentration at the shore

Time after release secs - 75	Concentration ppm	Time after release in seconds -75.	Concentration ppm
	0.06 (background)	160	6.8
0	0.58	170	7.4
10	0.12	180	6.2
20	0.52	190	7.2
30	1.0	200	6.2
40	1.1	210	7.4
50	3.6	220	8.2
60	2.2	230	12.2
70	5.2	240	8.0
80	7.5	250	9.6
90	8.8	260	10.0
100	6.7	270	8.6
110	7.5	280	10.4
120	7.0	290	8.4
130	7.8		
140	8.4		
150	9.6		

APPENDIX 9

Test No. 2. Raw dye concentration 1480 ppm released at rip base.

Time after release - 120 sec.	Concentration at the shore ppm	× 1.5**	Time after release sec -120	Concentration at the shore	× 1.5
Background	0.018	--			
0	0.026	(.039)	325	4.7	(7.0)
10	0.027	(.0395)	355	4.9	(7.35)
25	0.035	(.052)	385	3.3	(5)
55	0.26	(.39)	415	4.4	(6.6)
85	0.28	(.42)	445	4.3	(6.45)
115	0.38	(.57)	475	5.0	(7.5)
145	0.52	(.78)	505	4.8	(6.2)
175	1.2	(1.8)	In rip	24.6	(36.9)
205	2.0	(3)	In rip	22.0	(33)
235	2.3	(4.45)			
265	3.0	(4.5)			
295	3.4	(5.1)			

** Factor to compensate for lower discharge 1.64 compared to to 2.44 ccs/sec in Test 1. to make concentrations comparable.

APPENDIX 10

CALIBRATION OF WAVE RECORDER MARK 11

Purpose

The purpose of the test was to examine the response to pressure of the wave meter (Mark 11), which consisted essentially of a rubber air ring connected by hose to a mercury manometer and recorder.

Procedure

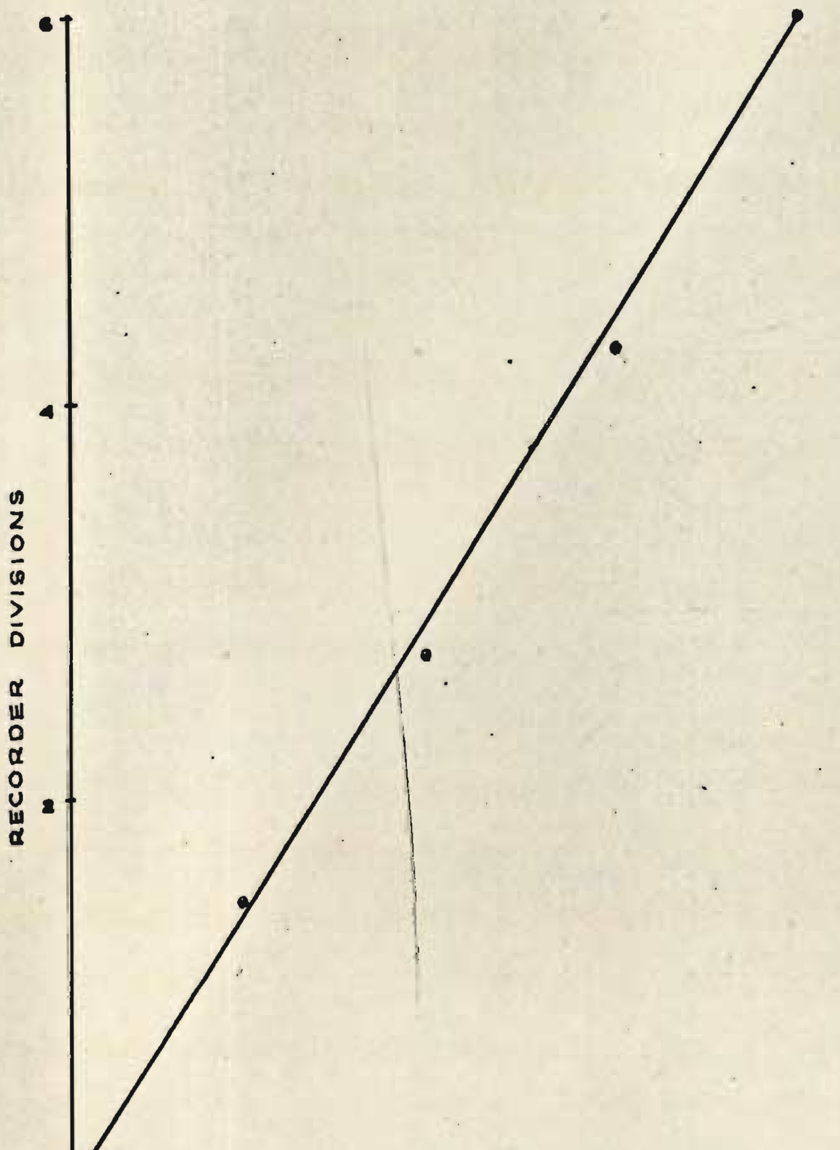
In the laboratory the rubber ring was immersed in a tank and pumped up to an arbitrary initial known pressure. It was then subjected to additional known hydrostatic loads.

