BEACH DEVELOPMENT AND MOVEMENT AS A FUNCTION OF WATER WAVES

Raymond Louis Menné

A thesis submitted in partial fulfilment
of the requirements for the degree
Master of Science in Engineering

Department of Civil Engineering
University of Cape Town

September 1978

DECLARATION OF CANDIDATE

I, Raymond Louis Menné declare that this thesis is my own work and that it has not been submitted for a degree at another University.

Signed by candidate

R.L. Menné

ABSTRACT

This thesis consists of three parts. In Part One a literature study is made of available knowledge regarding wave theory and sediment transport in the nearshore zone, with special emphasis on littoral drift. The linear Airy theory for water waves is developed for the computation of longshore currents, utilizing certain exact relationships for momentum flux. The nearshore beach environment is discussed in detail with regard to the wave forces acting on it, beach forms, and three-dimensional circulation patterns.

Part Two deals with experiments conducted in the hydraulics laboratory, Department of Civil Engineering, University of Cape Town, using the existing model wave basin. The experimental programme covers the development of equilibrium beach profiles as a function of certain wave characteristics, and seeks the relationship between different profiles in terms of the wave characteristics that formed them. Both normal and oblique wave attack programmes are reported on, the latter in terms of existing known relationships for littoral drift.

Part Three deals with experiments conducted in the field off the South West African coast. Beach profile changes are compared to the local wave climate and relationships are sought between these changes and certain wave characteristics. A fluorescent tracer study on the swash zone is reported on as a method of gauging littoral drift.

ACKNOWLEDGEMENTS

The author gratefully acknowledges the help and guidance given to him by his supervisor, Professor F.A. Kilner. To the following colleagues and fellow students, the author expresses his appreciation for their assistance with the many facets of the laboratory and field investigations:

P. Silbernagl

A.H. Stewart

B.J. Nicol

D.L.F. Gow

E. de Swart

P. Calothi

Finally, the author would like to thank his wife, Glynis, for her patience over a period of some considerable time.

LIST OF SYMBOLS

C	Chezy roughness coefficient ($m^{\frac{1}{2}}/s$)
D	Grain diameter (mm)
D_m	Mean grain diameter (mm)
E	Wave energy density (J/m^2 of plan area)
F	Total wave thrust in the longshore direction (N/m parallel to shore)
H	Wave height (m)
H^1	Wave height unaffected by refraction (m)
H_{rms}	Average wave height in a wave spectrum-root mean square (m)
H_{sig}	Significant wave height measured prior to breaking (m)
H/L	Wave steepness (ratio)
I_ℓ	Immersed weight transport rate (N/s)
K_r	Refraction coefficient (dimensionless)
L	Wave length (m)
P	Wave power or energy flow (J/s/m crest)
$P_a (= P_\ell)$	Longshore component of wave power (J/s/m length parallel to shore
P_x	On-shore component of wave power (J/s/m crest)
Q_s	Sand volume transport rate (m^3/s)
S	Shorewards flow of momentum (N/m)
S_{xx}	Momentum flow in on-shore direction (N/m)
S_{xy}	Momentum flow in longshore direction (N/m parallel to shore)
T	Wave period (s)
V	Longshore current velocity (m/s)
V_{max}	Maximum longshore current velocity (m/s)
$\overline{V}_s$	Immersed weight of mobile sand (N)

a'	Sand volume concentration
c	Wave celerity (m/s)
c_g	Wave group velocity (m/s)
d	Undisturbed or still water depth (m)
g	Gravitational acceleration (m/s^2)
i	Beach gradient (ratio)
n	Wave transmission coefficient (dimensionless)
u_{max}	Maximum water particle orbital velocity (m/s)
v_s	Terminal particle settling velocity (m/s)
w	Specific weight (N/m^3)
θ	Angle of wave crest to shoreline (degrees)
ϕ	Phi grain size classification ($= -\log_2 D$)
ρ	Density of water (kg/m^3)
ρ_s	Density of sand (kg/m^3)
μ_e	Horizontal eddy velocity (m/s)
$_o$(subscript)	Deep water wave condition
$_b$(subscript)	Breaking wave condition

LIST OF CONTENTS

P A R T   O N E

LITERATURE SURVEY

CHAPTER 1

WAVE THEORY

1.1 WAVE THEORIES

One of the earliest investigations into the nature of oscillatory wave motion was developed independently by Gerstner (1802) and Rankine (1863). Both investigators developed their theories on the assumption that the wave profile was exactly trochoidal in shape. Furthermore, mass transport was not allowed for, as the orbits of the particles were said to be exactly circular (in deep water).

In 1845 Airy presented a theory for oscillatory waves of small amplitude, and Stokes (1847) extended the theory and developed an approximate solution for waves of finite amplitude. Stokes' theory required both irrotational flow and mass transport, and was thus closer to reality than the simpler theory of Gerstner and Rankine. While the form of the waves in Airy's analysis was sinusoidal; for waves of finite height the form approximates to a trochoid.

At present time there exist over a dozen oscillatory wave theories, mostly analytical, some with empirical modifications and still others numerical. Various interpretations of the linear Airy theory have been made by Wiegel (1964) and Biesel (1952) together with an empirical modification of the Airy theory by Goda (1964). Stokes' wave theory has been developed to the second, third and fifth order of approximation by Wiegel (1964), Skjelbreia (1959) and Skjelbreia and Hendrickson (1962) respectively.

In 1949 Munk proposed the validity of the translatory solitary wave theory (developed by Boussinesq (1872) and McGowan (1891) respectively) for application in shallow water. Korteweg and de Vries (1895) proposed a new type of wave after a mathematical study of long waves. The theory was developed into the cnoidal wave theory of Keulegan and Patterson (1940) and the cnoidal theory of Laitone to the first and second order of approximation. Finally, of the numerical methods mention shall only be made of the stream function wave theory of Dean (1965), and Chappelear (1961). Numerical methods have only been feasible since the development of the high speed computer, although tabulation of parameters for numerical solution could eliminate the necessity of obtaining every solution.

Only general guidelines pertaining to the relative depth conditions for which a particular theory was developed, are available. Various investigators, Le-Méhauté et al (1968) and Dean (1970), have attempted to establish the relative validity of the wave theories under different shallow water and shoaling conditions. Dean reported that the linear Airy and first order cnoidal wave theories provide best fits for shallow water waves; Le-Méhauté also concluded that the linear theory was surprisingly good in shallow water for the shorter waves tested, while the best results were obtained from the cnoidal theory of Keulegan and Patterson. Both investigators stated that none of the theories were in exceptional agreement with the data.

It would thus seem that considerable investigation is still required in the field of wave motion, and an improved theory applicable under all conditions has yet to be developed. Further mathematical refinements of the existing theories would appear to be of little practical interest. Le-Méhauté notes that a rotational theory at a second order of approximation, with an arbitrary vorticity distribution (and, consequently, mass transport) remains to be established. This theory should take into account not only the viscous effect due to the bottom boundary layer, but also the effect of wind shearing stresses at the free surface.

Under the circumstances, it should be noted that knowledge concerning the sediment carrying capacity of waves in shallow water has suffered from the lack of information available on the basic mechanics of wave motion. The subject has thus become largely an empirical one, with no universal solution yet available.

The accuracy to which a theory (or theories) can be applied in practice relies on the accuracy of the observed wave parameters on which it is based. Accurate field data and to a lesser extent, laboratory data, requires extensive and expensive instrumentation. Such instrumentation was not available for this project. There must consequently be a considerable overlap in the commonly defined regimes of deep, intermediate and shallow water wave conditions. For the purposes of this project primary use of the linear wave theory has been applied; use of other wave theories or empirical modifications have been listed where used.

1.2 OCEAN WAVES

Water waves may be generated by any disturbance which affects the local velocity, surface elevation or pressure. In nature, waves are most commonly generated by the action of wind, but can also be a result of tidal action, seismic disturbances and barometric-pressure fluctuations. Once wave action has started, it is controlled primarily by gravitational forces.

Ocean waves get energy from the wind by the direct push of the wind on their upwind faces if they are running slower than the wind; the frictional drag of the air on the sea surface; and from pressure disturbances in the air above them. The growth of wind waves is thus a function of wind speed, direction and fetch on deep water, viscous forces are one of the main factors leading to the gradual dampening of waves or dissipation of wave energy. However, as long as the energy income of the waves from all sources is larger than the energy dissipated (by whatever means) the waves will continue to grow in wave height and wave length, forming combinations of harmonic oscillatory wave trains. Once free of the 'sea' that generated them, the wave trains can be propagated over vast distances, the energy losses being very low when travelling as swell.

A number of theories concerning the generation of wind waves have been put forward. As the transfer of energy from wind to waves, and the resulting wave growth, is not yet completely understood, the theories are of necessity semi-theoretical or semi-empirical. Less information is available on wind waves in shallow water than for deep water with regard to both theory and available data.

1.3 WAVE SPECTRA

Waves in the open sea are more complex in form than ideal waves as covered by the wave theories. In the open ocean there are nearly always a larger number of waves superimposed on each other, especially in the generating area. Because of the very nature of the fluctuations in their generating process, and the interaction of different sets of varying heights travelling at different speeds and in different directions, these waves are characterised by their irregularity and short-crested form. The transformation of these irregular waves in shoaling water, add further complications to an already complex process.

As there are, therefore, nearly always a large number of different wave heights and periods present at any one time at a certain location, the waves have to be interpreted in terms of a wave climate, or wave spectrum. The interpretation of the results of wave measurements depend on the development of statistical theory, illustrated by that as developed by Tukey and Longuet-Higgins (1952). Time series analysis, used in conjunction with other statistical methods establishes the wave spectra from which data concerning the significant heights, periods and power of waves can be obtained. The significant wave height has been defined as the mean or average of the highest one third of the waves under consideration, while the significant wave period is the period associated with this wave height. The maximum wave energy is concentrated around this period. (It was found from the analysis of wave records that the significant wave height was nearly equal to that height reported from visual observations) (Ippen 1963).

Recent wave studies have adopted the use of a more representative measure of the average conditions in a wave spectrum, known as the 'root mean square' term. In terms of the rate of energy transmission over a certain time period for a certain wave spectra, the latter term has been shown to give a better indication of the mean value. Longuet-Higgins (1952) has developed the following relationship between the significant wave height and the root mean square height.

$$H_{rms} = 0{,}71\ H_{sig}$$

Visual observation of the significant wave values, however, still serve to give the engineer a relatively good picture of irregular waves (Siefert 1970). For construction planning, naturally, characteristics such as mean, significant and maximum wave heights and periods are more useful than spectrum analysis.

1.4 DEVELOPMENT OF THE LINEAR WAVE THEORY

The limiting conditions for the linear theory are ill defined. The theory is based on the premise that motions are sufficiently small to allow the free surface boundary conditions to be linearised, in particular, terms involving the wave amplitude to the second and higher orders are considered negligible. The assumption is thus that the parameters of wave height to wave length and wave height to water depth are relatively small compared to unity.

If the assumptions of the linear theory are met the theory can be extended so as to be valid for all ranges of the relative water depth, defined as the ratio of water depth to wave length (d/L). Based on the Oceanographers definitions, as reported by Kinsman (1962), the limits of the three wave regimes are as follows:

Deep water	$d/L \geq 0{,}25$
Intermediate water	$0{,}25 > d/L > 0{,}05$
Shallow water	$d/L \leq 0{,}05$

These limits compare favourably with those as reported by Wiegel and Ippen, amongst others.

1.4.1 Wave velocity, wave length and period

For a simple harmonic wave train the period is independent of the water depth, hence

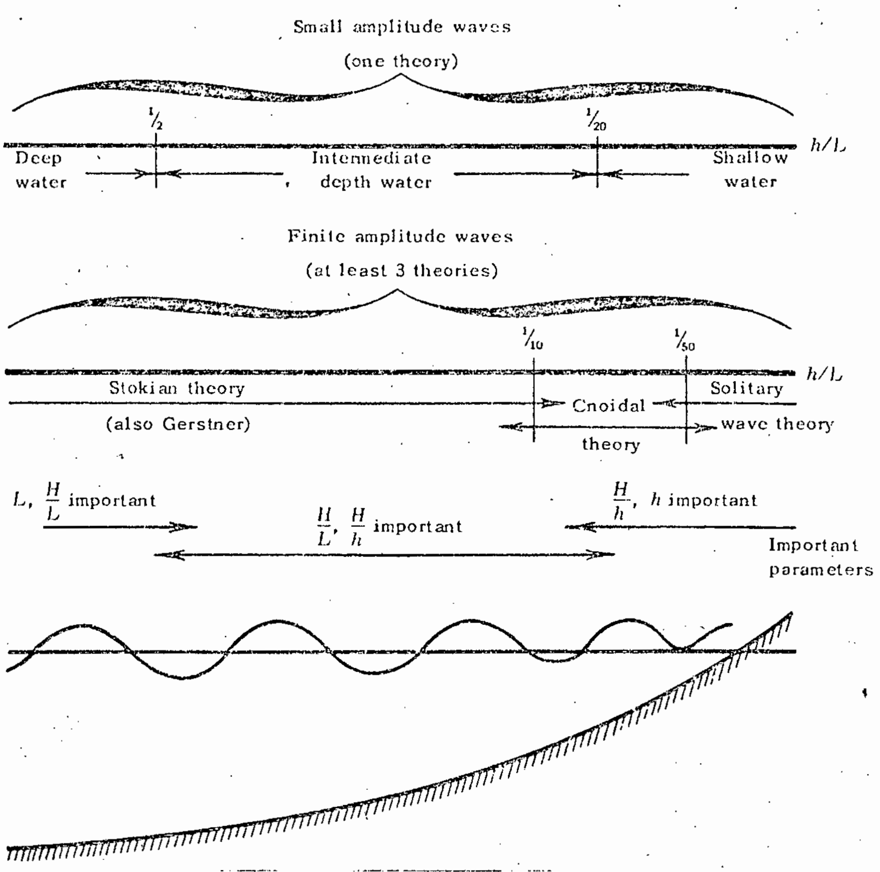


Figure 1 Classification of wave theories (after Eagleson and Dean 1963)

$$c_o = L/T \tag{1}$$

The linear wave theory expression for wave velocity is

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

Under deep water wave conditions the function tanh $2\pi d/L$ tends to unity. Thus,

$$c_o^2 = gL_o/2\pi \tag{3}$$

In shallow water the function tanh $2\pi d/L$ tends to $2\pi d/L$, yielding

$$c^2 = gd \tag{4}$$

The purpose in defining the three wave regimes becomes immediately apparent, in that the expression for wave velocity becomes simplified under deep or shallow water conditions. (The choice of the limits between the three regimes was based on the variation of the tanh function with depth, compared with the accuracy to which the wave parameters can practicably be gauged).

The expression for the wave length in terms of the wave period and water depth is

$$L = \frac{gT^2}{2\pi} \tanh \frac{2\pi d}{L} \tag{5}$$

For deep water this becomes

$$c_o = gT/2\pi \tag{6}$$

Manipulation of equations (2) and (3), and (5) and (6), leads to the following expression

$$(c/c_o)^2 = L/L_o \tanh \frac{2\pi d}{L} \tag{7}$$

If, as it is assumed, the period is an independent variable this expression is very useful. Once the deep water wave lengths and velocities have been obtained for a given wave period (equations (3) and (6)), values of d/L can be obtained as a function of even increments of d/L_o by successive approximations using the following expression

$$d/L_o = \frac{d}{L} \tanh \frac{2\pi d}{L} \tag{8}$$

This function together with various others has been tabulated by Wiegel (1954).

1.4.2 Wave energy and rate of energy transmission

The total energy of a wave system is divided into the potential and kinetic energy components and is generally expressed in terms of the energy per unit surface area (averaged over a wave length). This is also known as the energy density, and the energy in this form is found by determining the total energy (potential plus kinetic) in one wave length and in a unit width and then dividing this quantity by the wavelength. In small amplitude cases the potential and kinetic energy of a wave are always equal, and for a simple harmonic progressive wave their sum may be expressed as

$$E = wH^2/8 \tag{9}$$

where E is the average energy density per unit surface area.

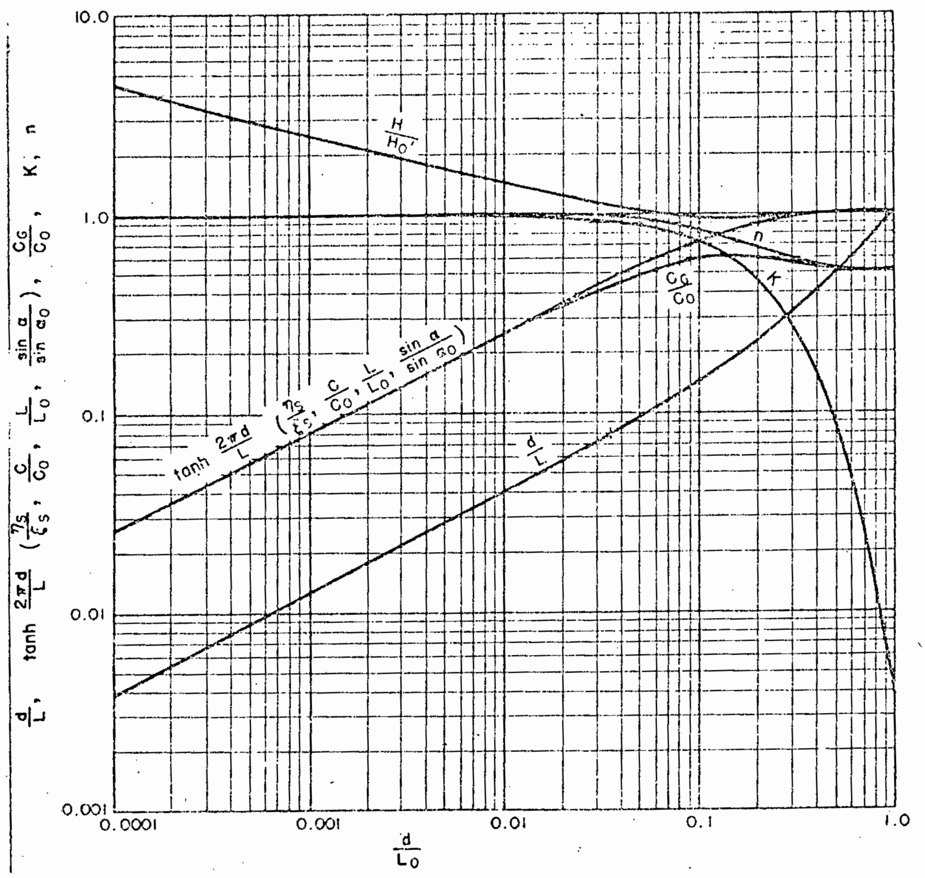


Figure 2 Progressive wave characteristics in transitional and shallow water relative to deep water linear theory (after Wiegel 1964)

Noting that for oscillatory waves the kinetic energy is retained at any location while the potential energy is transferred forward at the same speed as the velocity of individual waves, it is clear that energy is transported in the direction of wave propagation but with a velocity less than that of individual waves. The wave energy travels forward with a speed known as the group velocity, i.e.

$$c_g = n\, c \tag{10}$$

where n is known as the transmission coefficient, which depends on the relative depth, and is expressed as

$$n = 0{,}5 + \frac{2\pi d/L}{\sinh 4\pi d/L} \tag{11}$$

For deep sea conditions n tends to 0,5. In shallow water n tends to unity.

The average rate of energy transmission across a section, also known as energy flux or simply wave power, may be shown to be equal to the energy density multiplied by the group velocity per unit length of crest,

$$P = E c_g = w H^2 n c/8 \qquad (12)$$

Expressed in terms of its deep water values equation (12) becomes

$$P_o = w H_o^2 c_o/16 \qquad (13)$$

1.4.3 <u>Waves in shoaling water</u>

When waves travel into shoaling water a number of modifications take place. Taking first the simple case of a train of uniform waves approaching parallel to a coast where all the underwater contours are parallel straight lines, a platform for the more complex situations as found in nature may be developed.

The energy flow across any vertical plane parallel to the beach is given by equation (12). Unless energy is dissipated or stored, this energy flow remains constant. Measurements certainly indicate energy dissipation as minor until the wave begins to break. Outside the breaker line thus, wave height changes are governed by the consistency of the product H^2c_g, and equal to its deep water value. As the wave shoals, the group velocity first increases to a maximum value and then decreases steadily. The wave height thus decreases initially (to a value of approximately 0,91 H_o) and then steadily rises until the wave breaks. As the breaking wave moves in-shore it is usually assumed that the wave height remains proportional to the local water depth (the constant of proportionality being approximately 0,8).

Associated also with the orbital velocities is the shoreward flow of momentum, also referred to as a radiation stress. The following expression is exact and independent of Airy wave theory, and is valid from the refraction line to the shoreline.

$$S = w H^2 (4c_g/c - 1)/16 \qquad (14)$$

(N per metre length parallel to the shore)

The bracketed factor in equation (14) varies from 1 in deep water to 3 in shallow water. Calculations indicate that S increases steadily from its deep water value of $w H_o^2/16$ until the breaker line is reached, where S has a maximum value. From this line shorewards, the momentum flow reduces due to the reduction of wave height in the surf zone.

Between the refraction line and the breaker line there is a net outflow of shorewards momentum. In this zone the mean sea level is thus drawn down with the maximum effect occuring just before the breaker line. This is referred to as 'wave set down'. Between the breaker and the shoreline, there is a net inflow of shorewards momentum and this is balanced by a rising mean sea level, the maximum effect being achieved at the shoreline.

Figure 3 illustrates the changes in shoaling water of water height, energy and momentum flow, and wave set up and set down. The increased water depths associated with wave set up are of interest in various ways, i.e. set up has considerable bearing on longshore flows which lead to rip currents. If for any reason the wave height off-shore is not uniform along a line parallel to the shore, the resulting wave set up will vary significantly (set up is approximately proportional to wave height squared). The gradient of mean sea level in the longshore direction could thus generate longshore flows.

1.4.4 The effect of obliquity on waves in shoaling water

The effect of obliquity on waves in shoaling water is illustrated for the simple case of a beach for which the shoreline and all underwater contours are parallel straight lines by Figure 4. A train of uniform waves approaching such a beach has an angle between the crest lines and the bed contour of θ_o in deep water outside the refraction line and a general angle θ inside the refraction line.

In the refraction zone the wave crests are deflected towards the shoreline according to Snell's law, which, in the absence of currents, may be expressed as

$$(\sin\theta)/c \quad = \quad \text{constant} \quad = \quad (\sin\theta_o)/c_o \qquad (15)$$

Thus $\sin\theta$ (and θ) reduces as c reduces and a shoaling oblique wave swings more closely parallel to the shore as the wave moves shorewards. The changing angle of incidence causes the wave height to be generally smaller than would occur for a parallel wave approach. Inside the breaker zone, the similarity law of wave height proportional to local depth is again assumed.

Due to the incidence of the wave front, there are two effective components of momentum flow. If the axes normal and parallel to the shoreline are labelled x and y respectively, the two components may be given the symbols S_{xx} and S_{xy}. The first component produces wave set up and set down as in the parallel wave crest case, but the second component produces a longshore thrust on the water in the surf zone which may only be balanced by a longshore frictional reaction. (This in turn must be associated with longshore currents and sediment movements).

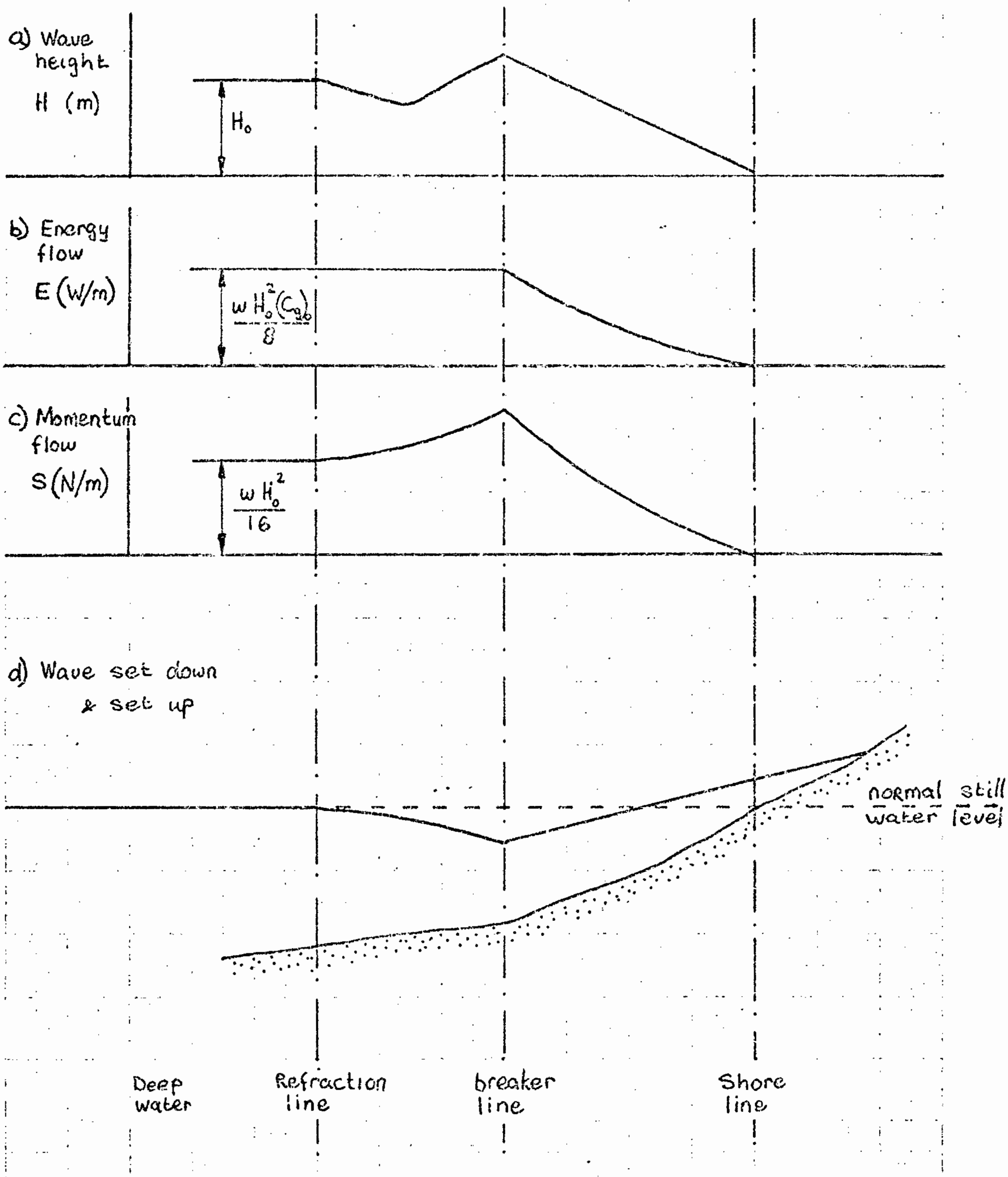


Figure 3 Illustrating changes in shoaling water

a) Wave height

b) Energy flow

c) Momentum flow

d) Actual profile of the mean sea level

(the horizontal axes on the above figures represent distances from the shoreline).

Outside the breaker zone the energy flux towards the shoreline in deep water is given as

$$P_x = \text{constant} = (E_o\ c_o \cos \theta_o)/2 \qquad (16)$$

The fluxes of energy and momentum have been shown to be related by the following relationship

$$S_{xy} = (\sin \theta/c)P_x \qquad (17)$$

Both the expressions on the right hand side of equation (17) are constant outside the breaker zone. Hence everywhere outside the breaker zone the wave exert a lateral thrust on the water and sediment inside the breaker zone. By manipulation of equation (17) in terms of equations (10) and (16) the relationship becomes

$$S_{xy} = E_o\ (c_g/c_o) \cos \theta_o \sin \theta_o \qquad (18)$$

$$= E_o\ (\sin 2\theta_o)/4 \qquad (19)$$

$$= w\ H_o^2\ (\sin 2\theta_o)/32 \qquad (20)$$

The value of S_{xy} is constant outside the breakers and reduces steadily in the surf zone as the wave energy is dissipated.

The momentum flow value in the on-shore direction is given as

$$S_{xx} = \frac{wH^2}{16}\left[(4\ c_g/c - 1) - (2\ c_g \sin^2 \theta)/c\right] \qquad (21)$$

(For $\theta = 0°$ the expression reduces to equation (14)).

The effect of the theta term is to reduce the values of S_{xx} by comparison to the parallel case, and thus reduce the magnitudes of wave set-up and down.

The pioneers in littoral drift studies developed a relationship very similar to equation (18), viz

$$P_a = P_\ell = E\ c_g \cos \theta \sin \theta \qquad (22)$$

P_a was termed a variety of names, amongst other, 'longshore power'. Longuet Higgins (1971) describes this quantity as having no obvious physical meaning, and advocates that P_a be replaced by the product S_{xy} c, both of

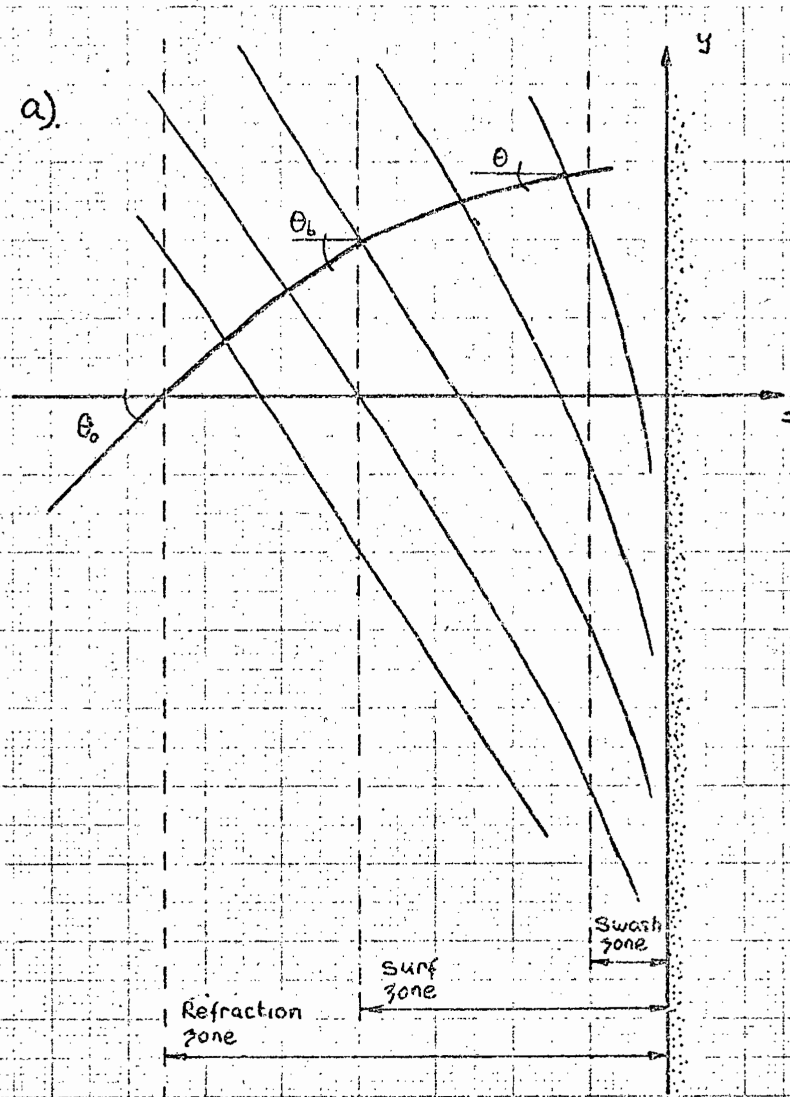


Figure 4a   Schematic view of waves approaching a straight coast-line

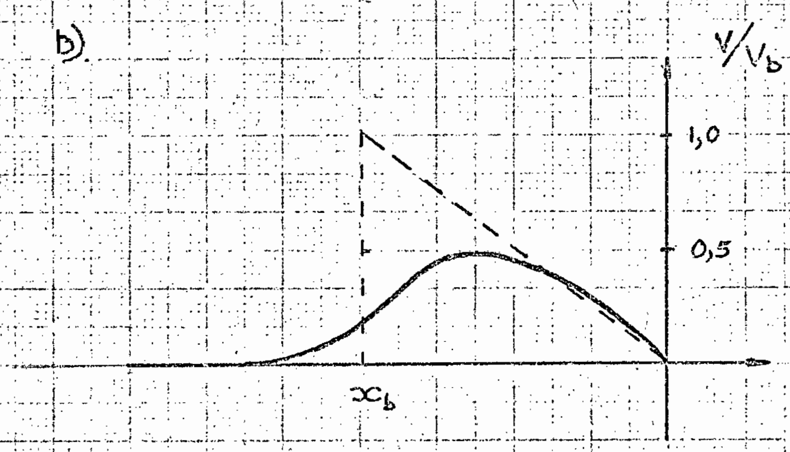


Figure 4b   Theoretical longshore velocity profile with no lateral mixing (dotted line), and with lateral mixing (full curve) (after Longuet-Higgins - 1971)

physical significance and to which P_a is formally equal.

The author feels that the derivation and form of the P_a expressions are worthy of brief note, as these expressions formed the basis of most littoral drift studies in the past. Up until the early 1970's, measured littoral drift rates were commonly equated to the P_a expressions.

The derivation of P_a assumed that the wave power transmitted between wave rays or orthogonals approaching the breaker zone remained constant and equal to its deep water value. The wave power at the breaker line was then expressed in terms of its longshore component, while the distance between the chosen pair of orthogonals was expressed in terms of a unit length of the shoreline. The product of these two expressions was termed the longshore power (or energy flux) per unit length of shoreline, and took the following form:

$$P_a = K_r^2 P_o \sin \theta_b \cos \theta_b \qquad (22a)$$

or

$$P_a = (K_r^2 w H_o^2 c_o \sin \theta_b \cos \theta_b)/16 \qquad (22b)$$

where K_r is the refraction coefficient (equal to $(\cos \theta_o/\cos \theta_b)^{\frac{1}{2}}$). Investigators often dropped the refraction coefficient by making use of an expression for the wave height, H_o^1, termed the deep water wave height unaffected by refraction, viz

$$P_a = (w(H_o^1)^2 c_o \sin \theta_b \cos \theta_b)/16 \qquad (22c)$$

or

$$P_a = (w(H_o^1)^2 c_o \sin 2 \theta_b)/32 \qquad (22d)$$

Certain investigators have acknowledged the difficulty in accurately measuring the breaker angle by minimising its effect, thus:

$$P_a = (w(H_o^1)^2 c_b \sin \theta_o \cos \theta_b)/16 \qquad (22e)$$

1.4.5 Breaking waves

Longuet Higgins (1971) reports that no fully satisfactory theory of breaking waves is yet available. Any non-breaking wave theory, such as the linear theory, is inadequate to describe violently plunging or surging surf conditions. Even the cnoidal theory only provides a description of periodic waves in moderately shallow water with gently spilling breakers.

Laboratory studies on models indicate that the two primary factors influencing the breaker characteristics are the wave steepness and beach slope functions. Several investigators have concluded that the theoretical limiting case of stability occurs when the local wave steepness becomes 0,143. Breakers are, however, more commonly encountered as waves move towards beaches where the relevant criterion is the ratio of water depth to wave height. Examination of available field data by Munk (1949) indicated that a curve derived on the basis of the theory of solitary waves compared favourably with the recorded data. Included in this theory was the concept that the ratio of the depth at breaking to the breaker height (d_b/H_b) is 1,28. This value agrees reasonably well with the average of all observations, but has been described by Komar and Gaughan (1972) as 'fortuitous'. The writers proposed the following relationship

$$H_b = 0{,}39\ g^{\frac{1}{5}}(T(H_o^1)^2)^{\frac{2}{5}} \qquad (23)$$

Komar and Gaughan noted that whereas the ratio of breaker depth to breaker height is known to vary with beach slope, as only a one-fifth power of the ratio is involved, the expected variations should not produce a marked change in the line slope of equation (23).

Guide values for the on-set of wave break are d_b/H_b less than 1,25 for a beach slope of 1 in 50 to about 0,9 for steep slopes of 1 in 10.

CHAPTER 2

WAVE CURRENTS

2.1 CLASSIFICATION OF BEACHES

Technically the shore is that strip of ground bordering any body of water which is alternately exposed, or covered by tides and/or waves (Wiegel - 1964). A shore of unconsolidated material is called a beach. Beaches are continually changing and at best are in dynamic equilibrium with sediment moving on-shore and off-shore, as well as longshore.

Beaches may be classified on genetic and evolutionary grounds, descriptively with respect to their constituent materials or a combined exposure and descriptive basis. The last mentioned classification is composed of the combination of a wave size characteristic, type of beach orientation, and size of beach materials. Thus a possible beach classification is 'straight medium sand beach on the South Atlantic'.

The most probable sources of beach material may be classified according to their origin, be it terrigenous, biogenous, hydrogenous or volcanic. Terrigenous, one of the most common sources can be further subdivided into material carried on to the beach by rivers or glaciers, cliff erosion and slides, or erosion of the beach itself, the on-shore movement of sand by wave action, and wind. The degree to which these factors (as well as any others) may be important on a certain reach of a shoreline is naturally relative to the local conditions. It remains convenient, however, to confine a study of a particular shoreline to a physiographic unit, whenever possible. Generally, the boundaries of physiographic units consist of such features as prominent headlands, man made littoral barriers, submarine canyons, or any other shoreline features which prevent the movement of sediment into and out of the shore area under consideration. That this is desirable is manifest in the observation that not only sediment size and density, but also sediment shape, affect the dynamic process of transport in or by the sea.

2.2 OCEAN CURRENTS

Johnson and Eagleson (1966) define ocean currents as fluid velocities which may be treated analytically as steady state phenomena. They may be periodic or even reversing, but the period of the motion is far greater than that of wind waves. The writers have divided ocean currents into the following groups:

a) Currents related to the distribution of density in the sea
b) Currents caused directly by the stress that the wind exerts on the sea
c) Tidal currents and currents associated with internal waves
d) Currents and transport induced by surface gravity waves
e) Local currents induced by fresh water entering the ocean at river mouths

All these systems are available for the mass transport of sediments in all the ways that fresh water streams are known to move sediments, whether suspended or bed load, or inbetween.

2.3 WAVE INDUCED CURRENTS AND PARTICLE MOTION

The shoaling effects of oscillatory water waves have already been considered with regard to the general wave characteristics. The shoaling process must now be considered with respect to the changes in individual water particle motions.

Application of the modified linear theory as well as finite amplitude wave theories have their origins in the existence of a mass transport of individual water particles in the direction of wave propagation not accounted for in the classical wave theory. Classical wave theory predicted horizontal and vertical particle motions in a closed circular orbit, resulting in zero mass transport. That there is in fact a mass transport velocity (also called mean drift velocity) is substantiated by observation and experimental verification, and is accounted for in the later wave theories. The orbital paths of the water particles within a wave in deep water are thus open circles. Two fluid motions thus need to be considered, namely the instantaneous particle velocities and the net particle drift or mass transport velocity.

The expressions for the individual water particle velocity, both vertical and horizontal, contain hyperbolic functions which cause an exponential decay of the velocity components with distance down from the free surface. At depths of half the wave length the velocities become negligible and below this depth there is essentially no motion. At depths of less than half the wave length the horizontal velocity has a finite value. Because of the conditions employed in deriving the velocity potential, however, the vertical velocity vanishes identically on the bottom boundary, regardless of the depth.

The orbital paths of the water particles within the wave are also modified as the waves enter shallow water. They change from being open circles to open ellipses with the long axis horizontal. These ellipses become flatter as the shoaling process continues, i.e. from the point which the wave has 'felt' the bottom (Wiegel - 1964). The orbital speed of movement no longer remains constant, but changes with the wave form. Under the sharp crest of the wave there is a short but rapid landward acceleration, while under the long, flat trough there is a much slower, but more prolonged, seaward movement of water.

The movement along the bottom is particularly significant in shallow water as this exerts a very strong influence on the movement of material along the shore. The elliptical orbit is larger than the deep water circular one along both axes and this also causes an increase in the orbital velocity. This increase is significant in the breaking of waves in shallow water. A wave will break when the increasing velocity of the water at the wave crest exceeds the decreasing velocity of the wave form.

Other factors also affect the breaking of waves. As the particle orbits increase in size, the water in the wave is reduced by the decrease in wave length. There is thus insufficient water remaining to complete the orbit, causing the front of the wave to become unsupported. The crest therefore collapses into the trough and the wave breaks.

When waves peak-up and break on a beach of very small slope, there is no appreciable reflection. Most of the energy in a wave form not dissipated by bottom friction appears to become concentrated in the kinetic and potential energy of the crest and this mechanical energy is reduced to sound, heat and turbulence induced by the break. Wiegel defines three types of breakers: spilling, plunging and surging. Spilling breakers are characterised by the appearance of 'white water' at the crest; they break gradually maintaining their identity, but gradually decrease in height until they become the swash on the beach. Plunging breakers are characterised by a curling over of the top of the crest and a plunging down of this mass of water. Surging breakers peak up as if to break in the manner of a plunging breaker, the base of the wave retains its form up to the beach face however, with the resultant disappearance of the collapsing wave crest. Wiegel likens the surging wave to that of a standing wave. The three types of breakers may grade into each other, and are dependent largely on beach gradient and wave steepness.

2.4 NEARSHORE CIRCULATION

Nearshore circulation patterns were first described comprehensively by Shepard and Inman (1950) following investigations on beaches near the Scripps Institute of Oceanography. The writers have divided the nearshore circulation pattern between the effects of the coastal currents and what they term the nearshore system.

The three-dimensional approach was necessitated from observations of what actually occurred in nature. While a two-dimensional approach is favourable in describing phenomena of 'wave set-up', it was found that the seaward return flow of the fluid mass carried into the surf zone by the breakers was assisted by localised rip currents. Furthermore, waves breaking at an angle to the shoreline injected the breaker mass into the surf zone with a longshore component of momentum, generating longshore currents.

Shepard and Inman defined the coastal currents as those flowing roughly parallel to the shore, constituting a relatively uniform drift in the deeper water in the off-shore zone. The nearshore system they described as being superimposed on the inner portion of the coastal current. In the absence of a coastal current the system could exist independently. The writers associated the nearshore system with wave action in or near the breaker zone. The system consists of four elements:

a) The shoreward mass water transport of water due to wave action (which carries water through the breaker zone in the direction of wave propagation)

b) The movement of this water parallel to the shore as longshore currents

c) Seaward return flows such as flow along concentrated lanes known as rip currents

d) The longshore movement of the expanding head of the rip current

The different systems are illustrated by Figure 5 in which the characteristic circulation pattern is clearly indicated.

Further observations and investigations have indicated that the circulation patterns were evident even under conditions of normal or near normal wave approach. The fact that as a result of the breaking wave the water level at the shore is set-up is well established both by theory and by experiment. There remains the problem of how the system will react to this equilibrium. The observations from nature tend to confirm that the seaward return flow is assisted by narrow fast flowing rip currents spaced at varying intervals along the shore. These currents seem to be fed by longshore currents flowing in from either direction, the various elements thus forming a horizontal cell of circulation. The formation of these cells seem to be thus explained by the fact that the seaward return flow after wave set-up is not completely counter balancing two-dimensionally, the concentration of a seaward return flow by means of rip currents constituting the balancing effect.

The implications of a nearshore circulation pattern are interesting. Formed, as it seems to be, by normal or near normal wave approach, the longshore

Figure 5

Terminology of nearshore current systems

(after Ingle - 1960)

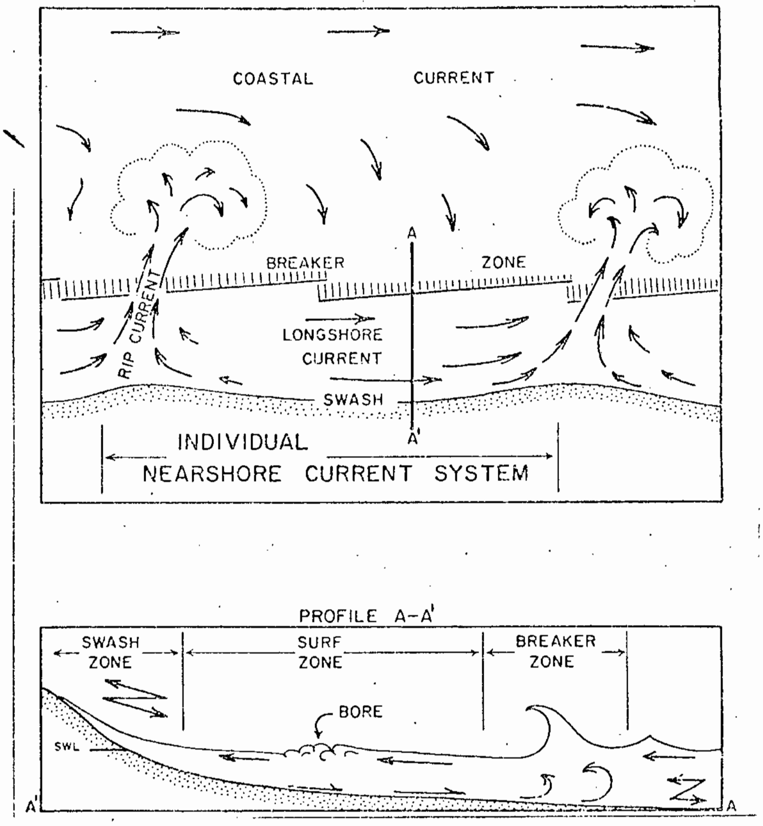
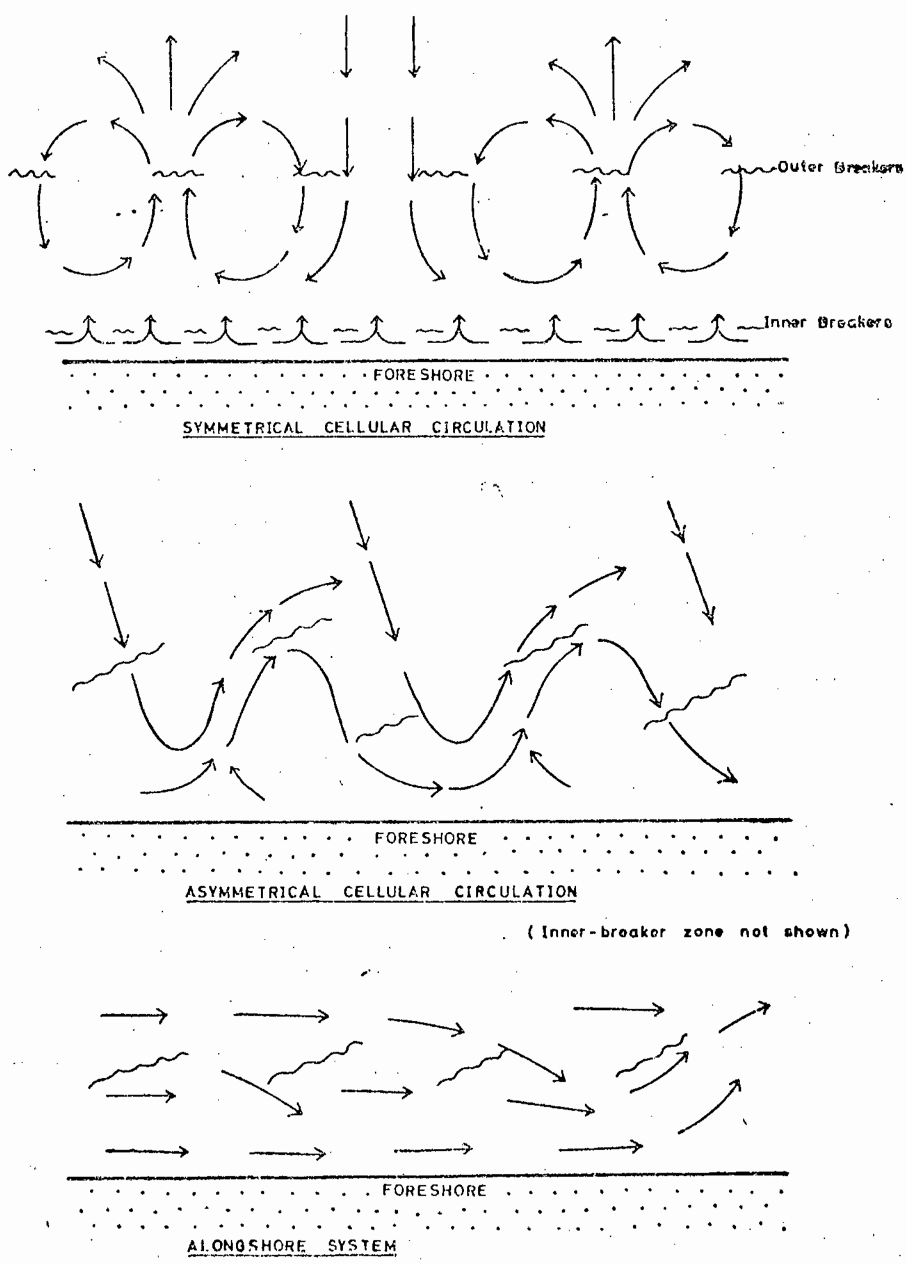


Figure 6

Nearshore circulation patterns

(after Harris - 1962)



currents are generated in such instances not by the obliquity of the waves but by the instability resulting from wave set-up. Furthermore, these longshore currents oppose each other in direction at the base of each rip current. With an increase in wave obliquity the systems tend from symmetrical to asymmetrical and finally to longshore (Shepard and Inman - 1950). Increasing wave obliquity seems to destroy the up longshore current until it is finally reduced to nil. The stage has now been reached where a true longshore system exists and the longshore current continues unidirectionally in the down wave direction, occasionally breaking away from the shoreline in the form of rip currents. The three systems are illustrated in Figure 6.

Summarising existing literature, Inman and Bagnold (1963) noted that while the spacing of rip currents varied greatly, the spacing tended to decrease with an increase in the intensity of wave action. Rip currents appear to discharge between 2% and 10% of the total volume of water transported between rips, which they estimate as the order of magnitude of the estimated longshore component of shoreward transport.

Not a great deal of attention has been given to the cause and mechanism underlying the nearshore circulation pattern. Longuet Higgins (1971) points out, however, that the theoretical framework provided by the theory of radiation stresses is capable of not only dealing with a straight and uniform coastline, monochromatic waves, a stable longshore current with no rip currents, but also with the more general situation. The writer details the more general situation as that found in nature where the wave amplitude may be non-uniform in space and time, owing to irregularities in the sea bottom, or to refraction by currents, or to wave breaking, or other causes.

CHAPTER 3.

SEDIMENTATION

3.1 INTRODUCTION

Bagnold (1964) uses sedimentation and sediment transport as synonyms. In the strictest sense sedimentation is the process by which grains become deposited. Bagnold points out, however, that since this process extends over a period of time during which the grains are being moved from place to place, sedimentation in this sense is inseparable from sediment transport.

Sedimentation in the nearshore zone is caused by a combination of shear stresses by wave action which 'breaks loose' the material, eddy currents which lift the material, and shear stresses by currents which transport the material in the direction of the current (Bruun - 1963). The static variables in the nearshore zone include the beach slope and any bottom irregularities. The dynamic variables include the effects of any local currents. Principally, these are the effects of wave induced mass water transport shorewards, the moving bores formed by breaking waves, longshore currents and the seaward return flow, including rip currents. The major parameters determining the direction and rate of sand movement are the breaker characteristics and angle to the shore, resultant longshore currents, and the degree of beach slope.

Certain aspects of sediment transport need to be covered before the effects of waves and/or currents can be considered as agents of sedimentation.

3.2 MODES OF SEDIMENT TRANSPORT

3.2.1 Transport during fall through the sea

The different behaviour of coarse and fine particles during deposition can be attributed to their differing vertical settling velocities through water. Whereas shape and density certainly affect the settling velocity of a particle, the following discussion will confine itself to particles of similar density and a basically rounded shape. These restrictions are largely met by natural beach sediments.

Dimensional analysis reveals that the drag coefficient of a sphere falling through water is a function of the Reynolds number. Through experimentation it has been established that the variation of the drag force coefficient may be approximated into three sets of power laws. The drag force laws are called respectively Stoke's law, valid for low Reynolds numbers where viscous forces dominate; Newton's law, valid for high Reynolds numbers where inertial forces dominate; and Allen's law, representing a transitional range where both inertia and viscosity would be involved.

The terminal settling velocity (v_s) of a particle falling through a body of still water may be calculated by the use of the drag laws applicable to the various Reynolds number ranges. Clay, silt and very fine sand obey Stoke's law, i.e. $v_s \propto D^2$; Gravel and very coarse sand obey Newton's law, i.e. $v_s \propto D^{\frac{1}{2}}$; Fine, medium and coarse sands obey Allen's law, i.e. $v_s \propto D$.

During a particle's fall through the sea, it is suspended by water which exerts an upward force on it equal to the immersed weight. The grains conform without resistance to any motion which the surrounding water might have, apart from inertial effects.

The settling velocity of clays and silts is very low due to viscosity effects. Therefore, any such particle falling through a body of moving water will have an almost horizontal vector. Mass gravity controls the settling velocity of very coarse sands, however, and therefore such particles fall with an almost vertical vector. Particles thus obeying Stoke's law are transported in suspension, whereas those obeying Newton's law are moved along the bottom as a bed load or rolled as a 'creep' population. Particles in the intermediate zone are moved in saltation.

3.2.2 Lateral movement and transport over the seabed

Transport over the seabed is a result of the combined action of fluid flow and of a gravity component parallel to the seabed. The phenomenon covers sediment transport by wind and water streams, and by sea currents with or without waves. It also covers (amongst others) turbidity currents and soil creep. The phenomena involves the 'flow' of dispersed granular solids within fluids (Bagnold - 1964).

The lateral movement is accomplished by the processes already mentioned (i.e. creep etc) and the upper size limits of each process is strongly affected by the degree of turbulence in the environment. Moss (1963) attributes the creep load as a result of the increase in current velocity with distance from the bed. This causes a greater force to be exerted on the upper surface of a grain than on its base and the grain is thus forced to roll forward.

During saltation, the particles are apparently lifted a few particle diameters from the base at a steep angle and accelerated forward by the currents as they settle back to the bed again. Moss describes the saltation particles as very well sorted, due to similar particles fitting into available spaces amongst deposited grains. Dissimilar grains land in unstable positions and are re-eroded and deposited elsewhere.

3.3 SEDIMENT TRANSPORT THEORIES

Das (1972) reports that there are no proven prediction methods of general validity for quantitative estimates of on-shore/off-shore and longshore sediment rates. Das attributes this to an inadequate knowledge of the turbulent flow field in water waves, and to the non-availability of reliable techniques to measure sediment transport rates in the nearshore zone. Longuet Higgins (1971) reports that at least a semi-quantitative theory for predicting both longshore currents and sediment transport is available. The writer acknowledges the lack of a really satisfactory theory of breaking waves and in particular the lack of a theory describing both the laminar and turbulent regions of flow in simultaneous detail.

It is agreed in principle that the oscillatory flow due to wave motion stirs up the sediment and makes it available for net movement by the mass transport current associated with the wave motion or by any other net current. It is also agreed that sediment motion occurs in two basic modes - suspension and bed load. Their relative contribution to total transport remains unspecified in both zones. Within the surf zone direct observations by Inman have shown that most of the weight of sediment is in bed load, not suspended load.

Theories on bed load transport have been developed from those based on unidirectional flow (Einstein - 1955 and Bagnold - 1963). Their application to the complex three-dimensional flow regime in the nearshore system is clearly difficult, relying as they do on idealized hydrodynamic analysis.

Following Inman and Bagnold (1963), Komar and Inman (1968) define the immersed weight transport rate I_ℓ by the relationship

$$I_\ell = a' (\rho_s - \rho) g Q_s \tag{24}$$

where Q_s is the sand volume transport rate, ρ_s the sand density and a' the volume concentration (equal to about 0,6 for closely packed sand). I_ℓ is related by definition to W the immersed weight of mobile sand by

$$I_\ell = W \bar{V}_s \tag{25}$$

where $\bar{V}_s$ is a mean longshore sand velocity.

From experimental data Komar and Inman (1970) have further proposed the relationship

$$I_{\ell} = K F c \tag{26}$$

where F is the total longshore thrust exerted on the surf zone, c is the local wave velocity and K is a dimensionless constant (equal to about 0,77). Expressing equation (26) in terms of the sand volume transport rate, Q_s:

$$Q_s = 0{,}77 \times 10^{-3} F c_b \tag{27}$$

(where Q_s is in m^3/s and $F c_b$ in N/s)

Equation (27) is, to the best of the author's knowledge, one of the most commonly accepted relationships for calculating the littoral drift rate.

3.4 SEDIMENT TRANSPORT BY WAVE INDUCED CURRENTS

Before breaking, the oscillatory motion of the waves move the bottom material back and forth, in and against the direction of wave propagation and subsequent mass transport. As wave asymmetry increases, the passing of a wave crest is associated with a rapid acceleration landwards at the bottom, while the passing of the wave trough has a slower though longer seaward acceleration. Some of this material is moved in suspension by the turbulent eddies generated as a result of the interaction between the oscillatory motion and the sand surface of the bottom, while some of the material is moved in the sand ripples of the bottom. These motions may have a longshore component because of the obliquity of the wave energy.

During breaking, the beach material is placed in suspension by the turbulence of the breaking waves. The waves will produce a current along the shore if the angle of approach is oblique. Material placed in suspension by the turbulence may be carried by this current. This motion will be superimposed on the basic on-shore or off-shore movement of material, the net movement normal to the shore being a function of the beach and wave characteristics

During and after breaking, a mass of water is propelled up the foreshore after each wave. Material is moved on-shore by the swash, and off-shore by the backwash. Again, should the mass of water have a longshore velocity component, the material will move in a slanted direction up and down the foreshore, both in suspension and rolling on the bottom as bed load. Generally, depending on the waves prevailing at the time, either a seaward or a landward movement of sand will predominate. Superimposed on this will be the longshore movement, if any. A longshore current prevailing will result in the characteristic zig-zag pattern of sediment movement in the direction of the current, with the basic tendency for the net movement to be either seawards or landwards.

3.5 SEDIMENT TRANSPORT BY WIND INDUCED CURRENTS

Apart from the major function of generating ocean waves, wind also plays an important part in the movement of beach material. The occurrence of frequent strong and enduring winds at the coast thus needs mention as a further factor affecting sediment transport.

One factor that can cause a seaward current along the sea bottom even in the off-shore zone, is an on-shore wind. The landward movement of the surface water as a result of an on-shore wind may be compensated by a seaward current at a lower level. Model studies in a narrow wave tank by King (1959) confirmed the existence of return flow on the bottom to compensate the water blown shoreward on the surface. This bottom flow was strongest in the shallowest water. Wind and waves combined tended to cancel each other so that the gross volume of sand moved was reduced. Under the action of constructive waves with a strong on-shore wind the landward movement of material was reduced in both the in-shore and off-shore zones. The effects of destructive waves was enhanced in the in-shore zone by an on-shore wind.

Observation in the field have tended to confirm that the same effects applied on a natural beach when the wind was blowing on-shore. Off-shore winds caused constructive action on the foreshore (King - 1970). Wiegel reports that in certain areas winds are also of importance as they are responsible for the loss of sand from the beach and its movement inland as dunes. The grading of foreshore sands in such regions can be expected to show a deficiency of fine grained sands.

3.6 SEDIMENT MOVEMENT IN THE NEARSHORE ZONE

In the off-shore region the mass transport velocity of the water in the direction of the wave movement has already been discussed. Superimposed on the current induced by passing oscillatory waves are the other currents also mentioned which may or may not be a factor in sediment transport. In general, the sediment movement tends to follow the water particle motions, thus a basic landward movement of material could be expected in this zone along the sea bottom. Bed load sediment movement involves the shearing of the sediment grains over each other and the bed. This naturally increases with increased orbital motion of the waves. Model experiments on the movement of material seaward of the break point reported by King (1970) show that this movement is nearly always landwards. The volume transported increases as the waves become higher, as their period lengthens, as their energy increases and as the beach becomes flatter and the bottom rippled. The last mentioned is no doubt due to the increased turbulence of the bottom layers under rippled conditions. Wave steepness does not appear to be fundamental to sediment transport in the off-shore zone.

3.7 SEDIMENT MOVEMENT IN THE IN-SHORE ZONE

Landwards of the break point in the in-shore dynamic zone the flow conditions are far more complex than in the off-shore zone. Valuable information on the movement of sediment normal to the beach has been obtained from two-dimensional studies. Fundamental to the direction of sediment in the in-shore zone is the wave steepness of the impinging waves. Waves of a certain steepness and greater have been found to move sediment seawards towards the break point, while for waves with lower steepness values the predominant motion of sediment was found to be landward.

Even the steepest waves appear to move little sediment outside the breakpoint, whereas the seaward motion inside the breaker zone was greatest just landward of the breakpoint. From this point on the amount of material moved decreases with depth towards the beach face.

3.8 BEACH ACCRETION AND EROSION

While coastal accretion and erosion can be attributed to a number of factors, such as changing sea level, this section will confine itself to the effects of constructive and destructive wave action. Furthermore, the effects of waves on unconsolidated beaches only will be considered. Coarse material is more mobile than fine, hence the effect of material size is basically one of degree of erosion or accretion.

3.8.1 Accretion due to waves

Wave steepness has been shown to be the most significant factor on the foreshore. A flat wave tends to move material landwards and build up the beach profile, whereas a steep wave has the opposite effect. The critical wave steepness at which waves change in character from constructive to destructive varies from beach to beach, and seems to be dependent on the physical properties of the beach material. There seems to be a tendency for the critical steepness to increase as the beach material becomes coarser. The values for natural beaches seem to be lower than those established in tank experiments, although there are few precise values available (King - 1970).

The dimensions of the waves are also important because their energy depends on their height and length. Because of their lower energy, long, low waves require more time to build up a beach than the high steep waves require to erode it. (This factor is counter balanced in nature by the short duration of high waves on most occasions). Lafond (1931) has shown from field studies in California that low steepness waves, associated with lower energy, did not deposit as much material in seven days as was removed by high steeper waves in one day.

Accretion of beaches is affected not only by the wave steepness, but also by the direction of approach of the waves. Longshore material transport resulting from oblique waves may in some instances be much more important than the material transported normal to the coast. This may be illustrated by the construction of groynes, the function of which is to trap material moving along the shore, encouraging beach accretion in areas where the natural supply of sand is small. Other obstructions may have less desirable results by trapping too much sand on the updrift side of the barrier, thus causing erosion on the other side. Accretion may even take place under destructive wave conditions if more longshore material is moved into an area than is being moved seaward by the destructive forces.

Similarly, coastal wind is another factor which may even reverse the movement to be expected from wave characteristics. As already discussed, a strong on-shore wind may more than counteract the effect of waves which would otherwise be constructive on the beach.

As can be seen from the above description, constructive wave action on the beach results from the action of waves which move material towards the shore, other factors for the moment considered negligible. On some costs the building up of beaches due to constructive wave action was found to be seasonal, as in parts of California. The variation of wave type with the seasons was found to be marked in this area. The beaches were found to build up during the summer, producing a profile with a wide berm at high tide and relatively smooth profile off-shore. This was termed a summer profile, and the constructive waves which formed it, summer waves. Constructive waves have also been called swell waves.

Whatever the terminology, constructive, summer, low step forming or swell waves are defined as waves which tend to move material towards the shore. Such waves are long relative to their height, and have a low steepness ratio.

Figure 7 serves to illustrate the influence of the grain size of the beach material and physical wave height expressed as a dimensionless ratio on the critical wave steepness. The generation of a longshore bar has been taken as the criterion between constructive and destructive wave action.

3.8.2 Erosion due to waves

Above a critical steepness value, dependent on the physical properties of the beach material, waves are destructive. The erosive effect of these steep waves result in the lowering of the foreshore and material transport seaward.

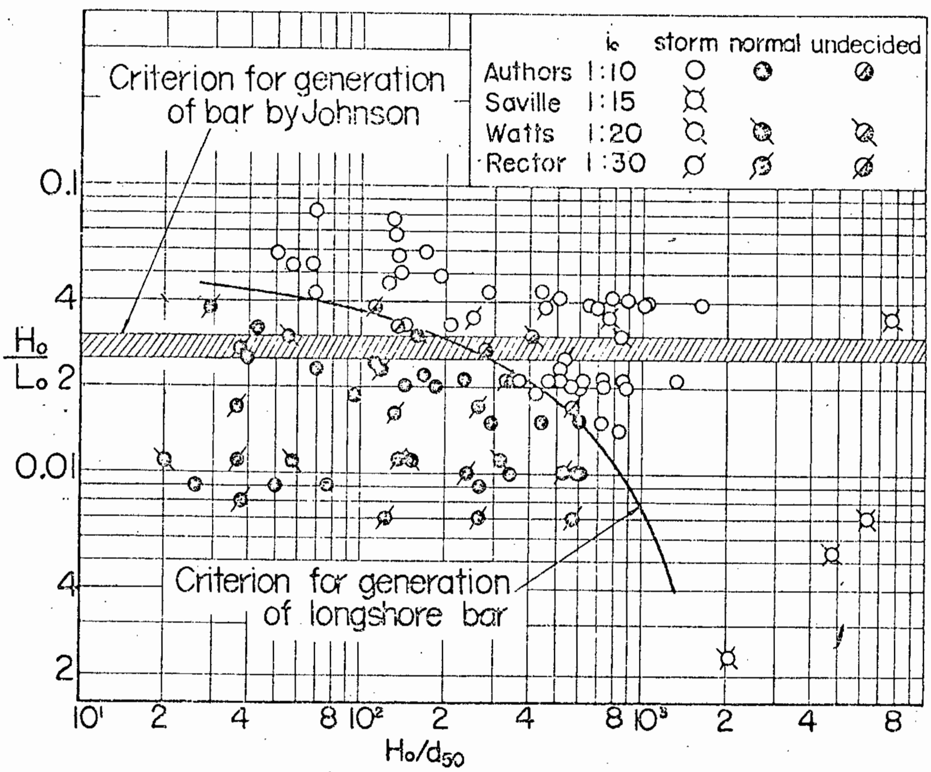


Figure 7 Criterion for generation of longshore bars (after Iwagaki and Noda - 1962)

The larger the waves are, provided they are above the critical steepness for destructive action, the greater will be the removal of material from the beach. This may be attributed to their greater energy. Destructive waves are normally associated with storms and high wind velocities. Strong on-shore winds, in this case, may assist the destructive tendency of the waves not only by the induced bottom seaward return flow, but also by raising the water level above its normal height. The sea will thus be able to attack zones normally beyond its reach.

There is a significant difference between the effect of steep waves on shingle and sand beaches. On shingle beaches such waves cause rapid seaward transfer of material, although some may be thrown above the reach of ordinary waves to form long lasting shingle ridges. On sand beaches steep waves are entirely destructive in character and take the sand out to deeper water off-shore. The coarser the sand, the greater the amount removed seawards under similar wave conditions (King - 1970).

In some areas whole beaches may be removed by destructive wave action, as reported by Shepard (1950) from observations along the Californian coast. The most serious beach depletion and coastal erosion is however usually associated with abnormal conditions of weather and water level.

Directly or indirectly, the movement of material longshore is responsible for nearly all coastal erosion (King - 1970). Destructive waves, acting normal to the shore, can only move the beach material a relatively short distance off-shore. Most of this material can thus be returned to the beach by constructive waves acting during periods of normal weather. The permanent absence of a beach however can usually be explained by longshore movement, unless the water is so deep off-shore that this prevents a beach forming. Coastal erosion is most likely to take place, therefore, where more material is moved longshore out of an area than is being moved into it from the updrift direction. This situation is likely to occur on headlands or man made structures where a strong along shore movement of material is interfered with. Erosion will occur on the downdrift side of the obstacle.

Destructive wave action, or destructive waves have been similarly termed steep, storm, bar forming or winter waves. The last mentioned is again a reference to the seasonal change experienced on some Californian beaches, likewise associated with so-called 'winter profiles'.

CHAPTER 4

BEACH PROFILES

4.1 BEACH TERMINOLOGY

The position at which a wave approaching the shore along a shoaling bottom peaks up and breaks, depends on the bottom slope and the wave steepness. At and after breaking, the dynamic properties of an oscillatory wave undergo considerable change. It is thus convenient to divide the shoaling region into two distinct zones, termed repectively the off-shore and in-shore zones. The two zones together form what is also termed the nearshore region. The terminology used for the shore profile is given in Figure 8.

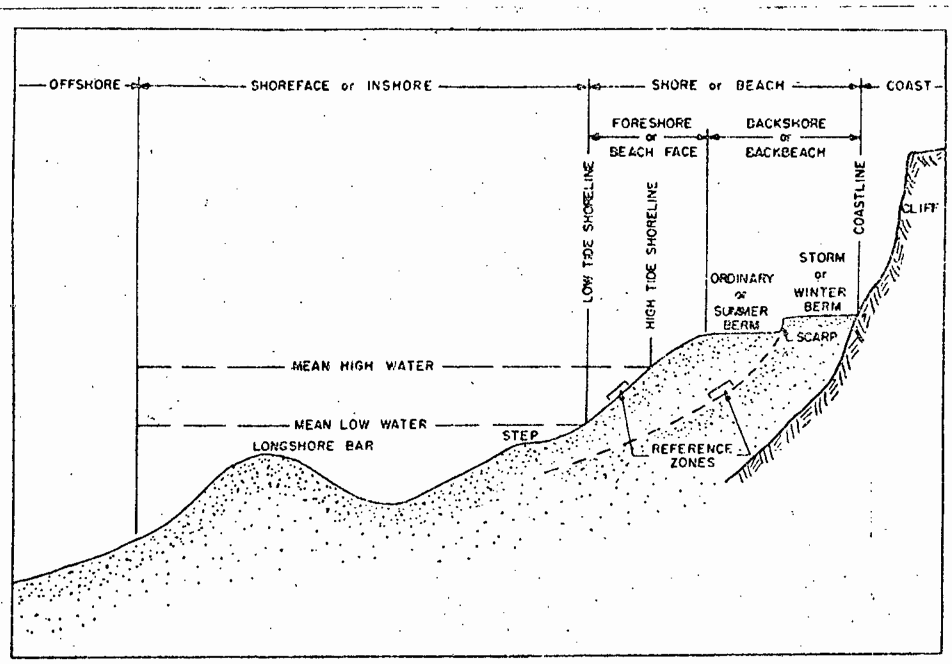


Figure 8 Shore profile terminology (after Wiegel - 1958)

4.1.1 The off-shore zone

This zone has its landward limit at the location of the breaking waves. Its seaward limit, being the limit of appreciable sediment motion, is less easy to locate. Sediment motion has been observed by divers down to depths of 50 m (King - 1972). To what depth net sediment transport occurs depends not only on wave and other currents, but also sea bottom topography. Radio-active tracer studies in progress at the time of writing should provide valuable information on the subject. The objectives of such studies are the understanding of the mechanism of sediment motion - both entrainment and transport, patterns of movement, and the volume of sediment movement.

4.1.2 The in-shore zone

This zone is further subdivided into four distinct subzones, based once more on dynamic considerations. These are the breaker zone associated with wave collapse and high energy; the surf zone with waves of translation and an energy gradient towards the beach; the transition zone characterised by collision between the surf and the backwash; and the swash zone associated with alternate swash and backwash.

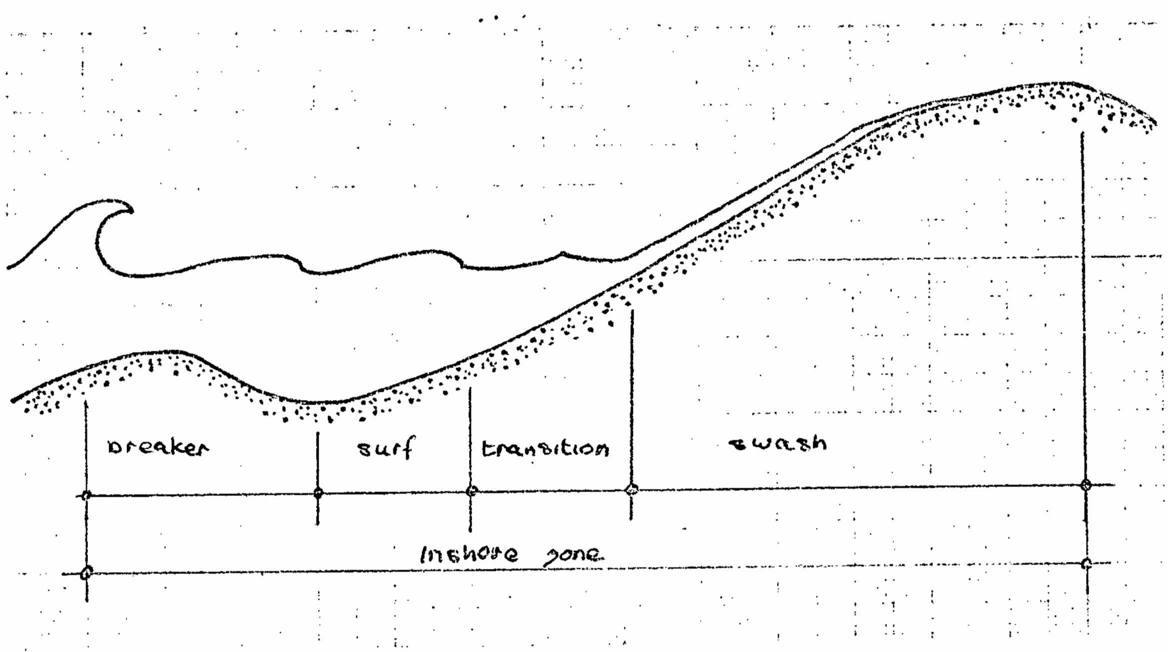


Figure 9 In-shore zone terminology
(after Ingle - 1962)

At this stage the author would like to stress the following point of relevance to the longshore sediment transport system. Under a given set of wave conditions the longshore velocity of sediment transport will differ from zone to zone, as well as within a zone. It is generally believed that the transport seaward of peaking-breaking waves is less than the transport on the beach face, and both of the former being less than the transport in the breaker and surf zone (Duane - 1962). Because of these differences it should be realised that tracer surveys confined solely to the in-shore or off-shore zones produce data only partially indicative of the transport in the whole.

In the above context King (1970) introduces two new concepts termed surf base and wave base respectively. 'Surf base' is the term used to delimit the zone of substantial sediment movement. King restricts this zone to shallow water of not exceeding a depth of about 12 m. 'Wave base', on the other hand, is applied to the slow and small changes that occur in deeper water and are not thought to involve the movement of substantial amounts of sediment over considerable distances.

Associated with the wave base and surf base concepts is the equilibrium line or neutral line, first proposed by Cornaglia in 1887. Landward of this line sediment movement is on-shore, while seaward of it the movement is very slowly off-shore. Theoretical and experimental results support this concept, the latter indicating that the off-shore movement is often negligible. The position of the neutral line would seem to be determined not only by the character of the waves but also by the slope of the bottom and the size and density of the bottom particles. For a given set of wave conditions, bottom slope and sediment density, the position of the neutral line depends on sediment size, being closer to the shore for larger material.

The movement of particles associated with the equilibrium line will cause both sorting of material and modification of the bottom profile. The equilibrium line is important in that it determines the depths at which sediment begins to move. Only sediment in water shallower than this can be moved by waves and is available to feed beaches from off-shore. Complications occur when particles are small enough to be carried in suspension, and the existence of other than wave induced currents. Superimposed on the fundamental on-shore/ off-shore sediment motion is the existence of longshore sediment movements.

4.2 BEACH FEATURES

Beaches are dynamic and sensitive to small changes in natural forces. They are composed of constantly shifting groups of particles which move and orientate themselves to fit changing waves and currents. The particles make up two major beach forms, the berm (above water) and the bar (below water). The transfer of material between the two seems to be dependent on the wave steepness (Bascom - 1953). The relief of these features and the slope of the beach face are related primarily to the particle size and to the character of the waves which formed it.

Model experiments in laboratory wave tanks have successfully reproduced similar profile configurations to those observed on natural beaches, in spite of the scale effects which ordinarily do not allow the formation of a substantial surf zone (Ingle - 1966). While flow conditions seaward of the break point are likely to be very different in nature and in model, shoreward of the break point fully turbulent conditions exist in both, hence dynamical similarity is good.

4.2.1 Bar profiles

Destructive wave conditions (or any other factors resulting in a net seaward movement of beach material such as on-shore winds) result in the formation of a sand bar slightly seaward of the breaking waves. The formation of a bar is thus associated with steep waves, and generally is accompanied by a trough landward of the bar beneath the plunge point of the breakers. The bars and troughs generally align themselves parallel to the shore, the crescentic formation observed by some will be discussed in a later section. Local discontinuities may occur where rip currents penetrate through the in-shore zone. These generally tend to a seaward increase in bar volume and a loss of identity of the trough.

Bars are considered to form as a result of the neutralising effect of currents carrying material landward and the seaward return flow. In the process, material is deposited and a bar forms. Model studies have indicated that subject to a non-varying wave attack an equilibrium bar size is reached, and the bar crest can never grow above the sea level. Once the equilibrium height is reached, any further growth would appear to result in deformation of the impinging waves and resulting dynamic imbalance. Certainly waves are filtered by the bar, and only waves with a height less than some critical value appear to pass without breaking. When these smaller waves reach some appropriate depth they also break and a second bar may be formed. Small waves appear unable to obliterate the outermost bar, although the bars in-shore may show considerable change. The occurrence of more than one ridge of bars may also be a result of tides and the build-up of once broken waves to reform and break again.

The bar position, height and size appear to be functions of the wave height, wave steepness and the beach gradient. An increase in wave height is associated with the formation of a larger bar in deeper water. The wave steepness is fundamental in determining whether a bar will form or not. The position and height of the fully formed bar is independent of the beach gradient. The gradient, however, is a function of the beach material and as such determines the critical wave steepness value between constructive and destructive waves. As the gradient becomes steeper, associated with an increasing coarseness of beach material, so the critical steepness increases.

King (1970) has reported the results of laboratory experiments with changes in wave size and water level. A slow increase in wave height caused a bodily migration of the bar seawards. Decreasing wave height shifted the bar landwards. Provided the wave steepness remained above the critical value, smaller waves did not appear to effect great changes in an already formed bar. Changes in water level affected migrations to the bar similar to those caused by changes in wave size. A rising tide had a similar effect to that of a reduction of wave height, and vice versa.

It should, however, be noted that on a tidal beach a bar probably never has time to develop fully. The bar is dependent on the stability of the break point for its formation and, as already mentioned, can only form when the wave steepness is above a critical value.

The formation of a semi-confined channel parallel to the shore as associated with a bar and trough are thought to have a big effect on the longshore drift phenomena (Eaton - 1950). Certainly, field measurements have tended to confirm the existence of high velocity longshore movements in these channels.

4.2.2 Berm profiles

The berm forms what is also commonly referred to as the 'beach'. It is a nearly horizontal formation of material brought ashore by the waves. Berms represent a depositional sand formation above mean water level as opposed to the underwater erosional formation constituting sand bars. After the initial seaward slope, the berm may be nearly horizontal and is dependent for its width on the amount of sediment available, and to a lesser extent by the amount of material being removed by wind.

Berms are associated with the action of constructive waves on the foreshore. The depositional height of a berm appears to be a function of the wave height in terms of the swash and backwash, the tidal range, and indirectly by the beach material. The wave steepness is again considered fundamental, determining whether a berm will form or not (King - 1970). The formation of a berm is associated with the sediment carrying capacity of the swash as it surges up the beach slope. This material is thus moved landwards and accumulates at the top of the beach where it is raised above the still water level and builds-up to the limits of the swash. The height of the berm furthermore appears to increase with decreasing wave steepness. Flatter waves are thus considered more constructive; steeper waves causing an immediate reduction of volume of the berm (King - 1970). King also considers the formation of a berm only if the original beach gradient is less than the gradient being built-up by the swash in its constructive action. Berms should thus be associated with an attempt by the waves to produce an equilibrium swash slope gradient, suitable to their dimensions, on a beach the overall gradient of which is flatter than the equilibrium gradient. A lagoon may form landward of the berm crest.

4.2.3 Additional beach forms and features

A further beach form associated with constructive wave action has been termed a 'step'. The step exists below water level and is presumed to be formed by the vortex produced by the backwash. After breaking a wave becomes translatory, carrying sediment as bedload or in suspension with it. The uprush of the swash slows down due to gravity, friction and percolation. When motion has ceased, water not percolated returns as gravity flow down the beach carrying sediment with it. When this backwash comes in contact with the swash of the next wave, coarse material is deposited to form a low seaward facing step. The step adjusts itself to changing wave characteristics. During a period of erosive wave action the great agitation of waves breaking over this step on reaching the beach face result in the deformation of the step into the characteristic bar and associated trough indicative of destructive waves.

Destructive waves eroding a beach with a prominent berm performed by constructive wave action will result in the formation of a cliff or 'scarp' at the limit of the swash. Destructive waves tend to cut the beach face back and down, the beach face thus tends to flatten somewhat. The overall beach gradient follows this trend, an additional contributing factor being the disappearance of the step and the formation of the off-shore bars. The process has been described by Kemp (1961) as an elongation of the beach profile with the crest moving landwards and the step moving seawards becoming, in the process, a bar.

Beach cusps are a beach form appearing on both accreting and eroding beaches and on beaches subject to both normal and oblique wave attack. Beach cusps are associated with wave set-up, edge waves and nearshore circulation patterns. If conditions are suitable cusps form very rapidly and a considerable change of conditions appear to be necessary to remove them (King - 1970).

Johnson (1919) first reported the apparent relationship between the size of the cusps and the size of the impinging waves. Cusps were also noted to form most readily under conditions of normal or near normal wave attack. Bagnold (1940) noted how the swash piled up against the horn of a cusp, dividing the flow into each adjacent bay to unite with its opposite number and forming a strong backwash down the centre of the cusp bay. Kuenen (1948) stressed the relative regularity of cusps on many beaches, and their tendency to migrate in the direction of longshore currents, if at all. After an extensive field study Longuet Higgins and Parkin (1962) failed to correlate wave period and cusp spacing, but thought cusp spacing and wave height to be related. The writers found a positive correlation between the cusp spacing and the swash length.

During the course of the author's model study certain observations of cusps and cusp behaviour were made which are recorded in Part 2 of this document.

4.3 EQUILIBRIUM BEACHES

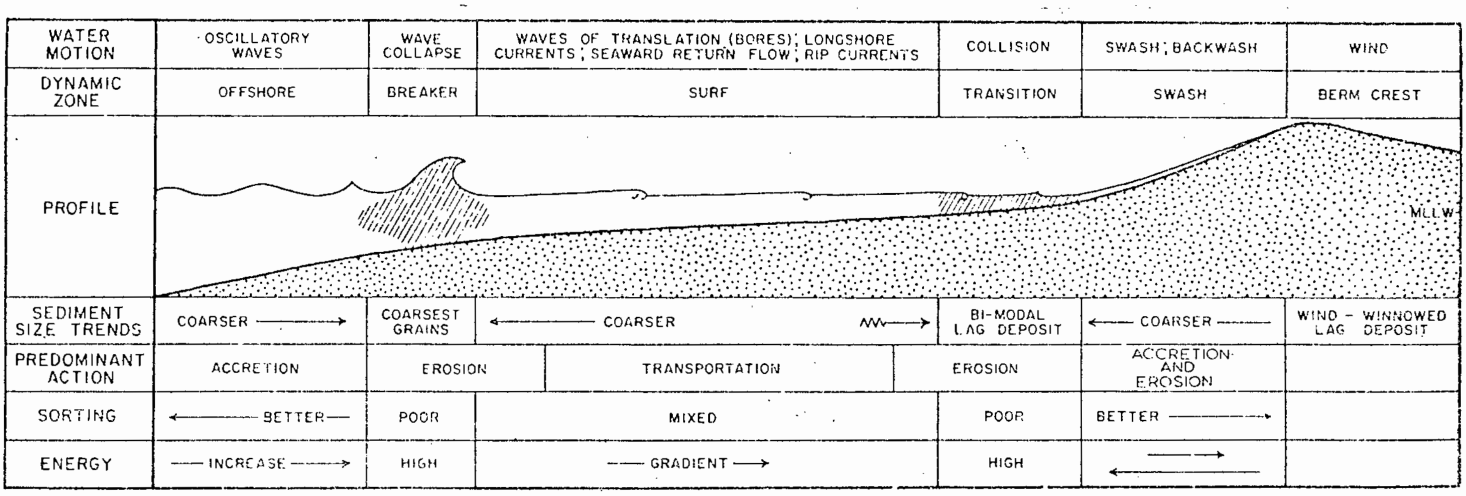
4.3.1 Beach materials

On natural sand beaches the most common method of classification is one of size distribution analysis. Shape analysis is also used to a smaller degree, but has generally been confined to shingle or cobble beaches, as these do not require microscope analysis. Several methods have been developed to study the size distribution in sands and other sediments by Task (1932), Inman (1952), Falk and Ward (1957) and McCammon (1963). These investigators have been primarily concerned with the grain size statistics and have presented various methods for obtaining the mathematical and graphic moment measures for sediment analysis.

Various classifications have been proposed to define the categories, from fine clay particles to chunks of rock, of which beaches may be formed. Amongst these are the BSCP and Wentworth scale classifications. Krumbein (1934) first proposed the ϕ unit classification, based on the transformation $\phi = -\log_2 D$, where D is the diameter of the particle in millimetres. This classification resulted from the observation that particle size distribution within an individual sample tended towards a log-normal distribution. Use of the ϕ transformation thus enabled investigators to work with a normal distribution of ϕ, statistically much easier than working with a non-normal distribution in terms of the direct grain size. The ϕ classification is that in most common use at the present time. The success of a grain size analysis remains however dependent on the accuracy of the sample as an unbiased representative of its environment. To this end various sampling schemes have been proposed. Krumbein (1957) has listed the most common techniques as that of random, stratified, systematic or stratified systematic sampling.

4.3.2 Sorting

The difference between the shoreward and seaward acceleration due to wave action, and resulting sediment transport has the effect of sorting the sand in particles sizes parallel to the shore. This sorting involves the rearrangement or adjustment of particles of varying size (and relative density) on a sloping surface in accord with specific dynamic parameters such as wave height, wave steepness or current velocity. Sorting should vary with depth, and has been found to become progressively better in an on-shore direction towards the beach point. The basic tendency in the off-shore zone is for the finer particles to move off-shore while the larger and coarser material moves towards the break point. As the water becomes progressively shallower, still larger particles will be capable of movement and will move on-shore. The coarsest material is found at the high energy zones, namely the break point where the waves collapse, and the transition zone where the backwash collides with incoming bores. These high energy zones are also associated with the poorest sorting because of the tendency of particles of all sizes to collect in these zones. Sorting parallel to the shore is shown graphically in Figure 10.



. Summary diagram schematically illustrating the effect of the four major dynamic zones in the beach environment. Hatchered areas represent zones of high concentrations of suspended grains. Dispersion of fluorescent sand and electromechanical measurements (SCHIFFMAN, 1963, 1965) indicate that the surf zone is bounded by two high-energy zones; the breaker zone and the transition zone. MLLW = mean lower low water.

Figure 10 Beach sand sorting
(after Ingle - 1966)

The most important element that accounts for the sorting of beach materials is the energy of the waves. If different character of wave motion, leading to a variation in the wave energy, is thus encountered along a beach, a degree of transversial sorting can also be expected. It does seem however that good sorting parallel to the sea can only occur where longshore movement is restricted and the amount of material on the beach is more or less static. Material can then move to that part of the beach where it is in equilibrium with the prevailing conditions. This applies to sorting both normal and parallel to the shore.

4.3.3 Gradient

Various investigators have regarded the beach gradient normal to the shore as indicative of the wave and material characteristics associated with a particular beach or stretch of coastline. The gradient referred to is generally that of the swash zone, and has been described by King (1970) as primarily dependent on the size of the beach material, the length and steepness of the waves.

Coarser material beaches are much steeper than those of fine sand. As a result, these beaches are more mobile due to the concentration of wave energy over a relatively narrow zone induced by a steep slope. Model studies by Bagnold (1940), and field observation by the Beach Erosion Board (1933) and Bascom (1951) have confirmed the increase of gradient with increasing material size. The basic cause of the phenomenon is attributed to the variation of the percolation rate through beach material. Coarser materials are more permeable than fine, allowing a larger percentage of the swash to sink into the beach. The backwash is correspondingly reduced. The force of gravity, which is proportional to the slope tends to equalise the force of the swash and backwash. When the two are very different as they are on coarse grained beaches, the slope must be steep to render gravity more effective. On a fine sand beach, because of reduced permeability, the opposing forces of swash and backwash are more equal. Gravity need not be so powerful and a flatter slope is formed.

An increase in gradient corresponding to a decrease in wave length has also been reported by King as a result of field and model studies. Controlling the wave steepness variable by means of partial correlation, King found the correlation between wave length and beach gradient to be almost linear under conditions of more or less constant material size. As the material remains constant, and consequently the rate of percolation, King attributes the flatter gradient produced by longer waves in terms of the volume of backwash. If it is assumed that a larger volume of backwash leads to a flatter gradient, King argues that a larger wave produces more swash and consequently more backwash if the percolation remains constant.

Although the slope of different parts of the beach depends on the material size, for any one size, the slope is related to the wave steepness (Rector - 1954). As the wave steepness increases so the gradient of the beach decreases. This relationship is borne out by Meyers, King and Ingle. Steep waves move sand seawards from the upper beach thus reducing the mean gradient above sea level. Explained in terms of the volume of the backwash, increasing wave steepness under constant wave length and percolation conditions lead to increased backwash and a consequently flatter gradient.

4.3.4 Equilibrium profiles

Natural beaches adjust themselves to a change in the incident wave through various mechanisms of net sediment transport, which continue until local slopes and median sizes satisfy everywhere the bed condition of oscillating equilibrium. Ippen (1966) defines the resulting 'equilibrium profile' of a sand beach as that profile which the water would eventually impart if allowed to carry its work to completion. Because of the constantly changing character of the waves, tides and currents acting to shape a natural beach, it is doubtful that such beaches ever reach a true equilibrium configuration. The equilibrium condition even when it does form is dynamic in nature and will be continually tending to adjust itself to the changing variables on which it depends.

Various analyses have been carried out to assess the relative importance of the variables associated with the equilibrium condition. Apart from the effects of material size, wave length and wave steepness on the gradient, material sorting, tidal range and the measure of exposure have also been considered as relevant to the equilibrium profile. Data analysed by King using simple, partial and multiple correlation confirmed that material size is the most important variable affecting beach slope. This relationship was also shown by trend surface analysis using sand size and wave energy values as the independent variables in relation to beach gradient (King - 1970). The minimum gradient was found to be associated with the finest material and the maximum value of wave energy or fetch. These are fine-sand beaches exposed to long swells in exposed situations. At the opposite extreme are the steepest beaches where material size is greatest and wave energy lowest. The dynamic equilibrium of the different beaches will fluctuate around a mean point according to the mean material size and the beach exposure. Figure 11 illustrates trend surface analysis as developed by King using the beach gradient as the dependent variable.

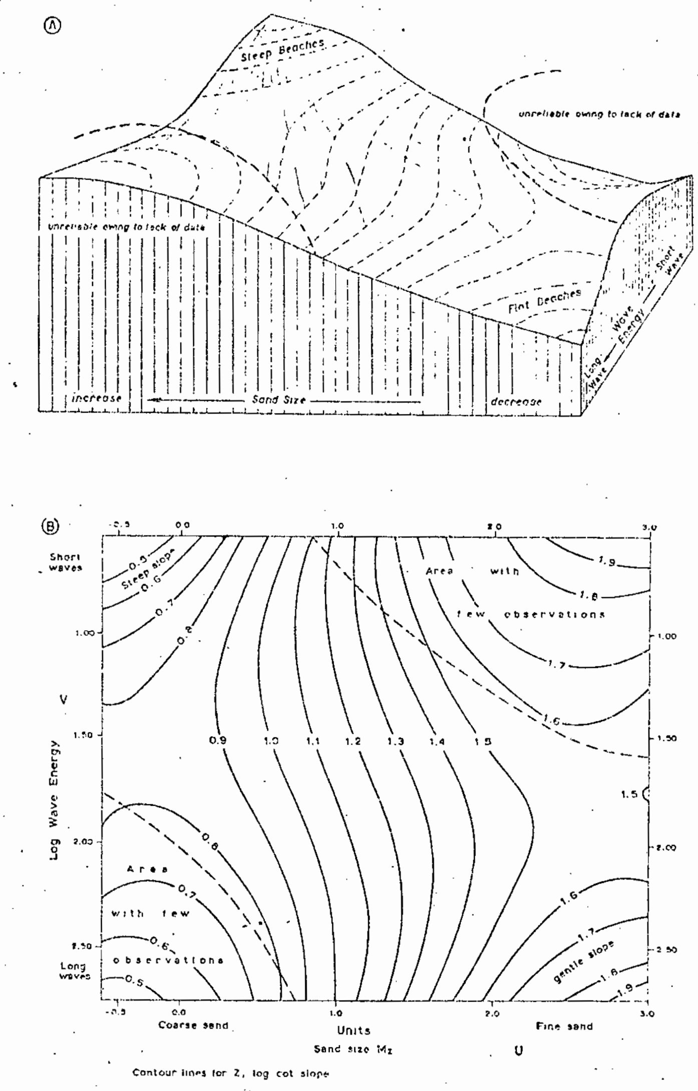


Figure 11   Trend surface analysis of the relationship between sand size, wave energy and exposure as the independent variables, and beach gradient as the dependent variable

(after King - 1970)

An important consideration when reviewing the concept of an equilibrium profile must be the amount of material available for the formation of the condition. (Bruun - 1955) goes as far as considering three different types of profiles, namely that of an overnourished condition, associated with an irregular profile with shoals and bars, a sufficiently nourished and an undernourished condition, both associated with a smooth equilibrium form. (The author, however, is of the opinion that an irregular profile is not necessarily the product of an overnourished beach but a feature of the equilibrium form under any condition).

Model studies have enabled investigators to control the variables upon which the equilibrium condition is dependent. By holding variables not under examination at constant values the effect of other factors may be assessed independently and with relative ease compared to conditions in the field. Laboratory models have displayed considerable scale effects when compared to prototype conditions and the elimination of these effects is still the subject for dispute and can only be achieved by the sacrifice of certain established scale laws. Models remain, however, important in their own right when used as a tool for understanding and gauging the effects of the various different conditions on the equilibrium profile.

The use of laboratory models in wave tanks and flumes have largely demonstrated the independence of the final or equilibrium gradient relative to the initial slope in the in-shore zone. Slopes steeper than the equilibrium profile are immediately combed down, while flatter slopes are built-up by the action of the swash. The initial slope does however control the amount of material available for movement by the waves. The effect of the initial slope on the equilibrium profile in the off-shore zone and at the break point is less clear, and remains a subject for controversy. The author attributes this to the basic lack of knowledge of the sediment carrying capacity of oscillatory waves under shoaling conditions over a movable bed.

Whether in models or in the field, it remains reasonable to suppose that if waves of a given set of characteristics act on a beach composed of particles which are capable of being moved under the action of the wave forces, then the beach will take up a configuration or profile characteristic of the waves and of the beach material. The effects of constructive or destructive waves have long been noted in the field. Johnson (1949) first expressed the limit between the two zones in terms of the wave steepness. Higher than the critical steepness the waves are erosive or destructive, while constructive wave action was associated with wave steepnesses below that of the critical steepness. Associated with the particular type of wave action have been the resultant profiles sometimes termed normal and storm profiles. The critical wave steepness between the two actions has long been the subject of study which have shown the limit to be primarily a function of the waves (expressed by the wave steepness) and a material characteristic.

Iwagaki and Noda (1963) first noted that the effect of material size on the beach process was remarkable, at least within a certain range of wave height to median sediment size ratio. They presumed that the reason for this was due to a change in the mode of transport of the breakers. Small values of H/D_m (wave height to median sediment diameter) were associated with bed load transport, while that of large values of H/D_m were associated with suspended sediment. Large scale experiments by Saville (1957) showed that under the correct conditions, 'storm' beaches having bars appeared to be in equilibrium even when the wave steepness was small. Zwamborn and Van Wyk (1969), as a result of model studies, concluded that the deformation of a beach was a function of the wave steepness and an expression containing the wave height and the sediment characteristics. By including in the latter term an expression for the settling velocity of a particle, they took into account not only the particle size but also the specific weight. Nayak (1970) derived a similar expression for the sediment characteristic directly in terms of the particle size and specific weight. Figure 12 provides a comparison between some of these critical values.

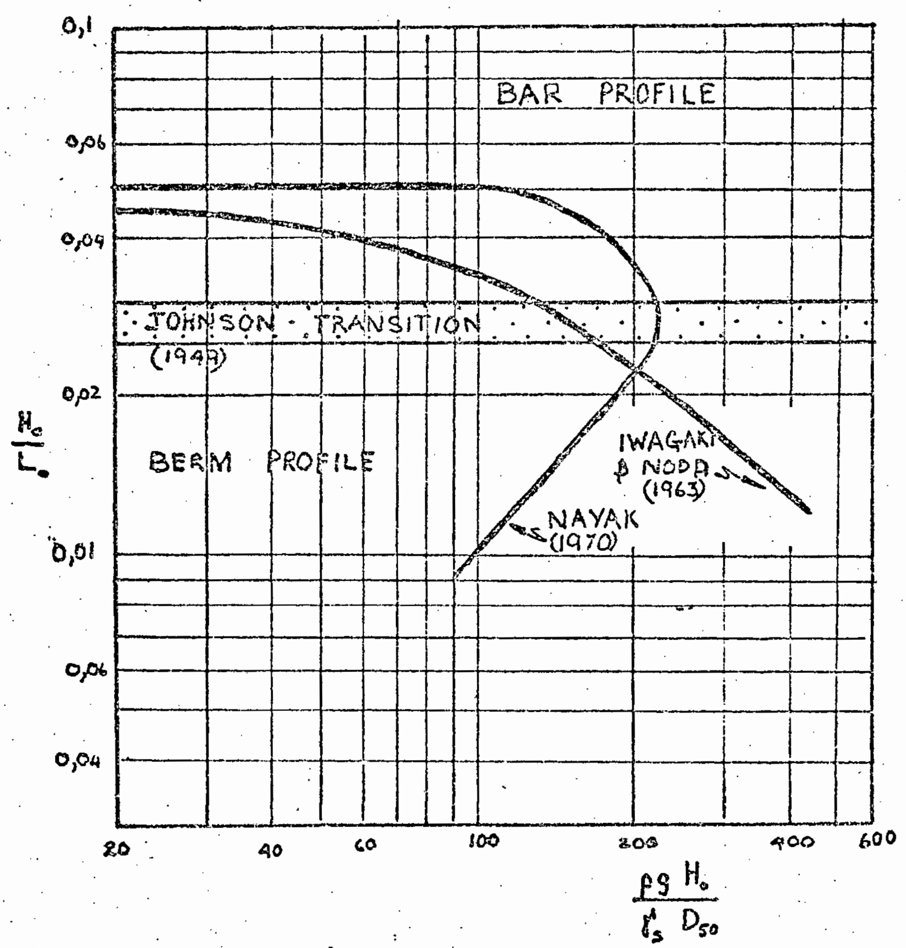


Figure 12 Profile types as a function of wave steepness and a beach sand characteristic

(after Nayak - 1970)

4.4 PROFILE RESPONSE

The variability of beach profiles have been studied by repeated observations of the profile over time spans of varying duration. The interpretations of the variations have been attempted in terms of the controlling process variables measured at the same time as the profiles were recorded. The type of short term fluctuations on the beach has been found to be dependent on the variability of the waves. The extent of the fluctuations is however dependent on the nature of the beach material. As already mentioned, the steep gradient of coarse beaches causes the greater mobility and variability due to the concentration of energy dissipation over a narrower zone.

Many beaches have been found to experience seasonal or cyclic changes associated with similar changes in the wave climate. Certain Californian beaches have exhibited this tendency, reaching a maximum width in summer under calm wave conditions while eroding in winter as a result of the more destructive winter wave conditions. Long term changes may also occur when a beach undergoes a more continuous erosion or accretion sometimes induced by artificial beach structures, or occasional severe storms. The former is associated with the longshore movement of material, while the latter is associated with the movement of material normal to the shore, or both.

Harrison (1969) produced empirical equations relating short term changes in profile to process variables. Harrison used linear multiple regression to provide prediction equations which naturally only apply to the type of beach analysed and conditions during measurements. The analysis as a whole indicated the importance of taking the ground water conditions into account in explaining the response of the foreshore to the processes operating upon it.

Field observations on Del Monte Beach, California by Thompson and Harlett (1969) were made on the inter-tidal part of the beach over a period of sixty days by making daily observations of the beach level on fixed poles. Wave conditions were measured over the same period. The site itself was selected on a stretch of beach subject to normal wave approach only, in an attempt to eliminate the effects of longshore movement from the study. The writers found that wave power operated in the same way as wave steepness, with beach cutting associated with an increase in wave steepness or power, or both. It was noted that cutting often occurred on the upper beach and filling on the lower beach and vice versa. Erosion was also found to take place more rapidly than accretion.

Field observations of profile variations in the surf and off-shore zones are difficult to obtain. Hydrographic surveys using echo sounder and leadline techniques may be used with a reasonable degree of success in the off-shore zone but less favourably in the surf zone. Man made piers such as the Scripps pier facilitate measurement in the surf zone, as has the use of aerial photography. Periodic and regular aerial photographs taken under favourable conditions may distinguish features of the submarine topography. Use of the latter technique by Hom-ma and Sonu (1963) led to their observations on the rhythmic pattern of longshore bars. Rocket beach profilers have also been developed by the CSIR. (This technique incorporates a pressure transducer which is shot by means of a rocket over the breaker zone. The transducer is then hauled back to shore with the pressure recording giving the depth, and hence the profile).

The theories of profile equilibrium discussed in the previous section consider basically a two-dimensional relationship between wave action and topographic response. The validity of these theories holds if waves arrive normal to the shore and nearshore contours are straight and parallel. In a study dealing with the behaviours of subaqueous beach profiles at Nags Head, North Carolina, Sonu and Russell (1965) first suggested that abrupt changes in profile configuration observed in a stationary traverse could be attributed to migration of sand waves across this traverse as a result of oblique waves. The writers reported that when this effect was suspected the beach change observed in the transverse was many times more pronounced than the change associated with higher waves arriving normal to the shore. These changes were restricted to the sub-aerial zone. Further analysis by Sonu (1968) led to the conclusion that the collective movements of sediment occurred in the form of two types of sand waves in the in-shore zone. The first type of sand wave he attributed mainly to wave orbital motion. It was generally observed that these bar-type sand waves were formed during a period of wave decay immediately following storm activity. Sonu noted that the effect of bar-type sand waves could be traced up on to the foreshore slope. The second type of sand wave Sonu attributed to longshore currents. In the absence of accepted terminology the writer termed them 'cusp type' sand waves. These sand waves developed periodic crests and troughs in a series along the shore and tended to migrate in the direction of existing along shore currents. Unlike bar-type sand waves, the crests of cusp type sand waves were found to be orientated obliquely to the shoreline, often extending across the entire width of the surf zone.

The implications of the existence of these different forms of sand waves are obvious. With reference to bar-type sand waves, it is suggested that the response of a beach profile to wave action depends not only upon wave characteristics but also upon the characteristics of an existing profile configuration. (Sonu has classified six major types of profiles). The presence of cusp-type sand waves would similarly cause variability in beach profiles along the shore. The migration of these sand waves may well produce pulsational transfer of material along the shore. Based on conclusions drawn from their investigations, Sonu and Young (1970) have proposed that the process of beach change is stochastic in nature, with the resulting beach profile partly a function of the proceeding profile. This theory awaits verification. The writers hope that the basic concept of the model will explain beach changes in various types of world coasts.

The author notes that while profile response might well be affected by the existing profile, full development of the equilibrium profile would seem to be independent of the degree of critical 'unequilibrium', provided sufficient time may elapse for the full development of the equilibrium profile. The author also suggests that the resultant equilibrium profile is furthermore dependent on the relative position of the profile on a cuspate foreshore, which in itself should be recognised as a feature of the equilibrium condition. This cuspate condition also seems to be a feature of the nearshore circulation pattern associated with a normal wave attack. These observations are the result wave model studies and will be discussed in further detail in Part 2.

CHAPTER 5

THE LONGSHORE SYSTEM

5.1 INTRODUCTION

Longshore sediment transport, commonly termed littoral drift, is defined as the movement of material along the shore in the nearshore zone by waves and currents. Each of the four preceding chapters have made mention, in one form or another, of longshore sediment transport.

The rate and direction of littoral drift is dependent on the many factors associated with the nearshore zone, amongst others wave energy, currents, tidal range, availability of littoral material, sediment size characteristics and beach slope. Neither the precise mechanism of transport nor the interaction of the various factors is clearly understood (Mongul et al - 1970). Barjorunas (1970) goes as far as stating that general agreement between the various investigators seems to end at the point where oblique waves are considered to be the main cause of and account for the major portion of littoral drift. Longuet Higgins (1971) however, as already stated, claims that the mechanics of the generation of longshore currents by breaking waves has now been laid bare, and at least a semi-quantitative theory for the prediction of littoral drift is available.