The corresponding manometer readings were recorded on a moving paper clockwork recorder.

Results

The relationship between the depth of water over the ring and the number of small divisions on the recording paper are shown in the attached graph. The response was reasonably linear over the range tested.

When used in the field the response to a wave was correlated with its height as measured by lining up the wave crest, the horizon and a mark on a graduated pole held at the water's edge. (mean uprush position).



APPENDIX.10 CALIBRATION OF MARK II WAVE RECORDER

ONE RECORDER DIVISION = 0·3 FEET

APPENDIX 11

REPRODUCIBILITY TEST ON TRACER DILUTION METHOD

WEDNESDAY, 11TH AUGUST, 1965 ; VIRGINIA BEACH.

Object

To investigate the reproducibility of the tracer dilution method as applied to the longshore current in the surf zone.

Procedure

On four successive occasions 1 lb of sodium fluorescein was released into the surf and sampled at a station 375 ft. downstream. The samples were analysed in a Turner Fluorimeter. Sampling was at the rate of every 15 secs. for tests 1 and 2, and every 10 secs. for stations 3 and 4.

Conditions

Wave height at sandbar 3-4 ft.

Wave height at the foreshore 2-3 ft.

Period 6 seconds.

Wave angle 10-15° from North East.

Wind light North East.

Width of longshore current approximately 82' .

Depth at 82' offshore 10'.

Area of cross section of current 450 sq. ft.

Results

The results of the analysis are set out in the following Table.

APPENDIX 11
(continued)

WEDNESDAY, 11TH AUGUST, 1965.

Four 1 lb batches of dye released 375 ft. from sampling point

Sample No.	Conc. ppm	Sample No.	Conc. ppm	Sample No.	Conc.ppm
Test 1		Test 3			
2	0.000	28	0.000	79	0.108
3	0.068	29	0.125	82	0.074
4	0.76	30	0.4	83	0.053
5	0.97	31	0.76	100	0.025
7	0.55	32	0.69	117	0.021
		33	0.62		
8	0.39	34	0.48		
9	0.385	35	0.355		
10	0.322	37	0.29		
11	0.057	38	0.23		
12	0.030	39	0.17		
13	0.0128	40	0.15		
14	0.0073	41	0.051		
15	0.0061	53	0.044		
16	0.0061	52	0.024		
		19	0.020		
		62	0.0091		
Test 2		63	0.0092		
90	0.000				
92	0.081	Test 4			
93	0.385				
94	1.52	50	0.000		
95	0.52	51	0.17		
96	0.322	60	0.69		
97	0.10	61	0.69		
98	0.023	67	0.62		
99	0.255	69	0.97		
101	0.031	70	0.97		
102	0.0097	71	0.76		
103	0.0085	74	0.48		
104	0.0091	78	0.126		

Note: For Tests 1 and 2 sampling done every 15 secs.

For Tests 3 and 4, every 10 secs.

APPENDIX 12

Records of the analyses made of the samples taken at stations during the alongshore system tests A.1 - A.5.

Note figures in brackets after each station refer to its distance (in feet) from the point at which the tracer was released.

APPENDIX 12

ALONGSHORE SYSTEM TESTS

ANALYSES OF SAMPLES

TEST A.1 (2.6.61.)