The proper design of coastal structures, improvements or controls depends crucially upon an accurate estimate of the amounts of sand supplied to or lost from the shore region. The most significant portion of the sand supply appears to be littoral drift. The littoral drift rate and direction are the result of the summation of the forces imposed on the beach by the sea, and the reaction of the beach to these forces. The drift rate and direction are thus also important to the design of shore protection and navigation projects.

5.2 ASSESSMENT OF LITTORAL DRIFT

Assessment of littoral drift or littoral drift rates along a coast may be extremely difficult. Superimposed on the longshore transport is the movement of material normal to the shore. Most assessment methods are based on the assumption that the on/off-shore movement of material is limited to the in-shore zone (neglecting for the moment the material carried off-shore in suspension by rip currents and the constant diffusion of finer particles seawards). All data to date indicates that net sediment movement in the off-shore zone is predominately landwards. The constant diffusion of finer particles towards the sea cannot account for major recessions in a coastline. Off-shore movement in the in-shore zone by destructive waves without a long-shore current would result in the accumulation of material in the breaker zone and the establishment of system of dynamic equilibrium. Similarly, accretive tendencies of certain beaches or sections of beaches, while relying on the effects of constructive wave action, cannot solely depend on the off-shore zone as their source of material supply. Erosive or accretive tendencies in the long term indicate the effects of the longshore movement of material, in the first case as a mechanism for removing material from an area, and in the second case as a source of supply. Galvin recorded in 1970 that the best way of predicting littoral drift rates is to adopt the best known rate from a nearby site, with modifications based on local conditions. This method, if applicable, relies largely on engineering judgement. (The writer does not indicate how the data for the 'other' site is to be obtained, but the mere advocation of such a method is interpreted by the author as an indication of the uncertain accuracies inherent in the methods which follow).

If the rates from nearby sites are not known, Galvin states that the next best way to predict drift rates is to compute them from data showing historical changes in the topography of the littoral zone. The primary sources of supply for these direct measurement techniques come from charts, surveys and dredging records. Aerial photography for this purpose, whether on a short or longer term basis, has been used by Longfelder et al (1970) and is similarly advocated by Silvester (1971). Historical observations must be assumed to be of questionable accuracy, while extensive field measurement programmes are expensive and not always practical. The use of tracer techniques as an indirect method of studying the longshore sand movement have been developed by various investigators using radio-active or fluorescent tracers. Price (1969) has shown that some of the interpretations of tracer experiments may give misleading results due to the irregular dispersion so characteristic of tracer experiment results.

As littoral drift is attributed mainly to the effect of waves approaching a coastline at an angle to the shore, the application of wave refraction techniques and the computation of the longshore wave energy flux has resulted in the establishment of various empirical relationships between the two parameters. The success of the method depends on the adequacy of the data on the wave climate and on the reliability of the relationship between the wave energy and sediment transport.

Various investigators have attempted to establish relationships between the actual longshore current, induced by whatever means, and the littoral drift rate by means of sediment transport theories. These relationships are usually expressed in terms of the longshore current velocity.

5.3 LONGSHORE CURRENTS

Many different relationships have been developed for the longshore current velocity in the surf zone. Whereas some of the relationships were obtained solely by the empirical correlation of data, most were based on the longshore energy flux induced by wave action and were developed from considerations of the conservation of energy, momentum and mass (continuity) respectively. Figure 13 illustrates the widely varied contributions of the component variables contained in twelve known formulae of longshore current velocities.

AUTHORS	FORMULA	POWER OF COMPONENT VARIABLES: Wave Height H	Period T	Incidence θ	Celerity C	Bed Slope α, arctn m	Friction f	BASIC SCHEME OF ANALYSIS
Putnam-Munk-Traylor (1949)	$\frac{a}{2}\left[\left(1+\frac{4C_b}{a}\sin\theta_b\right)^{\frac{1}{2}}-1\right]$ $a = 8m\Omega_b\cos\theta_b/fd_bT$	1 •	1 •	$\cos\theta_b$ • and $(\tan\theta_b)^{\frac{1}{2}}$	$\frac{1}{2}$ •	$(\tan\alpha)^1$ •	Darcy Weisbach coeff. −1	Momentum Balance, Solitary Wave.
"	$\left(\frac{4\,sm\,C_bE_b\sin 2\theta_b}{f\rho d_b}\right)^{\frac{1}{3}}$	$\frac{2}{3}$		$(\sin 2\theta_b)^{\frac{1}{3}}$	$\frac{1}{3}$	$(\tan\alpha)^{\frac{1}{3}}$	$-\frac{1}{3}$	Energy Balance, Solitary Wave.
Inman-Quinn (1951)	$\left[\left(\frac{1}{4x^2}+y\right)^{\frac{1}{2}}-\frac{1}{2x}\right]^2$ $x = 108.3\,H_b\tan\alpha\cdot\cos\theta_b/T$ $y = C_b\sin\theta_b$	−2 •	2 •	$(\cos\theta_b)^{-2}$ •	1 •	$(\tan\alpha)^{-2}$	$\sim V^{-\frac{3}{2}}$	Momentum Balance.
Nagai (1954)	$\frac{1}{6}H_bC_b\left(\sqrt{1+\frac{16\sin\theta_b}{KH_b}}-1\right)$ $\times\tan\alpha/kd_b$	$\frac{1}{2}$ •		$(\sin\theta_b)^{\frac{1}{2}}$ •	1	$(\tan\alpha)^{\frac{1}{2}}$ •		Momentum Balance, Oscillatory Wave.
Brebner-Kamphuis (1963)	$8H_0^{\frac{2}{3}}/T^{\frac{1}{3}}\,[\sin 1.65\,\theta_0$ $+\,0.1\sin 3.30\,\theta_0]$ $\times(\sin\alpha)^{\frac{1}{3}}$	$\frac{2}{3}$	$-\frac{1}{3}$	$(\sin\theta_0)^1$ •		$(\sin\alpha)^{\frac{1}{3}}$		Energy Balance.
"	$14H_0^{\frac{3}{4}}/T^{\frac{1}{2}}\,[\sin 1.65\,\theta_0$ $+\,0.1\sin 3.30\,\theta_0]$ $\times(\sin\alpha)^{\frac{1}{2}}$	$\frac{3}{4}$	$-\frac{1}{2}$	$(\sin\theta_0)^1$		$(\sin\alpha)^{\frac{1}{2}}$		Momentum Balance.
Galvin-Eagleson (1965)	$kgT\tan\alpha\cdot\sin 2\theta_b$	0	1	$(\sin 2\theta_b)^1$		$(\tan\alpha)^1$	$K \doteq 1$ by lab. experiment.	Momentum Balance.
Inman-Bagnold (1962)	$\frac{2}{\sqrt{3}}\left(\frac{H_b}{d_b}\right)^{\frac{1}{2}}T^{-1}\cdot\sin 2\theta_b$ $\times(\tan\alpha)\cdot l$	0 •	−1	$(\sin 2\theta_b)^1$ •		$(\tan\alpha)^1$		Continuity, Regular Rip. Interval = l.
Bruun (1963)	$\frac{0.38\,H s(1/3)^2\cdot\cos\theta_b\cdot l}{AT}$	4 • $H s(\frac{1}{3})$	−1	$(\cos\theta_b)^1$		$(\tan\alpha)^{-1}$		Continuity, Regular Rip Outflow, Spectral Wave.
"	$C\sqrt{R\,\frac{Q_b\cdot\sin 2\theta_b}{2r\cdot L_b}}$	1 $H s(\frac{1}{3})$	−1 •	$(\sin 2\theta_b)^1$		R: Hydraulic radius. r: Bar separation.		Continuity, Straight Single Bar.
Sitarz (1963)	$0.125g\left(\frac{R}{D}\right)^{\frac{1}{6}}H^2\,T/S$	2	1			S: Cross-sectional area of surf zone.		Continuity, Sediment Entrainment.
Shadrin (1961)	$\frac{c}{\cdot}\sqrt{1.11\frac{d}{T}\sqrt{gd}\left(1-\frac{l_1}{l_2}\right)}$	$\frac{3}{4}$ •	$-\frac{1}{2}$			l_1, l_2: Distances between bar and shoreline		Surface gradient lunate bar, Rip outflow.

• Approximate; subscripts o and b for deep- and shallow-water equivalents, respectively.

Figure 13 Comparison of formulae for longshore current velocities (after Sonu et al - 1966)

The method of computation of littoral drift based on longshore currents is based on two basic assumptions. Firstly, the longshore current velocity resulting from the waves approaching the coast obliquely should be calculated. Secondly, starting from the longshore current and taking into account the effect of the wave motion on the bed shear due to this current, the littoral drift is calculated. For this calculation it is necessary to take suspended as well as bed load into account. The advantages of the longshore current approach over existing littoral drift formulae, based on the assumption that the littoral drift is some function of the wave energy flux towards the shore, are many. Bijker (1968) has pointed out that the longshore current approach shows more clearly the origins of possible inaccuracies in the results, and that further study may solve the unknown points. Other advantages include reference to the grain size and slopes of beach and foreshore, as well as the fact that it is also possible to take into account a longshore current not generated by the waves.

Longuet Higgins (1971) has summarized recent advances as being based on two fundamental groups of ideas contained within the framework of basic assumptions as stated in the preceding paragraph. These are the concept of radiation stresses (momentum flux) in water waves, developed by Longuet Higgins and Stewart, and Bagnold's theory of sediment transport. It is assumed that in the surf zone there is a balance of forces, such that

$$\text{driving force} + \text{bottom friction} + \text{lateral friction} = 0 \qquad (28)$$

Assuming firstly that lateral friction may be ignored, equation (26) may be developed to derive an expression for the longshore current velocities as a result of oblique wave attack. The driving force, or onshore component of energy flow may be expressed as

$$P_x = w\, H^2\, c_g (\cos\theta)/8 \qquad (29)$$

If in the breaker zone it may be assumed that $H = 0{,}8\ d$; $\cos\theta = 1$ and $c_g = \sqrt{gd}$, it may be shown that the change in S_{xy} over a short distance is given by

$$f = 0{,}2\ w\ d\ i\ \sin\theta \qquad (30)$$

where i is the beach slope. Equation (30) is the average thrust on a layer of water in the surf zone between two vertical planes.

From frictional considerations, the second term in equation (28) may be expressed in terms of Chezy's law

$$f = 2 C \rho u_{max} V/\pi \tag{31}$$

where C is a Chezy roughness coefficient (about 0,01),

V is the local longshore velocity of the water, and

u_{max} is the maximum orbital velocity (estimated at 0,4 $\sqrt{gd}$)

Equating stress equations (30) and (31) and expressing them in terms of the local longshore velocity, leads to

$$V = \frac{\pi}{4} \cdot \frac{\sin \theta_o}{c_o} gd \frac{i}{C} \tag{32}$$

When the bottom slope is constant equation (32) represents a triangular velocity distribution, zero at the shoreline rising to a theoretical maximum at the breaker line of

$$V_{max} = \frac{\pi}{4} \frac{\sqrt{gd}\ i \sin \theta_b}{C} \tag{33}$$

This velocity distribution is unrealistic, as lateral friction will take place, but the order of magnitude appears correct.

Lateral friction, or horizontal mixing, was first introduced into the theory of longshore currents by Bowen (1969).

Estimates of the effect of lateral friction on the theoretical triangular distribution of longshore velocity as expressed by equation (33) vary between a reduction of 1/6 to 1/2. The expression for horizontal mixing as proposed by Longuet Higgins takes the form

$$\mu_e = N \rho |x| \sqrt{gh} \tag{34}$$

where μ_e is the horizontal eddy viscosity and N a dimensionless constant. Equations (28) and (34) yield a family of solutions. These however tend to over emphasize the effects of lateral friction and bottom friction outside the surf zone. (The general form of the longshore velocity distribution with and without lateral friction may be seen in Figure 4).

The author feels that while the calculation of littoral drift based on the longshore current velocity may yet provide a practical means of assessment, at present the method makes use of too many unverified assumptions and simplifications. Along beaches where wave action is attributed to be the major factor in littoral drift, the purely empirical approach based on wave thrust should be favoured, should no direct method of assessment be available.

5.4 . EMPIRICAL ESTIMATES OF LITTORAL DRIFT BASED ON LONGSHORE ENERGY FLUX

There have been a number of field studies of the relationship between what was called 'longshore wave energy flux' and sand transport along beaches, but the difficulties involved in obtaining wave characteristics, especially the angle of approach, and the inherent difficulties involved in surveying water depths in the surf zone have made the results difficult to assess. Wave refraction techniques have in general not been wholly successful in prèdicting breaker angles, probably due to the variability of nearshore topography, and the apparent 'batch' process of transportation (Longfelder et al - 1971). The off-shore zone changes sporadically, or even continually, so that spot hydrographic measurements may be quite deceiving. (This is supported somewhat by Sonu et al (1966), in their attempts to relate measured longshore currents to wave energy).

Field measurements have been supplemented by the use of model studies in three-dimensional wave tanks, in which wave characteristics may be controlled and kept constant. Angles between the wave generator and beach may be adjusted at will, but the effects of refraction must still be considered. This is not easy to do even by photographic methods. Models are also subject to scale effects which usually do not allow the formation of a substantial surf zone. This probably accounts for the dominant aspect of swash zone sand transportation in some model studies (Ingle - 1966).

The use of empirically derived formulae relating the littoral drift rate to the local wave conditions can thus once again only be recommended when no direct measurements of sand transport are available. Calculation of the needed wave statistics follow an established routine, even though gross simplifications and assumptions are required, and there is a tendency amongst investigators to apply this method where possible. The degree of certainty in the final product is however, considerably less than with direct measurement techniques, and are commonly associated with a factor of two.

5.5 EMPIRICAL ESTIMATES OF LITTORAL DRIFT - HISTORICAL REVIEW

Eaton (1950) first reported the results of the U.S. Corps of Engineers, Los Angeles District, who experimented with the concept of wave work. Caldwell (1948 and 1949) had previously conducted field measurements at Anaheim Bay California. Hydrographic surveys of the area indicated the rate at which sand was moved in or out of a catch basin. Watts (1952) studied littoral transport at South Lake Worth Inlet, Florida. Periodic measurements were made of the volume of material in a catch basin which was considered to represent the littoral drift. Both Watts and Caldwell then correlated their respective littoral drift data with observations of the wave characteristics after the periods of study. They expressed their relationships in the form

$$Q_s = x P_\ell^y$$

where x and y were dimensionless coefficients, and P_ℓ was termed the 'longshore component of wave energy flux', or 'longshore wave power'.

Savage (1962) presented a review of laboratory and prototype measurements of the littoral drift rate. The review covered laboratory studies by Krumbeim (1944), Saville (1950), Shay and Johnson (1951), Savage and Vincent (1954) and Savage (1959). The principle contribution of the model studies were in determining the factors affecting the littoral drift rate. Saville was the first to show that the drift rate did not vary linearly with the deep water wave steepness. Shay and Johnson showed that the drift rate varied with the angle of wave incidence. Savage found that a comparison of laboratory data with that of field work indicated a consistently lower drift rate for the former. He surmised that the considerable data scatter was indicative of the effects of other variables, as yet undefined, on the drift. Figure 14(a) compares the results of field and laboratory data, expressed in metric units. Caldwell's relationship is based on field data extrapolated in the laboratory zone, while Savage developed his relationship as the line of best fit for all data, both field and laboratory.

Inman and Bagnold (1963) pointed out that the existing relationships summarizing the available facts were empirical only and thus dimensionally incorrect. Bagnold suggested that the transport rate could be corrected from a volume to an immersed weight basis by expressing the littoral drift rate as

$$I_\ell = (\rho_s - \rho)\ ga'\ Q_s \text{ constant } x\ P_\ell$$

(see equation (24)). The new constant would be dimensionless and independent of the unit system used, providing the units were consistent.

Bruun (1963) noted that it was not likely that any fixed angle existed for maximum littoral drift, as the angle would vary with wave, topographic and material characteristics (for instance, breaker angles of 30° and 40° can correspond to deep water wave angles of 45° to 70°, depending on wave and depth characteristics).

Inman, Komar and Bowen (1968) conducted simultaneous field measurements of the energy flux of breaking waves and the resulting littoral drift in the surf zone. The littoral drift was calculated by means of a fluorescent tracer programme. From their results the best fit curve, expressed in the manner suggested by Inman and Bagnold could be represented by

$$I_\ell = 0{,}77\ P_\ell$$

(see equation (27)). For an equal drift rate the writers found that, for what was at that stage referred to as the longshore wave power, was about half of that reported for previous studies. This discrepancy they described as the result of the erroneous use of the significant wave height for the earlier studies. The writers expression for wave power was based on H_{rms}, which they suggested is also the wave parameter measured in the laboratory for simple waves of steady amplitude. Use of H_{sig} in place of H_{rms} gives a wave power that is erroneously high by a factor of 2.

It was at this stage that Longuet Higgins (1971) reported that previous use of the term 'longshore power' was unfortunate, and advocated the substitution of the term by the product of the total longshore wave thrust and the wave phase velocity at the breaker line. Figure 14(b) shows the littoral drift rate expressed as a function of F c_b.

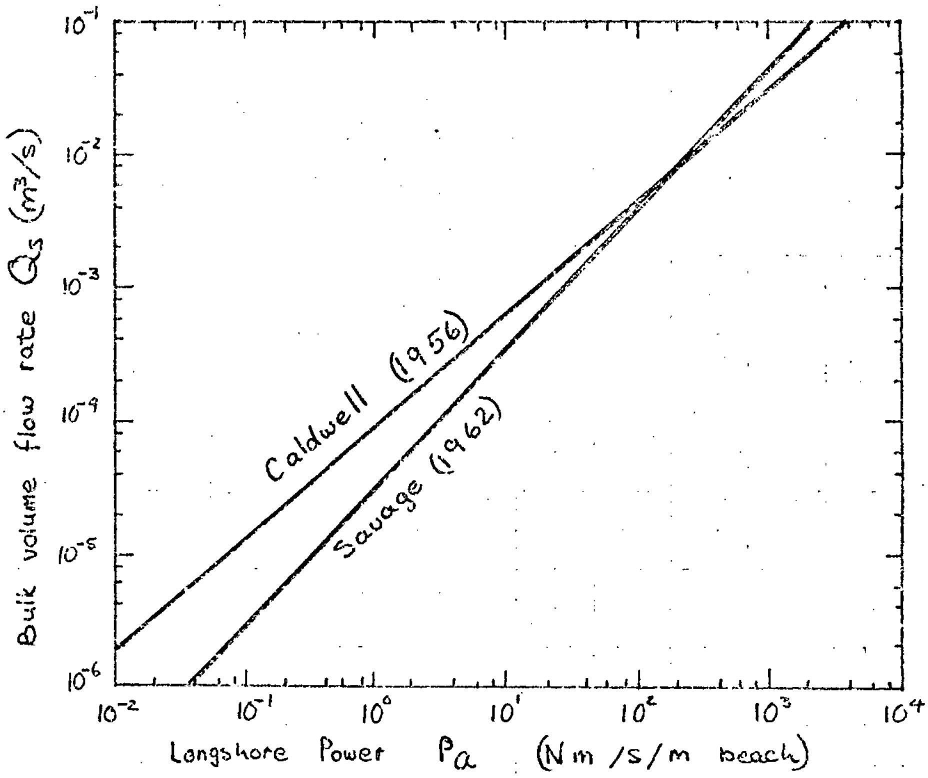


Figure 14a Littoral drift in terms of longshore wave power (metrification by author)

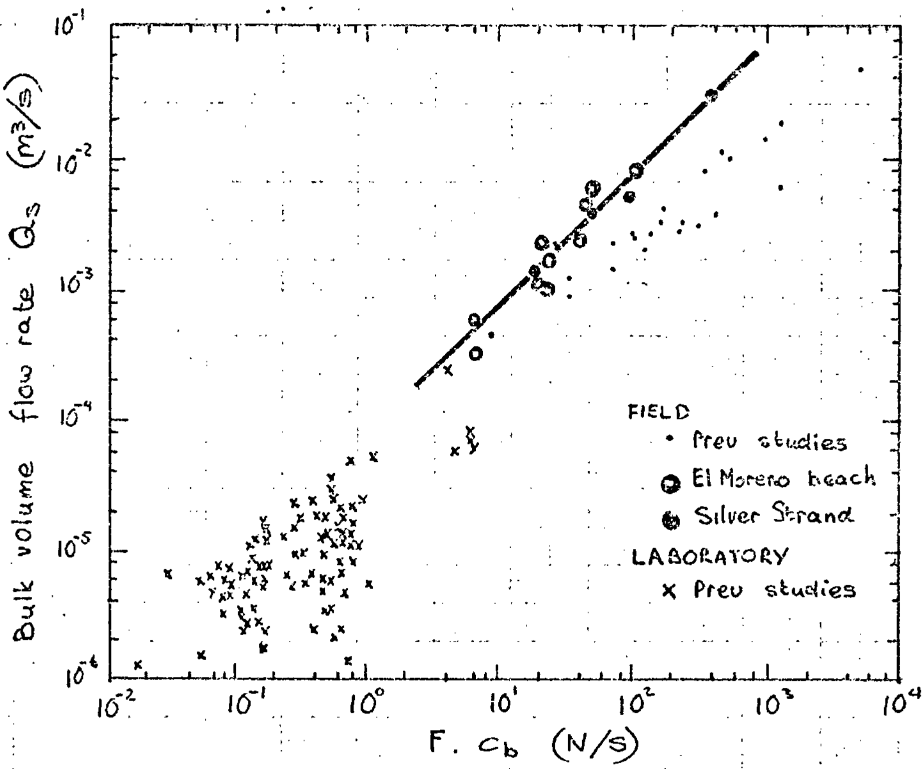


Figure 14b Littoral drift in terms of total longshore thrust and breaker velocity. Experimental verification of the relationship $Q_s = 0{,}77 \times 10^{-3} F\ c_b$ (after Komar and Inman - 1970)

PART TWO

THE LABORATORY PROGRAMME

CHAPTER 6

OBJECTIVES OF THE PROGRAMME

NORMAL WAVE ATTACK PROGRAMME

a. The establishment and recording of equilibrium beach profiles under conditions of controlled wave attack.

b. The interpretation of resultant beach profiles as a function of the controlling wave characteristics.

c. The comparison of recorded beach profiles created under conditions of different wave attack in terms of resultant beach movement.

d. A study of phenomena observed during the course of the experimental programme.

ANGLED WAVE ATTACK PROGRAMME

A study of the suitability of the model for the measurement of littoral drift due to angled wave attack.

As the experimental programmes for normal and angled wave attack differed considerably from each other, the procedures and results of each are presented separately.

CHAPTER 7

THE NORMAL WAVE ATTACK PROGRAMME

7.1 APPARATUS

The experimental programme was conducted in a model wave basin in the Civil Engineering Laboratories of the University of Cape Town. The basic layout and dimensions of the basin and wave generating mechanism are given in Figure 15.

An oscillating type wave maker powered by a variable speed motor fixed in position along one end of the basin was used for wave power generation. The wave generating mechanism was developed by the University of Cape Town and several innovations on the apparatus are believed to be unusual. The oscillating wave panel or drum was provided with a vertical face towards the direction of wave generation, while the rear face formed a circular arc with the pivotal rod as circle centre. In this manner disturbance of the water behind the paddle, and hence reflection, was reduced to a minimum as the rear face of the paddle sheared through the water.

The power transmitted to the wave paddle was provided by a variable speed motor. By making use of an independent motor and gearing system the main motor could be programmed to cycle its output between chosen speed limits or to run at any constant speed. The lever and gearing system between the main motor and wave paddle allowed for a variable stroke which could again be cycled automatically or manually between chosen limits, or pre-set at any desired stroke.

The wave basin was equipped with an automatic tide-producing mechanism which could similarly be used as a control of the water level in the basin to any desired depth. The mechanism operated in a stilling well joined to the basin by means of a pipe along the model floor.

The wave generating apparatus allowed for the formation of waves of varying period and power. By altering the height of water in the basin, the power transmitted to the water by the oscillating paddle could similarly be altered. No direct method of producing a wave of a certain height was available.

Throughout all experiments fresh water was used in the wave basin. The sand used for all model tests was kept constant throughout the range of

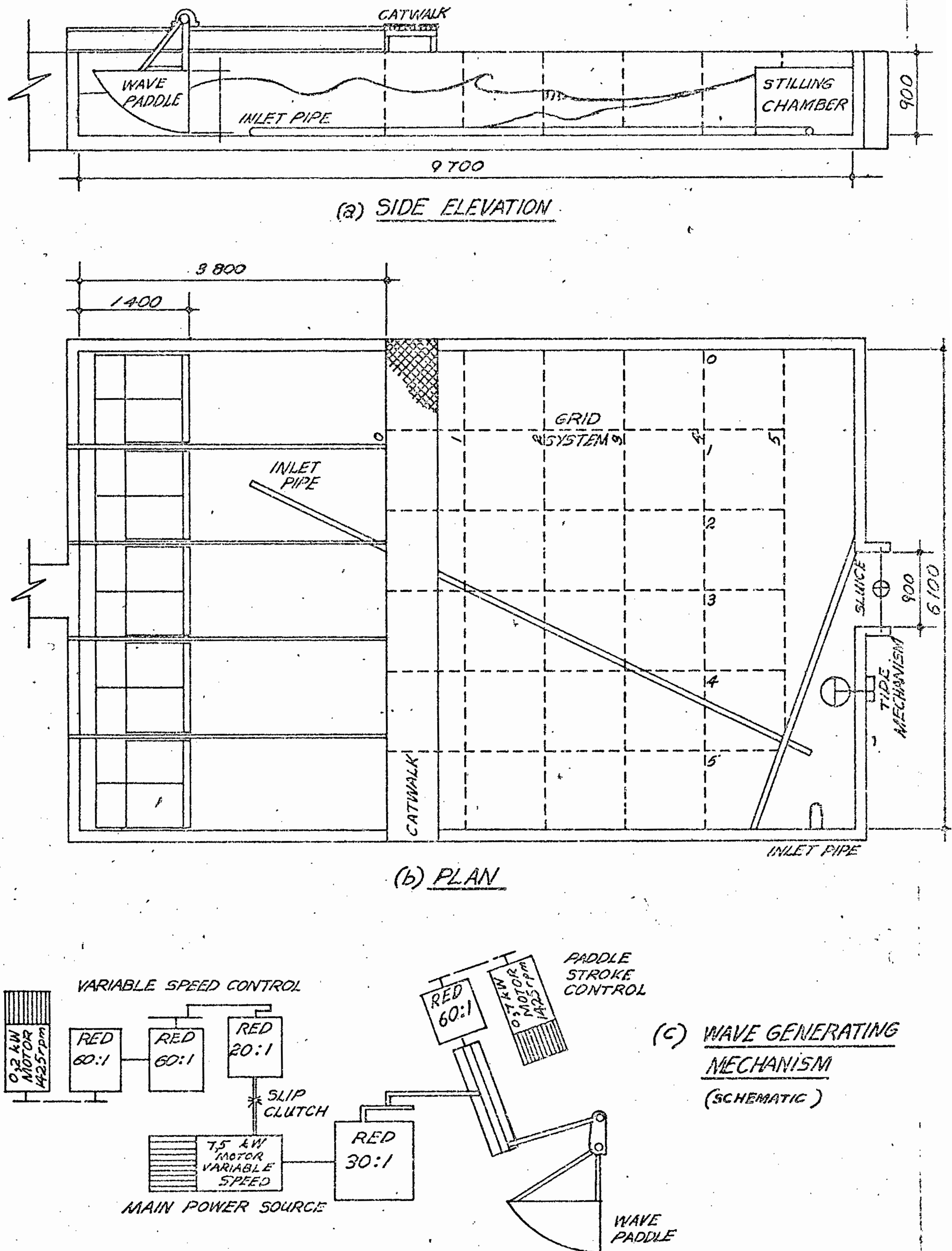


FIGURE 15

Model wave basin and wave generating mechanism.

experiments. A well graded beach sand with all material larger than 1 mm screened off was used. A grading analysis of the sand is presented in Appendix A. Quartz sand was used with a relative density of 2,65.

7.2 EXPERIMENTAL PROCEDURES

With the wave generating apparatus adjusted to preselected periods and strokes, and the water level in the wave basin held constant at selected levels, a series of tests was run to cover what was considered a representative range of wave conditions bounded by the limitations of the wave generating mechanism.

Recordings were made of the wave characteristics in the following manner. Throughout the duration of each test the mean water level was checked regularly and at frequent intervals. All still water level and wave measurements were related to a preselected vertical datum which was kept constant throughout the whole range of experiments. Wave period was determined by stopwatch and gauged from the oscillations of the wave generating paddle, taking an average of 10 strokes. These values were checked frequently. The wave height was measured by means of a point gauge as far seaward of the break point as possible. Initial wave height recordings were done on an electrical resistance type meter, coupled to an automatic plotter, but results were not successful. Wave height recordings were always related to the height of the wave crest and trough above and below the still water level and were coupled to readings of the water depth at point of measurement. Wave heights were measured at frequent intervals to gauge changes. Once the beach was assumed to have reached its equilibrium state, the wave height recordings were continued and then averaged out, to give a representative value.

Measurement of the resultant beach configuration was made by means of a grid system extending across the width of the model and between the seaward side of the break point and the furthest landward extent of the swash zone. The measurements were made at half metre intervals from the preselected datum, and in each case related to the still water level in the tank. The positions of the measuring grid were marked by small diameter brass rods driven into the beach sand at the required positions. These positions were unaltered throughout the experimental programme. Beach measurements were made at representative positions throughout the course of each test run, both above and below the still water line, to determine the stage at which equilibrium was reached. After completion of each test, the basin was drained, allowing the precise measurement of the beach at each point of the grid.

In order to present a profile for each experimental run, which would give a representative value of the average elevation along the length of the profile suitable for comparison with the other tests, a 'tank average' value was calculated. This was necessary due to distortions on the beach as the result of cusp formation. Expressing this value as the mean value per unit breadth along the length of a profile, it was possible to calculate the volume of material per metre width of the profile relative to the still water level.

For each individual test, standard visual observations were made throughout the extent of the test. The type of breaking wave and its position were recorded, as well as the swash zone flow pattern. Observations of the beach face itself allowed the recording of a visual assessment of the building up or destruction of the existing beach form, and the formation and characteristics of any visible secondary beach forms.

7.3 TIME TO REACH EQUILIBRIUM

The whole experimental basis of producing equilibrium profiles was based on the premise that a beach, subject to a constant wave attack, would adjust its profile to suit the wave conditions, regardless of the form of the original profile. All tests were repeated in such a way that the initial beach profile at the beginning of each run was a random variable.

The time required to reach equilibrium was initially gauged from a literature survey of previous model experiments. This period appeared to be approximately 20 hours. Successive measurements of profile changes during tests verified that the time to reach equilibrium varied inversely with the power of the impinging wave, and averaged between 12 and 20 hours for the wave power range studied. For the most powerful waves, the profile reached equilibrium after 5 hours but this period was generally increased considerably in order to ensure the equilibrium state below, as well as above, the mean still water level.

Initial swash zone changes were remarkably rapid - generally within an hour the beach face had taken up its new shape.

7.4 SECONDARY BEACH FORMS

The presence of cuspate formations across the breadth of the water basin - to a larger or smaller degree - was found in all the experimental runs subject to the normal wave attack programme. The formations appeared to move about at random, but retained their basic dimensions for each particular test. On each recorded profile where a cuspate formation was found to be significantly noticeable, both a horn as well as a rip profile recording was superimposed on the tank average profile for later comparison.

7.5 EXPERIMENTAL DATA

7.5.1 Measured wave characteristics

a. Wave period

The experiments were conducted between a range of 2,1 and 3,8 seconds. (It was found that shorter waves overran the beach, while longer waves created an insignificant surf and swash zone).

b. Wave height

The experimental range of wave heights varied between 40 mm and 200 mm before breaking. Depth of water at point of measurement was also recorded.

7.5.2 Calculated wave characteristics

Calculation of the deep water wave length was made by applying equation (6). Using the relationship established in equation (8), the deep water wave height could be calculated (the solution of equation (8) is available in Table form as calculated by Wiegel). The deep water wave steepness, as well as the local steepness, were now available. The deep water wave power was calculated from equation (13). (Under conditions of normal wave attack, ignoring second order transmission losses, the deep water wave power will be equal to the wave power just prior to breaking).

7.5.3 Data presentation

i) Appendix A presents the beach sand grading as used throughout the experiments.

ii) Appendix B is a record of the profiles studied together with all relevant measured and calculated data.

iii) Appendix C presents plan views of a typical bar and berm profile respectively.

iv) Appendix D presents a record of relevant photographs depicting various aspects and features of the test programme.

v) Appendix E presents a Table in chronological order of all tests carried out, together with relevant measured data.

7.6 PROBLEMS ENCOUNTERED DURING THE EXPERIMENTAL PROGRAMME

Before reviewing the results and interpretations of the normal wave attack programme, it is necessary to record various problems encountered with the experimental programme, and the manner in which they affected both the programme and the interpretation.

7.6.1 Constant still water level

Considerable effort had to be exercised to maintain a constant water level in the wave basin. At the beginning of a run this was expected due to water absorption of the sand while attaining a saturated state, but after a number of hours persistent drops in water level indicated leakage losses from the tank. To compensate for this, the tide generating apparatus was set at the desired level and switched on, theoretically thus maintaining the water at a constant level. Still water levels were monitored during a run by means of a point gauge behind (seaward) of the wave paddle, which, by nature of the paddle design, formed a type of stilling pool. Minor changes in water level were found to persist during each run, however, and differences in water levels were also encountered between the various runs.

7.6.2 Wave height recordings

Mention has been made earlier of the abortive attempt to record wave heights by means of a continuous record electronic recorder. Wave heights were subsequently measured by means of a point gauge. Initially, and while a wave attack was reshaping a foreign profile, wave heights varied drastically. Once the beach had been adjusted to the equilibrium profile however, the wave heights were found to remain fairly constant. As variations still occurred, the recorded wave heights tended to be the mean of a series of readings.

Of paramount importance while measuring wave heights, was the recording of water depth. These recordings were made by metre rules. All measurements were made in the ripple zone, thus minor differences frequently occurred due to the ripples themselves. Wave heights and water depths could not always be measured at the identical spot for different experimental runs, due to the varying position of the breaking wave under different sets of conditions.

7.6.3 Reflection

The model wave tank made no provision for the absorption of reflection. With a normal wave attack, it was reasoned that reflection from the tank sides would be minimal, as would be reflection from the beach face, once the equilibrium profile had been allowed to establish itself.

7.6.4 Cuspate formations

To a larger or lesser degree, cusps or sections of cusps formed under every set of wave conditions tested. (In cases where these formations were especially prominent, profiles of the horns and embayments were recorded together with the tank average - see Appendix B).

In spite of the cusps, the equilibrium beach formations recorded were exceptionally regular, the exception being in cases where the cusp wave length appeared to be close to or larger than the width of the tank. Because of the prominence of cusps in the experimental programme, the author refrained from including any graphical representations of beach gradient versus wave characteristics in this study. It was found that the beach gradient varied considerably across each cusp.

7.6.5 Mechanical abrasion on wave generating mechanism

During the course of the experimental work, friction resulting in metal fatigue and consequent relative movement between the wave paddle and the power stroke arm developed. As soon as this was noticed, it was repaired and the tests were repeated. In a few cases repeat tests showed significant differences from the originals. It was concluded that in these cases the relative movement in the wave paddle had generated waves of dissimilar characteristics.

7.6.6 Profile changes below the still water level

Whereas changes above the still water level were easily monitored and recorded during a test run, profile changes below the water appeared to be much slower and were largely invisible while the test was in progress. To allow the development of the equilibrium profile, long test runs were used. Certain discrepancies below the water level persisted however, and will be discussed in a later section.

7.6.7 Repeatability of tests

Apart from the few discrepancies between profiles created under similar conditions of wave attack attributed to wear and tear on the wave paddle, the repeatability of tests appeared to be good, apart from a certain phase shift encountered on a few occasions. Bearing in mind the constant mass of sand in the wave tank, the author expected not only the profile configuration, but also the relative position of the profile in the tank, to be constant under conditions of similar wave attack, irrespective of the initial profile preceding the test. This was found not to be the case, and certain repeated profiles were found to have moved seawards or landwards of the original. The sequence of tests thus played a role. When comparing profiles, therefore, it is felt that comparisons can only be made of sand volume transfer relative to some fixed datum if the profiles concerned were performed in sequence with no intervening experiments having been conducted. Alternatively, profiles can be compared relative to some inherent profile characteristic, for instance the still water level - beach face interface.

7.6.8 Model scale effects

The model scale effects reported by numerous investigators were observed during the experimental programme. Most noticeable of these was the compressed surf zone. No attempt has been made to apply model laws to the programme and the results have been presented per se. The linear wave theory has been applied unconditionally to all wave data recorded.

7.7 EXPERIMENTAL RESULTS

7.7.1 Profile configuration and formation

Appendix B presents a record of the results of the equilibrium profile studies defined in terms of the wave characteristics which formed them. The profile range obtained included both berm and bar type profiles, as well as secondary beach features such as cliffs, lagoons and ripples. Spilling, surging and plunging waves were all successfully reproduced in the programme.

Initial profile changes were found to be dramatic and rapid, with changes in the initial profile being apparent within minutes. After the initial demolition of the beach, changes were much slower, as the new profile established itself. Initial refraction of the impinging wave, as a result of irregularities below wave level on the old profile, did not persist for any length of time and were soon superseded by a regular breaker line as the new beach form developed. Only once the new profile had established itself did a cusp system form with subsequent minor refraction of the impinging wave.

It was noted that when a wave, more destructive than that of the preceding test, acted on its predecessors equilibrium profile, and its own inherent profile was steeper, albeit displaced shoreward of the former, the initial reaction of the beach was to build up to the gradient required. Deposition thus occurred first, followed by subsequent erosion, as the profile retained its new form but was displaced landwards of the original. The net result was thus one of erosion.

Throughout the programme it was evident that accretion and erosion could take place on both bar and berm profiles, without a transition from one regime to the other. This was to be expected. What was not expected, however, was the tendency for a beach in the berm regime to actually erode with increasing wave steepness even before the transition from bar to berm was reached. This phenomenum is discussed in the following sections.

7.7.2 Bar - berm profiles as a function of wave characteristics

From the basic data presented in Appendix B, conclusions regarding the interaction between basic wave characteristics and resultant profile form, could be drawn. The results could then be compared to that of previous investigators, namely Johnson (1949), Iwagaki and Noda (1963) and Nayak (1970).

Figure 16(a) shows the author's experimental results superimposed on a graph depicting limits between bar and berm profiles established by Johnson, and Iwagaki and Noda. The wave characteristic used is that of wave steepness plotted against wave height and a beach sand characteristic, D_{50}. Within the range of the experimental programme the results confirm Iwagaki and Noda's adaptation of Johnson's original limit between the two profile regimes.

No pattern between spilling or surging waves, both of which produce berm profiles, was discernable.

Figure 16(b) shows the author's results plotted on a graph adapted by Nayak from that presented in the previous figure and on which the previous limits have been reproduced. (The same code as for Figure 16(a) is used). Nayak introduced more beach sand characteristics thus also allowing for a comparison of model results utilizing beaches composed of different materials. The author's results do not support Nayak's limiting curve within the range of experiments conducted. The author noted no tendency from the experimental results for waves of low steepness together with small wave height to produce either plunging waves or a bar profile. It was evident from the results however, as will be discussed under Section 4.3., that increasing wave steepness, while still in the spilling wave - berm profile regime, produced erosion on the beach face. The implication is thus that the spilling to plunging wave transition and associated berm to bar profile response, does not, in fact, represent the transition between an accreting or eroding beach, erosion beginning some time before the berm-bar transition.

It is interesting to note the rather narrow spectrum of conditions reproduceable by the wave generating mechanism. The limited diagonal zone of results, as reproduced in either of the two preceding Figures, serves as illustration. This represents a limitation imposed by the mechanics of the system, whereby, for a short wave period, it was impossible to produce a wave of small wave height, and vice versa.

7.7.3 Cusp formation

Cusp formation to a greater or lesser degree occurred over the full range of the experimental programme. After a study of the experimental results, the author is convinced that cusp formation is a natural phenomenum of hydrodynamic nature and not a topographical one. Cuspate formations will

Graph (a)

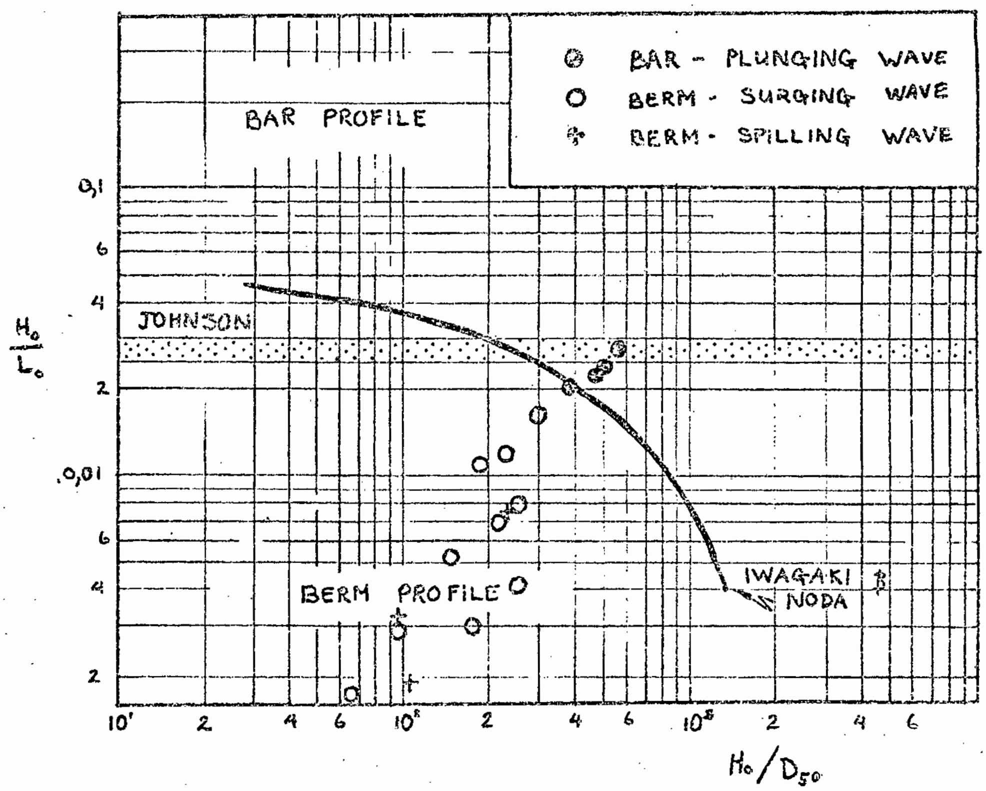
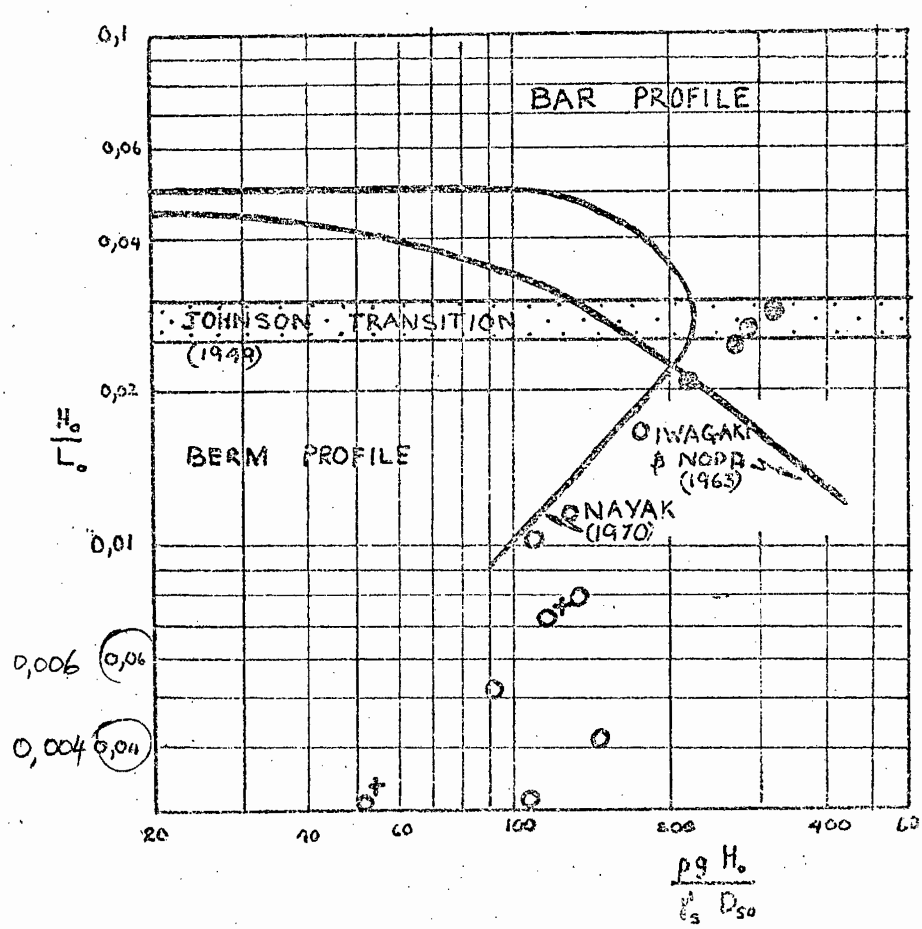


Figure 16 Wave steepness versus a) H_o/D_{50}

b) $w\ H_o/\gamma_s\ D_{50}$

Graph (b)



develop in any movable bed model tank of sufficient width for their occurrence to be detected.

During the experimental programme it was clear that cusps develop on both accreting and eroding beaches. Initial profile configuration and irregularities played no role whatsoever in the development, position or wavelength of subsequent cusp formations. The sequence of cuspate formation appeared to be as follows: the initial build up or erosion of the original profile (as the case may be) to a regular profile section similar in configuration to that of a horn profile (see Appendix B). Only thereafter did selective erosion occur in what developed into the embayments of the cusp above the still water line, and deposition similar to a delta below the still water line. The net result was a continuous regular deformation of the beach profile along the beach face of a repeating nature.

Appendix C presents plan views showing contour lines of sand elevation for a typical berm and bar profile respectively, a definite bar profile, when measured longitudinally along the centre line of an embayment exhibits all the characteristics of a berm profile to an unwary observer. The trough no longer exists and the bar itself is elongated in a seawards direction and can easily be mistaken for a berm.

The flow pattern of the swash zone was observed as follows. The swash surged up over the whole zone, but was deflected to either side at the apex of each horn section. This deflected water added to that already in the embayment, and once the swash had exerted itself, resulted in an increased back-wash. A slight time lag was noticeable between the reduced back-wash returning down the horn and that in the embayment, as would be expected. Inevitably the embayment back-wash was intercepted by the incoming surge of the following wave, on the beach face for the shorter period waves, and in the surf zone for the longer periods. Deposition thus occurred seaward of the embayment, and a delta was formed. Noticeable also was the formation of small rip currents in this region, collinear with the centre line of each embayment. These rips did not extend seaward of the breaker line but their effect, combined with that of the underwater deltas, resulted in minor refraction of the impinging waves.

The experimental results indicated that the basic cusp spacing or wave length was primarily a function of the impinging wave period. Within the range of the experimental programme the cusp spacing showed no clear correlation with impinging wave height. When cusp spacing was plotted against wave steepness however, a definite trend could be noticed.

Figure 17(a) presents the results of recorded cusp spacings plotted against the corresponding wave periods. For this test the wave periods were selected at random and the still water level in the tank was kept constant throughout. The cusp wave length was found to be independent of the profile form and maintained their spacing from initial formation without change. Random movement of the cusps along the beach face, did, however, occur, but no particular pattern could be determined.

The graph clearly indicates the tendency for the cusp wave length to increase with a corresponding increase in wave period. The relationship does not appear to be linear, but rather of an exponential form.

Figures 17(b) and 17(c) represent plots of cusp spacing versus deep water wave steepness. These results are based on data from Appendix B in which the equilibrium condition was allowed to develop. The results are presented separately, as each set represents a different maximum still water level in the wave basin. Because of the data scatter the best fit lines are indicated in dashed form only.

Both Figures 17(b) and 17(c) exhibit the same trend, namely that of a decrease in cusp spacing or wave length with increasing wave steepness. This trend is naturally in phase with the conclusions drawn from Figure 17(a), as the wave length is a function of the wave period and appears in the reciprocal form in the latter two graphs. The Figures also reveal that within the limits of the experiments conducted, a minimum cusp spacing is reached after which it remains constant for further increases in wave steepness.

Because of the data scatter in Figures 17(b) and 17(c) it is not possible to gauge any definite effect of the maximum still water depth on cusp spacing. Within the limits of the small amplitude wave theory however, the effect of an increase in water depth would be the establishment of a set or family of curves, with a tendency for the curves to be displaced to the right and up on the graph for increasing maximum water depths. A careful study of the best fit curves on the two Figures suggests that this does in fact take place.

7.7.4 Sand volume transfer considerations

As noted before, volumetric or sand transfer measurements in the on-shore off-shore direction between different profiles are complicated by two factors, viz., the presence of cusps, and the observation that a horizontal displacement occurred in certain cases for profiles formed under similar conditions of wave attack. The first factor was countered by using representative 'tank average' values for profile configuration where necessary. (i.e. where cusp proportions were of such a magnitude so as to be a factor for consideration). The second factor suggested two basic methods of approach. Firstly, comparisons to be based on sequential tests and utilising a fixed on-

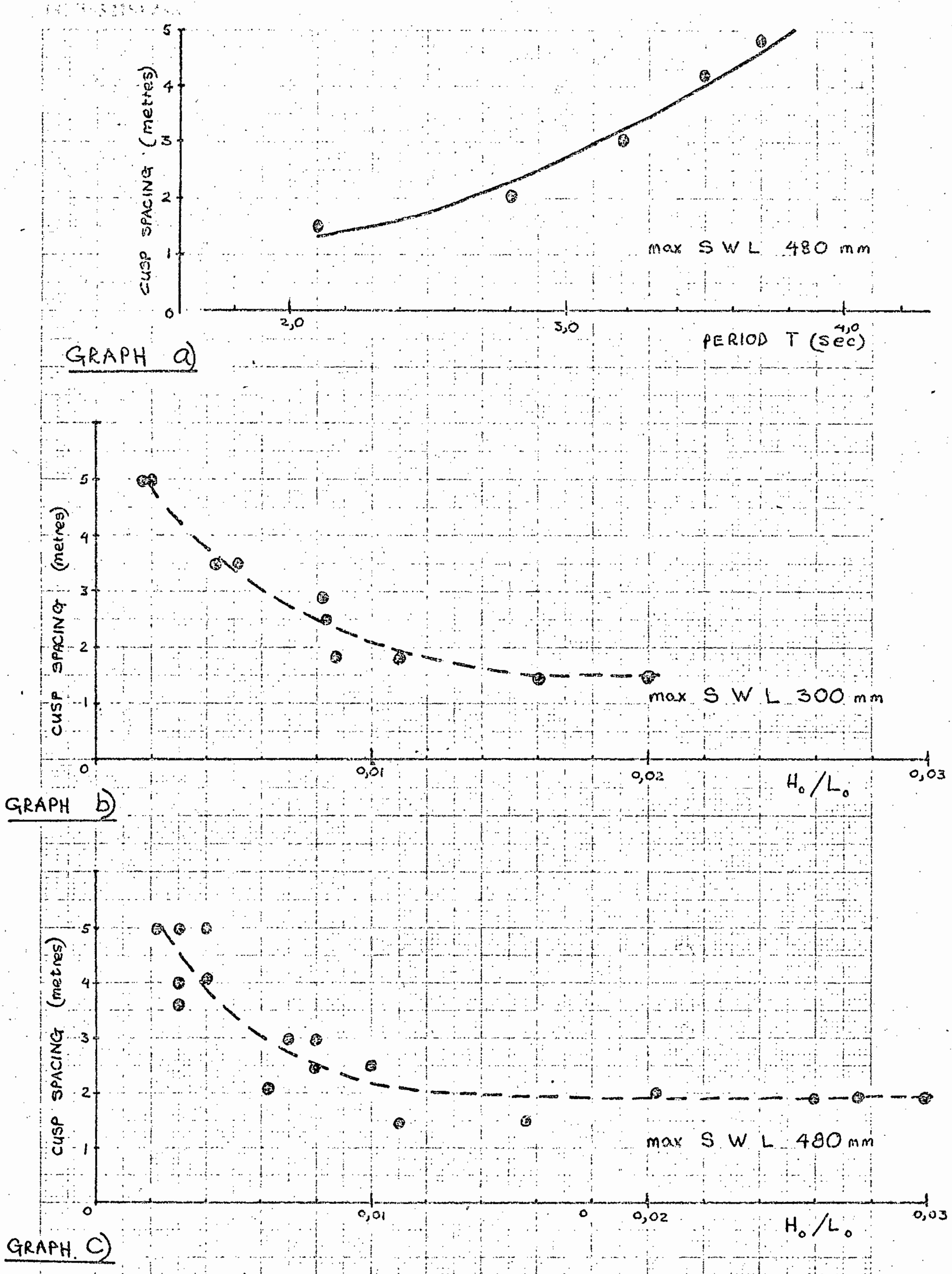


Figure 17 Cusp spacing versus

a) wave period (maximum SWL = 480 mm)

b) deep water wave steepness (max SWL = 300 mm)

c) deep water wave steepness (max SWL = 480 mm) shore datum line for horizontal measurements. Secondly, comparisons to be based on some inherent position on each individual profile and common to all from which horizontal measurements could be scaled. In the latter case, a line through the still water level and beach interface was selected.

A further physical limitation imposed by the model basin was one of water depth. This, together with the limited length of the wave basin, resulted in movement of sand along the whole length of the basin, considerable distortion occuring at the wave paddle end due to turbulence. It was thus not possible to record profiles to a point where the seaward extremity of the profile was stable and unaffected by the generated waves.

7.7.5 Sequential volumetric profile changes

By comparing the resultant equilibrium beach profiles formed under different wave conditions in terms of sand displacement normal to the shoreline, a study was made of sand volume transfer in the on-shore/off-shore direction.

Two series of tests were conducted, each series consisting of three individual equilibrium profile studies as recorded in Appendix B, and produced in the manner described in Section 2.2. Each of the two series had a constant but different wave period and a constant still water level, but wave heights varied in each case. The volumetric changes in profile as a result of wave changes are plotted in Figure 18. The two series, representing constant wave periods of 2,1 seconds and 2,7 seconds respectively, are placed side by side for comparison. (Individual profiles or tests can be identified in Appendix B from their test number as recorded on Figure 18). In Figure 18 the profiles have been divided up and plotted in segments or zones as illustrated by the key. The range of figures indicated are the horizontal location planes between which the volume is measured. The volumes are presented per unit width of beach.

As the still water plane was used as a datum for the vertical sand elevations, changes below the still water surface are represented in terms of changes in water volume, whereas above the water surface sand volumes are presented. For the horizontal measurements in the off-shore/on-shore direction, an on-shore line was selected, parallel to the beach line and landwards of the swash zone.

The series on the left of Figure 18 with a period of 2,7 seconds was solely in the berm profile regime with a spilling wave. The accretion tendencies of the sand above the water line, and the opposite action of the material below the water line, clearly indicate not only a build-up of the beach face with increasing wave height, but also the material exchange above and below water level. The maximum volumetric changes occur on the

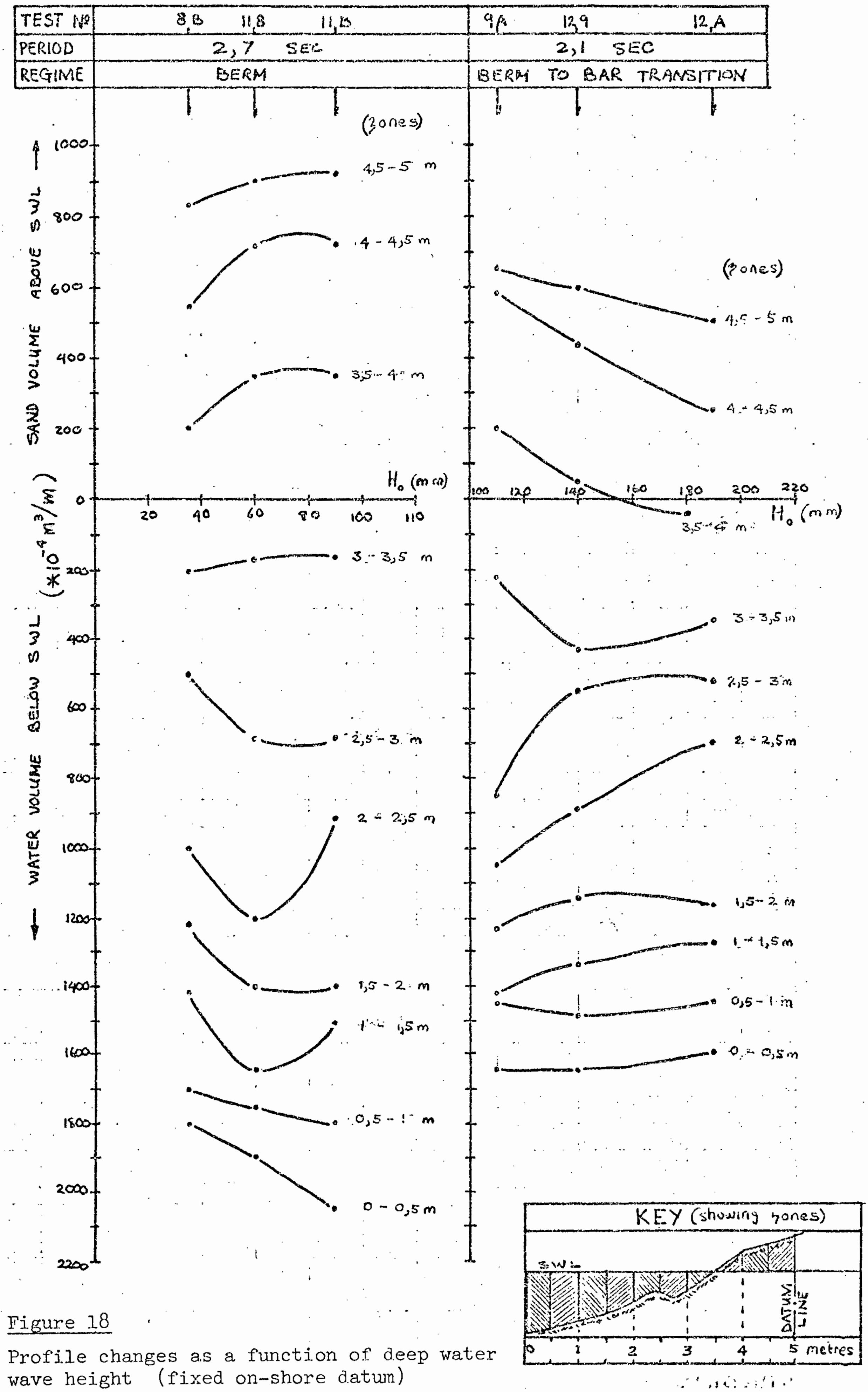


Figure 18

Profile changes as a function of deep water wave height (fixed on-shore datum) beach face in the mid-beach zone, and underwater in the proximity of the berm. The gradient of the line at the seaward extremities of the profile is indicative of the point raised in section 7.7.4.

The form of the curves indicate that towards the higher wave heights a transition has taken place and that a measure of erosion on the beach face has begun. This suggests that the optimum accretive tendency of an impinging wave is reached while still producing a profile in the berm regime, and thus, before the berm to bar transition has taken place.

The test series on the right of Figure 18 with a period of 2,1 seconds, represents a definite spilling wave - berm to plunging wave - bar transition. In this case, the erosive tendencies of the beach above the water line, and the opposite effect below water level, illustrate the destructive action of increasing wave height on the beach, and material exchange above and below the water line. The destruction of the beach is exemplified by the advancing 'sea' to the extent that the material in the 4 to 3,5 m zone has become negative when balanced with the advancing water volume. Once again, maximum changes occur on the mid-beach face and in the berm-bar zone.

In order to study the effect of beach gradient on sand volume transfer on the beach face, the same two series of tests as presented in Figure 18 are reproduced in Figure 19, with a slight modification to the horizontal datum. In Figure 19, instead of a fixed on-shore line being used for horizontal measurement, a plane through the beach face - still water level interface has been chosen. By comparing the tests in this manner the effects of an advancing or retreating sea relative to any fixed on-shore line are cancelled out.

Because the datum line as used in Figure 19 now represents a variable from test to test, it is preferred to represent each zone by means of a letter as opposed to the numbers used in Figure 18, which in turn relate to fixed points along the wave tank. The zones retain a horizontal dimension of 0,5 m.

In Figure 19, the left series remains identical to that of the previous figure, no significant advancement or recession of the interface having occurred. Varying beach volumes in this case are thus a function of beach gradient, both above and below the water level.

The right-hand series however, in which considerable beach recession did take place, differs appreciably from its predecessor. The tendency is still definitely destructive with increasing wave height, although naturally less significantly than before. This is indicative of a reduction of beach gradient, when attacked by waves of increasing steepness right into the plunging wave - bar regime.

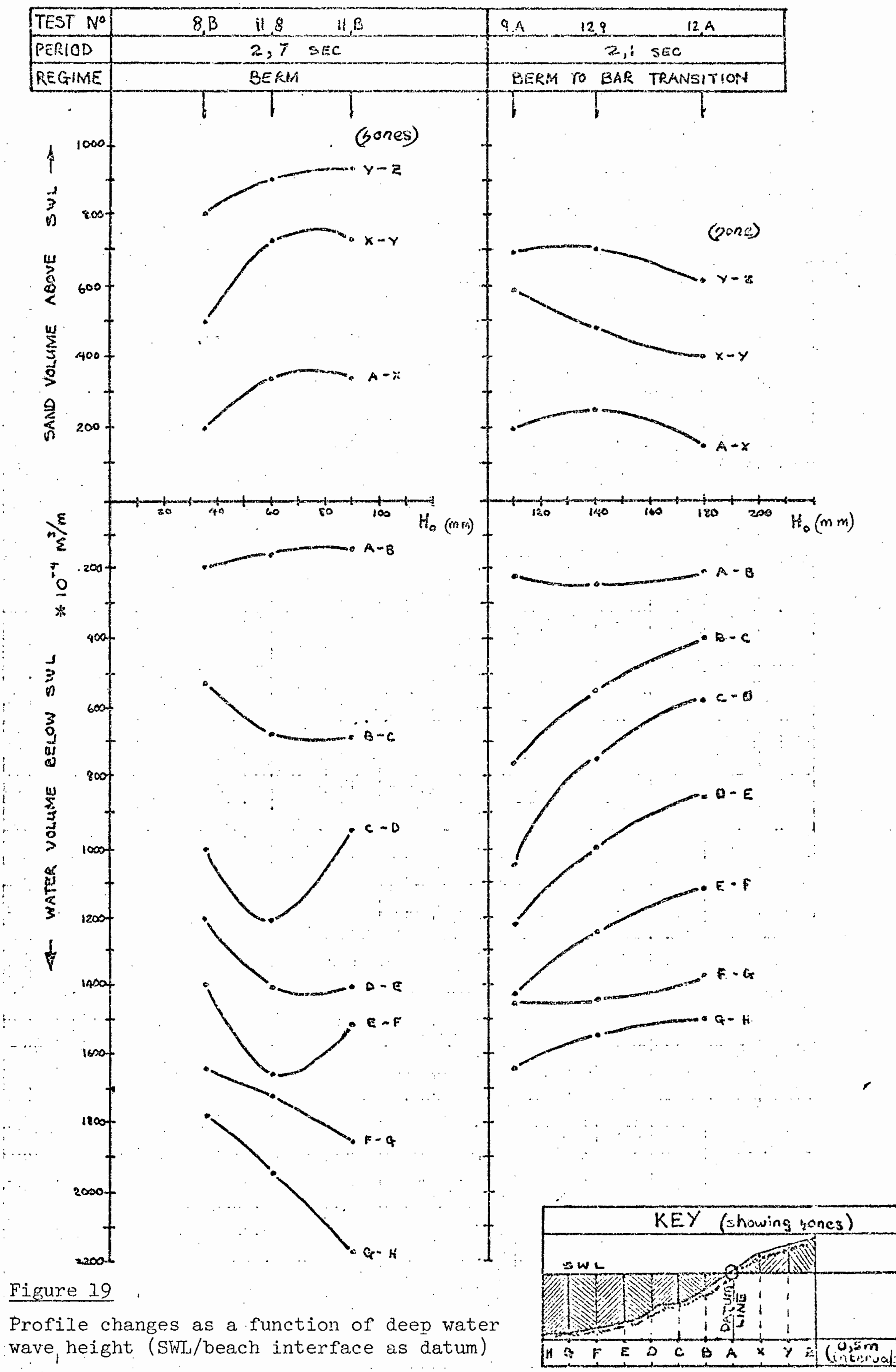


Figure 19

Profile changes as a function of deep water wave height (SWL/beach interface as datum)

7.7.6 Volumetric sand transfer as a function of wave characteristics

Although the series of tests conducted with a wave period of 2,7 seconds and varying wave heights gave rise to a volumetric trend indicative of a transition being reached where one was not expected, and inconclusive as the results might be, they are not necessarily incorrect. The test results thus indicate that within the berm regime, the constructive wave action has an optimum effect on the beach under a certain set of circumstances. When the wave steepness is greater than this optimum, a measure of beach face erosion takes place, while the profile nevertheless continues to display all the characteristics of a berm or accreting type beach. The bar or eroding type beach, due to destructive waves, appears at a later stage with further increases in wave steepness and wave height.

The ultimate aim of this test programme is to predict sand volume transfers in the on-shore/off-shore direction from observations made of the prevailing wave characteristics, which in turn have been related to their respective equilibrium profiles.

Concentrating firstly on the beach face only, and utilising a fixed on-shore datum, Figure 20 reproduces the basic data as used in Figure 18 for the series of tests with a 2,7 second wave period. The curves have been extrapolated for increasing wave height.

Manipulation of the constructed curves gives rise to the derived curve presented on the bottom half of Figure 20, which represents a sand volume transfer diagram in the off-shore/on-shore direction. For the different zones of the beach face, as demarcated, a series of curves allow the prediction of beach face accretion or erosion for a certain wave period and various deep water wave heights.

Figure 20 indicates, furthermore, that the various beach zones have different optimum wave heights for maximum build-up. The optimum wave height appears to increase with distance landwards along the beach face. The derived curves differ from one another, not only in gradient, but are also offset along the horizontal axis.

The same manipulations applied to the test series with a period of 2,1 seconds produce a derived curve of completely different form, as presented in Figure 21.

In this case, no interpolation of an optimum point is possible and extrapolation of the curve towards lower deep water wave height values would be purely speculative, although it is obvious that the optimum point would be in this region. The derived curves in this case can thus only be produced for the erosive tendencies over the wave spectrum under consideration. The rates of change on the beach face, as for Figure 20, indicate a maximum for

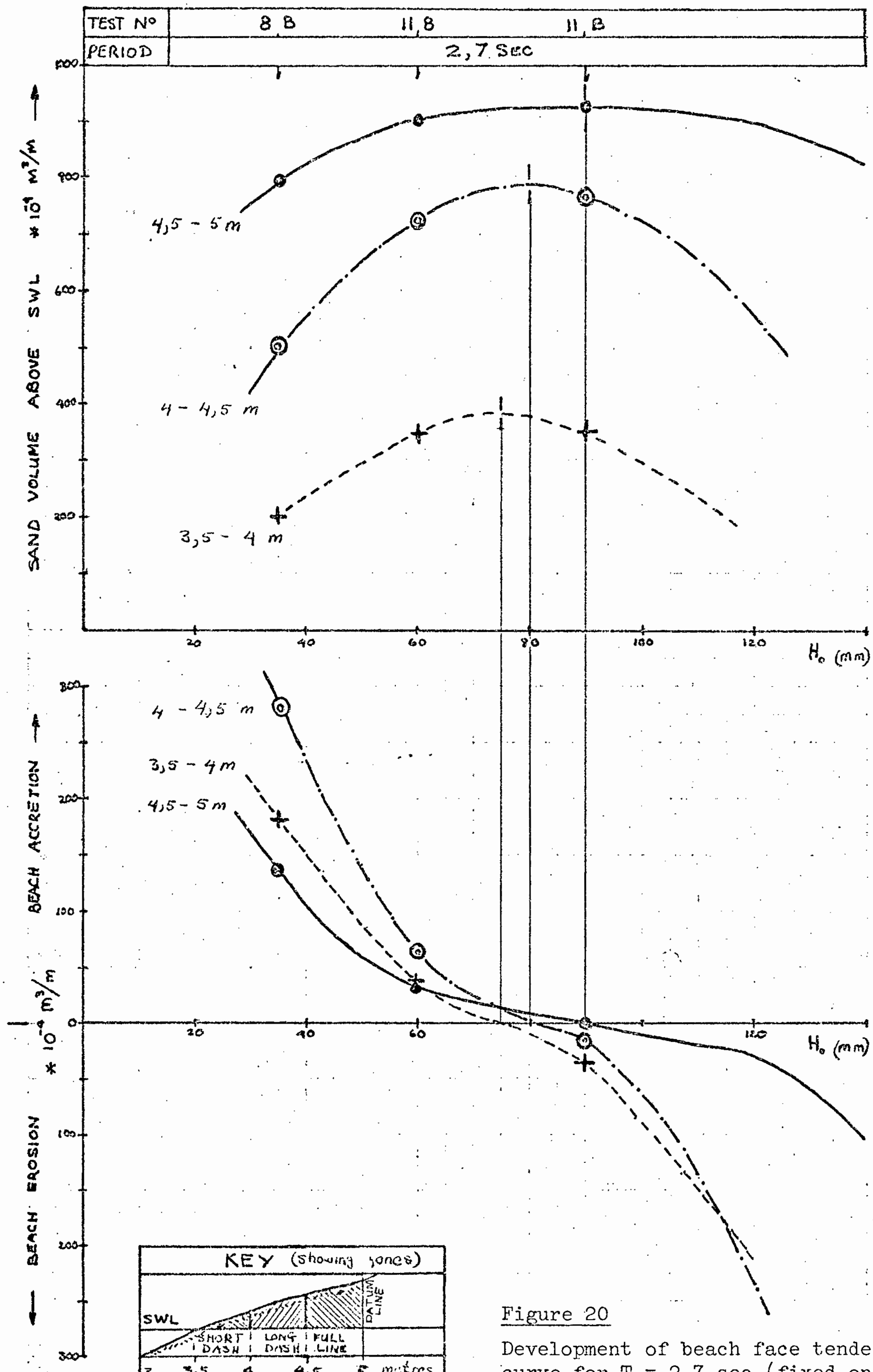


Figure 20

Development of beach face tendency curve for T = 2,7 sec (fixed on-shore datum)

TEST N°	9 A	129	12A
PERIOD	2,1 sec		

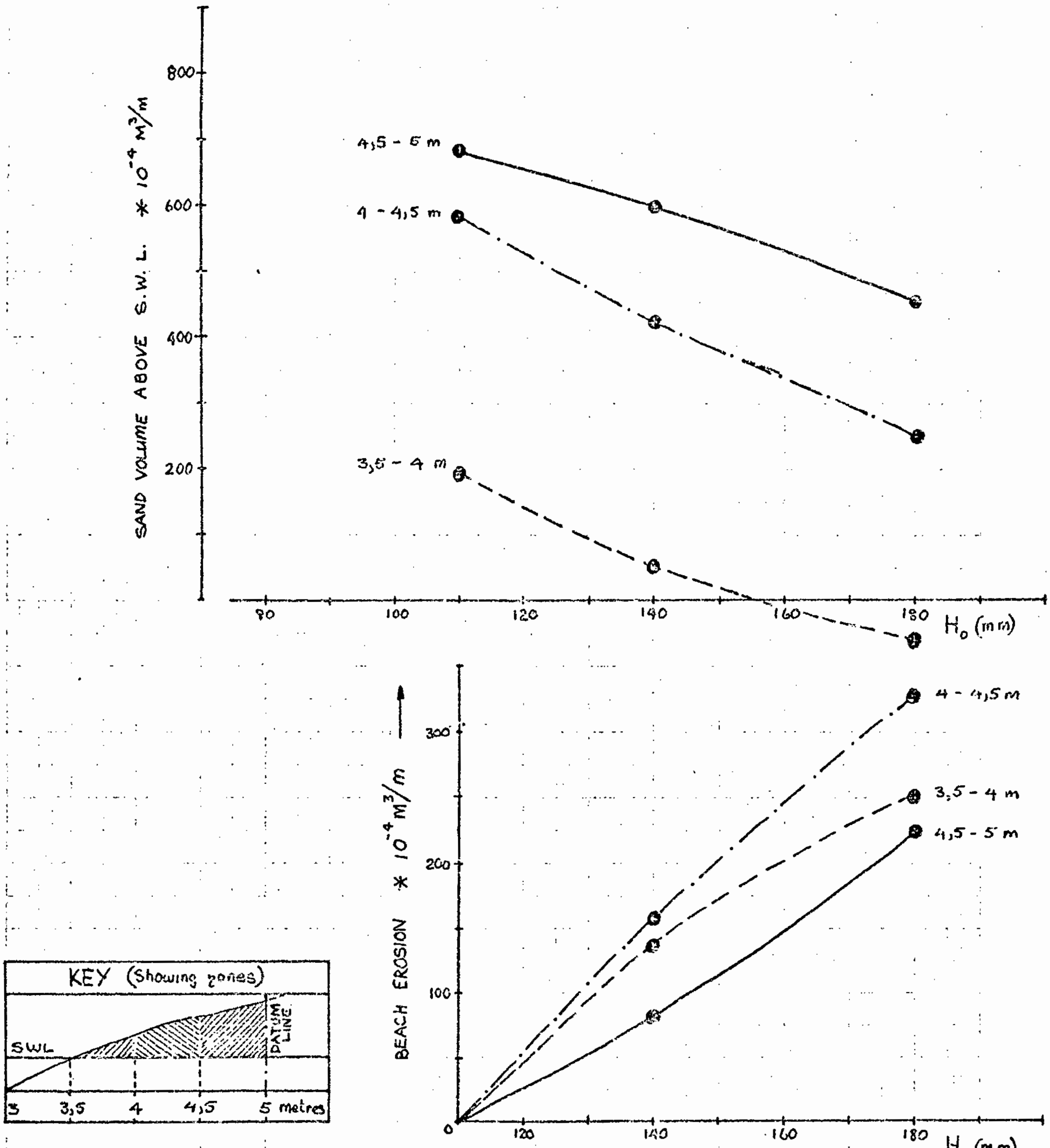


Figure 21 Development of beach face tendency curve for T = 2,1 sec (fixed on-shore datum)

the mid-beach zone, an intermediate rate for the nearshore, and the lowest rate for the farshore zones respectively. The rates of change are not, it should be remembered, time related, but represent magnitude of change from one set of equilibrium conditions to another.

The previous two Figures can be presented in a form of perhaps more practical value by expressing the changes in terms of total sand transfer in the off-shore/on-shore direction both above and below the still water level. Figure 22 applies to the case for the 2,7 second wave period test series.

In Figure 22 the fixed on-shore datum line for horizontal measurements has been retained. The figure represents a summation of the sand transfer previously measured in zones above and below the still water level respectively. As in previous figures, the volumes above water level (solid line) represent a quantity of beach sand, while the volumes below water level (dashed line) are those of the water above the submerged sand profile per metre width of beach. It will be noted that the respective turning points of the curves above and below water level coincide remarkably well (even allowing for experimental error).

A study of Figure 22 reveals a difference in magnitude of the sand transfer above and below the still water line for the test series considered. Volume transfer changes below the water line appear to be virtually double those above the water line. The reason for this discrepancy is not known, and as will be seen in a following figure the discrepancy did not reoccur.

Figure 22 in its derived form now provides a means of predicting sand volume transfer either towards or away from the beach face, under wave conditions of a given wave period but varying wave heights.

The same method as used for the derivation of Figure 22 was used to produce Figure 23. Figure 23, which presents the series of tests associated with a wave period of 2,1 seconds has of necessity a different form of presentation to that of Figure 22. The reasons for this are those as presented for Figure 21, from which Figure 23 is based in terms of total sand volume transfer above and below the still water line.

As there has been no attempt to extrapolate the curves presented in Figure 23, due to the reasons previously stated, the derived curve on the right-hand side of the figure has been produced to represent sand volume changes within the range of recorded wave heights only. Whereas in Figure 22 the horizontal axis in the right-hand figure is indicative of an eroding/accreting transition, in Figure 23 the horizontal axis is taken as the profile condition existing at H_o = 110 mm.

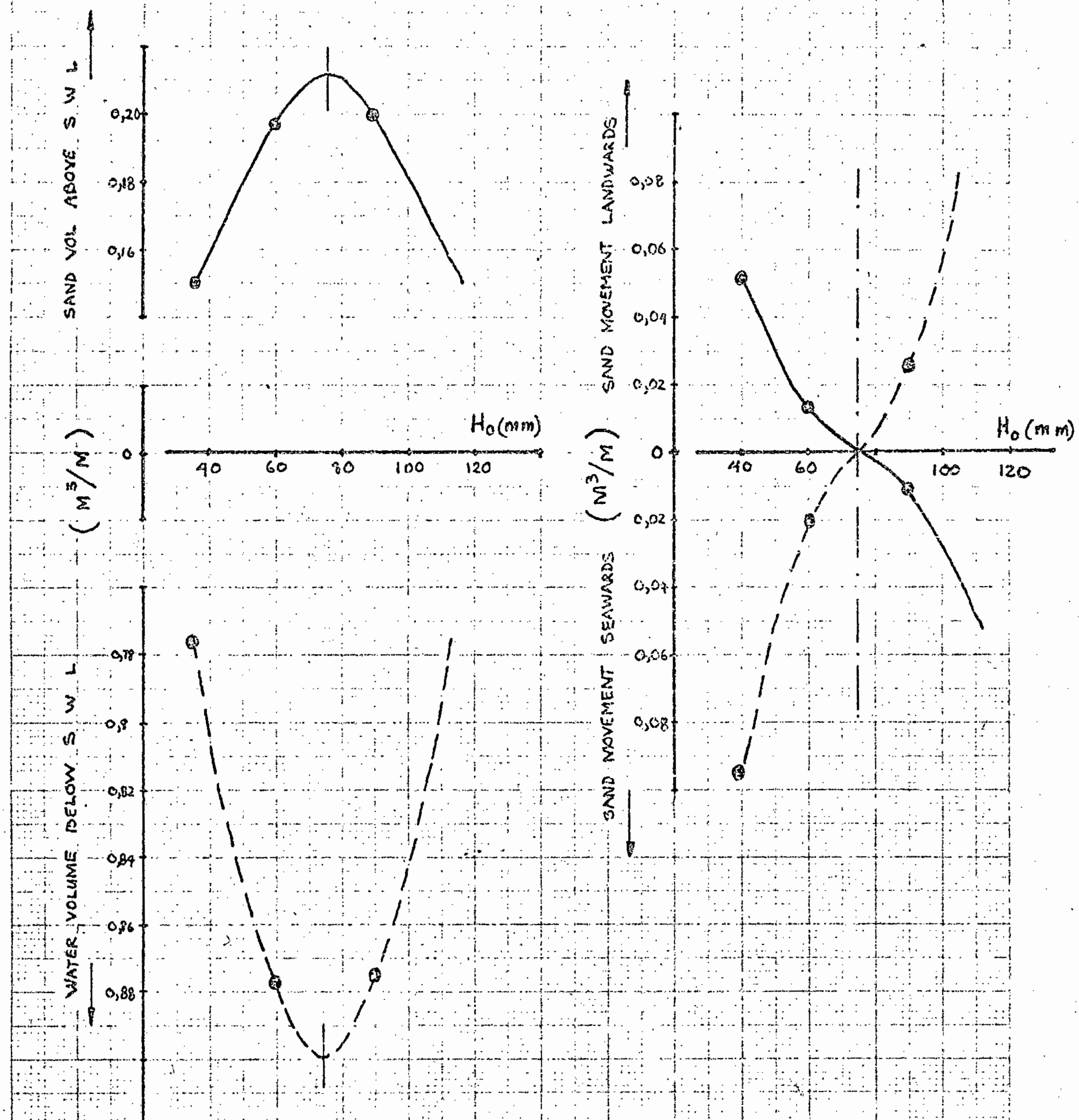


Figure 22 Beach tendency curves for sand transfer above and below SWL
T = 2,7 sec (fixed on-shore datum)

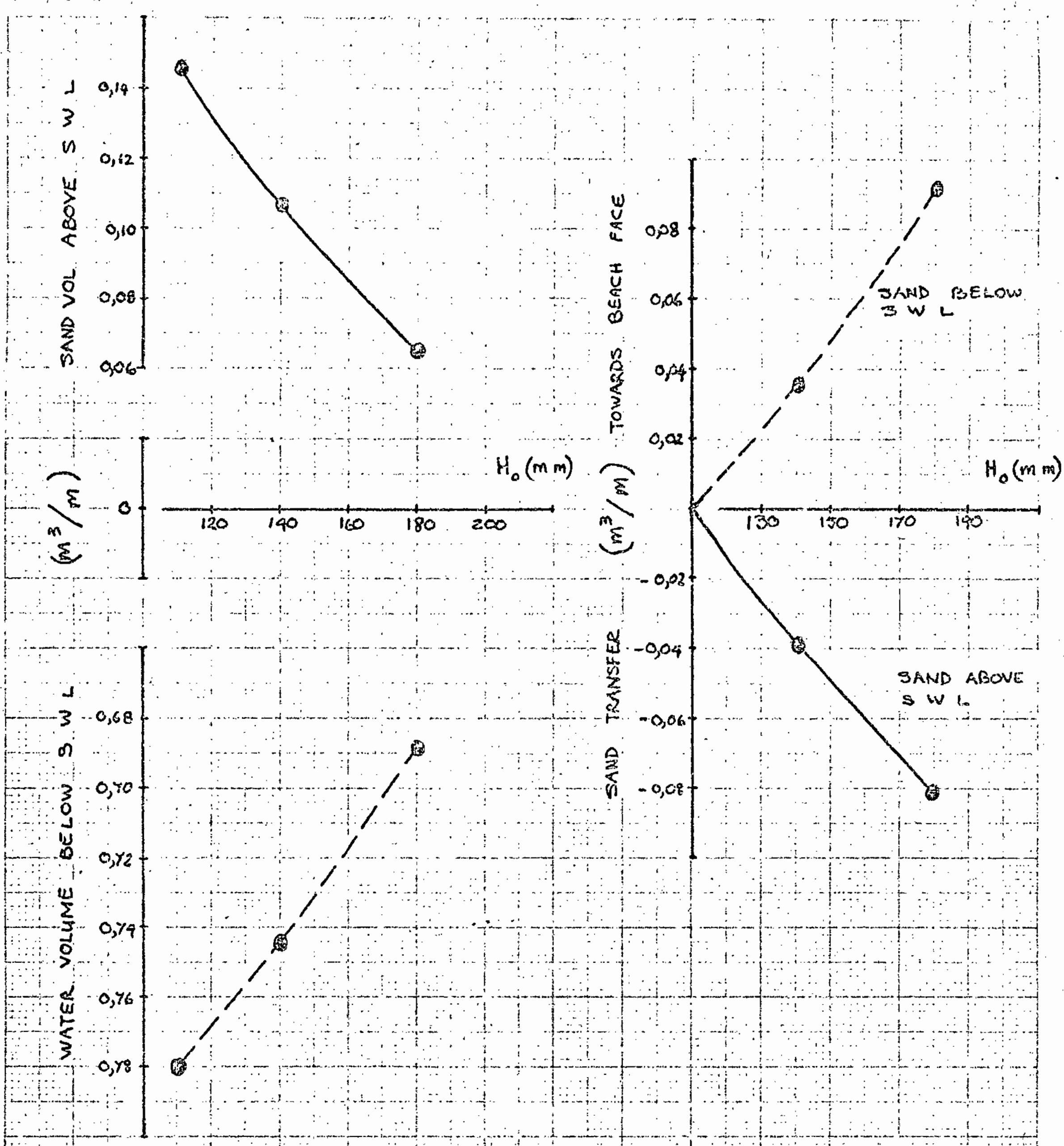


Figure 23 Beach tendency curves for sand transfer above and below SWL
T = 2,1 sec (fixed on-shore datum)

In Figure 23 the derived curve once again predicts the sand volume transfer in the on-shore/off-shore directions under wave attack conditions of constant wave period but varying wave height. In Figure 23 an increase in wave height leads to the erosion of the beach face. The material transfer above and below the still water line can be clearly seen, and in this case equate very well to each other.

7.7.7 Nonsequential comparisons of profiles

Figure 24 presents cumulative volume curves for various profiles plotted against the tank grid as ordinate. In Figure 24(a), two berm profiles are considered. From the figure it appears that in this regime beach build-up, due to constructive wave action, has the effect of a rotation of the beach mass around a node, somewhere below the still water level.

On Figure 24(b), where a berm-bar comparison has been made, the volume curves for the bar profile has been plotted, firstly, relative to the same fixed point datum as the berm profile and secondly, using the still water level and beach interface as the datum. The two bar curves, when compared to the berm, both show a negative rotation of the beach mass, in the one case negative displacement also being evident. ('Negative' is here defined as any erosive tendency). The curve forms are inconclusive as to whether any rotation occurred between the two bar curves.

The object of this graph was to determine, if possible, whether the two bar curves were similar in every respect except for a certain displacement. If erosion and accretion of a beach, when presented in this form, could be explained in terms of a rotation and a displacement, a method of comparing non-sequential profiles at least in terms of rotation angle would be possible, the displacement being determined by some other method. The ultimate aim of any such study would be the prediction of sand transfer on a beach between any two sets of constant impinging wave conditions.

7.7.8 Combination of results of different wave periods

The resultant sand transfer results presented thus far have all been comparisons based on different wave heights for the same wave periods. If, however, instead of basing the comparisons on wave height alone, a more comprehensive form of wave characteristic is used, i.e. that containing in it a measure of the wave period, direct comparisons between any combination of wave characteristics and beach sand volume transfer should be possible.

Figure 25(a) and (b) present such comparisons between sand volume on the beach face and wave steepness and wave power respectively. Waves of three basic periods are compared, and all sand volumes are relative to a fixed

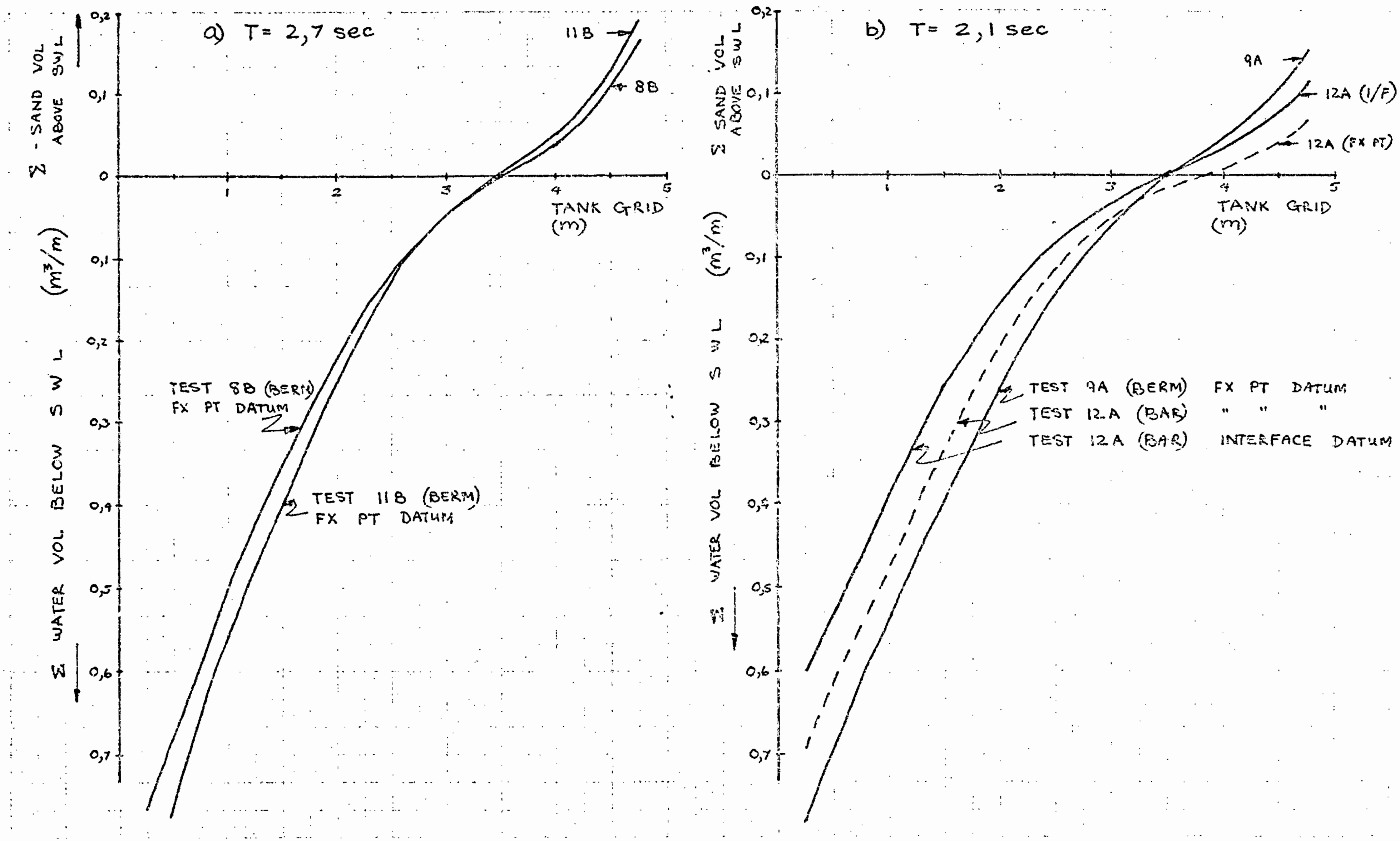


Figure 24 Accumulative volume curves

a) T = 2,7 sec b) T = 2,1 sec (datum as indicated) on-shore datum. In the figures only the sand volume above the still water line and relative to the swash zone is considered. The relative positions of points for each respective set of tests for both wave steepness and power, where made earlier, remain similar to the direct wave height comparisons.

It would be logical to assume that results based on wave steepness, which emphasizes wave period, should yield more consistent results than those based on wave power, with the emphasis on wave height (wave period being much more easily and accurately measured than wave height). Nevertheless, while the data scatter in Figure 25(a) is less than Figure 25(b), some distortion is evident for the longer periods which would require further investigation. Figure 25(b) thus, even with its greater scatter, shows a better trend in this particular case. The scatter is such in both graphs that no curve has been developed through the points. The data points have been left to indicate the trend. Were it possible to develop these curves, the graph would have been able to predict sand volume transfers on the beach face for a change in impinging wave characteristics as expressed by either wave steepness or wave power.

From Figure 25(a) and (b) it can be seen that the tendency of both graphs appears to indicate a steeper gradient on the accretive side of the 'curve', the erosive wing having a more gentle slope. The implication is that in the beach face building-up zone, small increments of wave steepness or wave power result in appreciable accretion on the beach face. Larger increments of wave steepness or wave power are required to produce the same measure of erosion. Results produced in the course of this experimental programme indicate that the peak of beach accretion does not necessarily represent either the transition between spilling and plunging waves, or the transition between berm and bar profiles. The results of this study have indicated that the beach tends to begin eroding before either of these transitions are reached.

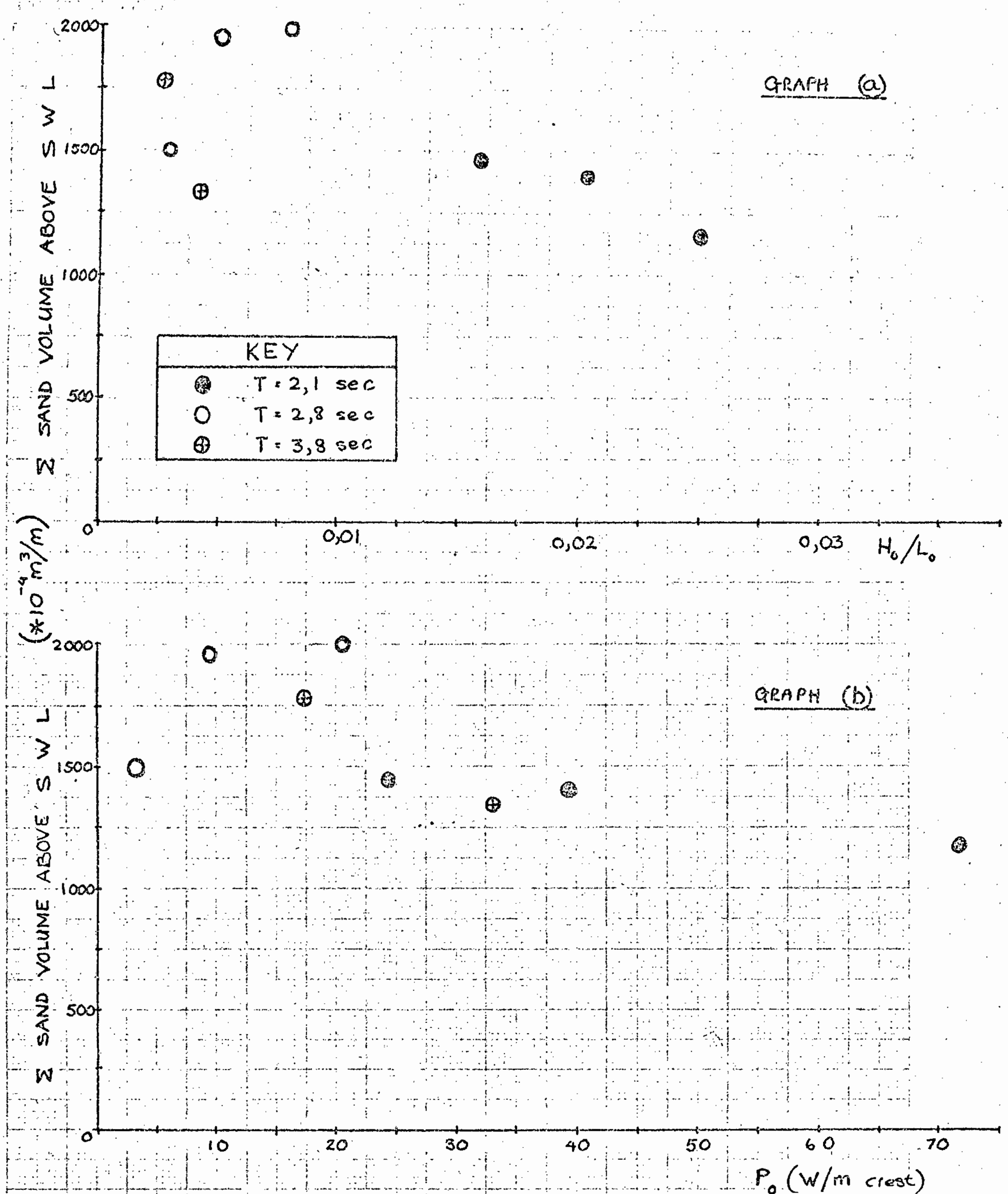


Figure 25 Beach sand volumes, versus

a) deep water wave steepness

b) deep water wave power

CHAPTER 8

THE ANGLED WAVE ATTACK PROGRAMME

8.1 APPARATUS

The experimental programme was conducted in the same model wave basin as used for the normal wave attack, with the principle aim of determining the suitability of tha tank to do littoral drift studies. As the model had a fixed angle wave paddle, an angled attack could only be achieved by laying and maintaining the artificial beach at the angle under consideration.

Wooden collector bins or hoppers were designed for mounting on the downdrift side of the beach. These were designed in units for classification and ease of handling. An adjustable set of wooden templets were designed to be arranged alongside the bins, adjustable to the particular profile configuration. No automatic system of emptying the bins was devised. The bins were cleared by using garden spades, weighed in containers and manually reintroduced on to the updrift side of the beach in approximately the same position, relative to the profile, from where it was collected

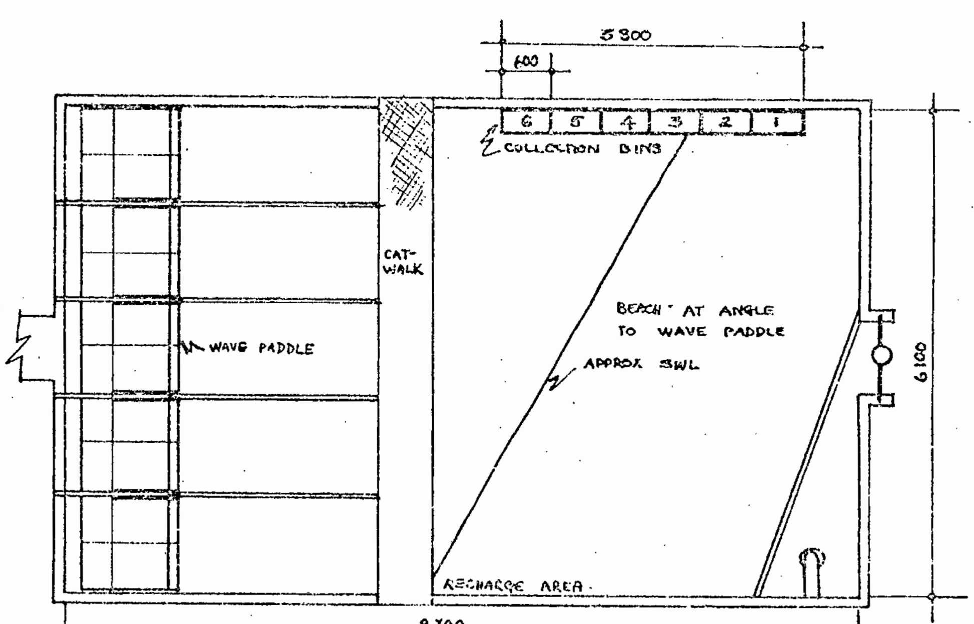


Figure 26 Plan view of model basin as used for the littoral drift studies

No reflection absorption measures were taken during the test programme. The same beach sand was used as in the normal attack programme, as well as fresh water. The wave parameters studies fell within the range of those of the normal attack programme.

8.2 EXPERIMENTAL PROCEDURE

Due to the exploratory nature of the investigation, as well as the difficulty in maintaining a constant set of conditions from an angled wave attack under the circumstances, only two sets of tests were carried out. A specific constant deep water wave angle was selected, a constant water level maintained and comparisons were sought on the basis of two different sets of wave periods. In both cases the angle of wave attack (wave paddle to beach laid) was 30°. Initially a one in ten slope was laid by hand on the beach, but before measurement was begun, the beach was allowed to form or at least approximate its own equilibrium profile. Thereafter each test was conducted for ten sets, each of ten minutes duration, the final measurements being the average of the ten sets.

The procedure during a ten minute set was as follows:

a) Measurement of wave height and depth at which recorded.
b) Measurement of water depth at wave break point.
c) Measurement of breaker angle.
d) Measurement of the speed of longshore current.
e) Continuous clearing, weighing, logging and reintroduction to the updrift side of the beach material collected in the littoral drift bins.

Visual observations of the flow pattern and beach formation were made in the course of each set of tests. At the conclusion of each set the beach profiles were measured. Measurements of wave period, wave height, water depth at point of wave height measurement and water depth at the break point were done manually as for the normal attack programme. Determination of the breaker angle was considerably more difficult to gauge, and recourse had to be made to a visual estimation. Longshore velocity was measured by timing the drift of dye introduced on the updrift side of the tank over a known distance.

During the course of each ten minute set, as each individual bin filled up with sand, its contents were shovelled into buckets and weighed. Before weighing all excess water was poured off. The material was than reintroduced to the updrift side of the beach at the same relative position along the beach profile from which it was collected on the downdrift side.

The measured wave and drift rates for each series of tests are presented in Appendix F. A study of Appendix F reveals considerable data scatter. However, as the scatter appears fairly random, it is assumed that throughout each series of tests the conditions remained relatively constant. The basic angle of wave attack was thus retained and the respective mean values drawn from the raw data may be regarded as a realistic interpretation of the conditions under consideration.

8.3 DATA PRESENTATION

Calculation of the unrefracted wave height leads from equation (8). (The author made use of Wiegel's tables of progressive wave characteristics in transitional and shallow water relative to deep water linear wave theory). The deep water wave height may then be calculated from the expression:

$$H_o^1/H_o = K_r = \left((\cos\theta_o)(\cos\theta_b)\right)^{\frac{1}{2}}$$

The total longshore thrust on the water and sand in the surf zone is given by equation (20), the expression for longshore momentum flux S_{xy} at the breaker line. The total thrust is equal to the difference in momentun flux at the breaker line and at the landmost limit of the swash zone. (The latter term is obviously nil). The wave velocity at the breaker line is given by equation (4). As is convention the littoral drift rate may now be plotted against the product of the total longshore thrust and the wave velocity at the breaker line.

The measured moist mass of sand was converted to a dry mass by means of a conversion factor determined by experiment.

A summary of the measured and calculated data is presented in Figure 27.

A graphical representation of the littoral drift study is presented in Figure 28. Also included on the graph is a plot of Komar and Inman's relationship as expressed in equation (27), as well as experimental work carried out by A.H. Stewart (UCT undergraduate thesis No 29 of 1973). Stewart used the same model wave basin as the author and followed the same basic experimental procedure. Stewart's original results were expressed in terms of P_a; the author prefers to express them in terms of the product $F\ c_b$, even although the latter form of representation necessitates an estimate of the breaker velocity. The author feels that Stewart's use of the beach inclination angle to the wave paddle when applying equation (22c) is not correct and leads to a greater margin of error than using the water depth at the point of wave measurement as an estimate for the water depth at the point of wave break.

Parameter	Units	Test 1	Test 2
θ_o	degrees	30°	30°
T	sec	3,8	2,1
H	mm	117	158
d	mm	250	255
d_b	mm	190	200
θ_b	degrees	10°	15°
H_o^1	mm	84	146
H_o	mm	90	154
F	N/m	2,2	6,3
c_b	m/s	1,4	1,4
F c_b	N/s	3,1	8,8
I	kg/s	0,32	0,42
Q_s	m^3/s	$1,7 \times 10^{-4}$	$2,2 \times 10^{-4}$

Figure 27 Longshore model study measured and calculated data

(moist to dry mass conversion: 0,827)

(relative density of sand: 2,65)

(void ratio: 0,6)

8.4 EXPERIMENTAL RESULTS

The author's test series 1 yielded a spilling wave and associated berm type beach configuration. The resultant beach was slightly irregular across the width of the model tank, and showed evidence of two distorted cusp systems. Distortion also occurred in the vicinity of the beach recharge area. It was noted that the reflection of the waves from the downdrift side of the tank caused no appreciable local profile distortion. Visual observations confirmed the existence of both bed and suspended load transport during the experiment. The swash zone was observed as a predominantly bed load transport zone. After draining the tank it could be seen from the configuration of the beach profile against the collector bins that suspended load played a large role in surf zone transport.