STATION I (500 ft.)		STATION II (1,000 ft.)		STATION III (2,000 ft)	
Minutes after Release	Fluorescein Concentration in ppm.	Minutes after Release	Fluorescein Concentration in ppm.	Minutes after Release	Fluorescein Concentration in ppm.
3	0.029	6	0.041		
3½	2.53	6½	0.41	10	0.000
4	2.98	7	2.28	11	0.14
4½	2.18	7½	2.78	12	0.42
5	0.47	8	1.38	13	1.08
5½	0.037	8½	0.33	14	0.45
6	0.0165	9	0.076	15	0.23
6½	0.015	9½	0.069	16	0.068
7	0.0045	10	0.08	17	0.055
7½	0.002	10½	0.034	18	0.033
8	0.003	11	0.039	20	0.022
8½	0.018	11½	0.031	22	0.036
9	0.007	12	0.009	24	0.00
9½	0.0015	12½	0.010	26	0.003
10	0.0015	13	0.0005	28	0.001
10½	0.00	14	0.000		

STATION IV (3,500 ft)		STATION V (5,000 ft)		STATION VI (6,000 ft)	
17	0.000	25	0.000	30	0.000
18	0.185	27	0.044	31	0.009
19	0.23	28	0.049	32	0.017
20	0.20	29	0.037	33	0.035
21	0.16	30	0.068	34	0.062
22	0.14	31	0.062	35	0.034
24	0.105	32	0.058	36	0.027
26	0.085	33	0.056	37	0.051
28	0.039	34½	0.052	40	0.047
30	0.037	36	0.049	43	0.037
32	0.032	37½	0.04	46	0.039
35	0.011	39	0.0405	49	0.024
40	0.01	41	0.035	52	
45	0.011	43	0.029	55	0.025
		45	0.021	58	
		48	0.018	61	0.0105
		51	0.012	64	
		54		67	0.007
		57	0.011	70	
		60		73	0.006
		63	0.007	76	

NOTE: Distance of sampling points from the release points are given after the station number.

APPENDIX 13

ANALYSIS OF SAMPLES

TEST A.2 11.7.62.

Station	Distance from point of tracer release - ft.	Concentration of 10 micron material - ppm.
1	300	1,320
2	900	660
3	1,500	335
4	2,100	184
5	2,700	134
6	3,450	85
7	3,900	74

APPENDIX 14

ANALYSIS OF SAMPLES

TEST A.3 27.8.64

STATION I (360 ft.)		STATION II (780 ft.)	
Time after release - seconds	Fluorescein Concentration ppm	Time after release - seconds	Fluorescein Concentration ppm
180	0.87	300	0.300
195	1.35	315	0.480
210	0.71	330	0.545
225	0.22	360	0.270
240	0.14	405	0.016
255	0.04	420	0.020
270	0.02	435	0.055
285	0.018	450	0.055
300	0.016	465	0.010
315	0.010		
330	0.007		
340	0.005		

APPENDIX 15

ANALYSIS OF SAMPLES

TEST A. 4 14.7.65.

STATION I (240 ft)

Time after release - minutes	Fluorescein Concentration ppm.
2.0	0.0966
2.25	0.875
2.5	0.750
2.75	0.625
3.0	0.066
3.25	0.086
3.5	0.0165

STATION II (660 ft)

Time after release - minutes	Fluorescein Concentration pm.
5	0.0217
5.25	0.0852
5.5	0.166
5.75	0.29
6.0	0.216
7.0	1.00
8.0	0.093
9.0	0.0014
10.0	0.0003
11.0	0.0003

STATION III (960 ft)

9.0	0.0006
9.25	0.1166
9.5	0.045
9.75	0.058
10.0	0.047
10.5	0.1033
11	0.093
11.5	0.083
12	0.100
12.5	0.130
13	0.05
13.5	0.06
14	0.0189
14.5	0.023
15	0.0008

STATION IV (1,272 ft)

12	0.0079
13	0.0429
13.5	0.0711
14	0.065
14.5	0.10
15	0.073
15.5	0.066
16	0.06
16.5	0.0556
17	0.0466
17.5	0.0177
18	0.0087
20	0.0008

STATION V (1,852 ft)

18	0.0006
18.5	0.0214
19	0.0088
20	0.0405
21	0.0264
22	0.0276
23	0.0288
24	0.0370
25	0.0441
26	0.0429
27	0.0169
28	0.0617
29	0.0148
30	0.0088
31	0.0060
32	0.0074
33	0.0008
34	[illegible]

APPENDIX 16

ANALYSIS OF SAMPLES

TEST A.5 2.3.60.

STATION I (250 ft.)

Time after release - seconds	Concentration ppm
0	0.00
15	16.4
30	0.29
45	0.27
60	0.05
75	0.15
90	0.01
180	0.00
STATION II (1,000 ft.)	
270	0.24
300	0.30
315	0.50
330	0.39
390	0.28
420	0.10
450	0.075
690	0.00
STATION III (2,750 ft.)	
915	0.00
930	0.08
960	0.085
990	0.12
1020	0.14
1080	0.005*
1102	0.12
1140	0.19
1185	0.005
	*rejected