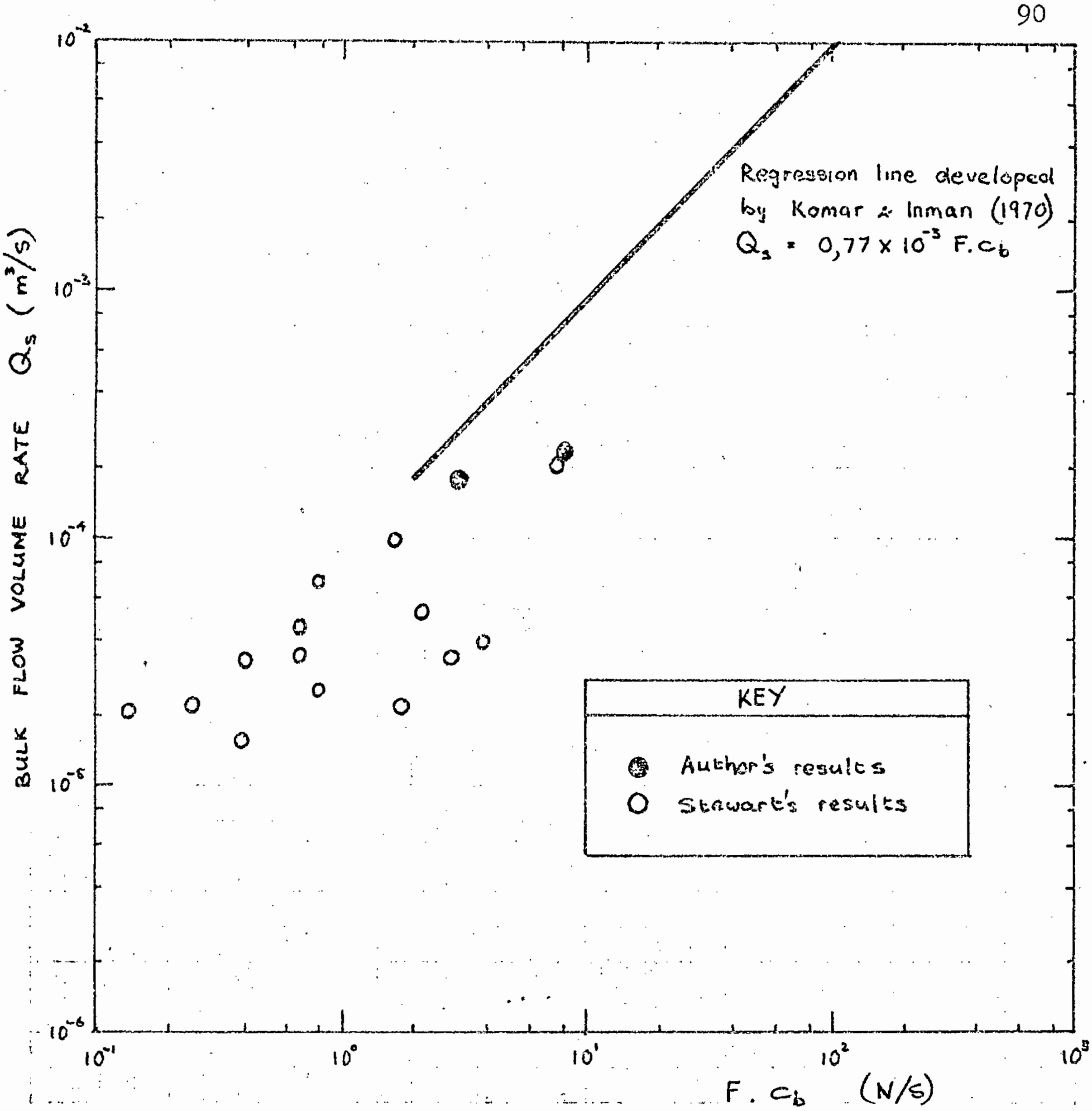
The author's test series 2 in turn produced a distinct plunging wave and associated bar type beach profile. Traces of a cusp system were apparent but indistinct. The reflection of the wave striking the downdrift side of the tank caused considerable disturbance in this case, and consequent local profile distortion. The updrift recharge area was also slightly distorted. In this test series a considerable mass of sand appeared to be placed in suspension by the plunging wave condition. Bed load movement, especially in the swash zone was nevertheless also evident. Littoral drift appeared to take place even slightly seaward of the collector bins, thus a pile up factor had to be applied to the measured drift to allow for this to be taken into account.

The author acknowledges that because of the short duration of each test series, the true equilibrium beach condition did not, in all probability, develop fully. The laborious method of manually recharging the collected littoral drift on the updrift side of the tank did not allow test runs of any length of time. The beach was however, assumed to at least approximate its equilibrium condition. (Measurement for each test series was preceded by a dummy run to allow the initial shaping of the beach to take place).

A study of Figure 28 reveals that the test series 1 results coincide reasonably well with the Komar and Inman relationship, while test series 2 appears to yield a bulk volume flow rate of approximately a quarter of that of Komar and Inman. In the case of test series 2, which covered an aggressive plunging wave condition, it is possible that the author failed to collect the full littoral drift as the bins filled up very rapidly and sand may have spilled out of the bins to a greater extent than compensated for in the author's pile up factor.

The data scatter of Stewart's results as plotted on Figure 28 is considerable, being of such an order that the author is hesitant in developing a regression line through the points on the graph. The extrapolation of Komar and Inman's relationship would in fact provide a reasonable upper limit of Stewart's results.

It should be noted at this stage that Komar and Inman's relationship was developed from the results of field studies and not model studies. Very few investigators have successfully correlated the results of both field and model studies, probably due to the scale effect in models resulting in a compressed surf zone. On average model studies have tended to point to a lower drift rate than would be expected from the results of field investigations. Also evident from a study of previous model studies is the considerable data scatter evident even when using the most refined laboratory techniques.



<u>Figure 28</u> Relationship between Q_s and F c_b

(The effect of data scatter is especially significant when the scales of the double logarithmic graph presentation is taken into consideration).

The author's test results as well as the higher values of Stewart's test series indicate two significant points. Firstly, they encroach into the regime of the field study values thus implying that some particularly mild longshore attack systems were studied in the field. Secondly, few laboratory studies have resulted in values of the same order of magnitude, thus implying that the UCT tests allowed for drift studies larger than those of the vast majority of previous model studies. The author suspects that the UCT wave generator could produce considerably larger waves than are usually used in three-dimensional model studies.

CHAPTER 9

EXPERIMENTAL PROGRAMME CONCLUSIONS

9.1 NORMAL WAVE ATTACK PROGRAMME

1. The model tank and wave generator could successfully reproduce the full range of wave conditions (spilling, surging and plunging), and related profile configurations (bar and berm features). The wave generator did have a failing in that the mechanism could not completely divorce wave period from wave height. This resulted in a narrow spectrum of conditions being studied, as it was not possible to produce a short period wave of low amplitude or vice versa.

2. Stabilized equilibrium beach profiles could be created under conditions of constant wave attack. The stabilization period of a beach appeared to be an inverse function of the attacking wave power. Three-dimensional flow patterns were observed in most of the experiments due to the development of beach cusps.

3. The repeatability of equilibrium profiles was established as to the basic profile form and average gradient, but not to location. The final equilibrium profile is thus not wholly independent of the original profile configuration. The original profile configuration appears to determine to a large extent its successor's final position relative to a fixed datum.

4. During the course of the experimental programme cusp formation featured prominently. Cusp formation appears to be a natural phenomenon of a dynamic nature, completely unrelated to initial beach configuration. Cusp spacing appears to be primarily a function of the impinging wave period, as well as being related to the wave steepness. Cusps developed on both berm and bar beach profiles. The formation of a cusp system appears to start with an initial beach accretion after which concentrated return flows carve out embayments leaving a cusp horn inbetween consecutive embayments.

5. Consecutive beach profile measurements under conditions of various wave attack programmes reveal an exchange of beach material above and below the still water level largely confined to the in-shore zone, but sea bed levels seaward of the breaker line also showed a measure of movement. The tendency of a beach to erode or accrete is reflected throughout the length of the beach profile in the in-shore zone, the only distortion appearing to be in the vicinity of the submarine bar or berm where local sand transfers invariably indicate a reciprocal action.

6. Graphs may be produced for predicting the average rate of material transfer above and below the still water level for different wave characteristics. These changes may be expressed in certain beach zones or as totals of the in-shore profile sand volume stored above and below the still water line. Sand volume transfers may be adequately defined as a function of wave steepness or wave power.

7. The gradient of a model beach is difficult to assess when cusps are in evidence. Cusps appear to take on a prominence in model studies far larger than their status on natural beaches. This is in part due to the compressed surf zone as found in model studies. Because of the regularities of the distortions due to cusps however, it is possible to gauge the average gradient on a model beach, as well as the average amount of material stored relative to a chosen horizontal datum. The average gradient of a beach remains indicative of total beach erosion or accretion, and is a function of the wave steepness as well as of the beach sand.

8. Erosion appears to take place on a beach berm profile with increases of wave steepness not yet within the recognised berm to bar transitional zone (as defined in terms of wave steepness and a beach sand characteristic).

9.2 ANGLED WAVE ATTACK PROGRAMME

1. While recognising the limitations imposed by the UCT wave basin, and in particular the unsophisticated experimental procedures adopted, the experiments conducted appeared to produce relatively satisfactory results supporting the tendency of an increased littoral drift rate due to increases in the longshore wave thrust acting on the water and sand in the surf zone. Even the data scatter of the experiments appears to fall well within that of previous investigators using considerably more sophisticated

model basins and experimental methods. Nevertheless, a less laborious method of recharging the model basin with collected littoral drift will allow more flexibility to further use of the UCT basin, and measures should be taken to reduce wave reflection on the downdrift side of the basin. Where the model basin does lack real flexibility however, is in the fact that the wave producing mechanism cannot be automatically aligned to produce wave attack of varying angles.

2. By observation, both bed and suspended load littoral transport were in evidence during the course of the tests. Both modes of sediment transport were observed to occur under both plunging and spilling wave conditions. Under conditions of plunging wave attack, the mode of transport appeared to be predominantly that of suspended load in the surf zone. Bed load transport occurred for both wave conditions in the swash zone. (The high velocities in the swash zone do not however exclude the possibility of suspended load transport).

3. Cusp formations were in evidence under conditions of angled wave attack The formations were distorted at an angle to the beach inclination and the spacing was irregular. No major rip current system extending past the breaker zone was in evidence in the author's spilling wave test. In the case of the author's plunging wave test the distortion evident on the downdrift side of the tank was attributed more to a pile up of drift material rather than a rip current in its proper sense. (The reflection of the waves against the downdrift side of the tank made visual observation difficult).

4. The author's assumption, that although the angled wave attack tests were of short duration, the beaches in question at least approximated their equilibrium condition, is borne out by the appearance of cusps in the course of each test. It was observed during the course of the normal wave attack programme that cusps only started developing on a beach after the initial reshaping of the beach configuration had been completed.

CHAPTER 10

EXPERIMENTAL PROGRAMME REVIEW

The experimental programme as conducted was designed to investigate all basic beach responses to attacking wave characteristics. By establishing beach equilibrium profiles for various wave conditions, the ultimate aim of the study was to produce meaningful sand volume transfer predictions based on some fundamental wave characteristics. In retrospect, the author feels that although the programme established certain trends and tendencies, the variables under consideration should have been limited to allow a more in-depth study of certain of these trends and tendencies.

It is felt that the programme illustrated the fundamental importance of wave period as a wave characteristic for the determination of resultant beach profile configuration. The ultimate study should therefore include a wide range of wave periods, and each particular period should be developed over a full range of wave height values. A more limited study should concentrate on a smaller range of wave periods but should still include a full range of wave height values resulting in coverage of the full transition from berm to bar profiles, where possible. (The amount of wave height values studied for any particular period should be such that the sand volume transfer curves may be developed with accuracy). The effects of different depths of water to still water level can be considered secondary at this stage.

The sand volume transfer concept is considered a useful method of predicting beach profile response to wave attack, worthy of considerably more attention. The difficulty in predicting prototype magnitudes from model studies notwithstanding, model studies should be able to shed light on the processes at work and lead to a better understanding of the principles involved. Also requiring further attention would be the non-sequential relationships of identical profiles with regard to relative position from a fixed on-shore datum, and thus associated volume flow.

The formation and occurrence of three-dimensional flow patterns in the in-shore zone, viz rip currents and beach cusps, are also subjects worthy of further study. Associated with the formation of in-shore circulation patterns are concepts such as wave set-up and set-down, non-uniformity in wave height along a beach and the presence of edge-waves. The author considers a study of beach cusps on a moveable bed model as the subject matter of a thesis in its own right.

With regard to all the unsolved occurrences and phenomena associated with a normal wave attack programme, the author considers doubtful whether a study of an angled wave attack programme in the existing UCT model basin can lead to any meaningful results. It is felt that approximate estimates of littoral drift as a function of total longshore wave thrust (or longshore wave power, for that matter) are now available to the same order of accuracy as can be determined in the UCT wave basin, even incorporating the most sophisticated refinements and techniques. There remains, certainly, an inadequate knowledge of the turbulent flow fields in water waves, but a model basin of this nature is unsuited to the study of bed and suspended load sand transport which would require a detailed study of the boundary layer turbulence and exchange coefficient.

P A R T   T H R E E

FIELD INVESTIGATIONS

CHAPTER 11

OBJECTIVES OF THE PROGRAMME

The field programme was initially designed to gather information on, and study, littoral drift in the short term under field conditions. The programme was subsequently extended to investigate various other aspects of the nearshore zone, viz:

a) Sediment sample analysis in the surf zone
b) Beach sand analysis
c) Correlation of beach profile fluxes as a function of the impinging ocean wave conditions
d) Fluorescent tracer study of littoral drift in the swash zone

Two field trips were undertaken, the former of 4 weeks duration and the latter of 8 weeks. The initial visit was undertaken by Professor Kilner, a postgraduate colleague and the author. The purpose of this visit was to establish a work site, design a programme and develop and test techniques to be applied on the later visit. After a period of 2 months to allow equipment to be made and collected, the second trip was undertaken by the aforementioned in the company of 4 final year B.Sc. engineering students. These students subsequently submitted facets of the investigation as theses for their Bachelor degrees. The author and his colleague concerned themselves with overall supervision and coordination on their particular spheres of interest

CHAPTER 12

LOCATION OF INVESTIGATIONS

The field study was undertaken on the extreme southern portion of the South West African coast in the restricted diamond concession area near Oranjemund. The beach itself was situated approximately 12 km north of the Orange River mouth, between two CDM owned diamond recovery plants known as Plant 4 and Plant 100 G, respectively south and north of the beach. (see Figure 29).

The beach itself was chosen on the basis of being a relatively straight, unrestricted piece of coast-line with no indication of rocky outcrops in the vicinity. The initial site visit had revealed the beach to display a wide range of bathymetry, the whole coast-line being accessible to unrestricted ocean action. The coast-line was assured of an abundant supply of littoral material from an artificial stockpile of overburden to the immediate south of the test beach. The stockpile was accessible at all times to wave action and formed a dune approximately 8 m high and 1 km long along the coast-line.

12.1 WAVE AND CLIMATIC CONDITIONS

Measured by usual standards, the wave and climatic conditions active along the stretch of coast-line under consideration were extreme. The beach was directly exposed to the Atlantic Ocean with the cold Benguela current off-shore. The prevailing winds were South Westerly. Daily temperature ranges were large. Fog along the coastal belt occurred regularly at night, while clear skies dominated during the day.

Wave heights recorded during the study period averaged at 1,2 m. The waves in the nearshore zone tended to plunge and the surf zone was often vicious. Surging and spilling wave conditions were also observed. The surf zone tended to vary between 100 m and 200 m in width. A marked discolouration of the ocean due to suspended silt was frequently observed in the nearshore zone up to 1 km wide.

It should be noted that during the period of study the Orange River mouth was largely silted up, with relatively little river water flowing directly into the ocean. An abundant supply of littoral material was

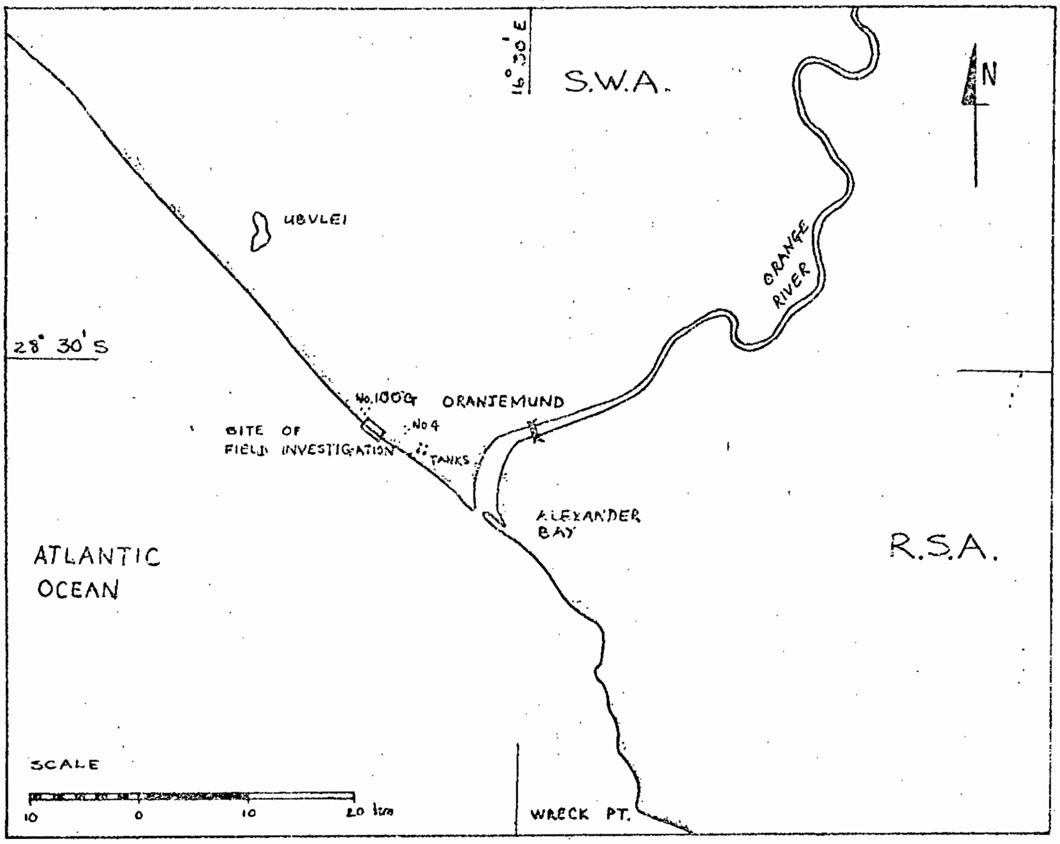


Figure 29 . Location of field study

available from the artificial dunes already mentioned, an additional source being the slimes outfalls from both recovery plants north and south of the test beach. The slimes originated from the diamond bearing gravel washing process. Of the two plants, Plant 4 to the south of the beach carried the greater volume. The volume of slimes as a source of littoral material was considered insignificant when compared to the artificial dunes. No east wind conditions (a possible source of considerable littoral material) were observed during the study.

Over the period of investigation, observations of conditions were made on an average of twice a day. The equipment available for the observation and recording of data was unsophisticated and limited. Methods employed were thus similarly so. Appendix H presents in graphical form a summary of collected data over the observation period.

Wave periods were gauged by stopwatch over a set of ten waves, the average of three such sets being recorded as the prevailing condition. Wave heights were estimated by eye by independent observers over a period of time, the mean being accepted as the significant wave height prior to breaking. Breaker angles were similarly gauged by eye, as was direction of littoral drift. A measure of the longshore current velocity was gauged by means of dyes and all manner of floating objects. Wind velocity and direction were measured by a simple hand held wind vane. Tides and tidal ranges were interpolated from tide tables available for Port Nolloth and Luderitz Bay.

12.2 BEACH SAND CHARACTERISTICS

It was considered necessary to establish the physical characteristics of the sand making up the beach in order to apply meaningful interpretations of beach fluxes as a result of external forces. Intensive sieve analyses and specific gravity tests by means of standard methods and a sampling programme worked out to avoid bias were undertaken by D. Gow of the B.Sc. engineering team.

The beach sand was found to be moderately well sorted ranging from very coarse sand to silt. Gravel fractions were not common, although cobbles and very coarse pebbles were occasionally in evidence and could frequently be heard in the near surf zone. Crushed shell and other marine animal debris was virtually non-existent on the beach. The grading analysis indicated bimodal tendencies at the landward fringe of the swash zone and at the berm crest. At both these points coarse material tended to collect. Surf conditions were such so as to prevent regular collections of samples from the surf or breaker zones, thus preventing comparison with the swash zone as regard size ranges and degree of sorting of the different zones.

When cusps occurred the increase in particle size towards the berm crest continued for both cusp horn and embayment profiles. Particle sizes along the embayment tended to be slightly finer than the horn. In general, however, the grading analysis showed little correlation between beach gradient and mean grain size, and no correlation between grain dimensions and beach erosion or accretion tendencies.

The beach was found to consist primarily of quartzitic sand, of relative density 2,65 and mean diameter of 500 microns. An increase of relative density for the smaller size sand fractions indicated the presence of magnetite with a relative density of 4,99. The magnetite itself was well graded with a mean diameter of 240 microns, considerably finer than the quartz. The percentage of magnetite in individual samples varied considerably, but generally tended to increase in the swash zone away from the sea.

A considerable amount of sand available to the beach undoubtedly came from the overburden dunes. This material was not foreign to the ocean regime, having in previous times formed part of the seabed. Whereas wind action would have robbed this stockpile of fine material from the surface, the depth from which this material was excavated and the manner in which it was stockpiled made this effect negligible.

Foreign material was in fact being introduced from the slimes outfalls. The volumes in question were, however (as has already been stated), considered relatively insignificant in comparison with the other sources of littoral material.

CHAPTER 13

BEACH VOLUME CHANGES AS A FUNCTION OF WAVE CHARACTERISTICS

In this section of the field investigation short term changes of the beach in the swash zone were correlated with the ocean wave characteristics in an attempt to establish the inter-relationship between the two. It should be noted that in addition to the wave data, a full record of the tide, wind velocity and wind direction conditions were recorded. The effects of these factors are similarly discussed when reviewing the results of the investigation.

13.1 MEASUREMENT OF BEACH VOLUME CHANGES

Fixed reference points on the test beach were established by means of 37 mm steel pipe section driven into the beach at regular intervals. The steel pipes, 1,2 m in length, were driven into the beach by means of a hammer to a depth of approximately 1 m. A 25 mm pipe section, similarly 1,2 m long was then joined to the implanted pipe. Figure 30 shows the relative grid positions, each number on each of the 5 grid profiles indicating a pipe position.

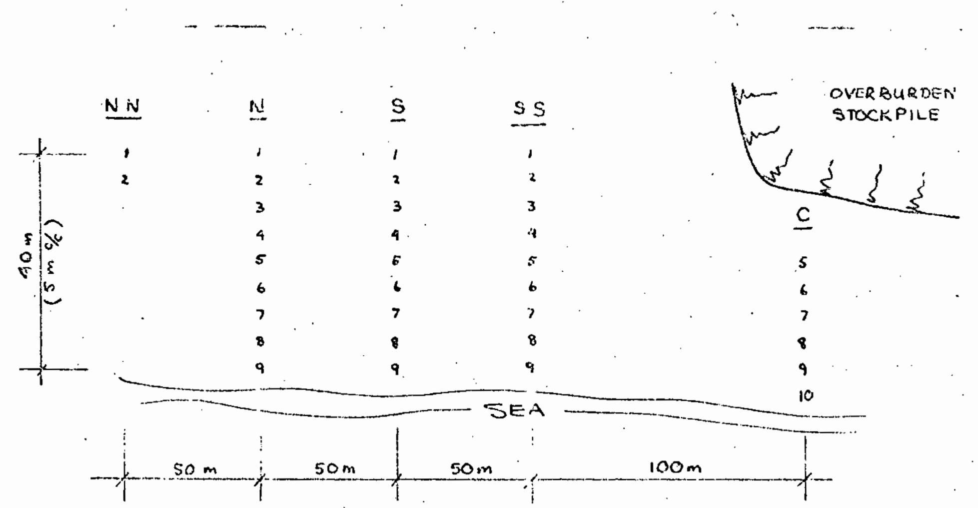


Figure 30 Grid reference system

Once the various grids had been established, measurement of profile changes were made by means of an automatic level. Initially (once the individual pipes had been coordinated by level), beach changes were gauged by measuring tape from fixed points on each individual pipe. After a number of pipes had either been broken at the joint by the power of the waves or covered by sand movement, the level and staff method had to be resorted to for each daily reading. In cases where pipes were no longer visible, positions were located by means of a measuring tape.

Figure 31 presents the beach change envelopes along 4 of the profile lines for the period of observation. Because of heavy and dangerous surf conditions it was not possible to record with any regularity daily changes of the two seaward most grid positions. The first three landward most grid positions for profiles N, S and SS were subsequently also disregarded due to disturbances caused by the investigating team. (During the period of observation only exceptionally high tides caused the swash to reach these positions).

The daily changes on the grid system are indicated in Appendix J. Beach profile contours versus time plots were developed for the N, S and C grids. Dashed lines indicate interpolated values based on the daily profile form in cases where level readings towards the seaward side of the profiles were unobtainable. It should be noted that the levels as indicated in Appendix J were based on an estimation of the mean sea level as datum. Also indicated on Appendix J is a time plot of the calculated deep water wave steepness, to be discussed later.

13.2 METHOD OF APPLICATION OF DATA

In the absence of continuous wave records a system had to be devised whereby recorded wave data could be applied to changes on the beach face. In general, the beach profiles were measured daily at low tide. (Exceptions to this rule were made when tidal ranges were small). Wave conditions were measured daily at both low and high tides. It was decided that, when the tidal range was 1 m or greater, the conditions prevailing at the previous high tide be accepted as representative of causing beach changes over that period. When the tidal range was less than a metre, the average of low and high tide conditions were selected. Appendix K presents chronological records of the recorded wave period and wave height data. The times as underlined in the table represent the values selected on the basis of the above mentioned method. The recorded wave conditions were expressed in terms of their deep water equivalents utilising linear wave theory and known empirical relationships in the following manner. Calculating H_{rms} from the relationship $H_{rms} = 0{,}71\ H_{sig}$ and depth at breaking from $d = 1{,}28\ H_{b(rms)}$, and calculating

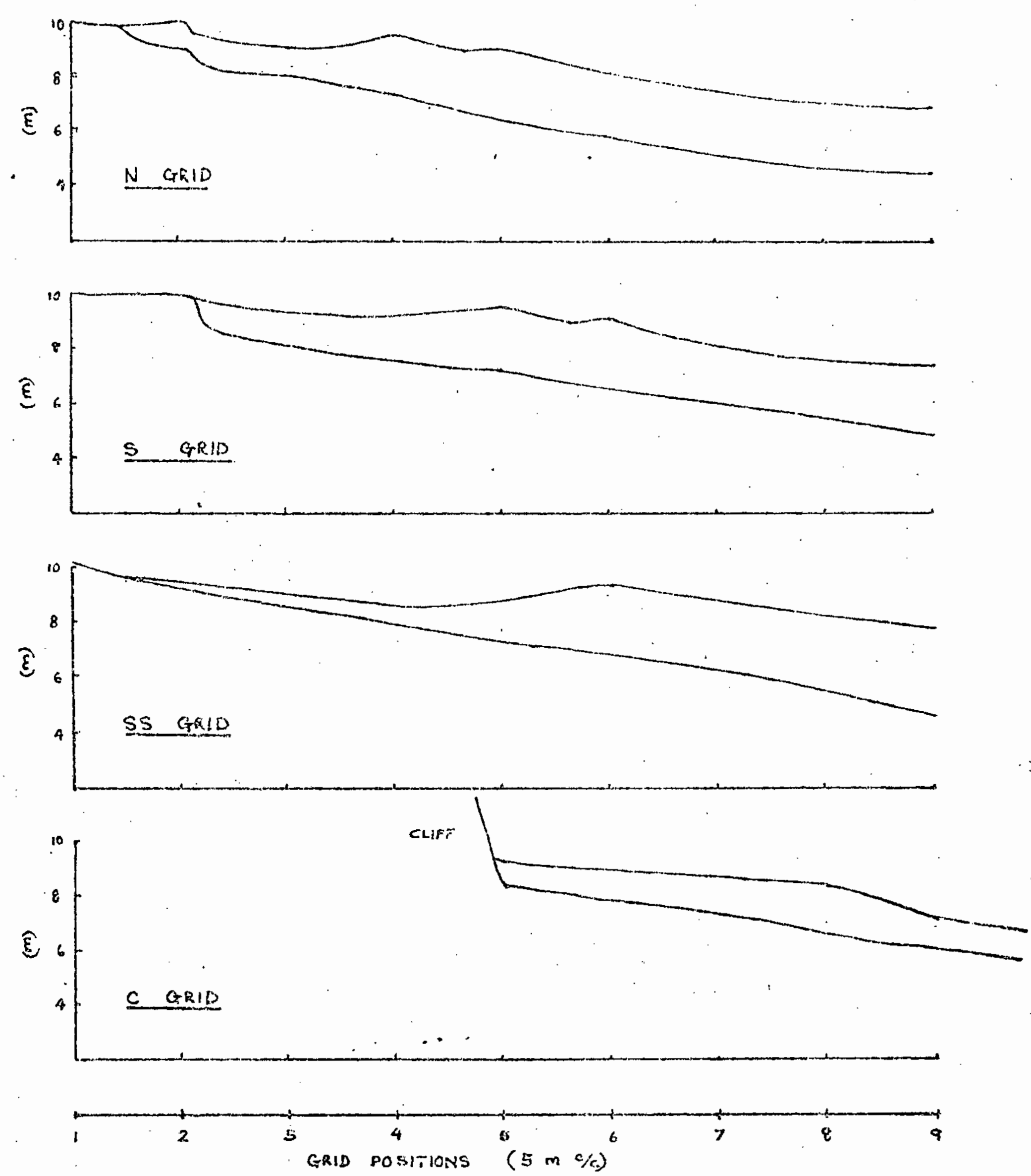


Figure 31 Beach profile changes over period of observation

the deep water wave length from equation (6) of Part 1, Wiegel's tables could be used to find the unrefracted wave height. From the above manipulation the deep water wave steepness was computed. No attempt was made to determine the refracted wave height as the angle of wave incidence was unknown. Over the period of investigation the breaker angle was observed to be small, thus differences between refracted and unrefracted values were expected to be marginal. The processed wave data is tabulated in Appendix L.

The changes in cross-sectional area of profiles N and S, the profiles for which the longest period of observations were available, are tabulated in Appendix M. As the investigation was aimed at recording changes in the beach volumes only, an arbitrary baseline was chosen as datum. It should be remembered (for reasons already explained) that the changes per cross-sectional area of each profile as represented in Appendix M are relative to a

20 m stretch of the mid swash zone of the beach. Quantitative measurements of the beach volume changes were confined to the section of beach confined by the N and S grids respectively. This situation resulted from the above mentioned grids being those first established and thus providing the longest continuous record. Also highly significant were the occurrence of cusp formations on the swash zone both at the beginning and end of the observation period. In the manner employed, volumetric changes on the beach while cusps were in evidence could not be gauged with any accuracy. The author furthermore suspected a possible imbalance of the system on profiles SS and C as a result of the close proximity of the overburden stockpile.

Appendix M presents in tabular form the cross-sectional areas of the N and S grids over the period of investigation. Changes are expressed in terms of the first recorded profile areas as datum, as well as in terms of daily changes. Horizontal lines across the table dividing the period 3 October to 5 October, and 24 October onwards, from the intervening period, demarcate the period over which cusp formations were present in the swash zone. Over the intervening period of 6 October to 23 October the beach was undistorted and regular. By computing the mean change in cross-sectional area of the two profiles and multiplying this by the distance between the two, a measure of volumetric change over the test beach could be gained. In the graphs to follow these fluxes are expressed in terms of volumetric change per unit width of beach.

13.3 RESULTS OF BEACH CHANGE INVESTIGATION

13.3.1 Beach change as a function of wave period and wave height

Figure 32(a) represents a plot of sequential beach volume changes plotted against the representative period of the waves over the time of formation. Each daily change is numerated on the graph by the date of formation. Only volume changes relevant to the non-cuspate beach are plotted.

The graph indicates only a vague trend associating increasing wave periods with deposition on the swash zone and shorter periods with erosive tendencies. The author feels that the data scatter is such that development of a regression line would be presumptuous. Sand volume changes associated with a wave period of 10,5 seconds vary between + 5 m^3/m beach and - 10 m^3/m beach respectively. Nevertheless, within the limits of the investigation the expected basic trend for longer waves to produce beach accretion and vice versa is in the author's opinion apparent from a study of the graph.

Figure 32(b), of similar construction to that of it's counterpart but with deep water wave height as the relevant wave characteristic, appears

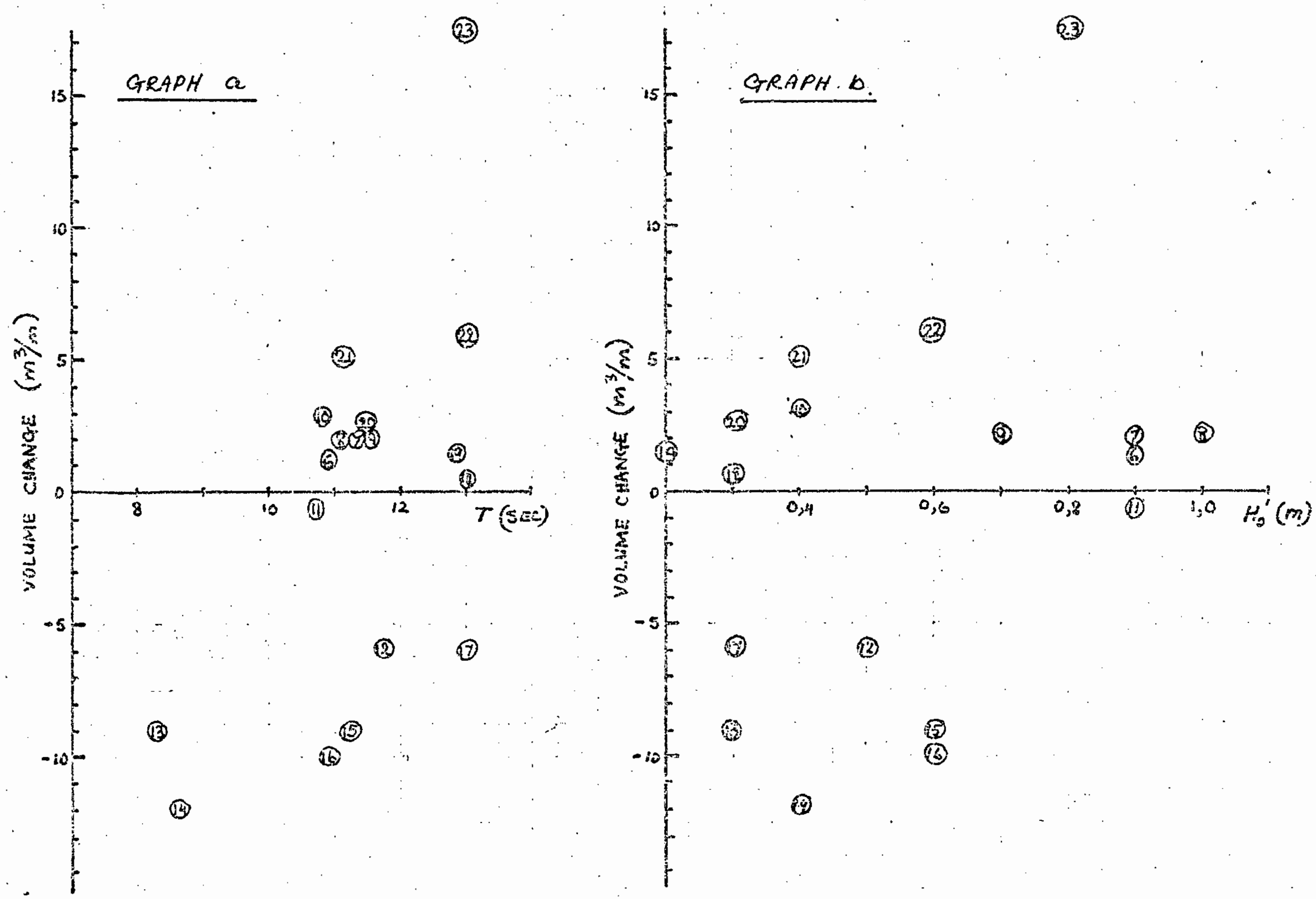


Figure 32 Sequential volume changes, versus a) wave period b) wave height

to indicate no basic trend. Data scatter is such that a study of the graph cannot predict or even imply a basic relationship between beach accretion/ erosion and wave height by the methods employed in the investigation. Under the circumstances, further comparisons were based on wave steepness and not wave power (the latter with its exaggerated effect of wave height, less easily measured than wave period).

13.3.2 Beach change as a function of wave steepness

Figure 33 depicts beach volume changes expressed in terms of deep water wave steepness. In this figure again as in the previous figures readings are indicated by a figure depicting the relevant date of the reading. In this case however the full range of observations are plotted, i.e. cusp and non-cuspate beach values.

A study of the graph in detail reveals certain apparent relationships and non-relationships. The volume changes on a cuspate beach reveal no relationship with wave steepness. This was to be expected as the method determining volume change was not inherently accurate for irregular profile beaches. Whereas the non-cuspate beach volume changes do not predict a pattern as a whole, sequential values studied in isolation indicate certain trends, especially when viewed with due consideration to the prevailing tidal conditions. Up to a deep water wave steepness of approximately 0,0035, increments in wave steepness for an increasing tidal range reveal swash zone accretion, whereas similar increments but with a decreasing tidal range appear to result in swash zone erosion. For steeper waves, in the range 0,0035 to 0,0052, the beach tends to a considerably less determinable trend for a rising tidal range than that at the lower steepness values.

Expressed in terms of daily changes, a beach flux versus wave steepness plot as depicted by Figure 34 reveals little more than a progressive tendency towards erosion with increasing wave steepness. It is noteworthy that even the cuspate volume fluxes appear to follow this trend. During the period of observation the beach cusps showed little inclination to migrate, and the short term beach fluxes over the period of their occurrence indicate tendencies in phase with that of plane beaches.

13.3.3 Beach change with compensation for tides

In view of the trends discernible in Figure 33, an attempt was made to nullify the effect of varying tidal ranges when making comparisons of beach changes and a wave characteristic. In all the comparisons previously done in the investigation a static baseline was used as horizontal datum when calculating profile volumes. A new method was applied in which the baseline

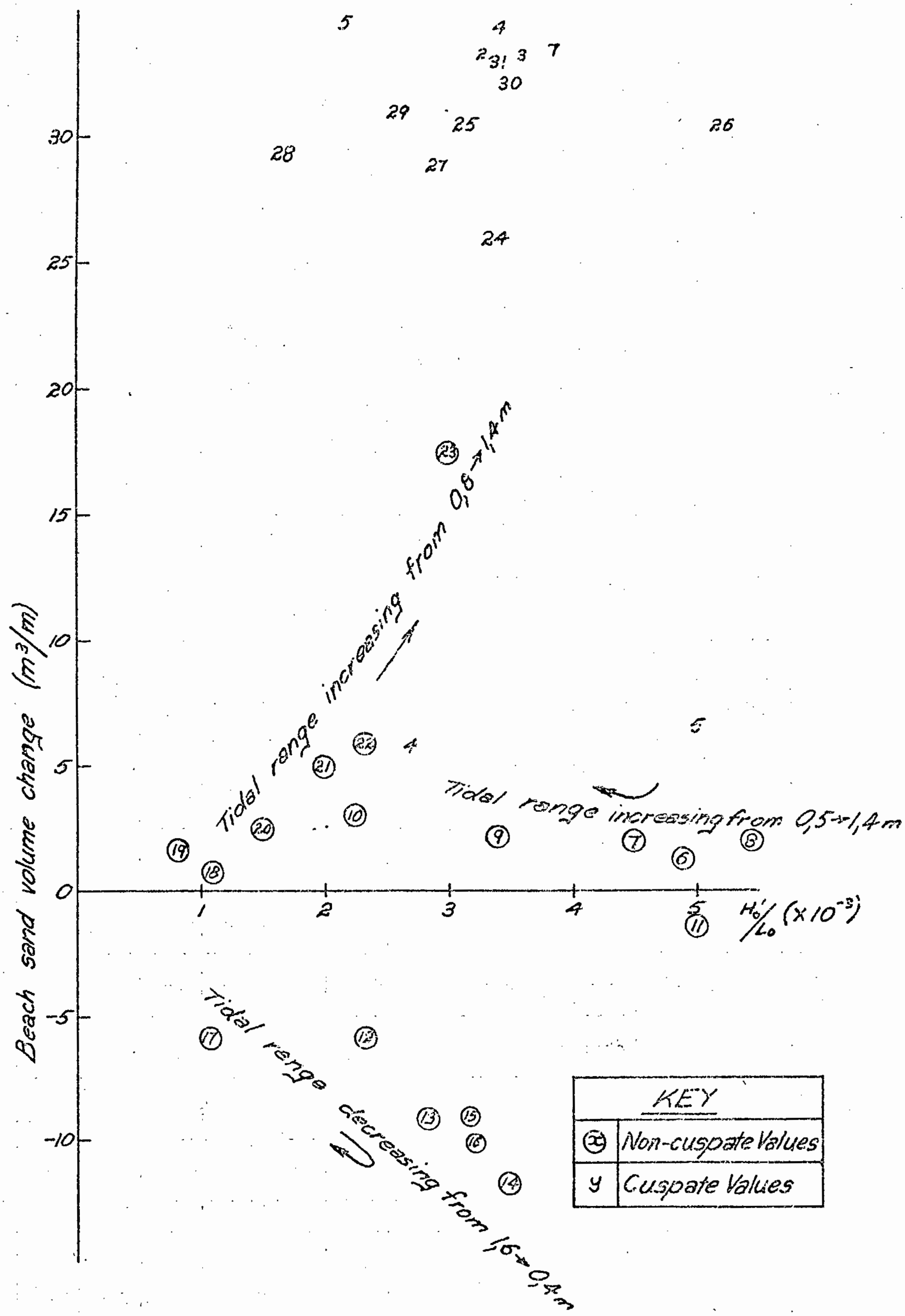


Figure 33 Sequential beach volume changes versus wave steepness

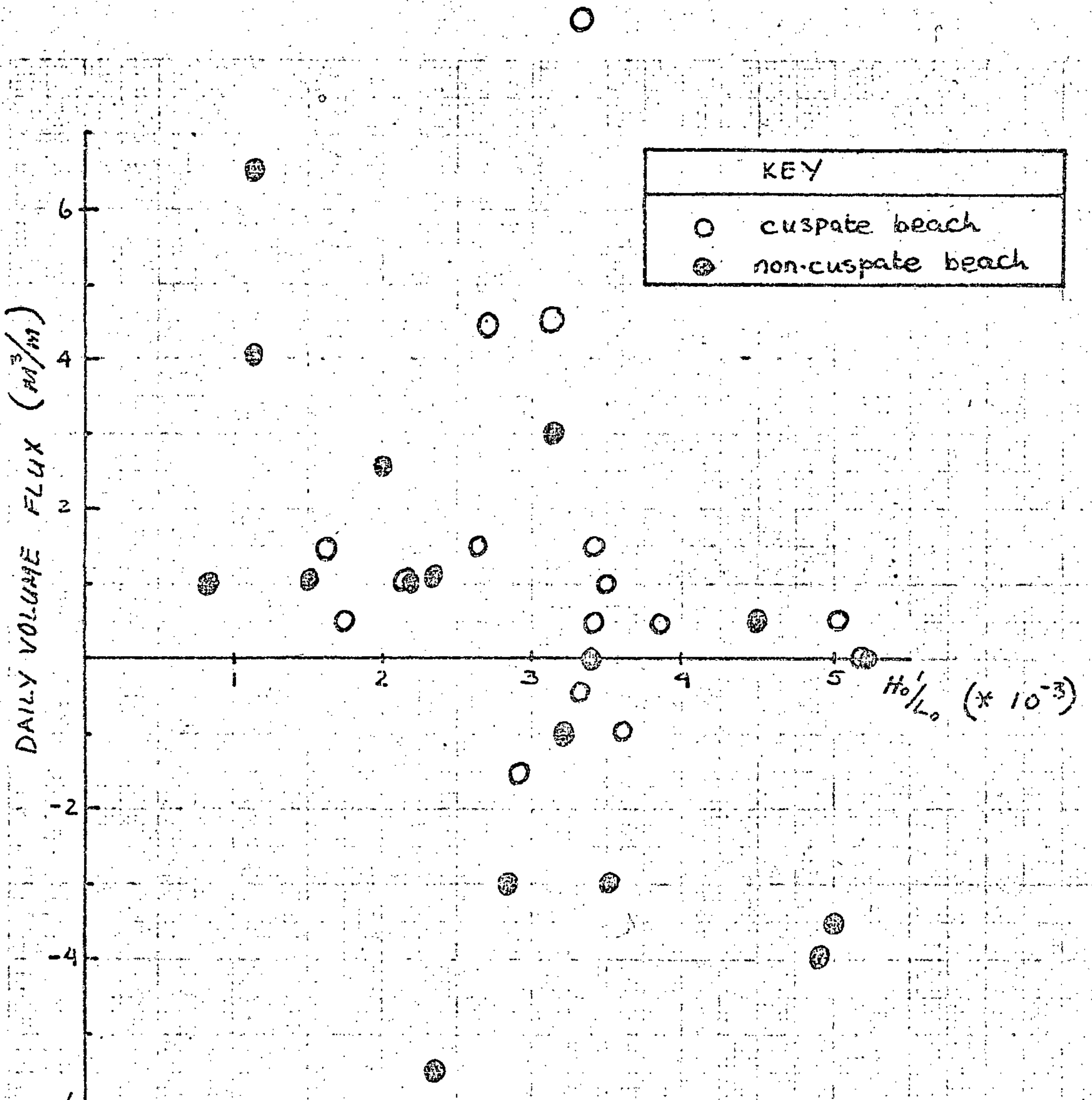


Figure 34 Daily beach volume fluxes versus deep water wave steepness

was correlated to the tidal range in such a way that an increasing tidal range raised the datum and vice versa. When applied the method had the effect of reducing volume changes on an increasing tidal range, and increasing volume changes on decreasing tidal range. No data scatter was eliminated and trends in effect became more vague. The method was regarded as unsuccessful in its object and discarded.

Regarding the effects of tides themselves as opposed to tidal ranges, on one particular day of the investigation, i.e. 23 October, profiles were gauged first at low tide and later at high tide of the same day. The tidal range at the time was 1,2 m. The basic beach gain was 5 m^3/m beach. This occurred over a period of 8 hours, and corresponded to a deep water wave steepness reduction from 0,0025 to 0,0022. Thus during the period in question 250 m^3 of sand was added to the beach between grids N and S, at a rate of approximately 31 m^3/hr (or 0,6 m^3/m beach).

13.3.4 Maximum daily rates of erosion and accretion

During the period of observation the maximum net gain to the beach took place from 24 October to 26 October. This gain took place over a period of apparent increase in wave steepness. Expressed in terms of a daily flux (i.e two tidal cycles), the maximum rate of accretion between grids N and S was 8 m^3/m day. The accretion took place over a period of increasing tidal range.

The maximum net loss to the beach took place from 13 October to 14 October. This loss occurred over again an apparent increase in wave steepness, although in this case the tidal range was on the decrease. The measured rate of erosion was 3 m^3/m/day between the N and S grids. Over the period of observation thus beach accretion took place at a considerable faster rate than beach erosion when measured in terms of daily beach fluxes.

13.3.5 Cusp formations in the swash zone

During the 35 day period of observation cusps were identified for 46% of the period. The cusps, which were present at the beginning of the study, disappeared after the first neap tide and re-appeared on the following spring tide. Cusps were apparent on both eroding and accreting beaches. The cusps appeared relatively static and showed little positive tendency to migrate either up or down the beach. The maximum amount of lateral movement of one particular horn over a period of 13 days was 20 m south. On occasion the cusps appeared to align themselves at an angle to the ocean line, these occasions co-inciding with times of observed longshore currents. The cusp spacing or wave length varied between 25 and 50 metres, often cusps of varied spacing were observed along the beach at the same time. One interesting observation was made when a cusp sequence of 25 m spacing superimposed itself over already existing cusps of 50 m spacing. The two sequences appeared out of phase with each other. Cusp pitch, or vertical difference between horn and embayment varied greatly. Observations indicated to the author the appearance of first the cusp horns with subsequent and progressive erosion of the embayments.

CHAPTER 14

FLUORESCENT TRACER STUDIES IN THE SWASH ZONE

The choice of fluorescent tracer instead of radioactive tracer for the study of littoral drift in the swash zone was based on the budget available for the project. Both techniques have their obvious advantages and disadvantages. The choice of fluorescent tracer as a working medium was made with due recognition of the probable superiority of the radioactive tracer method, the greater hazards of application of radioactive tracer notwithstanding.

14.1 APPARATUS

Certain innovations of scanning process developed by Professor Kilner of the University of Cape Town are worthy of note. The obvious restrictiveness of night time scanning by means of a portable generator and ultra-violet lamp led to the development of a light and compact daylight scanning box with a viewing area of 0,08 m^2. The light-proof visual scanner was attached by a 30 m lead to a portable generator giving the viewer considerable range even without moving the generator. In anticipation of the difficulty of making accurate visual tracer counts, especially when large concentrations of tracer were encountered, a mobile camera unit was successfully designed. Made of plywood with a built-in 35 mm camera, the unit included a visual observation bay. The unit could be carried by two men. (see plate 12). Colour slides representing an area of 0,4 m^2 could be produced, allowing tracer counts to be made with a high degree of accuracy. A system was anticipated whereby the smaller and lighter visual scanner would be used to reconnoitre the beach, the photographic unit would then only be applied to strategic points on the grid.

The basic system of pipes on the test beach were used throughout the study as reference points. Tracer dumped at various point on the beach was related to the grids, as were all subsequent scanning operations.

14.2 THE MANUFACTURE OF TRACER SAND

Sand to be used as tracer was collected directly from the test beach as well as the overburden stockpile. Both sources were considered to form part of the littoral environment with similar physical and hydrodynamic properties. The sand was oven-dried before being placed in a mechanical mixing pan to which fluorescent paint and thinners were added.

The fluorescent paint was supplied by Dulux in the following colours: fire orange, signal green and arc chrome. The paint had a relative density of 1,1. Thinners, trademark 'Toluol' with a relative density of 0,8 was used. For the moderately well sorted medium size sand under consideration the most successful mix proportion proved to be 2,5 litres paint plus 1,8 litres thinners per 60 kg dry sand.

Once the sand and paint was properly mixed (approximately 8 minutes per 20 kg), the tracer was placed in shallow galvanised steel trays and dried in the sun. Although the sand was continuously turned the drying process was found to take up to 5 days, dependent on weather conditions. Due to the inflammable thinners in the mixture it was not considered advisable to oven-dry the tracer. Once dry the tracer was sieved to break up lumps and batched in 20 kg bags ready for dumping. Prior to dumping, the mixture was thoroughly wetted with sea water mixed with a commercial liquid detergent as wetting agent.

14.3 METHODS OF TRACER DUMPING AND RECOVERY

Predetermined amounts of tracer were dumped onto the beach at certain positions of the grid system. Tracer was dumped on an incoming tide between successive backwashes. After unloading, the tracer was immediately spread out so as not to form an obstruction to the swash or backwash. At all times steps were taken not to contaminate the beach with tracer from the investigators' bodies or equipment.

After dumping the tracer, two full tidal cycles (approximately 24 hours) were allowed to take place before scanning operations began. The procedure then was to scan the beach with the visual scanner in such a manner that the extent of tracer spread could be determined both as to distribution and position on the beach. Scanning points at 10 m intervals normal to the shoreline were viewed. At each view point an area of 0,16 m^2 was scanned (i.e. two views through the small scanner). Because of the area to be covered before the succeeding high tide a greater area of scanning per view point was not possible.

The photographic scanning unit was not extensively used during the scanning programmes because of certain practicalities. The most important of these was the exceedingly low tracer recovery. Low tracer counts allowed for accurate visual assessment and photographic methods were thus not required.

14.4 TRACER STUDY RESULTS

14.4.1 Identification

The three basic tracer colours proved distinguishable from the natural fluorescence of the beach sand with varying degrees of success. Most brilliant was the fire orange tracer, closely followed by signal green. Both the aforementioned were judged to be entirely satisfactory. Arc chrome coloured tracer was less conspicuous and not as easily identifiable, resulting in considerable eye strain during scanning operations.

14.4.2 Representative sample

Tests made in order to compare the scanned area per view point as a sample population of a metre squared yielded progressively poorer results in inverse proportion to the amount of tracer recovered. As the overall tracer recovery was very low, the variability was relatively high. Of 3 tests done on a selected square metre of beach the variance of the samples was found to be up to 50% of the mean for the lower tracer counts, reducing as the tracer count increased.

14.4.3 Depth of disturbance

The maximum depth of disturbance measured by tracer counts at different depths under the surface on the test beach was found to be 125 mm. Observations of this nature were difficult to make because of the non-static nature of the swash zone. Very rarely had no erosion or deposition not taken place between the initial tracer dump and the scanning process 24 hours later. During the period of observation beach accretion of up to 1 m per 24 hours was measured. After one such period of accretion tracer was discovered 365 mm below the surface.

Attempts to establish a correlation between tracer visible on the beach surface and buried tracer were hampered by the changes in beach elevation between viewings. Under a period of relative beach stability a sample population of 11 test pits revealed a remarkably constant 73% of the visible surface tracer located buried within the top 70 mm of the beach. Because of changing conditions however, no accurate correlation between relative amounts of buried and surface gracer could be established. Graphical representation of tracer spread was thus limited to visible surface tracer.

14.4.4 Hydrodynamic and physical properties of the tracer sand

During the course of the programme, investigations by B.Sc. student B. Nicol revealed certain changes in the hydrodynamic properties of the tracer sands. Under the circumstances the significance of the changes could not be determined. Nicol found that tracer-coated sand particles of diameter greater than 0,71 mm showed an increase in relative density. As the paint itself had a relatively low relative density, he attributed the increase as due to finer particles of magnetite being trapped in the paint layer on the larger quartz sand particles.

In general, the coating of smaller particles with paint caused a reduction in relative density as expected. Tracer samples showed an increase in the larger particle size range and a consequent decrease in the smaller size range when compared to untreated sand samples.

Supplementary tests revealed no deterioration of tracer brilliance with either time or when subjected to heat treatment. The detergent used for wetting purposes did not appear to effect the tracer brilliance.

14.5 INDIVIDUAL TEST PROGRAMME RESULTS

In all, 5 tracer dumping and viewing programmes were undertaken, the details of which appear in Figure 35.

	Date	Tracer Dump Load	Position	Conditions	Drift	General
1	5.10 - 7.10	20 kg red daily	N 5,5	Eroding	-	No cusps, low tidal range
2	10.10 - 16.10	20 kg red daily	N 5,5	Eroding	North	No cusps, high tidal range
3	18.10	120 kg chrome	N 5,5	Accreting	-	No cusps, low tidal range
4	23.10 - 28.10	60 kg green daily	Mid NS 5,5	Accreting	North	Cusps, high tidal range
5	5.11	240 kg green	SS 5,5	Accreting	-	Cusps, medium tidal range

Figure 35 Tracer programme details

14.5.1 Programme 1

No usable concentrations of tracer was discovered on the beach near to the release point or elsewhere. Little trace of buried tracer could be found. The test period coincided with a period of beach erosion. The tracer appeared to be moved directly off the swash zone into the surf zone with little or no diffusion taking place on the beach. No isopleth plots could be made. It was decided to increase the period of dumping for the subsequent test programmes.

14.5.2 Programme 2

This programme in which the daily dumping of 20 kg of tracer was extended to 7 consecutive days took place under similar conditions to that of the preceding programme. The erosion over the period of observation was less extreme however, and the presence of a northerly longshore current in the surf zone was detected.

The surface tracer counts for the programme were found to be very low. Isopleths for the net period are shown in Figure 36. The isopleths reveal a measure of tracer diffusion in the immediate vicinity of the dumping area but no evidence of littoral drift. Northerly longshore currents were in evidence over the test period, and visual observations indicated that littoral drift was taking place. Tracer movement in the swash zone, as recorded in the preceding test programme, appeared to be directly into the surf zone.

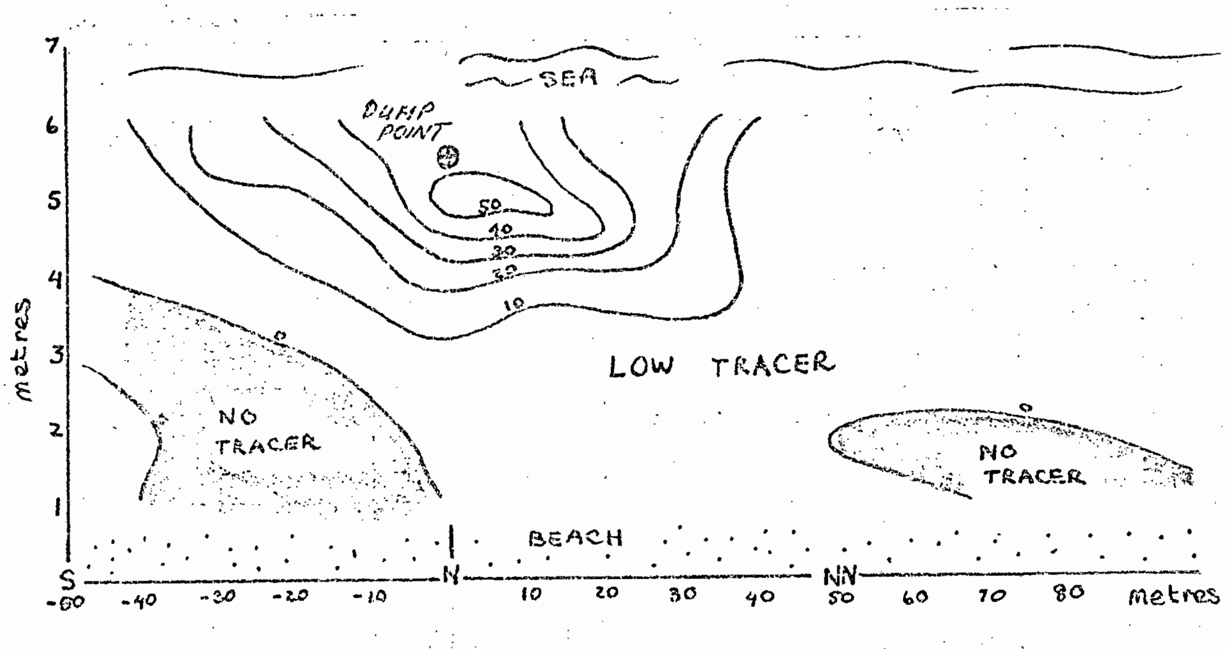


Figure 36 Surface tracer distribution -- Programme 2

14.5.3 Programme 3

The programme took the form of a single mass dumping of 120 kg of arc chrome tracer, the scanning process taking place after 24 hours. The beach showed signs of accreting over the intervening period, and no drift was in evidence. The scanning process failed to locate any significant surface tracer concentration anywhere on the beach. Test pits revealed the presence of tracer 360 mm below the surface of the beach in the immediate vicinity of dump spot. This was taken as indicative of the amount of accretion having taken place, rather than the depth of disturbance in the swash zone.

On the strength of the programme results it was decided to discontinue the use of arch chrome as a tracer medium. While still distinguishable from the natural fluorescence on the beach, the colour difference was not as distinct as with the other tracer sands.

14.5.4 Programme 4

The low recovery results of the previous programmes let the investigating team to treble the dump load of programme 2 over the same period of 7 constructive days. The investigation period coincided with an extensive build-up of the beach and the appearance of cusps in the swash zone. Strong northerly longshore currents were in evidence over the period, with current strengths of up to 2 m/s. Littoral drift was in evidence. Isopleths for the net period are given in Figure 37.

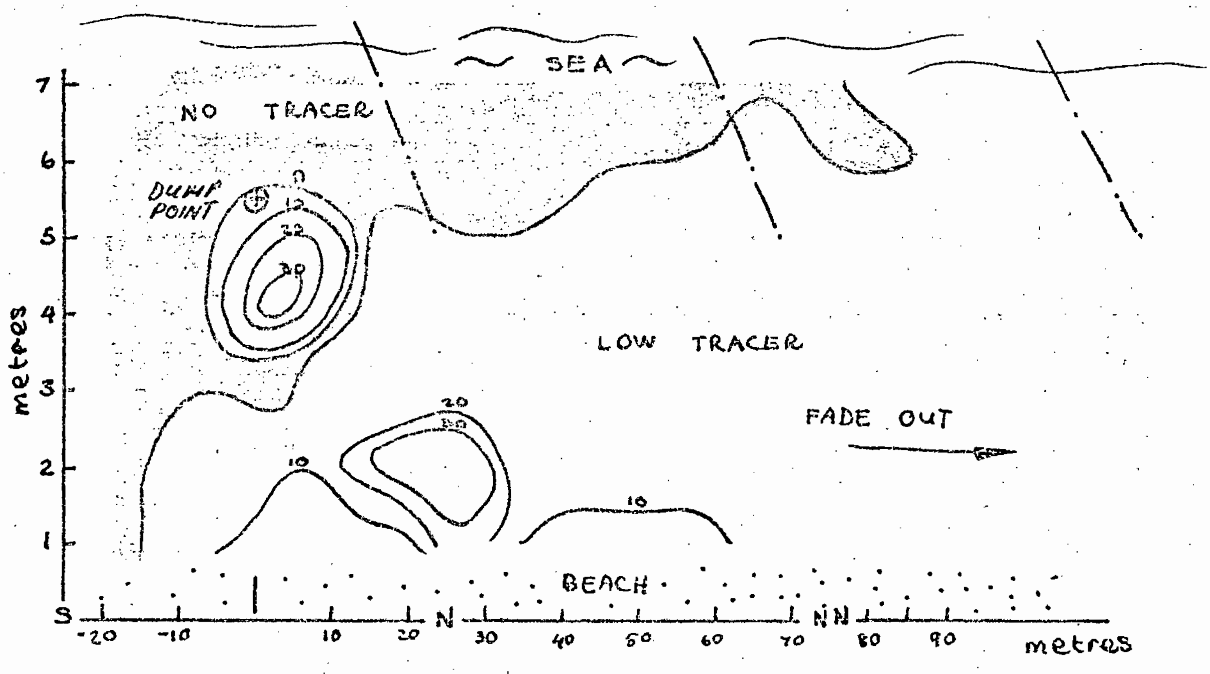


Figure 37 Surface tracer distribution - Programme 4
(contours equal tracer grains per square metre)

Figure 37 reveals that the increased amount of tracer released did not result in greater tracer counts. The beach showed aggressive accretion tendencies over the period of observation. A berm was quickly built-up in line with grid pole position 5 parallel to the beach, with the formation of a lagoon landward of the berm. Into this lagoon the larger waves carried quantities of tracer which were subsequently deposited and played no further part in the swash action.

Low tracer counts in a northerly direction along the beach, although inconsistent and of minor proportions, for the first time supported visual observations of the prevailing northerly littoral drift. The tracer counts faded out within 100 m north of the dump spot. The dashed lines on Figure 37 show the position and inclination of the prevailing cusp horns. The tracer appeared to disappear directly off the beach from a cusp embayment and reappear on the cusp horns. The tracer spread was thus attributed to transport in the surf zone and not to a direct cross-flow from one cusp to another.

14.5.5 Programme 5

The last test programme took the form of a mass dump of 240 kg of tracer into the landward extremity of the surf zone. Scanning was begun within 30 minutes of dumping. Although tracer counts had to be made quickly between successive swashes it was hoped that more tracer would be recovered. The results of the programme are given in Figure 38.

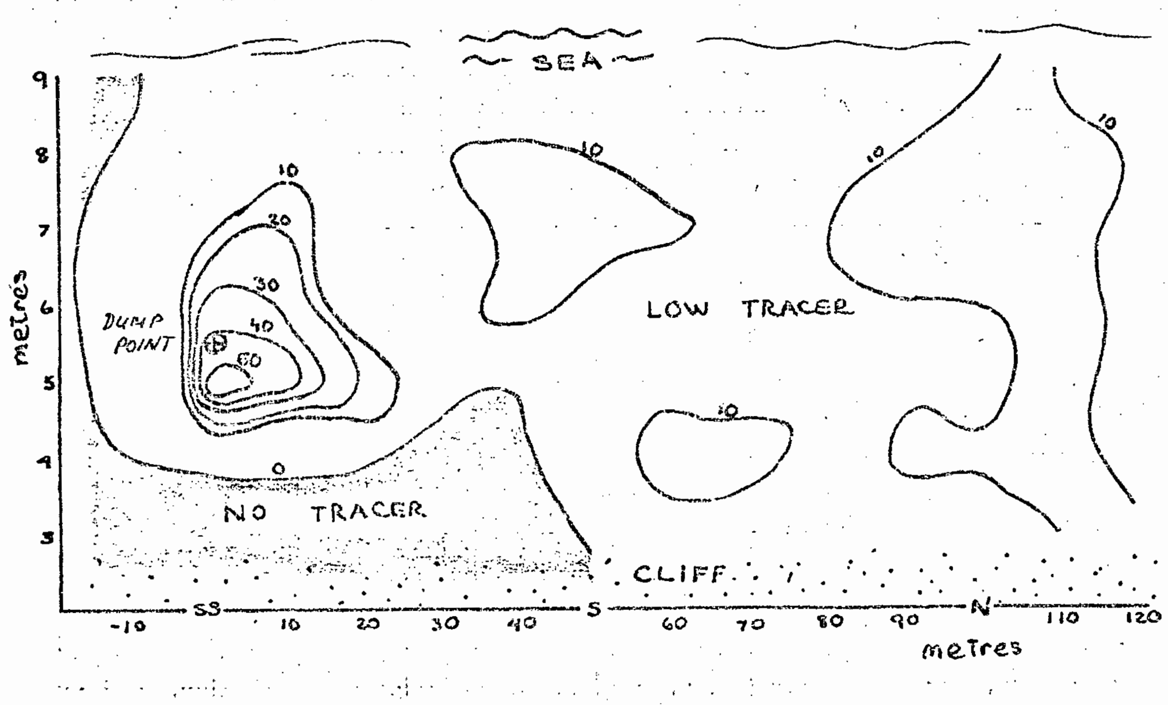


Figure 38 Surface tracer distribution - Programme 5 (contours equal tracer grains per square metre)

The test programme took place over a period of relative beach stability. Only a slight accretion tendency was in evidence and no drift was apparent. Cusps were in evidence on the beach.

In spite of the change in tactics, low tracer counts were again in evidence. Once again the tracer tended to disappear directly into the surf zone. Whereas no longshore current or littoral drift was apparent in the surf zone, the tracer distribution revealed a measure of littoral drift in the swash zone in a northerly direction. The scanning programme had to be abandoned after a relatively short period of time because of the incoming tide. Scanning was resumed the following day at low tide. Extremely low tracer recovery was found local to the dumping spot with no basic trends in evidence.

C H A P T E R 15

FIELD PROGRAMME CONCLUSIONS

15.1 BEACH VOLUME CHANGE PROGRAMME

The beach volume change programme was aimed at developing relationships between representative wave characteristics and changes on the beach face attributed to these characteristics. Certain wave conditions were expected to produce a beach condition in equilibrium with the wave forces acting on it. Subsequent changes in these forces would result in beach volume change and by measuring these it was hoped to correlate the volume changes in a positive manner to forces governing them. The relationships thus developed could then be compared to similar relationships developed in the model study under controlled conditions.

With the methods employed, the field investigation failed to establish the well defined relationship expected between beach volume changes and the deep water wave steepness.

The results indicated a basic trend towards swash zone erosion with progressively steeper waves when daily fluxes were considered. As expected, a beach volume change comparison based on wave period showed swash zone accretion with increasing periods. Wave height studied in isolation revealed no underlying trend. No constructive beach effect due to increasing wave steepness could be discerned from the daily flux graph. (Such relationships were successfully developed in the model study). Consequently, in the field study no frontier between constructive and destructive wave action could be established. Under the prevailing conditions it was not possible to investigate beach configuration in the surf zone and the occurrence of sub-marine bars and steps.

Net beach volume changes over a period of time indicated a possible effect of tidal ranges on beach configuration. Efforts to establish a relationship between tidal range, wave steepness and beach changes were unsuccessful. Neither did the method employed allow for a study of the effect of tidal phases and ground water conditions on beach changes.

The time series of beach changes represented by contour times and a plot of wave steepnesses as presented in Appendix J, shows little direct correlation between beach volume change and wave steepness. A period of erosion towards the beginning of the time series is associated with a relatively high and constant wave steepness. A period of considerable beach accretion from 22 October to 26 October was associated with wave steepness increases from 0,001 to 0,004. Unfortunately, as the period was associated with the appearance and growth of cusps, the beach contour time series cannot as a whole be accepted as truly representative of beach volume changes, as profile lines tended to coincide with cusp horns. All profiles plotted in this manner nevertheless revealed similar overall trends.

In general the overall results of the study revealed an extremely mobile swash zone. The beach profiles were in a constant state of flux, responding continuously to changes in wave and tide characteristics. In the light of the programme results the author is led to the following conclusions

a) Ocean conditions

It is probable that the methods used to collect the raw wave data and the application of this data to produce conditions subsequently accepted as representative of the prevailing wave climate acting on the beach over a period of time, were an over simplification of the true state of affairs. The local wave climate displayed considerable variability and the author is forced to conclude that the chosen wave characteristics could well have been unrepresentative.

b) Beach conditions

It is probable that because of the extremes and variability of the wave conditions acting on the beach in question, the equilibrium state was rarely, if ever, attained.

Beach profiles not in equilibrium with the impinging wave conditions and measured before a near to equilibrium condition was attained could possibly indicate initial beach changes not really indicative of the final result. It was frequently noted during the model study how constructive wave action, while eventually resulting in beach accretion, would initially have the effect of eroding or 'combing down' any irregularities foreign to the new wave conditions before accretion could begin. Seen in this light, measured beach changes might not be representative of the whole unless the beach is well established on its way to equilibrium.

The presence of the readily accessible and abundant stockpile of overburden material to the immediate south of the test beach resulted in a measure of profile distortion. This took the form of a delta effect as more material was available to the littoral environment than could be displaced by littoral drift. The distortion was greatest in the south, adjacent to the stockpile, and gradually disappeared towards the north.

15.2 FLUORESCENT TRACER PROGRAMME

The tracer programme results indicate that under the conditions experienced and the method as applied, fluorescent tracer studies as a means of gauging or even predicting littoral drift in the swash zone are not particularly successful. The programme results revealed the rapid transportation of a larger portion of the tracer sands directly off-shore with relatively confined dispersion in the immediate vicinity of the dump point. The scanning process revealed a very low incidence of surface tracer. The various test pits investigated revealed significant quantities of buried tracer not visible to a surface analysis. No correlation between surface and buried tracer could be established because of the mobile beach. Under the circumstances sophisticated statistical methods of analysis such as trend surface analysis were not justified.

The author considers that tracer studies on a mobile beach would possibly be more successful and certainly more conclusive if radioactive rather than a fluorescent tracer be used, (the extra hazards of radioactive tracer notwithstanding). A fluorescent tracer study is by its very nature not conducive to gauging the occurrence of buried tracer, a problem not encountered with radioactive tracer studies.

The fluorescent tracer study, of necessity confined to a tiny portion of the in-shore zone, could not, in the author's opinion, produce any quantitative results regarding littoral transport in the zone of immediate concern to coastal engineers. If such a study is to be extended into the surf zone as was done by Ingle (1966), the study area would have to be chosen with regard to the local wave climate. Both radioactive as well as fluorescent tracer studies would have limited application in an aggressive surf and breaker zone.

Regarding the method of production of the tracer sand, the observed tendency for considerable quantities of tracer to move directly into the surf zone, could possibly have been the result of hydrodynamic inequilibrium on the part of the tracer sands. The significance of the slight physical changes of the tracer sands when compared to untreated samples as to hydrodynamic properties would be best investigated under laboratory conditions. The mobility of the swash zone is well known however, and it is doubtful whether the hydrodynamic properties of the tracer sands were in fact significantly affected.

The problem of scanning buried tracer could in part be alleviated by collecting samples of the beach sands at various points and up to various depths. These samples could then be thinly spread out in uncontaminated pans thus promoting ease of tracer counts. Were this method to be applied, considerable experimenting would be required to establish the size of representative samples. In the author's opinion such a method would have to be resorted to for fluorescent tracer study results to be of any significance.

CHAPTER 16

REVIEW OF PARTS ONE, TWO AND THREE

It appears evident from a study of the literature that while considerable work has been done on the subject of wave theory and sediment transport in the nearshore zone, many of the processes at work are yet to be fully understood. Recent advances in the field have allowed a clearer theoretical picture of certain aspects, such as Bagnold's theory of sediment transport and Longuet-Higgins' concept of momentum flux, but the application of these ideas is limited in practice by the complexity of conditions in the surf zone. Current theory often relies on very inadequate mathematical approximations, the assumptions involved at times demonstrably invalid. Even so, these theories often give results in reasonable agreement with observation, and their use should certainly not be restricted (in the interim), provided their limitations are realised and they be regarded as the first step towards understanding a phenomenon. The success of the application of the linear Airy theory in the non-linear nearshore zone should be regarded in this light. An adequate theory for breaking waves, describing in detail both the laminar and turbulent regions of flow, has yet to be developed.

Bagnold's theory of sediment transport makes use of a coefficient of friction which is strictly speaking only applicable to bed load transport. Suspended load transport is still under investigation. Investigators still differ in their opinions of the relative importance of bed and suspended load transport.

Nearshore circulation patterns are an observed fact, the three-dimensional nature of which further complicates an already complex situation in the nearshore zone. Forms of sand accumulation and transfer in the beach zone occur in a great variety of sizes and shapes due to the complex interaction of changing wave conditions, changing water levels and even reversing tidal currents. Certain investigators feel, however, that there is a simplicity and order to be found with respect to sediment accumulation in the beach zone, although field work has really only begun on the problem of precisely determining the distribution and origin of the various sand accumulation forms.

The mechanics of the generation of longshore currents by breaking waves has been investigated to the extent that a semi-quantitative theory for predicting the currents and the sediment transport is available. The step from longshore currents to littoral drift involves empirical factors, provided either from models or prototype data. With rationalisation a simple relationship has evolved connecting wave and sediment characteristics, but application of any equation so derived should be made with due recognition of the natural variations in energy source, bed material and coastal configuration.

When comparing the experimental model results to those of the field programme, the differences in approach used for the two studies must be emphasised. In the model study it was possible to establish equilibrium profiles and comparisons could thus be made between various combinations of wave characteristics and their respective stable profiles. This could be done to the exclusion of all other factors with an influence on the profiles. Under field conditions it was possible only to work with the magnitude and rates of change of the beach profile under the assumption that the changes were the result of the profile moving towards the equilibrium state (enforced on it by a certain set of wave characteristics). The impinging waves in the field changed at random, the dominant conditions not necessarily constant for any length of time. Whereas the model study allowed control of wave direction, nature did not. Both studies were three-dimensional in nature. In each case, however, beach volume changes were expressed in terms of unit length of the beach. In the case of the model study this apparent contradiction of fact was overcome by calculating average profile configuration across the width of the model tank. An approach of this nature was not possible under field conditions. The study was thus confined to times when the beach did not visibly display discontinuities, such as cusps in the swash zone, being further limited to a selected area of beach displaying regular and similar profile changes at both extremities.

It is unfortunate that the field study could not define the limit between destructive and constructive wave action. Short term beach accretion cannot be judged the sole consequence of constructive wave action because of the nature of littoral drift. The author considers it feasible that beach accretion may take place under destructive wave conditions if more material is moved into an area (longshore) than may be moved seaward by the destructive waves. The close proximity of the overburden stockpiles to the south of the test beach could thus influence profile changes in a manner to which a normal beach without such a source of readily available littoral material might not have reacted.

It is the author's opinion that this study, incorporating both model and field investigations, while in itself having produced few results of a practical nature, has provided many pointers towards further more refined, and thus hopefully more conclusive, studies.

While it is accepted that the scale-determination and calibration of coastal movable bed models have, in general, not provided calibrated models yielding reliable projected prototype results, the value of model studies must not be under-estimated. Even accepting model distortion, invaluable insight into the processes at work are much easier observed and recorded in a laboratory. The basic knowledge of the current and wave regime, still sadly lacking, will in all probability have to come from model studies. Model wave basins of the type available for this study would be better employed for investigations utilizing normal rather than angled wave attack. Littoral studies would require more sophisticated models allowing for variable wave attack angles and eliminating sidewall reflection. Equilibrium profile studies require, in the author's opinion, considerably more attention, incorporating investigations into the mode of material transport in the near-shore zone. The effect of the tidal cycle on equilibrium profiles could be effectively studied in a model basin. Three-dimensional beach forms, especially the occurence of cusps, cusp growth and propagation, form a field of interest which has not yet been satisfactorily explained in the literature. The use of movable bed models, whether two- or three-dimensional, cannot fail to develop a better basic understanding of the interaction between water and sediment movement in the coastal environment.

With model studies in mind, but to a far greater extent necessary for field investigations, more sophisticated methods of collecting raw wave data would have to be utilized. Equipment is available and should be used to record on a continuous basis the total wave climate or spectrum to which time series analysis may be applied to determine the true significant wave heights and wave periods. A reliable method recording the impinging wave angle utilizing static floating buoys would also be required.

Finally, bearing in mind and assuming that a field investigation is aimed at obtaining information of a general nature, it would be wise to choose a location where extremes of wave climate are limited, and artificial or non-typical beach features are non-existent. An investigation confined to the swash zone of a beach is very limiting and every effort should be made by whatever means available to extend the study into the surf zone and possibly even beyond the breaker zone.

Many problems associated with coastal engineering and coastal hydraulics have yet to be fully met. Local and overall littoral drift rates, on-shore/

off-shore transport phenomena and beachfill stability are not yet fully understood. Continuous and continuing studies utilizing both model and field programmes, as well as computer techniques, will hopefully in the future allow the field of study true scientific status, to the exclusion of the present many and varied empirical relationships.

BIBLIOGRAPHY

BAGNOLD, R.A. 1963 Sedimentation. The Sea, Vol 3, ed M.N. Hill. Interscience, New York.

BAJORUNAS, L. 1960 Littoral Transport in the Great Lakes. Proc. Coastal Engineering (7), ASCE, New York.

1970 Littoral Transport and Energy Relationships. Proc. Coastal Engineering (12), ASCE, New York

BASCOM, W.N. 1953 Characteristics of Natural Beaches. Proc. Coastal Engineering (4), ASCE, New York.

BREMNER, M. 1970 Littoral Drift Studies. M.Sc. Thesis, University of British Columbia.

BRUUN, P. 1962 Longshore Currents in One and Multibar Profiles. Proc. Coastal Engineering (8), ASCE, New York.

1964 Quantitative Research on Littoral Drift in Field and Laboratory. Proc. Coastal Engineering (9), ASCE, New York.

CALDWELL, J.M. 1956 Wave Action and Sand Movement Near Anaheim Bay, California. Tech Memo No 68, Beach Erosion Board. U.S. Dept of the Army.

DAS, M.M. 1972 Longshore Transport and Suspended Sediment Review. Proc. Coastal Engineering (13), ASCE, New York.

DE SWART, E. 1973 Littoral Sediment Transport (Profile Analysis). B.Sc. (Eng) Thesis No 32, University of Cape Town.

EATON, R.O. 1950 Littoral Processes on Sandy Coasts. Proc. Coastal Engineering (1), ASCE, New York.

GALVIN, C.J. & EAGLESON, P.S. 1965 Experimental Study of Longshore Currents on a Plane Beach. U.S. Army Coastal Engineering Center, Tech Memo 10.

GALVIN, C.J. 1967 Longshore Current Velocity Review. Rev. Geophysics, No 5.

GOW, D.L.F. 1973 Littoral Sediment Transport (Beach Sand Analysis) B.Sc. (Eng) Thesis no 32, University of Cape Town.

HARRIS, T.F.W. 1968 Field and Model Studies of the Nearshore Circulation. Ph.D. Thesis, Dept of Physics, University of Natal.

HARRISON, W. et al 1970 Field Study of Tidal Inlet, Bimini, Bahamas. Proc. Coastal Engineering (12), ASCE, New York.

HOM-MA & SONU 1962 Rhythmic Pattern of Longshore Bars. Proc. Coastal Engineering (8), ASCE, New York.

INMAN, D.L. & BAGNOLD, R.A. 1963 Littoral Processes. The Sea, Vol 3. Interscience, New York.

INMAN, KOMAR & BOWEN 1968 Longshore Transport of Sand. Proc. Coastal Engineering (11), ASCE, New York.

IPPEN, A.T. 1966 Estimation of Littoral Sediment Transport. Estuary and Coast-line Hydrodynamics, Mc Graw-Hill.

IWAGADI & NODA 1962 Laboratory Study of Scale Effect in 2 D Beach Processes. Proc. Coastal Engineering (8), ASCE, New York.

KEMP, P.H. 1960 The Relationship between Wave Action and Beach Profile Characteristics. Proc. Coastal Engineering (7), ASCE, New York.

KING, C.A.M. 1959 Beaches and Coasts. First ed., Edward Arnold.
1970 Beaches and Coasts. Second ed., Edward Arnold.

KOMAR & GAUGHAN 1972 Airy Theory and Breaker Height Prediction. Proc. Coastal Engineering (13), ASCE, New York.

KOMAR & INMAN 1970 Longshore Sand Transport on Beaches. J. Geophysics Res., 75.

LE MÉHAUTE, B. et al 1968 Shallow Water Wave Comparisons. Proc. Coastal Engineering, (11), ASCE, New York.

LONGUET-HIGGINS, M.S. 1971 Recent Progress in the Study of Longshore Currents. Waves on Beaches, Academic Press, New York.

NAGAK, I.V. 1970 Equilibrium Profiles of Model Beaches. Proc. Coastal Engineering (12), ASCE, New York.

NICOL, B.J. 1973 Littoral Sediment Transport (Fluorescent Tracer Study) B.Sc. (Eng) Thesis No 32, University of Cape Town.

PUTMAN, MUNK & TRAYLOR — 1949 Prediction of Longshore Currents. Transactions AGU, Vol 30, No 3.

SAVAGE, R.P. — 1962 Laboratory Determination of Littoral Transport Rates. Journal, Waterways and Harbours Div, ASCE.

SHEPARD & INMAN — 1950 Nearshore Circulation. Proc. Coastal Engineering, (1), ASCE, New York.

SIEFERT, W. — 1970 Wave Investigations in Shallow Water. Proc. Coastal Engineering (12), ASCE, New York.

SILVESTER, R. — 1969 Beach Profiles and Littoral Drift Assessment. La Houlle Blance, Vol 6.

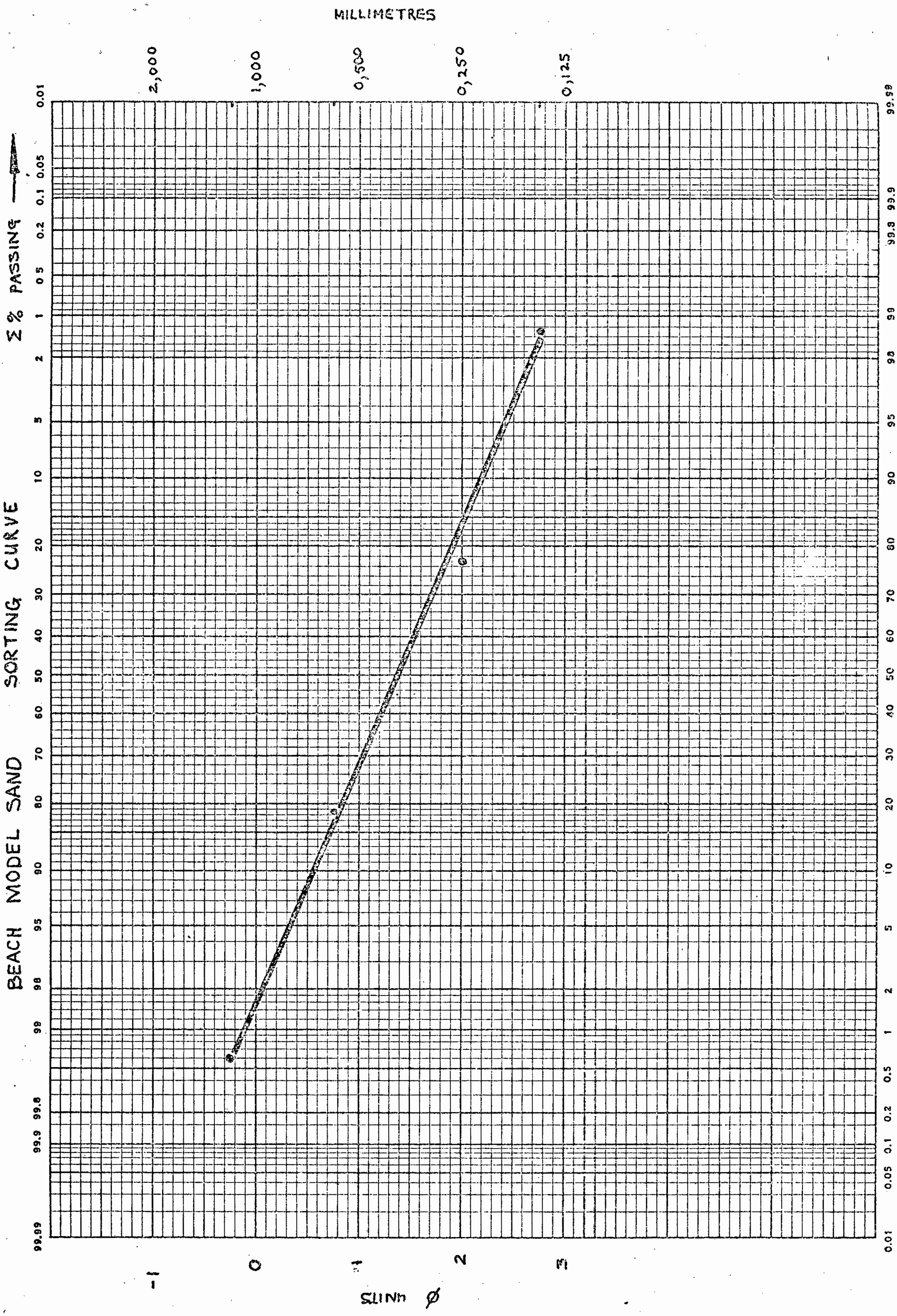
SONU & RUSSEL — 1965 Topographic Changes in the Surf Zone Profile. Proc. Coastal Engineering (10), ASCE, New York.

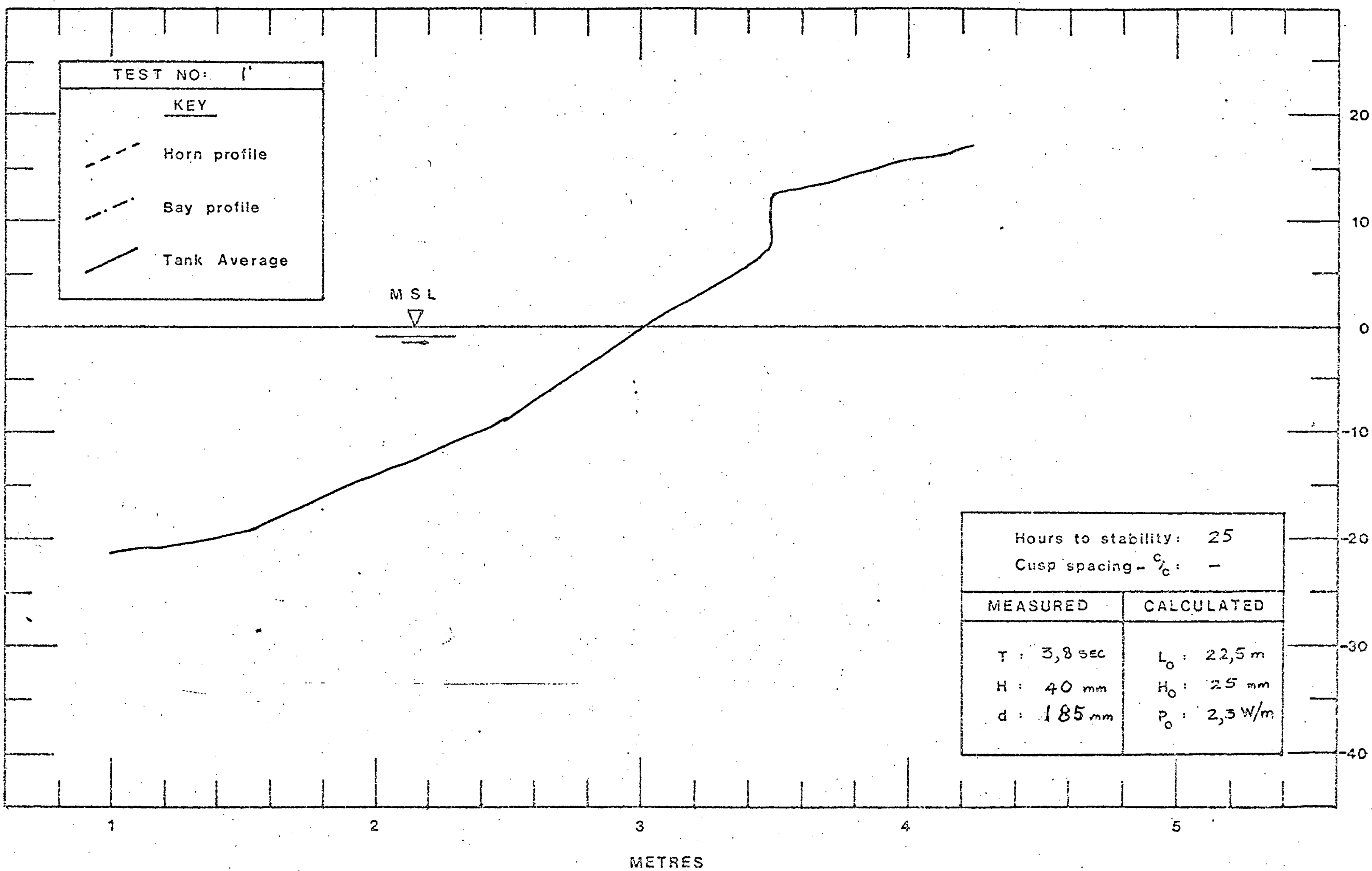
SONU, C.J. — 1968 Collective Movement of Sediment in the Littoral Environment. Proc. Coastal Engineering (11), ASCE.

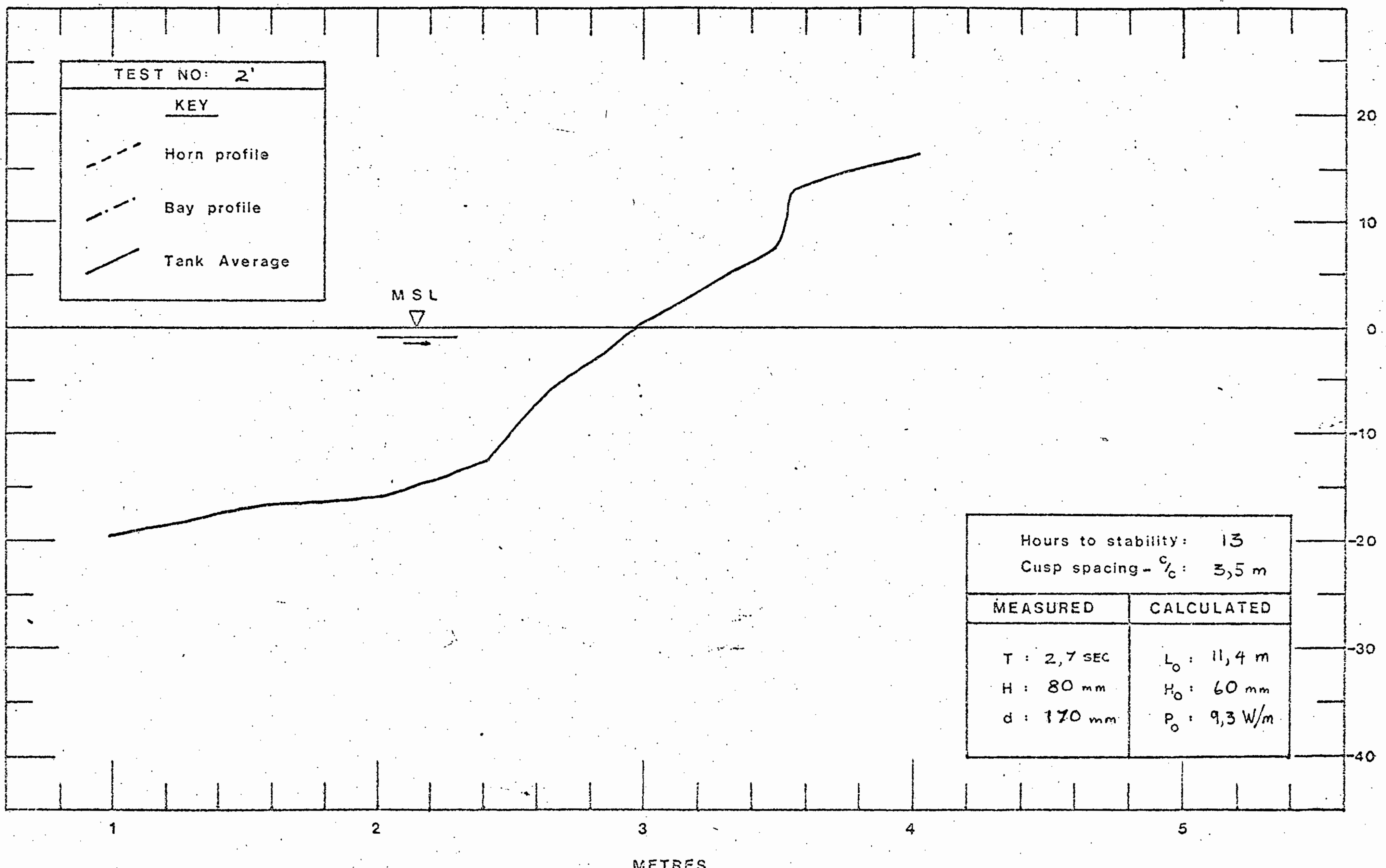
SONU & YOUNG — 1970 Stochastic Analysis of Beach Profile Data. Proc. Coastal Engineering, (12), ASCE, New York.

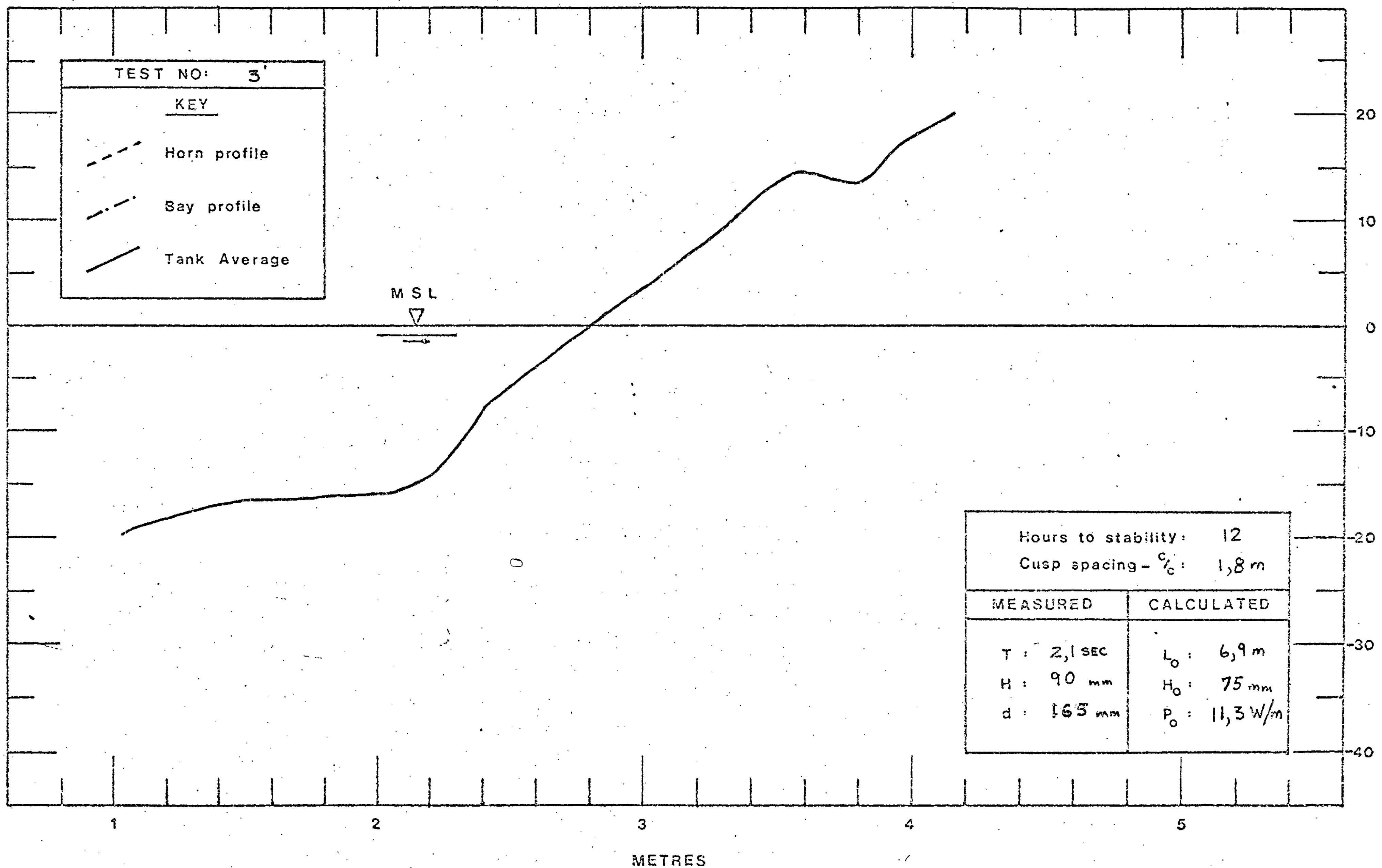
WIEGEL, R.L. — 1964 Oceanographical Engineering. Prentice Hall, New York.

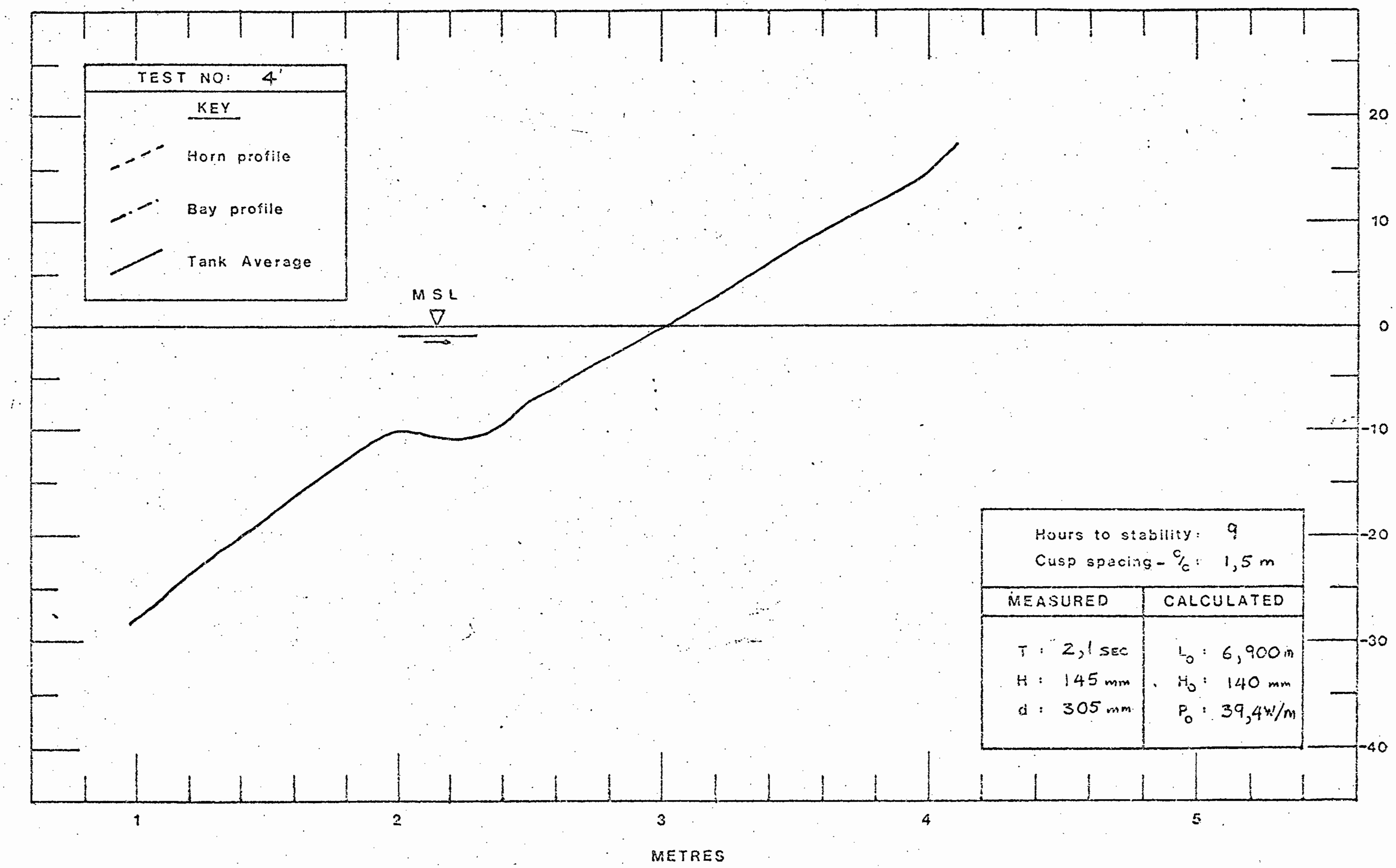
APPENDIX A

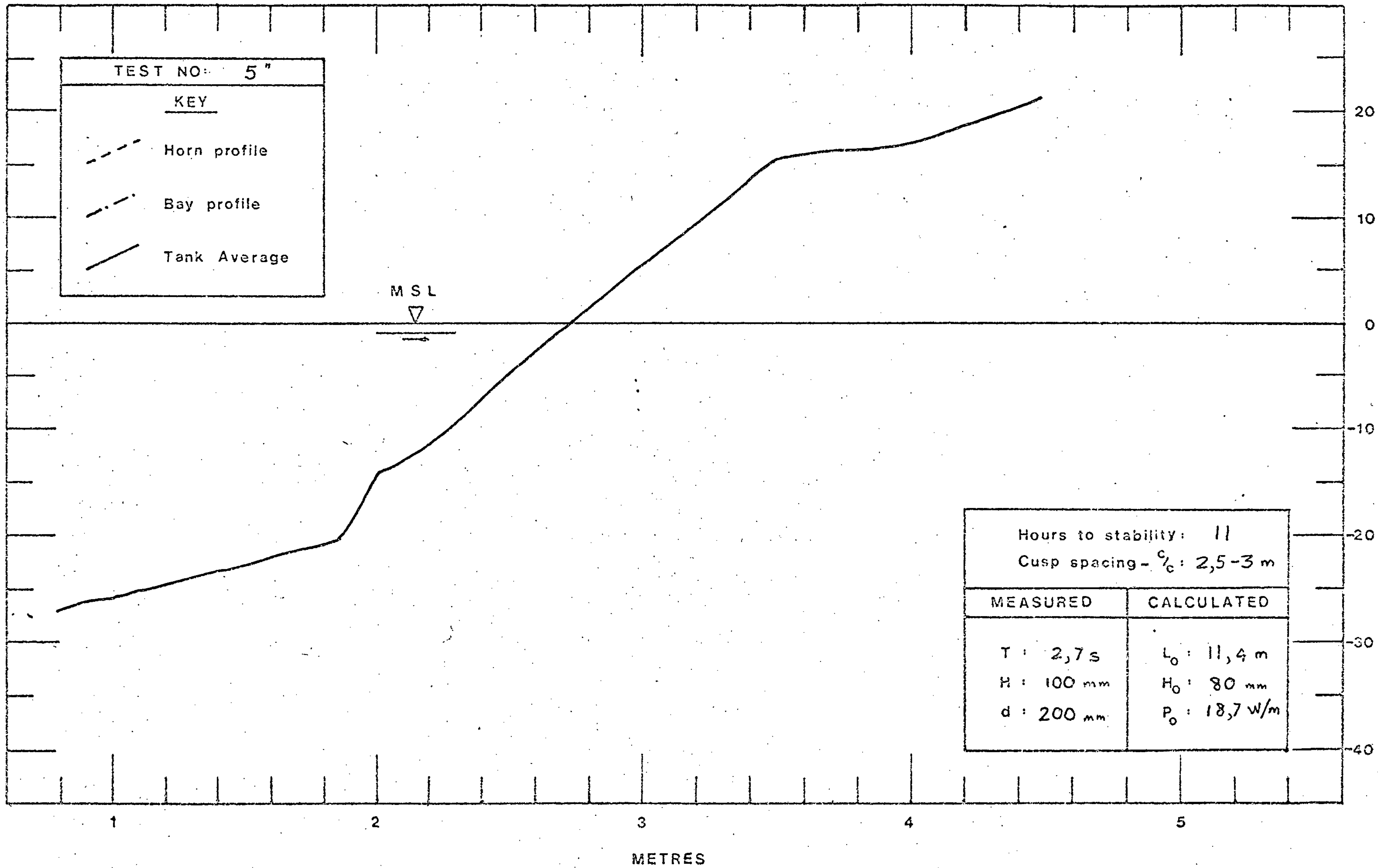

MILLIMETRES
2,000
1,000
0,500
0,250
0,125
Σ% PASSING
BEACH MODEL SAND SORTING CURVE
-1
0
1
2
3
ø UNITS


TEST NO: 1'
KEY
Horn profile
Bay profile
Tank Average
MSL
Hours to stability: 25
Cusp spacing - c/c: —
MEASURED
CALCULATED
T : 3,8 sec
H : 40 mm
d : 185 mm
Lo : 22,5 m
Ho : 25 mm
Po : 2,3 W/m
20
10
0
-10
-20
-30
-40
CENTIMETRES
1
2
3
4
5
METRES

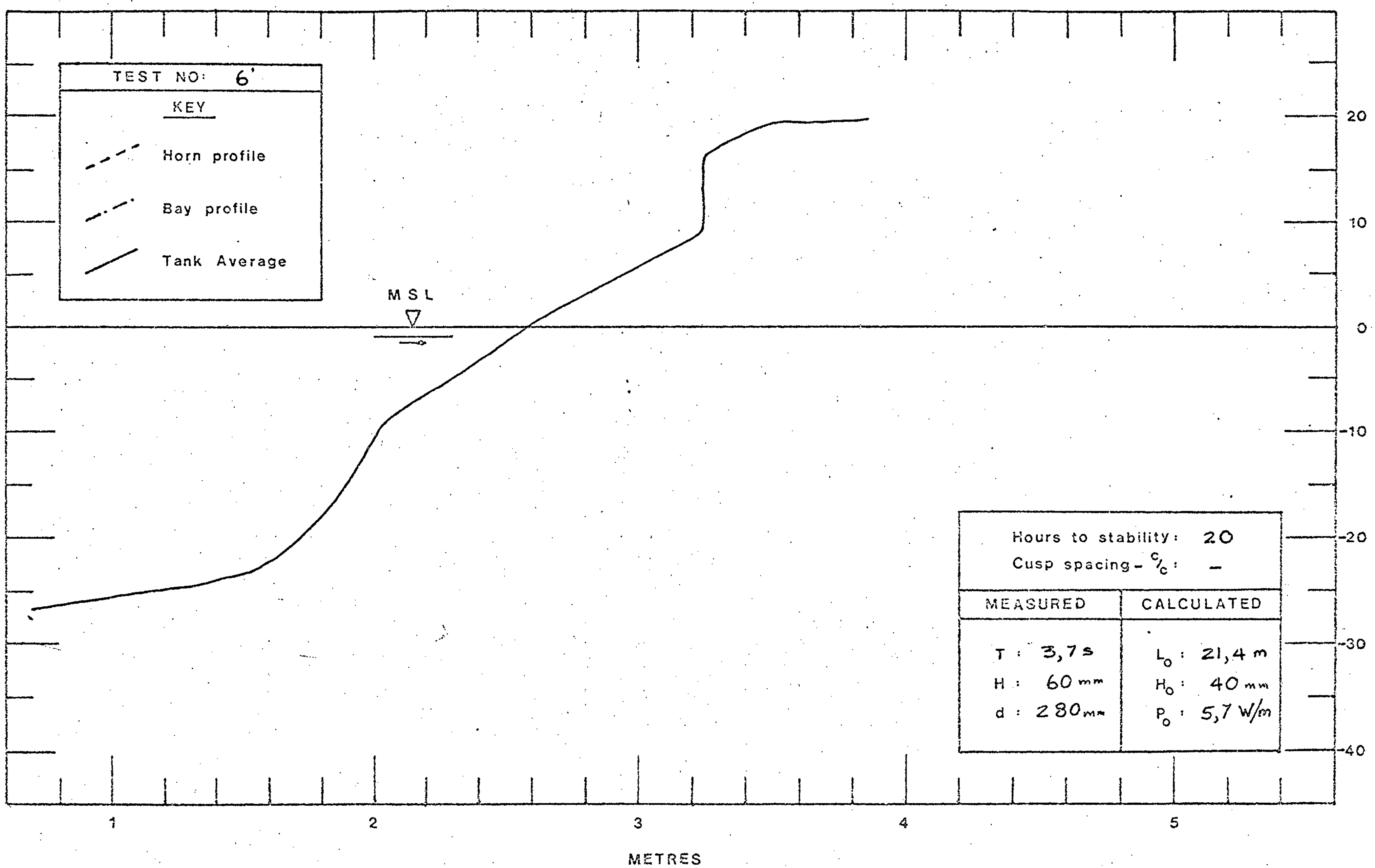

TEST NO: 2'
KEY
Horn profile
Bay profile
Tank Average
MSL
20
10
0
-10
-20
-30
-40
CENTIMETRES
1
2
3
4
5
METRES
Hours to stability: 13
Cusp spacing - c/c: 3,5 m
MEASURED
CALCULATED
T : 2,7 SEC
H : 80 mm
d : 170 mm
L_o : 11,4 m
H_o : 60 mm
P_o : 9,3 W/m


TEST NO: 3'
KEY
Horn profile
Bay profile
Tank Average
MSL
Hours to stability: 12
Cusp spacing - c/c : 1,8 m
MEASURED
CALCULATED
T : 2,1 SEC
H : 90 mm
d : 165 mm
Lo : 6,9 m
Ho : 75 mm
Po : 11,3 W/m
20
10
0
-10
-20
-30
-40
CENTIMETRES
1
2
3
4
5
METRES

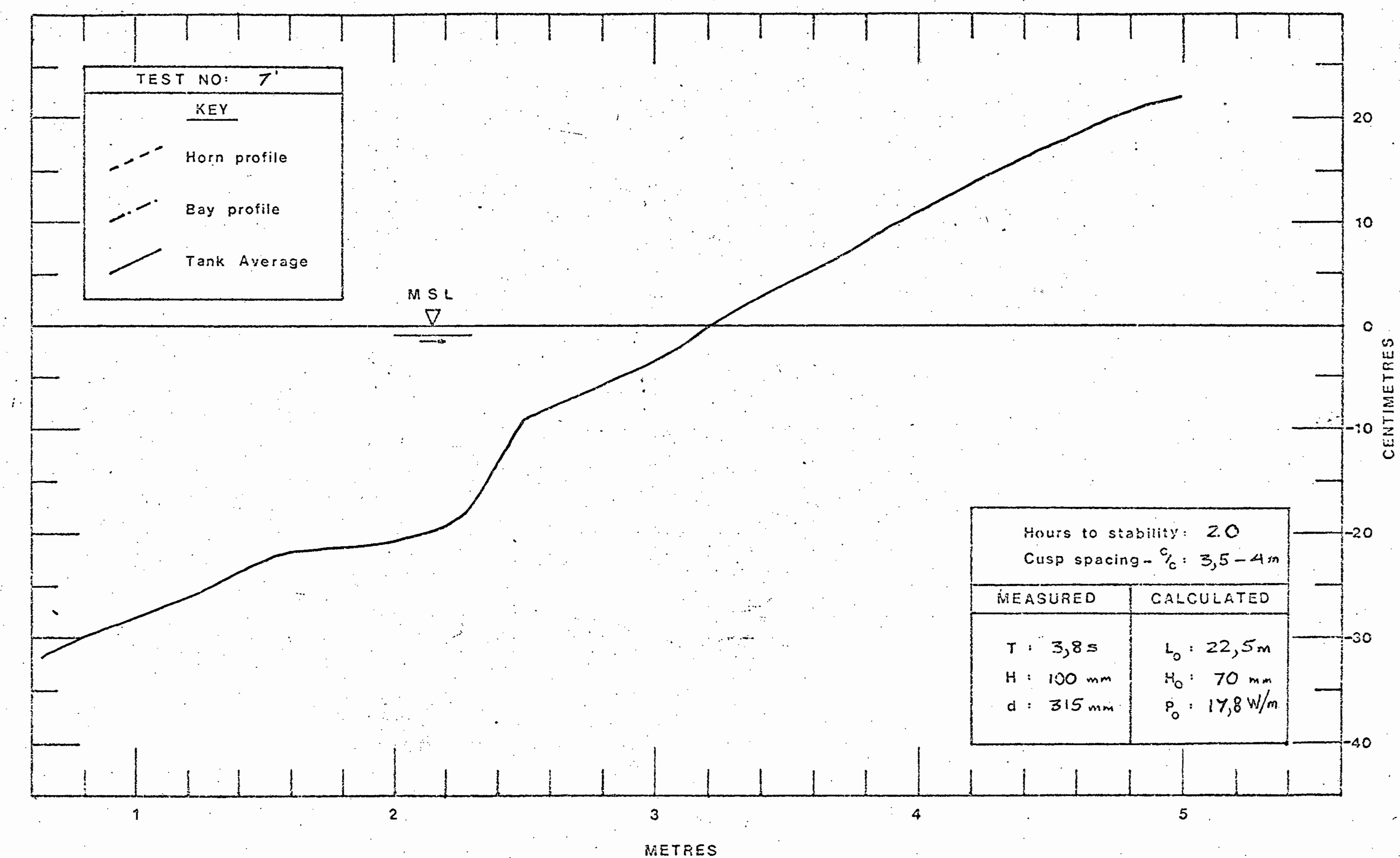

TEST NO: 4
KEY
Horn profile
Bay profile
Tank Average
MSL
20
10
0
-10
-20
-30
-40
CENTIMETRES
1
2
3
4
5
METRES
Hours to stability: 9
Cusp spacing - c/c: 1,5 m
MEASURED
CALCULATED
T : 2,1 sec
H : 145 mm
d : 305 mm
L0 : 6,900 m
H0 : 140 mm
P0 : 39,4 W/m

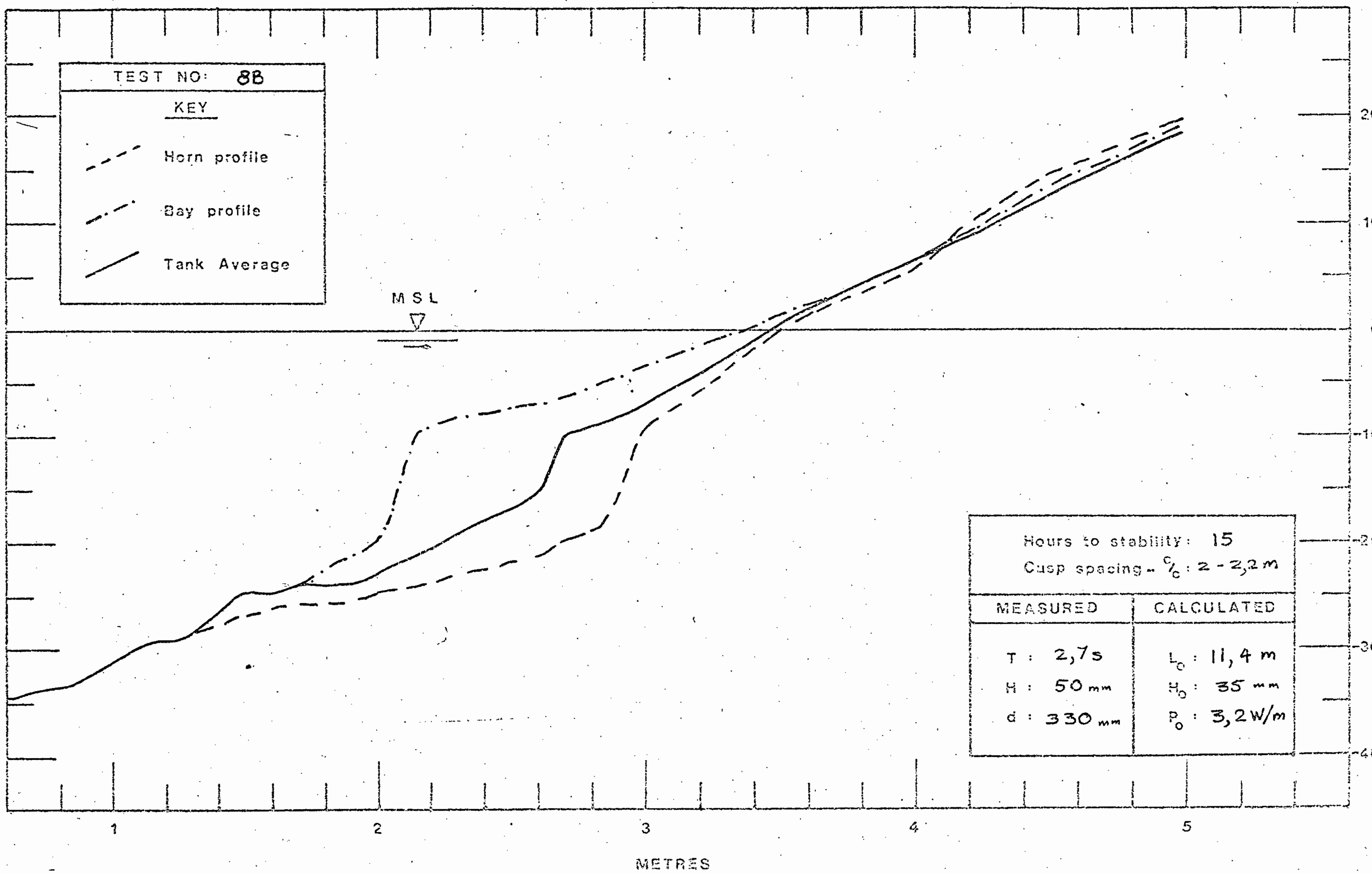


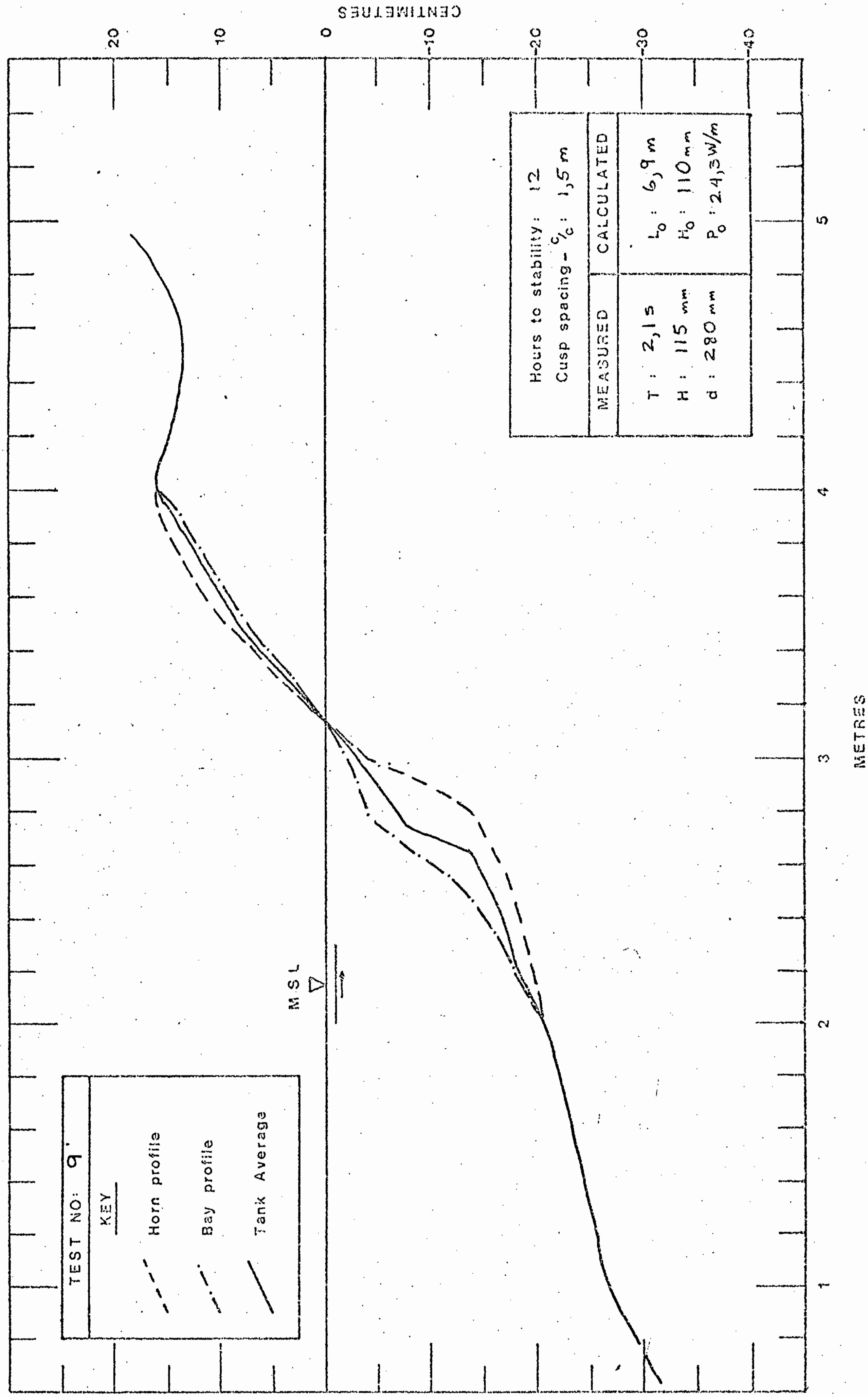
TEST NO: 5"
KEY
Horn profile
Bay profile
Tank Average
MSL
Hours to stability: 11
Cusp spacing - c/c : 2,5-3 m
MEASURED
CALCULATED
T : 2,7 s
H : 100 mm
d : 200 mm
Lo : 11,4 m
Ho : 80 mm
Po : 18,7 W/m
20
10
0
-10
-20
-30
-40
CENTIMETRES
1
2
3
4
5
METRES

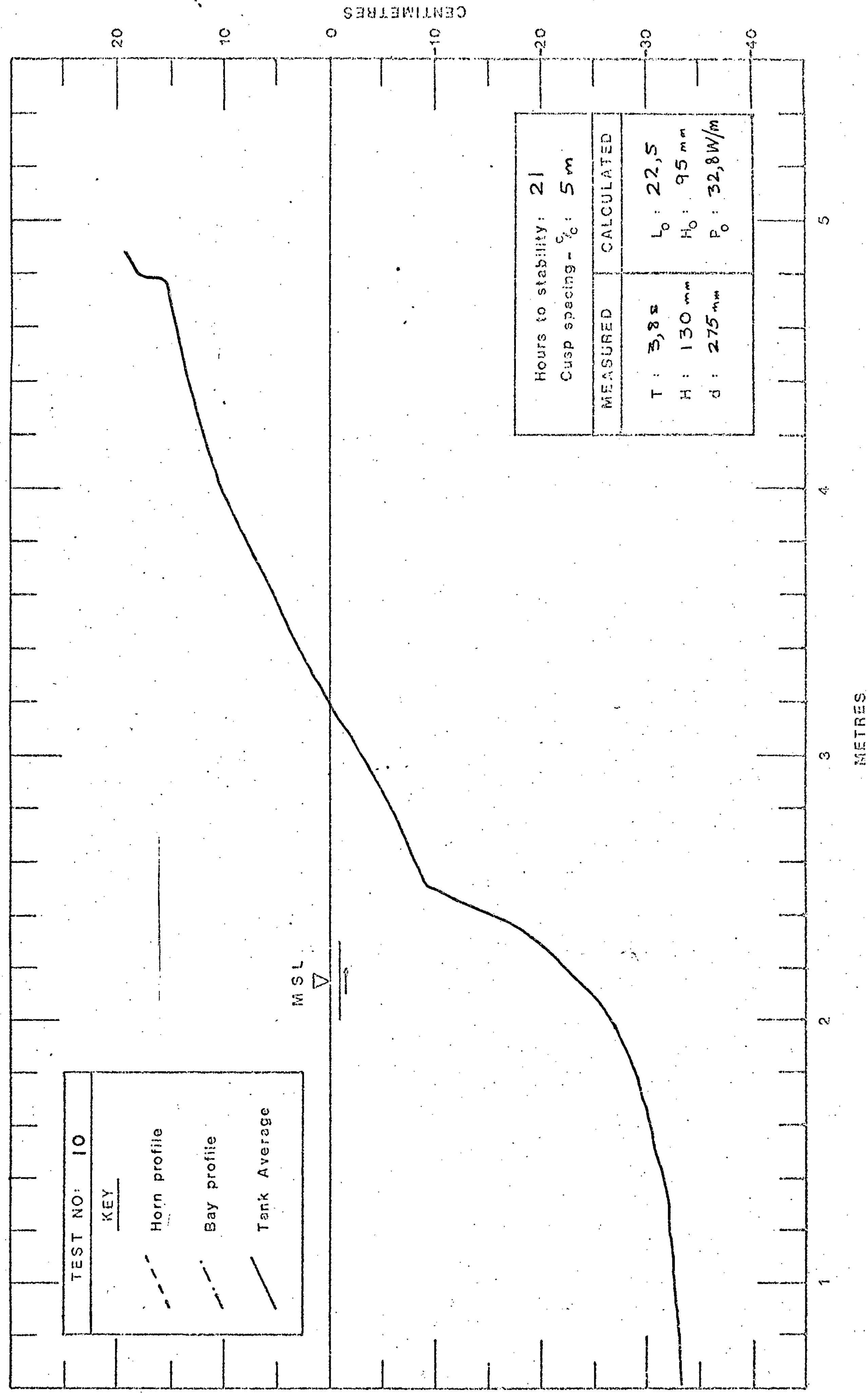


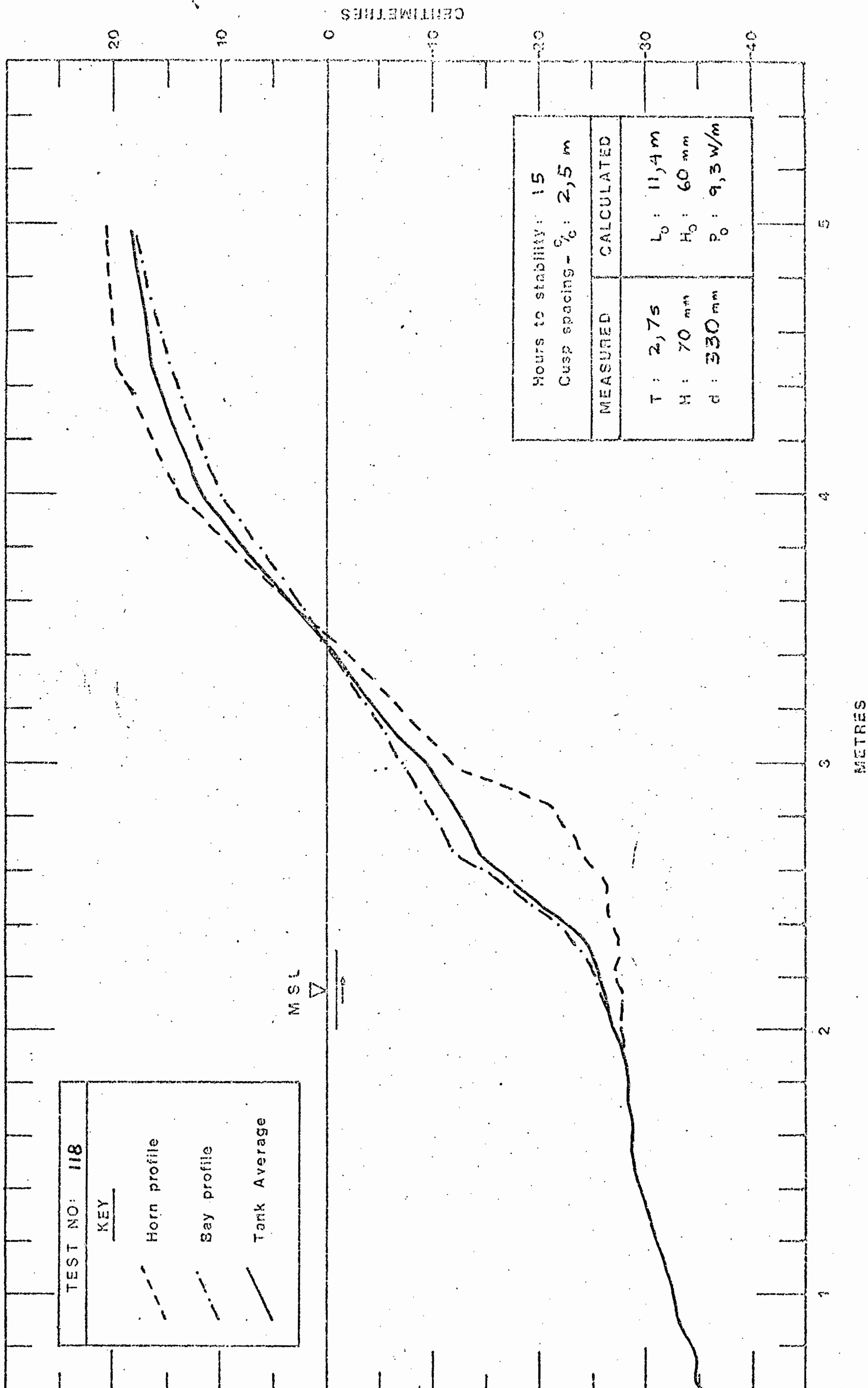
TEST NO: 6'
KEY
Horn profile
Bay profile
Tank Average
M S L
20
10
0
-10
-20
-30
-40
CENTIMETRES
1
2
3
4
5
METRES
Hours to stability: 20
Cusp spacing - c/c: –
MEASURED
CALCULATED
T: 3,7 s
H: 60 mm
d: 280 mm
L_o: 21,4 m
H_o: 40 mm
P_o: 5,7 W/m

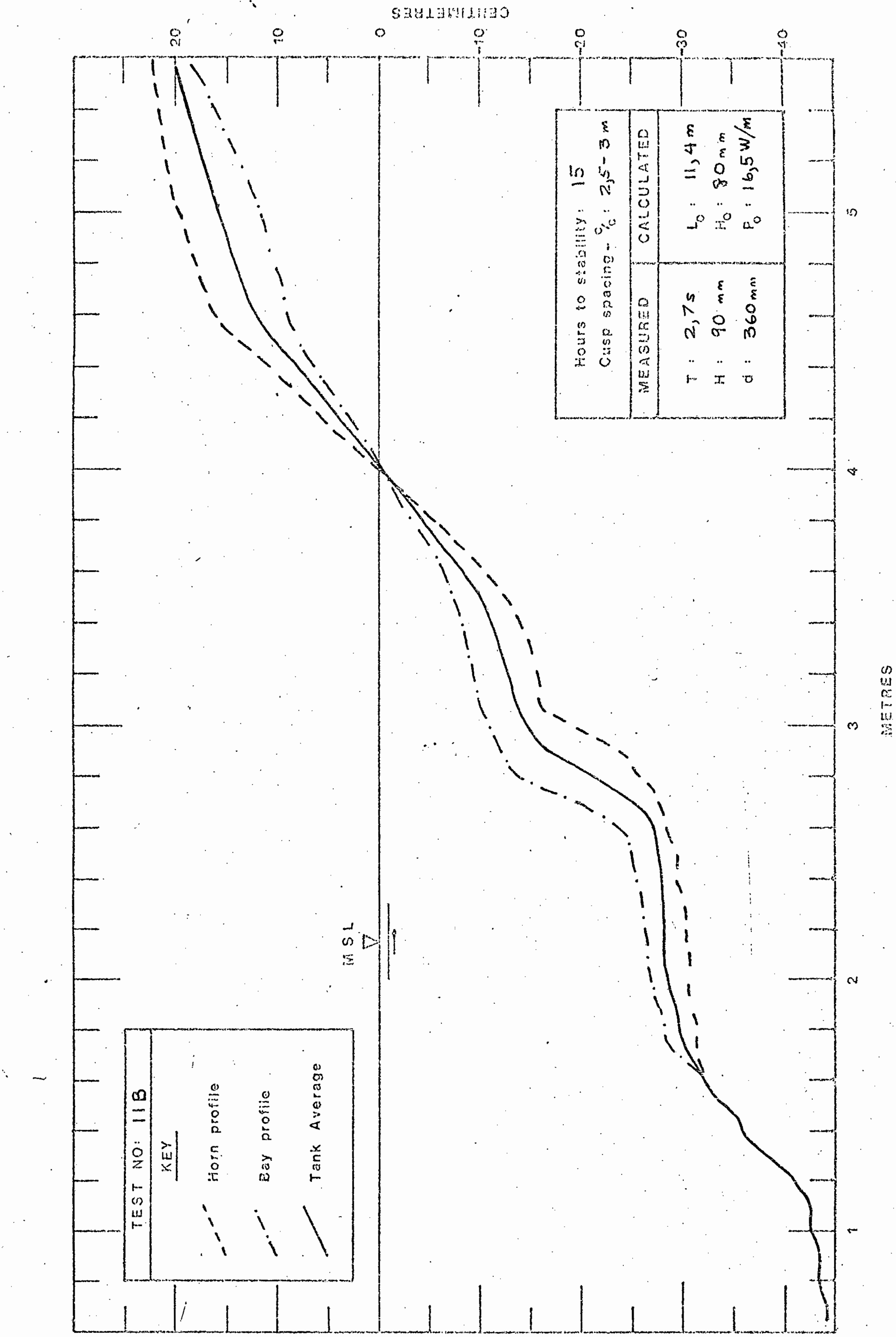

TEST NO: 7
KEY
Horn profile
Bay profile
Tank Average
MSL
Hours to stability: 20
Cusp spacing - c/c: 3,5 – 4m
MEASURED
CALCULATED
T: 3,8s
H: 100 mm
d: 315 mm
L₀: 22,5m
H₀: 70 mm
P₀: 17,8 W/m
20
10
0
-10
-20
-30
-40
CENTIMETRES
1
2
3
4
5
METRES

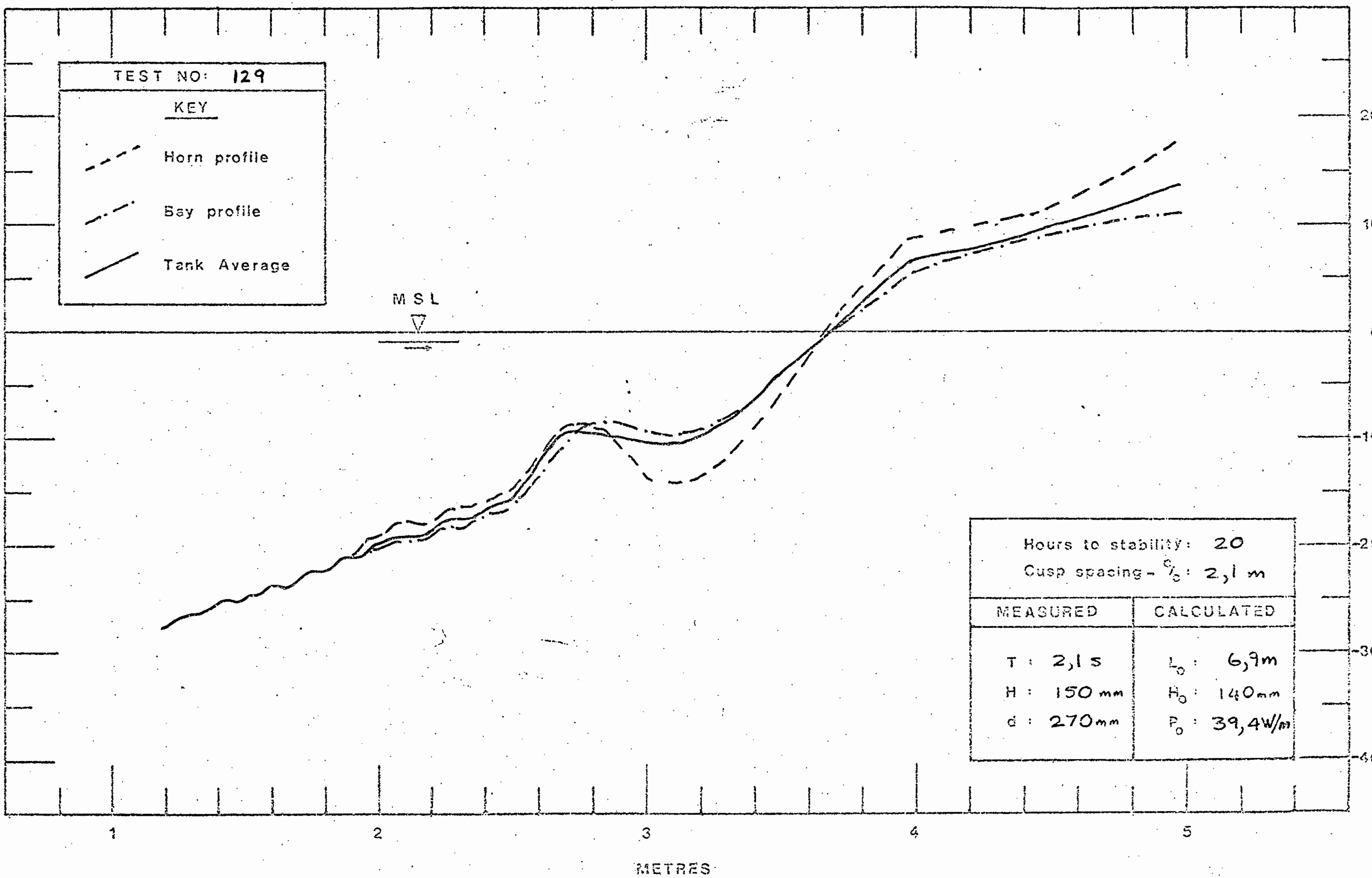

TEST NO: 8B
KEY
Horn profile
Bay profile
Tank Average
MSL
Hours to stability: 15
Cusp spacing - c/c : 2 - 2,2 m
MEASURED
CALCULATED
T : 2,7s
H : 50 mm
d : 330 mm
L_o : 11,4 m
H_o : 35 mm
P_o : 3,2 W/m
20
10
0
-10
-20
-30
-40
CENTIMETRES
1
2
3
4
5
METRES

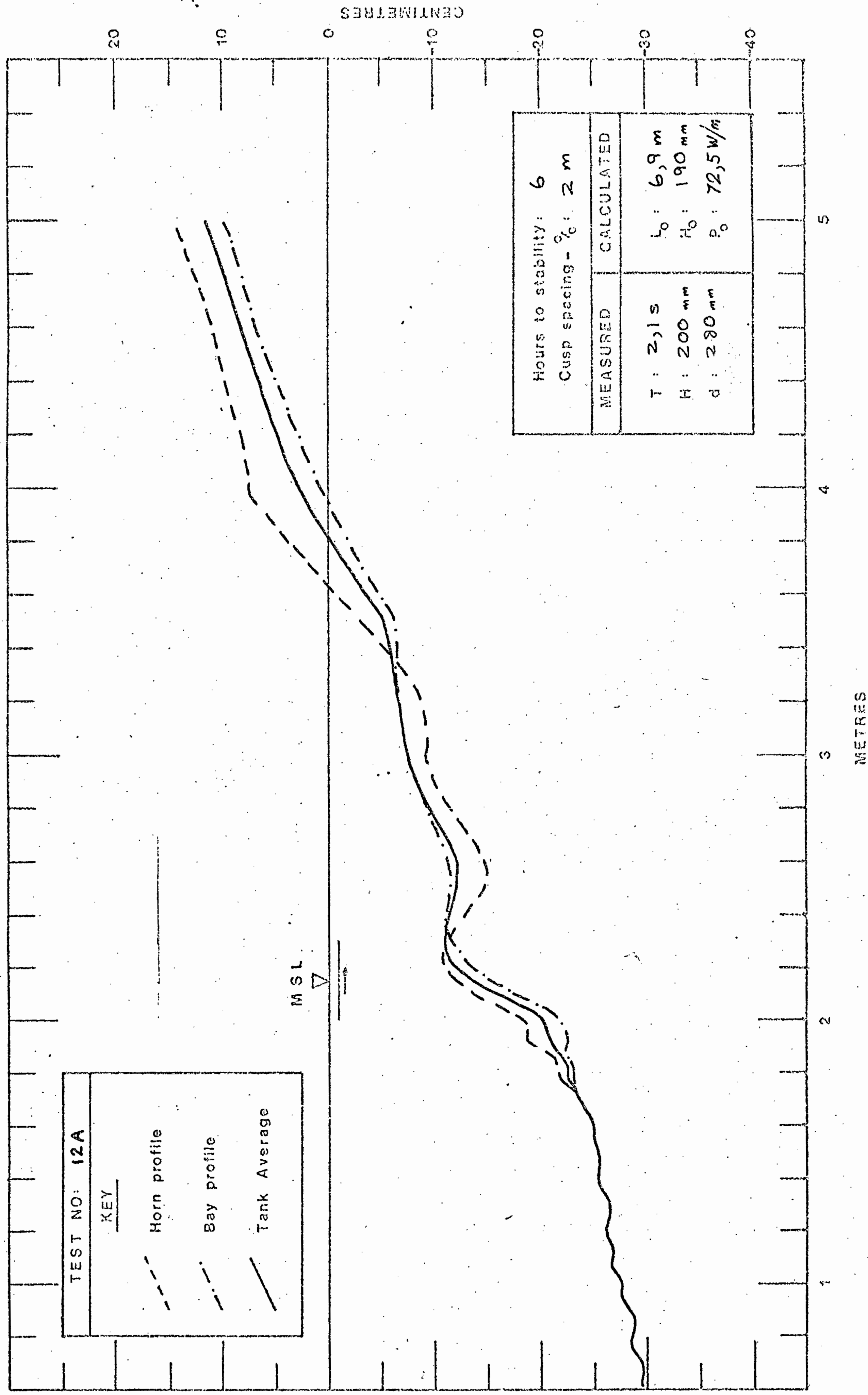

CENTIMETRES
20
10
0
-10
-20
-30
-40
METRES
1
2
3
4
5
M S L
Hours to stability: 12
Cusp spacing - c/c: 1,5 m
MEASURED
CALCULATED
T: 2,1 s
H: 115 mm
d: 280 mm
L0: 6,9 m
H0: 110 mm
P0: 24,3 W/m
TEST NO: 9
KEY
Horn profile
Bay profile
Tank Average

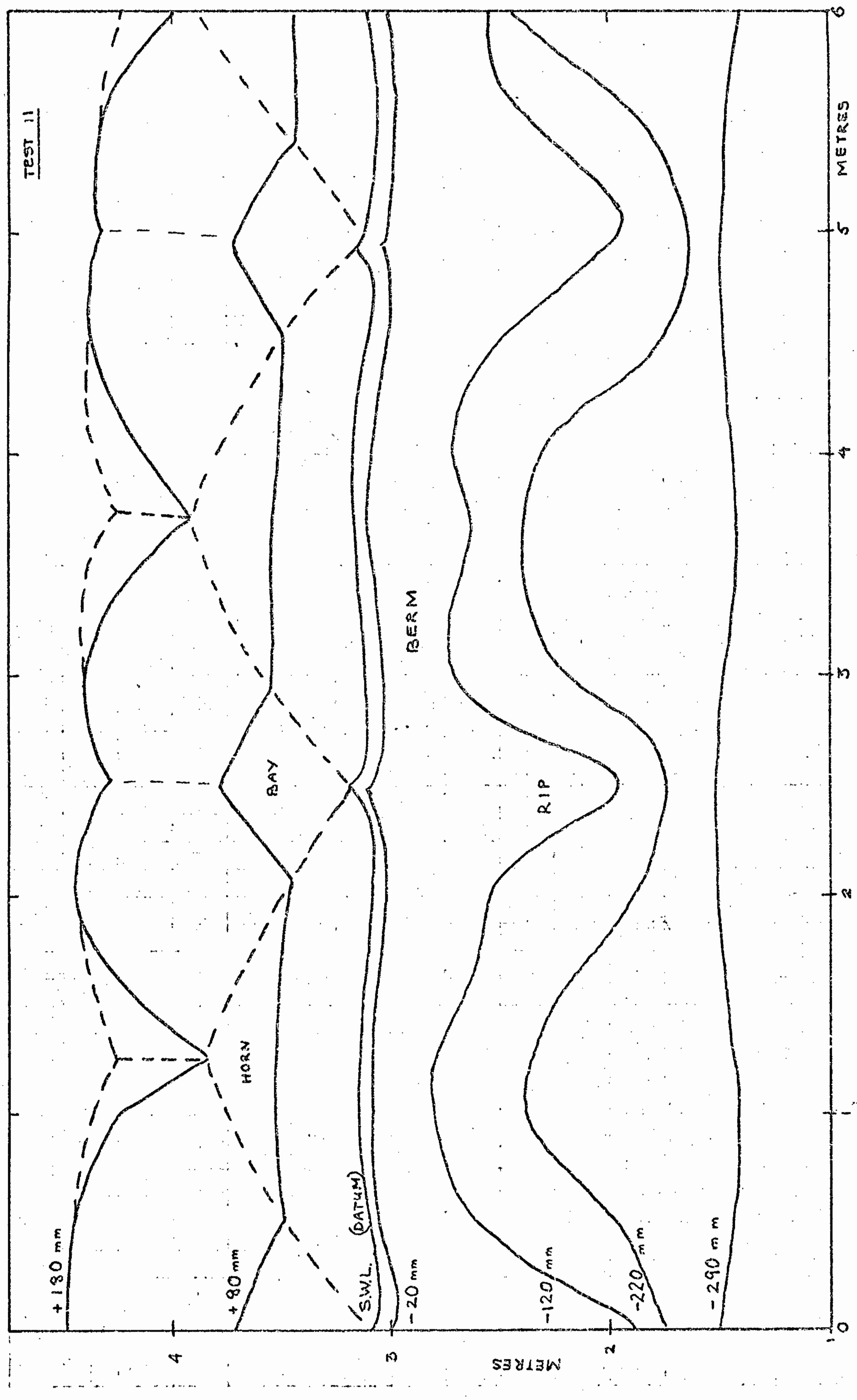

CENTIMETRES
20
10
0
-10
-20
-30
-40
Hours to stability: 21
Cusp spacing - c/c: 5 m
MEASURED
T : 3,8 s
H : 130 mm
d : 275 mm
CALCULATED
L_o : 22,5
H_o : 95 mm
P_o : 32,8 W/m
MSL
TEST NO: 10
KEY
Horn profile
Bay profile
Tank Average
1
2
3
4
5
METRES


CENTIMETRES
20
10
0
-10
-20
-30
-40
METRES
1
2
3
4
5
M S L
TEST NO: 118
KEY
Horn profile
Bay profile
Tank Average
Hours to stability: 15
Cusp spacing - S_c : 2,5 m
MEASURED
T : 2,7s
H : 70 mm
d : 330 mm
CALCULATED
L_o : 11,4 m
H_o : 60 mm
P_o : 9,3 W/m

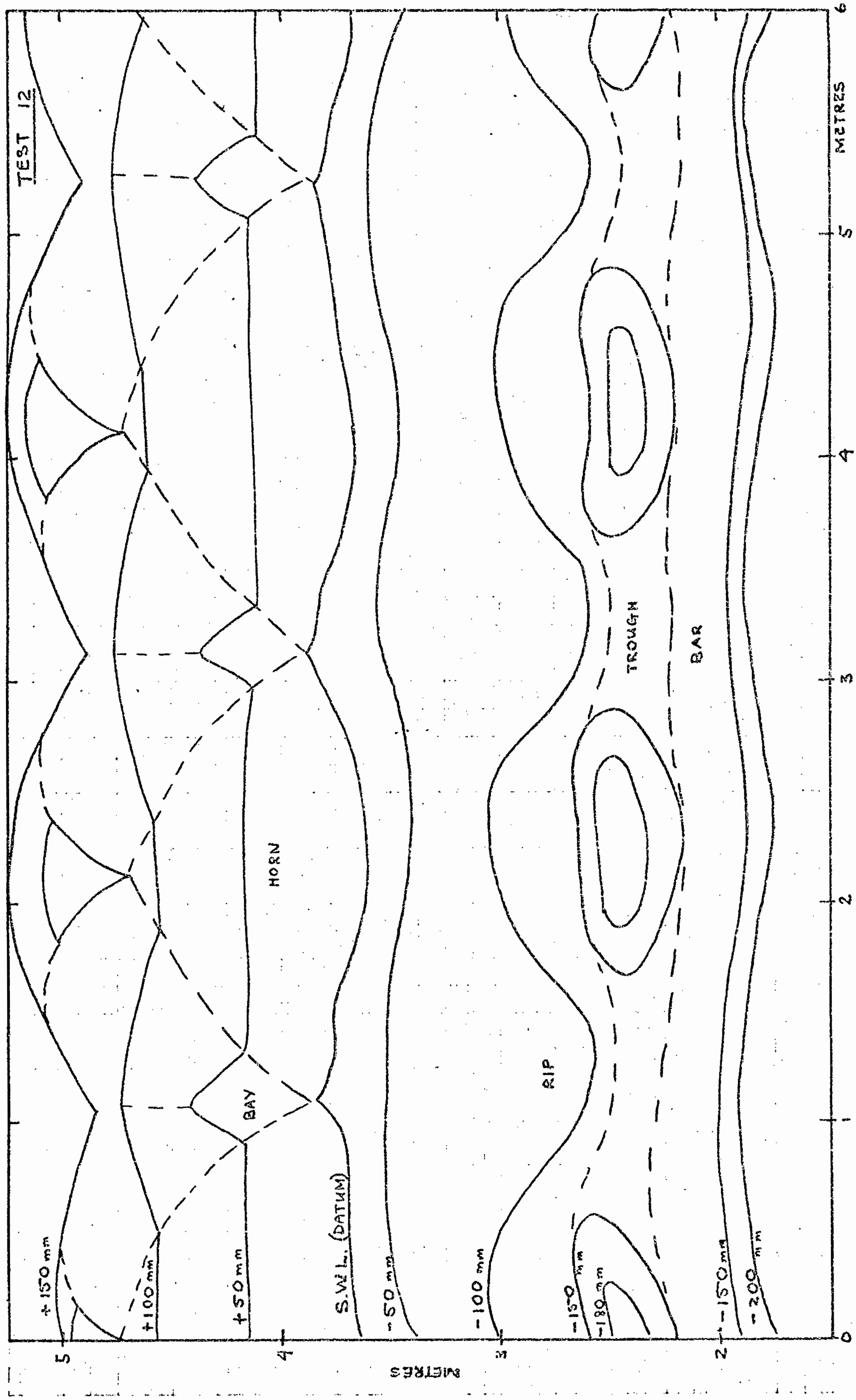

CENTIMETRES
20
10
0
-10
-20
-30
-40
METRES
1
2
3
4
5
MSL
TEST NO: 11B
KEY
Horn profile
Bay profile
Tank Average
Hours to stability: 15
Cusp spacing - %: 2,5 - 3 m
MEASURED
CALCULATED
T: 2,7s
H: 90 mm
d: 360mm
Lo: 11,4 m
Ho: 80mm
Po: 16,5 W/m


TEST NO: 129
KEY
Horn profile
Bay profile
Tank Average
MSL
Hours to stability: 20
Cusp spacing - $c/_o$: 2,1 m
MEASURED
CALCULATED
T : 2,1 s
H : 150 mm
d : 270 mm
L_o : 6,9 m
H_o : 140 mm
P_o : 39,4 W/m
20
10
0
-10
-20
-30
-40
CENTIMETRES
1
2
3
4
5
METRES


TEST NO: 12A
KEY
Horn profile
Bay profile
Tank Average
MSL
Hours to stability: 6
Cusp spacing - %: 2 m
MEASURED
CALCULATED
T : 2,1 s
H : 200 mm
d : 280 mm
Lo : 6,9 m
Ho : 190 mm
Po : 72,5 W/m
CENTIMETRES
20
10
0
-10
-20
-30
-40
METRES
1
2
3
4
5



PLAN VIEW OF BERM PROFILE



PLAN VIEW OF BAR PROFILE

PLATE 1: View of wave generating mechanism

PLATE 2: View of drained model tank showing regular discontinuities caused by rip currents. The profile shown here is the result of a test made with a short wave period.

PLATE 3: Backwash condition in model tank showing presence of rip currents

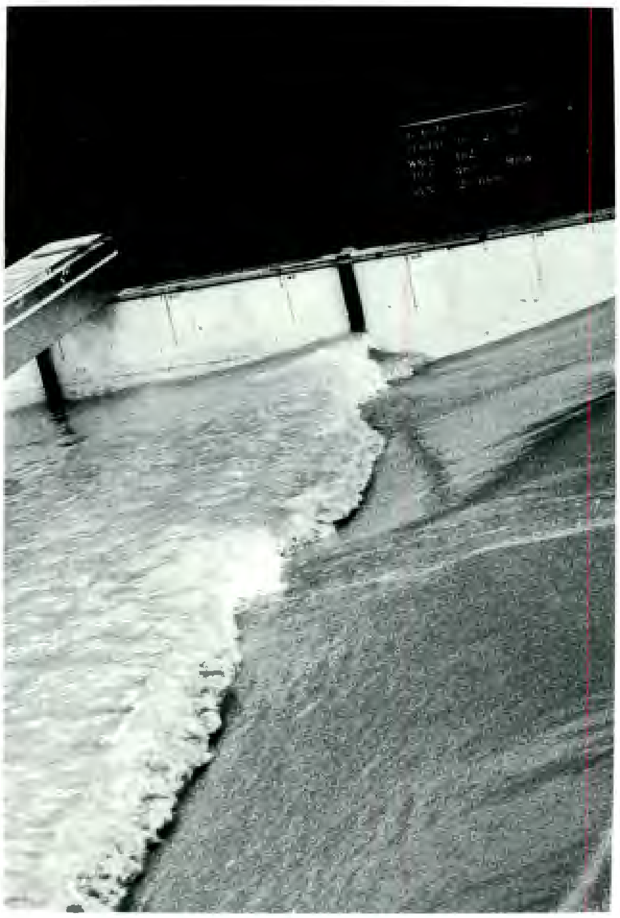


PLATE 4: Wave in model basin just after breaking. Note the deflection of wave crest due to rip currents.

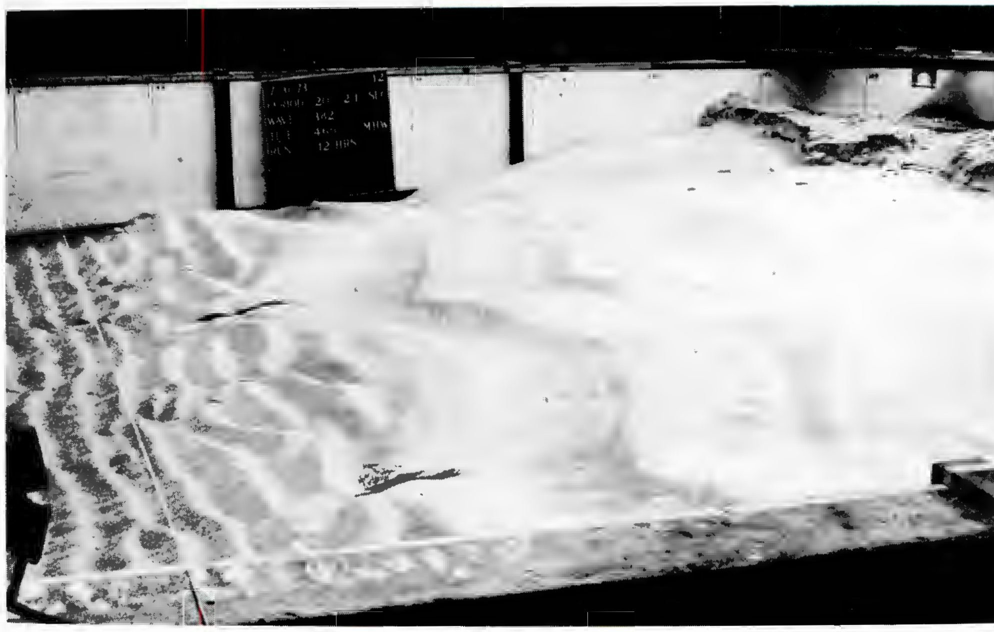


PLATE 5:

View of model beach tank drained to show a bar and trough profile associated with steep wave condition. The profile has been distorted by the presence of rip currents.

PLATE 6: View of model tank showing a plunging wave just after collapse.

PLATE 7: View of model tank showing a plunging wave acting on a beach orientated at an angle to the wave generating mechanism.

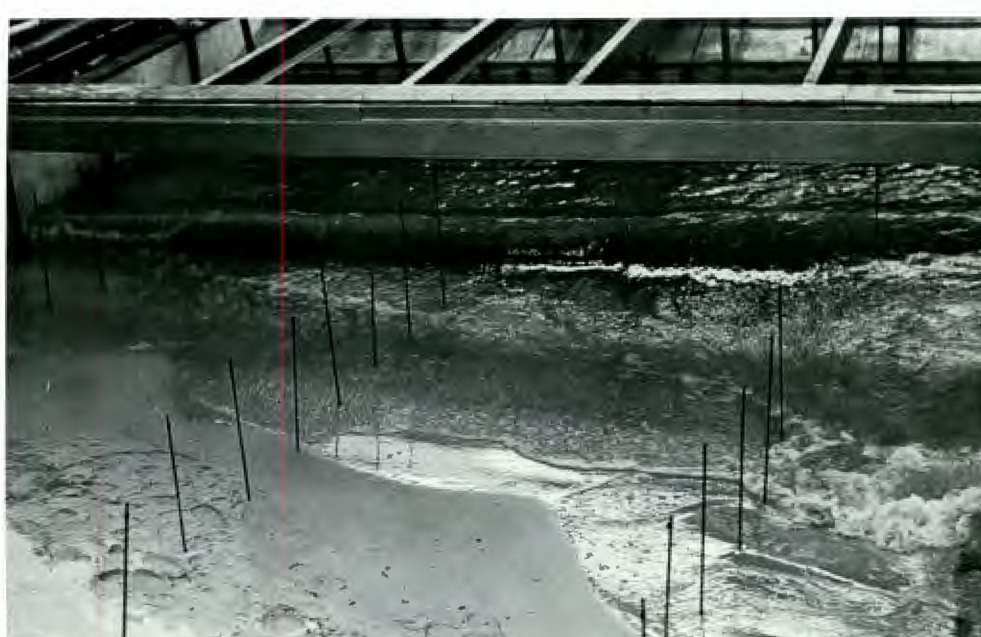


PLATE 8: View of model tank with the beach orientated at an angle to the wave generating mechanism.

PLATE 9: General view of test beach at Oranjemund

PLATE 10: View of test beach showing the steel pipe grid system.

PLATE 11: View showing plunging wave conditions and heavy surf right onto the beach

PLATE 12: Scanning for fluorescent tracer. Portable generator in the foreground with light-proof ultra-violet lamp and camera box in the background. The box also allows for visual scanning.

APPENDIX E

Normal Wave Attack Programme

Test No.	Tide	Wave Setting	T (sec)	H (mm)	d (mm)	Break Type	Beach Type	Max. Run up (m)
1	Low	Low	3,8	75	200	Surging	Step	3,2
2	Low	Low	2,7	60	200	Surging	Step	3,1
3	Low	Low	2,1	70	200	Surging	Step	3,0
4	Low	High	2,1	130	150	Plunging	Bar	3,1
5	Low	High	2,7	100	220	Surging	Step	3,2
6	Low	High	3,8	60	200	Spilling	Step	2,1
7	High	Low	3,8	90	320	Surging	Step	4,4
8	High	Low	2,7	50	320	Spilling	Step	4,0
9	High	Low	2,1	75	320	Surging	Step	4,1
10	High	High	3,8	130	275	Surging	Step	4,3
11	High	High	2,7	90	320	Surging	Step	4,5
12	High	High	2,1	190	270	Plunging	Bar	5,0
11'	High	High	2,7	90	305	Surging	Step	4,4
11''	High	High	2,7	90	300	Surging	Step	4,4
7'	High	Low	3,8	100	315	Surging	Step	4,0
10'	High	High	3,8	130	270	Surging	Step	4,5
9'	High	Low	2,1	115	280	Surging	Step	4,0
8'	High	Low	2,7	40	310	Spilling	Step	4,0
2'	Low	Low	2,7	80	170	Surging	Step	3,0
3'	Low	Low	2,1	90	165	Surging	Step	2,7
1'	Low	Low	3,8	40	185	Surging	Step	2,5
4'	Low	High	2,1	145	305	Plunging	Bar	3,0
5'	Low	High	2,7	100	240	Surging	Step	3,0
5''	Low	High	2,7	100	200	Surging	Step	2,6
6'	Low	High	3,7	60	280	Spilling	Step	2,1
129	High	Med.	2,1	150	270	Plunging	Bar	4,7
12A	High	High	2,1	200	280	Plunging	Bar	5,0
9A	High	Low	2,1	115	280	Surging	Step	4,5
12A	High	High	2,1	218	300	Plunging	Bar	5,0
8B	High	Low	2,7	50	330	Spilling	Step	4,5
118	High	Med.	2,7	70	330	Surging	Step	4,8
11B	High	High	2,7	90	360	Surging	Step	4,4
8B	High	Low	2,7	50	330	Spilling	Step	4,3

Angled Attack

Test 1

Period = 3,8 seconds

θ_o = 30°

Ten tests of ten minutes duration each.

Wave Height & Depth (mm)	Breaker Angle	Longshore Velocity (m/s)	Littoral Drift (kg)					
			1	2	3	4	5+6	Total
115 at 270	5°	0,16	25	76	90	22	4	217
125 at 265	10°	-	30	75	85	16	3	209
130 at 250	5°	0,20	29	65	50	6	3	153
125 at 260	10°	-	43	67	38	7	2	157
120 at 260	10°	0,20	75	73	81	28	2	259
130 at 250	15°	-	68	45	104	27	4	248
125 at 250	10°	0,27	31	32	76	22	4	165
110 at 240	10°	-	27	60	75	24	4	190
110 at 240	15°	-	35	43	64	21	3	166
95 at 250	10°	0,20	17	30	70	17	1	135

Average Values from Above

117 at 250	10°	0,21	38	57	73	19	3	190

Total littoral drift = 19 kg/minute

Breaker depth (gauged after draining tank) = 190 mm.

Angled Attack

Test 2

Period = 2,1 seconds

θ_o = 30^o

Ten tests of ten minutes duration each

Wave Height & Depth (mm)	Breaker Angle	Longshore Velocity (m/s)	Littoral Drift (kg)					
			1	2	3	4	5 + 6	Total
175 at 255	5°	0,27	71	79	40	39	53	282
160 at 255	10°	-	23	27	22	73	55	200
155 at 255	20°	0,27	16	30	34	65	64	209
145 at 255	15°	-	41	41	30	71	55	238
155 at 250	15°	0,22	37	62	39	41	35 + 22	236
170 at 255	15°	-	28	53	60	55	32 + 9	237
170 at 255	-	0,27	32	63	65	51	22 + 18	251
155 at 255	20°	-	35	62	75	24	20 + 24	240
145 at 245	15°	0,22	24	59	76	24	10 + 26	219
145 at 240	20°	-	33	58	93	26	11 + 31	252

Average Values from Above

158 at 255	15°	0,25	34	53	53	47	49	236

Total littoral drift	23,6 kg/minute
plus correction factor for pile up (estimated)	1,4
	25,0

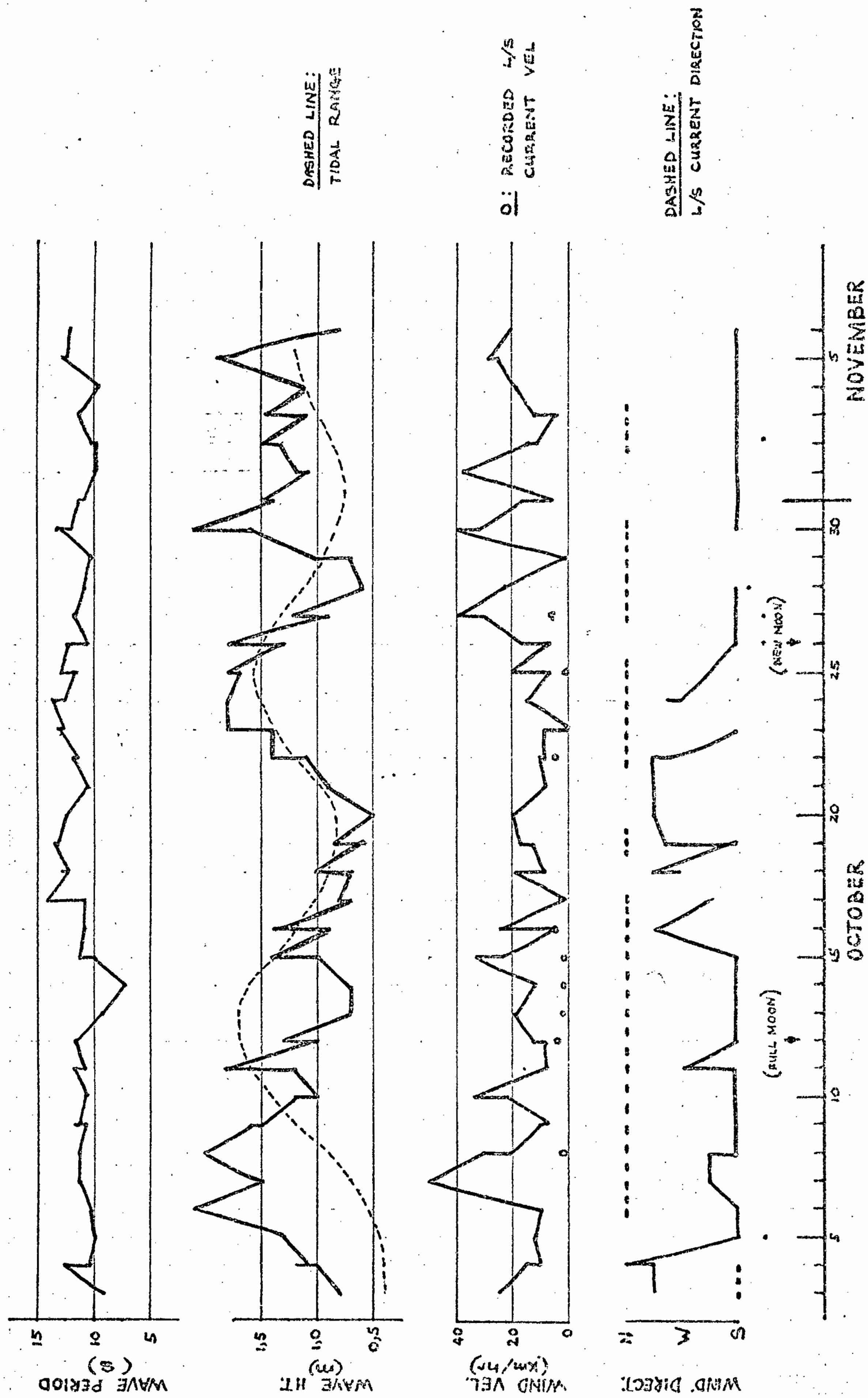
Breaker depth (gauged after draining tank) = 200 mm

APPENDIX G

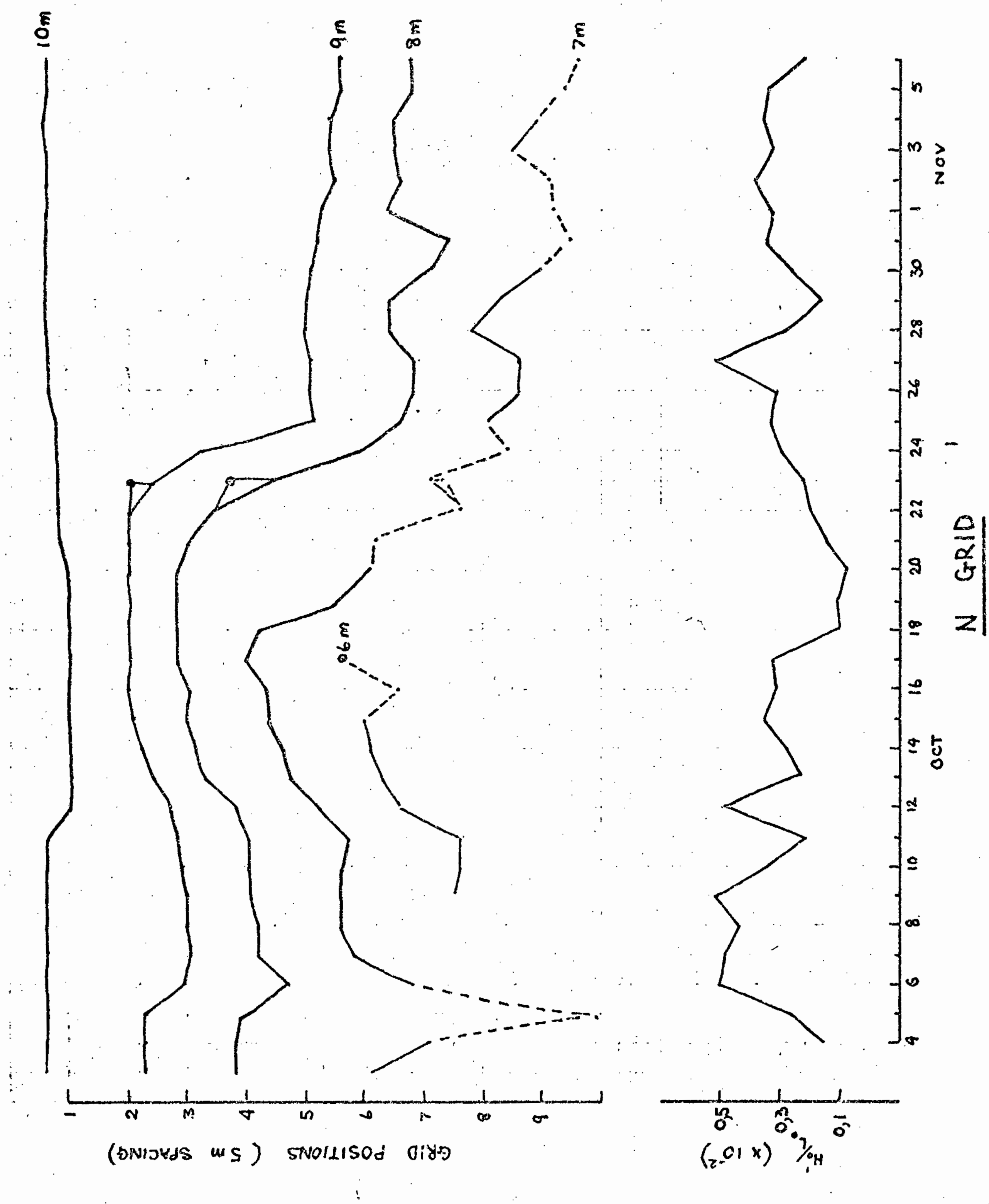
Littoral Drift studies by A. Stewart (UCT - 1973)

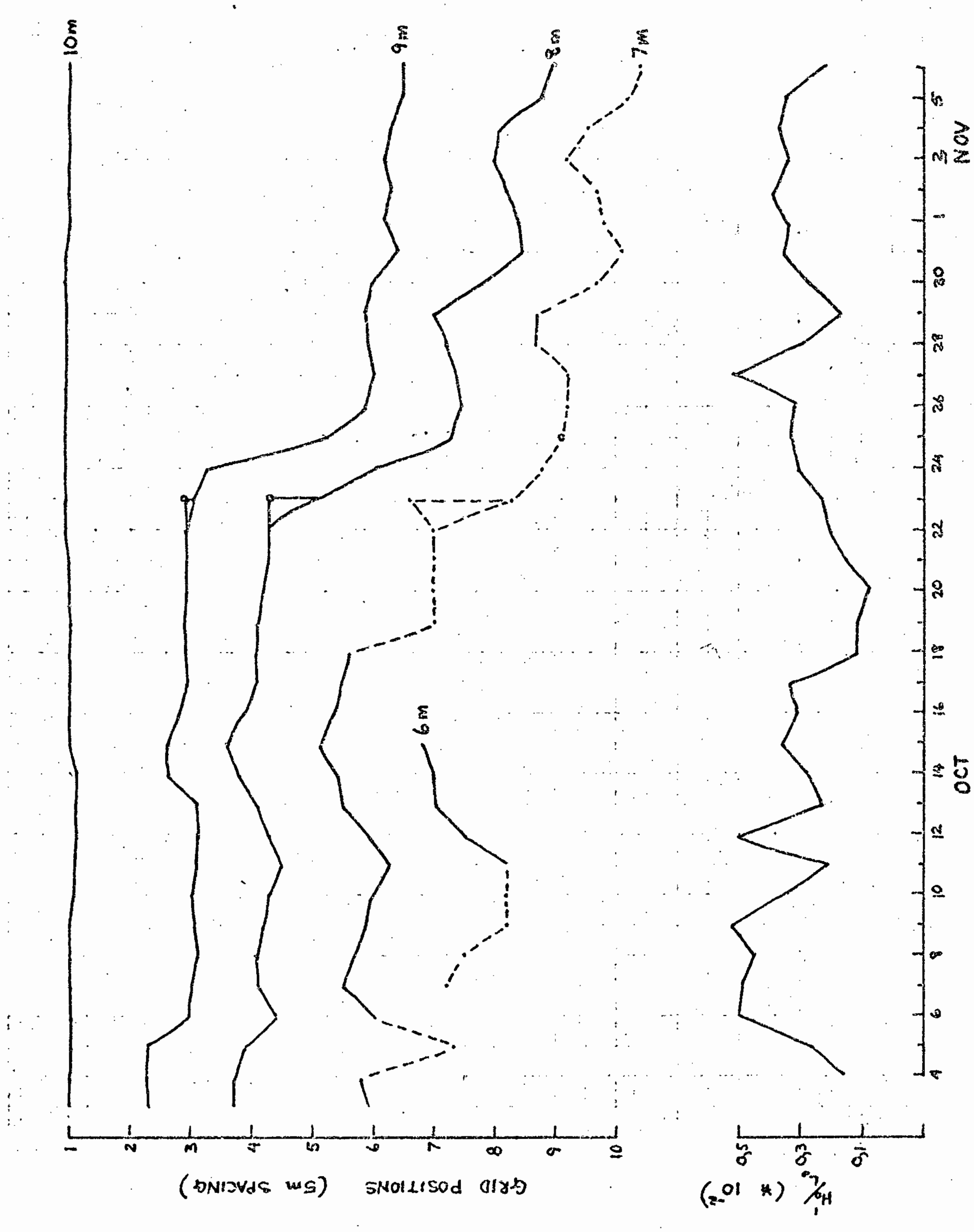
Test Number	Beach Angle	T (sec)	H (mm)	d (mm)	H_o^1 (mm)		c_b (m/s)	F (N/m)	F c_b (N/s)		I (kg/s)	Q $(m^3/s) x 10^{-4}$
1	10°	2,6	57	160	44		1,3	0,20	0,26		0,03	0,22
2	10°	2,1	102	130	83		1,1	0,72	0,79		0,08	0,7
3	10°	3,9	57	127	35		1,1	0,13	0,14		0,03	0,21
4	20°	3,8	72	195	49		1,4	0,47	0,66		0,05	0,47
5	20°	2,6	64	182	51		1,3	0,51	0,66		0,04	0,36
6	20°	2,8	115	195	89		1,4	1,6	2,2		0,06	0,52
7	20°	2,8	104	191	80		1,4	1,3	1,8		0,12	1,01
8	20°	3,8	62	172	40		1,3	0,31	0,40		0,04	0,34
9	20°	3,0	245	175	180		1,3	6,4	8,3		0,23	2,03
10	30°	2,7	82	240	68		1,5	1,2	1,9		0,03	0,21
11	30°	3,9	59	253	30		1,6	0,24	0,38		0,02	0,16
12	30°	2,0	90	252	84		1,6	1,9	3,0		0,04	0,35
13	30°	2,7	112	253	94		1,6	2,3	3,7		0,05	0,40
14	30°	2,7	55	235	45		1,5	0,54	0,8		0,03	0,26

APPENDIX H

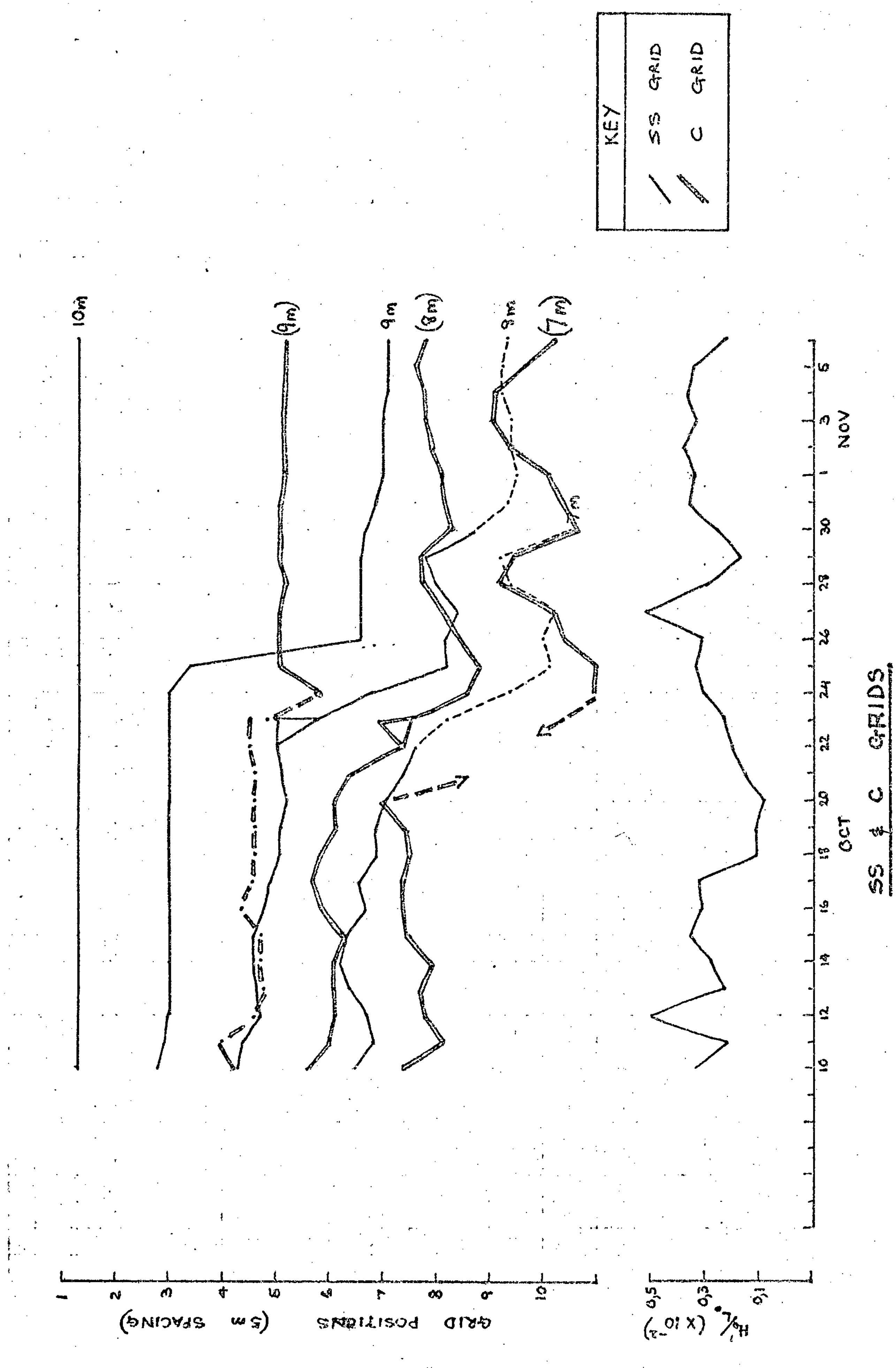


WAVE PERIOD (s)
15
10
5
WAVE HT. (m)
1,5
1,0
0,5
DASHED LINE: TIDAL RANGE
WIND VEL. (km/hr)
40
20
0
O: RECORDED L/S CURRENT VEL
WIND DIRECT.
N
W
S
DASHED LINE: L/S CURRENT DIRECTION
(FULL MOON)
(NEW MOON)
5
10
15
20
25
30
5
OCTOBER
NOVEMBER


10m
9m
8m
7m
6m
GRID POSITIONS (5 m SPACING)
OCT
NOV
N GRID



S GRID


KEY
SS GRID
C GRID
10m
(9m)
9m
(8m)
9m
(7m)
OCT
NOV
10
12
14
16
18
20
22
24
26
28
30
1
3
5
GRID POSITIONS (5m SPACING)
1
2
3
4
5
6
7
8
9
10
0.5
0.3
0.1
SS & C GRIDS

RAW WAVE DATA

NOTE: 1. times as underlined coincide with times of beach profile measurement

2. Units H_{sig} in metres

T in seconds

DATE	TIME	H_{sig}	T	DATE	TIME	H_{sig}	T
3.10	15.50	0,8	9,0	21.10	17.20	0,9	10,5
4.10	10.00	1,0	12,5	22.10	09.30	1,1	11,8
	14.30	1,2	10,6		18.00	1,4	11,6
	16.15	1,0	10,1	23.10	09.30	1,4	13,0
5.10	10.30	1,3	10,0		17.15	1,8	12,6
6.10	11.15	2,1	10,2	24.10	09.30	1,8	13,4
7.10	16.30	1,5	11,3		16.30	1,8	12,5
8.10	14.15	2,0	11,2	25.10	09.30	1,7	11,7
	18.00	2,0	11,3		16.30	1,8	12,8
9.10	10.10	1,6	10,9	26.10	08.30	1,3	12,3
	14.30	1,6	11,5		17.00	1,8	10,5
	17.15	1,5	11,2	27.10	09.45	0,9	11,6
10.10	10.20	1,2	10,7		16.00	1,2	11,5
	14.45	1,0	10,8	28.10	10.00	0,6	10,8
11.10	09.00	1,2	11,7	29.10	10.20	0,7	10,2
	15.30	1,8	10,7		17.30	1,0	10,3
12.10	09.10	1,0	11,5	30.10	11.00	1,6	13,2
	14.50	1,3	11,7		16.30	2,1	11,9
13.10	09.30	0,7	9,3	31.10	11.45	1,4	11,3
14.10	11.30	0,7	7,2		17.20	1,5	10,9
15.10	11.15	1,0	10,0	1.11	12.30	1,1	10,0
	16.40	1,4	11,2		17.00	1,2	9,8
16.10	12.15	0,9	10,9	2.11	10.00	1,3	9,9
	16.30	1,4	10,9		15.00	1,5	10,0
17.10	12.00	0,7	10,9	3.11	07.30	1,1	11,2
	16.45	0,8	14,2		15.00	1,5	11,1
18.10	10.00	0,7	12,4	4.11	15.10	1,1	9,5
	13.15	0,8	12,8	5.11	09.30	1,9	12,6
	17.00	1,0	12,4		16.00	1,8	12,3
19.10	09.10	0,6	13,4	6.11	07.20	0,8	12,1
	16.00	0,8	13,2		15.20	0,6	11,1
20.10	11.40	0,5	12,5				

FIELD STUDY

PROCESSED WAVE DATA

DATE	$T_{(ave)}$ (s)	H_{sig} (m)	L_o (m)	H_o^1 (m)	$Ho^1/_{Lo}$
3.10	10,8	0,9	182	0,3	0,0016
4.10	10,8	1,1	182	0,5	0,0027
5.10	10,1	1,7	159	0,8	0,0050
6.10	10,8	1,8	182	0,9	0,0049
7.10	11,3	1,8	199	0,9	0,0045
8.10	11,1	1,9	192	1,0	0,0052
9.10	11,5	1,6	206	0,7	0,0034
10.10	10,8	1,0	182	0,4	0,0022
11.10	10,7	1,8	179	0,9	0,0050
12.10	11,7	1,3	214	0,5	0,0023
13.10	8,3	0,7	107	0,3	0,0028
14.10	8,6	0,9	115	0,4	0,0035
15.10	11,2	1,4	196	0,6	0,0031
16.10	10,9	1,4	185	0,6	0,0032
17.10	13,0	0,8	264	0,3	0,0011
18.10	13,0	0,8	264	0,3	0,0011
19.10	12,9	0,7	260	0,2	0,0008
20.10	11,5	0,7	206	0,3	0,0015
21.10	11,2	1,0	196	0,4	0,0020
22.10	13,0	1,4	264	0,6	0,0023
23.10	13,0	1,8	264	0,8	0,0030
24.10	12,5	1,8	244	0,8	0,0033
25.10	12,8	1,8	256	0,8	0,0031
26.10	10,5	1,8	172	0,9	0,0052
27.10	11,5	1,2	206	0,6	0,0029
28.10	10,5	0,7	172	0,3	0,0017
29.10	11,2	1,1	196	0,5	0,0026
30.10	12,1	1,7	228	0,8	0,0035
31.10	10,7	1,3	178	0,6	0,0034
1.11	10,0	1,3	156	0,6	0,0038
2.11	10,8	1,4	182	0,6	0,0033
3.11	10,3	1,3	166	0,6	0,0036
4.11	11,5	1,6	206	0,7	0,0034
5.11	12,2	1,3	232	0,5	0,0022

FIELD STUDY

BEACH GRID CROSS-SECTIONAL AREAS

	N GRID units (m^2)			S GRID units (m^2)		
DATE	AREA	Change (total)	DAILY change	AREA	Change (total)	DAILY change
3.10	50	DATUM		49	DATUM	
4	53	3	3	49	0	0
5	57	7	4	55	6	6
6	59	9	2	54	5	- 1
7	53	3	- 6	50	1	- 4
8	52	2	- 1	52	3	2
9	50	0	- 2	54	5	2
10	50	0	0	54	5	0
11	50	0	0	56	7	2
12	46	- 4	- 4	53	4	- 3
13	38	- 12	- 8	50	1	- 3
14	36	- 14	- 2	46	- 3	- 4
15	34	- 16	- 2	42	- 7	- 4
16	35	- 15	1	47	- 2	5
17	31	- 19	- 4	49	0	2
18	38	- 12	6	50	1	1
19	45	- 5	7	56	7	6
20	46	- 4	1	57	8	1
21	47	- 3	1	58	9	1
22	52	2	5	58	9	0
23	50	0	- 2	62	13	4
24	67	17	17	68	19	6
25	74	24	7	80	31	12
26	78	28	4	83	34	3
27	78	28	0	83	34	0
28	75	25	- 3	83	34	0
29	76	26	1	82	33	- 1
30	80	30	4	84	35	2
31	81	31	1	85	36	1
1.11	81	31	0	85	36	0
2	82	32	1	85	36	0
3	81	31	- 1	85	36	0
4	81	31	0	85	36	0
5	83	33	2	86	37	1
6	83	33	0	86	37	0

APPENDIX N

EXAMINATIONS WRITTEN TO COMPLETE THE REQUIREMENTS OF THE DEGREE

Examination		Credit Rating
CE 518	Water Treatment (Project and Oral Examination)	10
CE 504	Probability and Statistics for Engineers	5
CE 520	Earth Dams	2
CE 521	Foundation Engineering (Project and Oral Exam)	2
CE 514	Nomography	4
Thesis		20

UNIVERSITY EXAMINATION NOVEMBER 1971

Probability and Statistics for Engineers

Time : 3 hours

External Examiner : Dr. D.M. Schultz
Internal Examiners : Professor G. v.R Marais
Mr. A.H. Money

Answer FIVE questions.
All symbols used have their usual meaning.

---

1. (a) State the axiomatic definition of probability.

(b) Show by using the three basic axioms of a probability distribution that

(i) If $B \subseteq A$ then $P(B) \leq P(A)$

(ii) For any event A $0 \leq P(A) \leq 1$.

(c) State and prove Bayes Theorem.

(d) Suppose an office has four secretaries handling respectively 20; 60; 15 and 5 percent of the filing of all government reports. The probabilities that they misfile such reports are 0,05; 0,10; 0,10; and 0,05. Find the probability that a misfiled report can be blamed on secretary No. 1. Comment on the result.

(3; 5; 7; 5)

2. (a) Define the following statistics (i.e. sample values.)

(i) Mean
(ii) Median
(iii) Mode
(iv) Variance and Standard deviation
(v) Coefficient of Variation.

(b) The lifetime of a sample of 100 transistors were tested, with the following results

Lifetime in hours	f Number of Tubes	Cum f.	$\frac{2n-1}{2m}$.
$500 \leq X < 600$	1	1	
$600 \leq X < 700$	0	1	
$700 \leq X < 800$	7	8	
$800 \leq X < 900$	12	20	
$900 \leq X < 1000$	31	51	
$1000 \leq X < 1100$	24	75	
$1100 \leq X < 1200$	18	93	
$1200 \leq X < 1300$	6	99	
$1300 \leq X < 1400$	1	100.	

Using this data calculate

(i) The mean
(ii) The mode
(iii) The median
(iv) The variance and standard deviation
(v) The coefficient of variation

(8; 12)

3. (a) continued.............

Clearly indicate the advantages and disadvantages of the two tests.

(b) Suppose it is desirable to check whether pinholes in electrolytic tin plate are distributed uniformly across a plated coil on the basis of the following distances (in cms) of 10 pinholes from one edge of a long strip of tin plate 30 cms wide

4,8 14,8 28,2 23,1 4,4 28,7 19,5 2,4 25,0 6,2

Test whether the pin holes are uniformly distributed clearly stating

(i) the null hypothesis (H_0)

(ii) the test statistic

(iii) the level of significance

(iv) the decision rule.

Note: For $n = 10$ and $\alpha = 0{,}05$ $D_{10;0,05} = 0{,}410$

(11; 9)

4. (a) Suppose $X_1, X_2, \ldots, X_n$ is a sample of size n from a population with true mean m_X and variance σ_X^2. Show that $E(\overline{X}) = m_X$ and $Var(\overline{X}) = \sigma_X^2/n$.

(b) State and prove the central limit theorem. (Assume the moment generating function exists.)

(c) (i) Suppose a random sample of size 100 is taken of inner diameters of certain lengths of seamless pipe. If the mean and standard deviation of such measurements are respectively $m_X = 34{,}1$ cms and $\sigma_X = 1{,}5$ cms, what is the probability that the mean of the sample will lie between 34,0 and 34,3 cms?

(ii) Suppose $\overline{x} = 34{,}5$ cms. Would you say that the sample came from a population having $m_X = 34{,}1$ and $\sigma = 1{,}5$.

(4; 8; 8)

5. (a) Complete the statements of the following theorems.

(i) If $\overline{x}$ is the mean of a random sample of size n taken from a normal population having mean m_X and variance σ_X^2 then

$t = \dfrac{\overline{x} - m_X}{s^*/\sqrt{n}}$ is the value of a random variable having the

.... distribution with .... degrees of freedom, where s^{*2} = ... is an unbiased estimate of σ_X^2.

(ii) If s_1^{*2} and s_2^{*2} are the variances of two independent random samples of sizes n_1 and n_2 respectively, taken from two normal populations having the same variances, then

$F = \dfrac{s_1^{*2}}{s_2^{*2}}$ is a value of a random variable having the..........

distribution with parameters ν_1 = ... and

ν_2 = ...

5. (a) (ii) continued...........

Further ν_1, is the degrees of freedom for the sample variance in the .... and ν_2 is the degrees of freedom for the variance in the ....

(b) A standard cell, whose voltage is known to be 1,10 volts (i.e. the true mean is claimed to be 1,10 volts,) was used to test the accuracy of the two voltmeters, A and B. Ten independent readings of the voltage of the cell was taken with each voltmeter, and the results were as follows

A	1,11	1,15	1,14	1,10	1,09	1,11	1,12	1,15	1,13	1,14
B	1,12	1,06	1,02	1,08	1,11	1,05	1,06	1,03	1,05	1,08

From these results is there any evidence of bias in either voltmeter? Also, is there any evidence that one voltmeter is more consistent than the other?

(7; 13)

USE A SEPARATE BOOK WHEN ANSWERING THIS QUESTION;

6. (a) Give graphical examples of sets of measurements which are

(i) imprecise-accurate
(ii) precise-inaccurate.

(b) An investigator measures the size of drops in an aerosol by three techniques to give

(i) diameter
(ii) surface area,
and
(iii) volume of drops.

27
8
4
39

He finds that the distribution of the diameters are normal but the other two distributions are skew to the right. Give a reason why these results are found and suggest a procedure by means of which they can be analysed.

(c) (i) A length of railway line is measured a number of times by means of a steel tape. What kind of distribution do you expect from the data and why?

(ii) A well mixed pond received an effluent containing bacteria. The bacteria die off according to the law of

$N = N_o e^{-KR}$ where R = retention time of pond (days)
N_o, N = concentrations of bacteria per unit volume in the influent and pond respectively
K = death-rate of bacteria (day^{-1}).

Assuming steady state conditions a number of tests on the concentration of the bacteria in the pond are done over a period of a month. What type of distribution do you expect and why?

6. continued..........

(d) A set of counts on the influent to a sewage works in the Tropics gives the following concentration of bilharzia ova per ml:-

25
35
80
600
70
8
70
17
170
80
17
250
80

Determine graphically the average concentration of the counts.

After anti-bilharzia measures are put into operation it is found from a set of counts equal in number to the previous that the geometric mean density has decreased by 50%, but that the geometric standard deviation has remained approximately the same. Using graphical methods determine whether the reduction is significant at 96% confidence level.

UNIVERSITY OF CAPE TOWN

DEPARTMENT OF CIVIL ENGINEERING

EXAMINATION FOR THE PARTIAL FULFILMENT OF

COURSE CE 5.20(a) - EARTH DAM DESIGN

Time allowed: 2½ hours

May, 1972

Note: The marks for the tutorials, the project and this written examination will together form the final mark for the course

100 marks will constitute full marks for this paper

1. With the aid of sketches, briefly describe the effect of filters and dam geometry on the stability of an earth or rockfill dam, and also indicate approximately the types of flow net pattern which can be expected for the different types of dams.

About one third of your discussion should be concerned with the stability aspects. (Suggested length of answer - about four or five pages of writing and sketches.)

[25 marks]

2. Briefly describe one of the following:

(a) The development and dissipation of excess pore water pressure due to dam construction and the effects of this excess pore pressure on the stability of a clay core earth dam. (Illustrate your answer with sketches wherever necessary.)

(b) The various stages in the planning of an earth dam project for irrigation, or municipal water supply purposes. (Suggested length of answer - about three or four pages.)

[20 marks]

3. Briefly, with the aid of sketches, describe one of the following topics: (Answer to be about two or three pages.)

(a) Membranes for earth and rockfill dams.

(b) The use of instrumentation in earth or rockfill dams.

(c) Hydraulic-fill dams.

(d) Floods and hydrology in relation to earth dam design.

(e) Floods and spillways.

(f) Properties and the testing of rockfill material.

(g) The properties, and the testing of materials for earth dams.

(h) Field and laboratory permeability tests.

(i) Site investigations for earth dams.

(j) The effect of seismic disturbances on the stability of earth dams.

(k) Practical aspects relating to grouting, membranes, filters and other design details for earth dams.

[15 marks]

4. The inner core of the dam shown in Figure 4 consists of a material A with a coefficient of permeability:

$$k_A = 2 \times 10^{-4} \text{ cm/s}$$

whereas the outer two zones consist of material B which has a coefficient of permeability k_B which is twice the value of k_A.

(a) Draw a flow net for the dam section.

(b) Briefly list the various construction rules or constraints which should apply to this flow net.

(c) Estimate the rate of seepage through the dam.

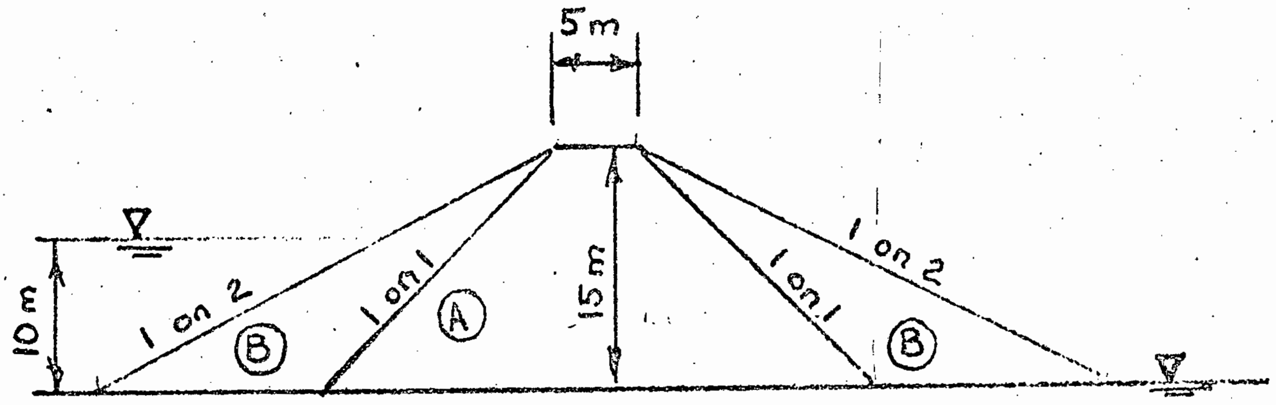


FIG. 4

[25 marks]

5. Consider a typical slice used in the method of slices for slope stability analyses. Draw all the forces which act on this slice and also draw the force polygon. Label and identify all the forces in the force polygon. Hence explain in words (and with the aid of sketches where necessary) the basic assumptions within, and the differences between, different methods of analysis.

For example the following methods could be considered:-

(a) the conventional simple method of slices;

(b) the Bishop method (described in Guthrie Brown's Book)

(c) the modified iterative method (Janbu and Bishop) mentioned in the 1967 edition of Terzagh and Peck's book.

(d) the method for non-circular composite surfaces (Terzaghi and Peck).

Particular attention must be given to the concepts involved. Formulae can assist your discussion but are not essential, unless in a simple form.

[30 marks]

6. (a) Briefly describe the method for plotting flow lines and equipotential lines from readings taken during a laboratory test on a model dam, in which a capillary seepage zone exists above the phreatic surface. How can one find the position of the phreatic surface in such a model?

[10 marks]

(b) Briefly provide a reason or justification for the use of $\phi = 0$ in certain soil stability analyses.

[5 marks]

/7. ....

7. A copy of pages 248 to 253 from Terzaghi and Peck's book is supplied for your assistance. Hence perform at least one cycle of calculations to determine the factor of safety F for the composite sliding surface shown in Figure 7. Use the slices which have been numbered in the sketch. If simplifying assumptions are made, list these above your table. Also list the formula which you have used. For your first try assume F = 2,0. Suggest the value of F to use in the second cycle.

A slip surface of the type shown in Figure 7 would be possible if the slope consisted of two different soils. The values of c and ø for the two soils are shown in Figure 7.

The pore pressures may be assumed to be zero. Both soils have a specific weight of 20 kN/m^2.

[35 marks]

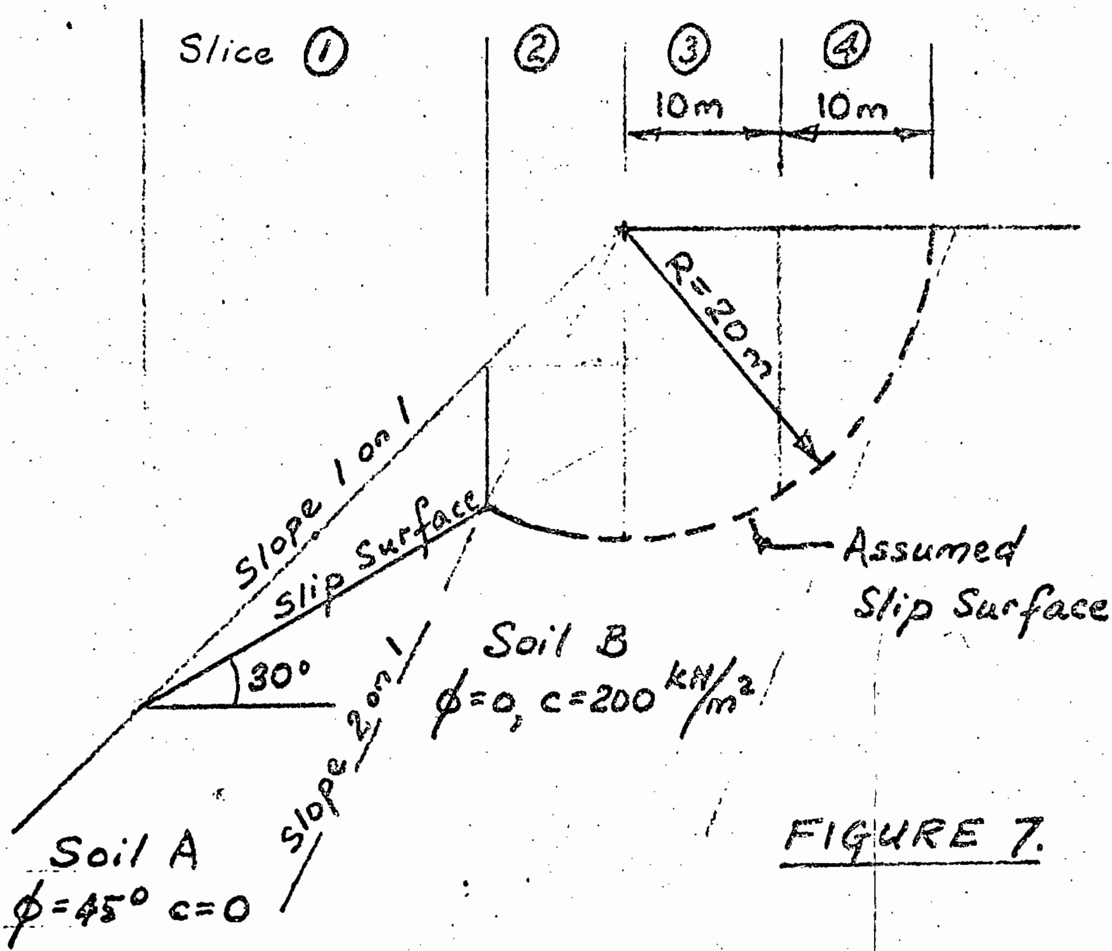


FIGURE 7.

TIME ALLOWED: 3 HOURS

UNIVERSITY OF CAPE TOWN

DEPARTMENT OF CIVIL ENGINEERING

COURSE CE 5.14 : NOMOGRAPHY

UNIVERSITY EXAMINATION

9.00 am, Saturday, August 5th, 1972

Q.1. Outline, with the aid of sketches, how the modified Lafay method may be used to set the scales of a rectified intersection diagram preparatory to its dualisation. Indicate in particular to what extent the distribution of the principal points of the scales may be controlled.

Q.2. A nomographer, interested in only a short section of the U scale near to its end where (U) has very high values, but concerned with the full ranges of (V) and (W) values from zero to infinity, begins with a circular nomogram solving (U) (V) = (W) in the form shown in figure 1 for which he uses the basic determinant

$$\begin{vmatrix} \dfrac{d\,U^2}{U^2+1} & \dfrac{d\,U}{U^2+1} & 1 \\ \dfrac{d\,V^2}{V^2+1} & \dfrac{-d\,V}{V^2+1} & 1 \\ \dfrac{d\,W}{W+1} & 0 & 1 \end{vmatrix} = 0$$

With the purpose of improving the lengths of the scales he plots a diagram in the YZ plane as shown in figure 2, and projects it on to the XY plane from a point of projection P whose coordinates on the X, Y and Z axes are:

$$\left(-d;\ \frac{3d}{2};\ \frac{3d}{2}\right)$$

EITHER (a) obtain the resulting alignment chart graphically, establishing five principal points on each scale.

OR (b) obtain the determinant for plotting the chart in the XY plane without the need for projective constructions.

Q.3. Draft a double alignment chart to solve the equation

$$t^2 = \frac{2\,W\,x}{M}$$

where t = thickness of a concrete road foundation in mm,

W = design wheel load in N,

x = variable depending on subsoil properties (dimensionless),

M = design modulus of rupture of the concrete in N/mm^2.

The ranges to be covered are:

W : from 9 000 N to 55 000 N

x : from 0,7 to 1,0

M : from 1 N/mm^2 to 2 N/mm^2

The range of t is to be deduced.

Do you know of any technique available for solving the problem with a single alignment?

Q.4. Obtain a disjunction for the equation:

$$(1 - U)\tan V + UW \sin V + (1 + U)(1 - W) = 0$$

Sketch the chart you would obtain from your determinant if the ranges are from 0,1 to 0,9 for both U and W and if V varies from 100° to 180°. Comment upon any disadvantages that the chart may have.

CE 5.111
8/72

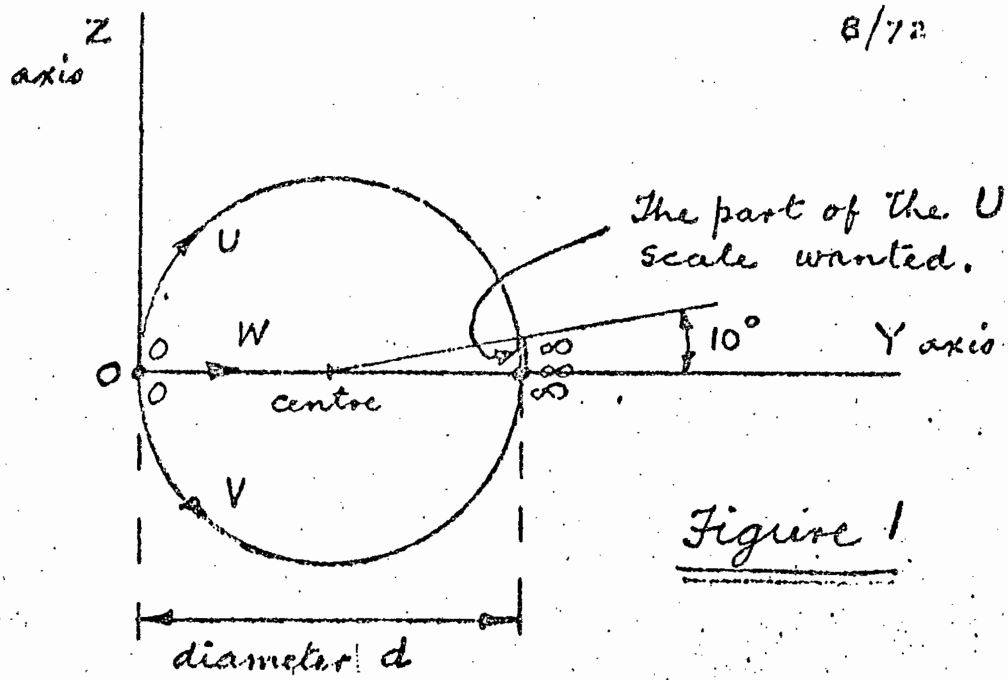


Figure 1

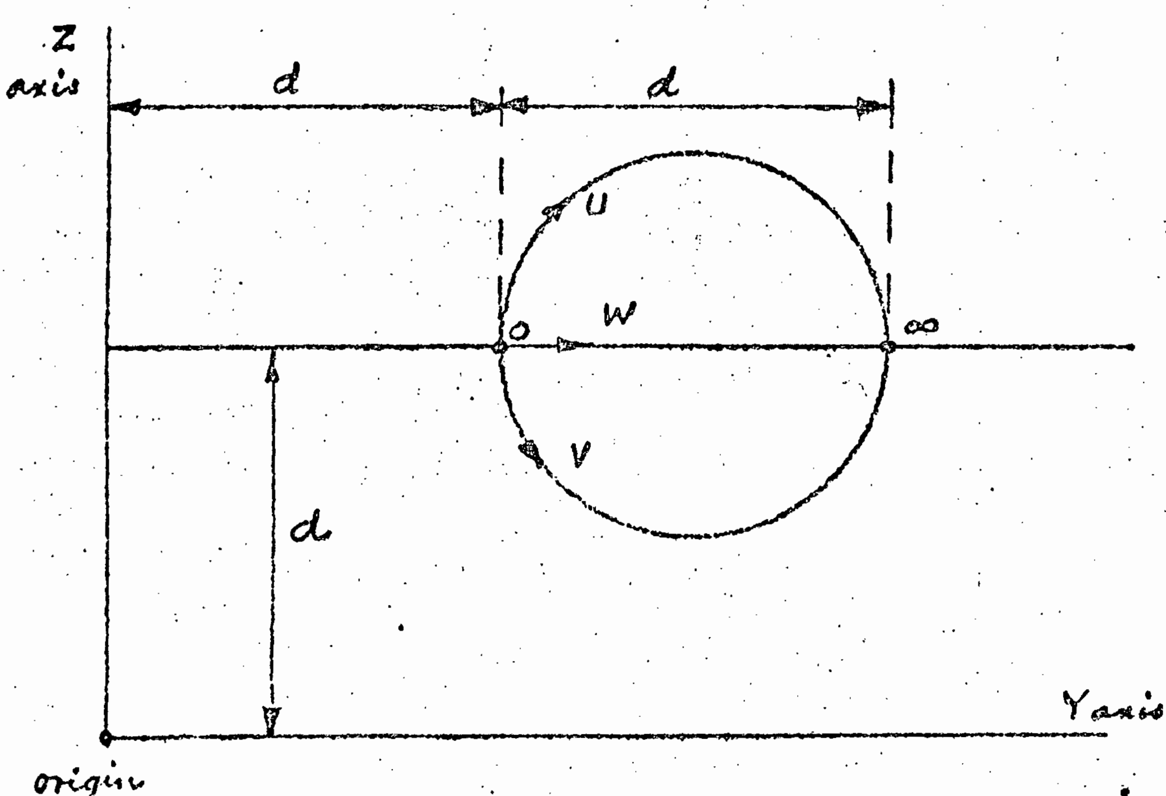


Figure 2

W.H.K.

N-12

PARSHALL FLUME (CHART II)

CORRECTION FACTOR FOR SUBMERGED FLOW

$$C = \left[\left\{\frac{H_a}{\left(\frac{1.8}{K}\right)^{1.8} - 2{,}45}\right\}^{4{,}57-3{,}14K} - 0{,}093K\right] W^{0{,}815}$$

WHERE $K = H_b / H_a$