

CHAPTER 108

RESEARCH FOR THE COASTAL AREA OF THE DELTA REGION OF THE NETHERLANDS

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ABSTRACT

Tidal computations, wave studies, refraction computations, and morphological studies are discussed and practical results are mentioned. Theoretical investigations on refraction computations, in particular the accuracy, are dealt with in detail, because difficulties occurred in the practical applications. A former study of Morra, based on Kalinske's work on sand transport, has also been discussed in some detail in the section on morphological studies.

1 Introduction

The closing of the various sea-arms in the Deltaplan, the building of sluices, and the construction of Europort Harbour will cause great changes in the hydraulic and the morphologic situation along the south western part of the Netherlands. The practical purpose of these studies is the determination of the future conditions for the beaches and dunes, the discharge of the sluices, and the navigation. A big hydraulic model has been built at the Hydraulic Laboratory at Delft for the planning of the construction of Europort Harbour. The length scales are 1:640 in the horizontal plane, and 1:64 in the vertical.

The changes in the tidal motion can be computed in a satisfactory way, based on Leendertse's computational scheme. Difficulties do not occur with respect to the stability of this scheme. The accuracy of the method depends on the gridsize and the timestep.

In section 3, a discussion is given about wave research and practical determination of the energy wave spectrum. A relation of depth and breaking of waves is obtained from the wave observations.

Informations are also obtained about the changes of the directions of the waves by means of refraction computations. These computations are based on the first order wave theory, which is an approximation of the irregular wave motion. Usually an iteration process has been applied for the determination of the celerity of the shallow water wave. In section 4 a direct formula for the celerity is derived by means of which the accuracy of the refraction computation is discussed.

The future depths must be determined in the coastal area. An iteration process must be carried out, because the changes in the currents and waves determine the morphological changes and vice versa. The determination of the morphological changes is a difficult affair. The studies describe the physical factors in a general way. Simplifications must be accepted and the empirical coefficients in these formulae have to be determined from

observations in nature

The relation between the sediment transport and the currents is much better known than for the wave action. The methods of determining of these relations will be discussed in section 5. The application of a physical scale model of the Haringvliet area for these studies has been considered, but rejected, because the sand in the Haringvliet mouths is very fine and the relation between sand transport along the bottom and that in suspension is unrepresentative in the scale model as compared to nature.

For the future a great research program has been set up for getting observations from nature, during the period in which the great changes in the coastal area will take place. By means of the use of radio-active tracers, the direction and the relative significance of the sand transport can be determined. The most important tool is however the study of the changes in the shape of the bottom obtained from periodically repeated soundings after every three months.

The programs for the computations on the electronic computer of the Rijkswaterstaat (Elliott 503) are composed in Algol by the Mathematical Physical Division.

2 Tidal computations in the coastal areas of the Delta region**2 1 The effect of the Delta works on the tidal motion in the North Sea**

The total quantity of water flowing to and from the sea-arms to be enclosed amounts about 2,000 million m³ during normal tide, while the flood volume passing through the Strait of Dover is about 19,000 million m³ and the ebb volume 17,000 million m³. The flood volume passing through a section crossing the North Sea from the isle of Walcheren to Harwich amounts about 50,000 million m³, so that the tide in the southern part of the North Sea comes mainly from the north and the effect of the tide passing through the Strait of Dover is limited. From these figures it is obvious that the closing of the sea-arms of the Delta-Works must have a small effect on the tidal motion in the North Sea and that the studies can be limited to the coastal area.

2 2 The tide along the Delta coast

The vertical tide changes considerably along the Dutch coast at Flushing (south of the Delta area) the mean tidal range is 3.8 m and at the Hook of Holland (north of the Delta area) 1.6 m, where the tide arrives 2.5 hours later.

The shoals along the coast of the Delta region are about 8 km wide. The transition from the 10 to 20 m contour line of depth can be considered as the boundary of the "underwater" delta. The greatest depth in the North Sea opposite the coast is 30 m. The bottom topography in the mouths of the sea-arms shows many gullies and shoals.

The pattern of the tidal currents in the mouths of the estuaries is complex because of the interference provided by currents flowing in the direction of the estuaries and tidal currents running parallel to the coastline caused by the prevailing tidal motion in the North Sea. The shape of the tidal current diagram changes from elongated in the direction of the coast to elliptical and then again to elongated, more or less perpendicular to those of the

North Sea currents

For the tidal studies in the coastal areas an extensive measuring programme has been set up in order to obtain a detailed insight into the vertical and horizontal tide in a strip 120 km along the Delta coast. The boundaries of the strip are shown in figure 1. Such programs were carried out on 27 and 28 June 1967 and on 22 May and 9 September 1969.

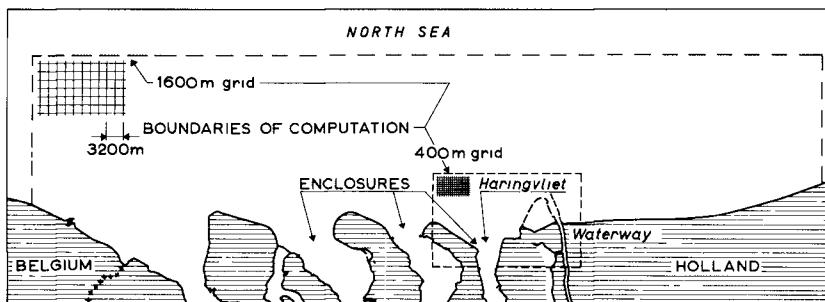


Fig 1 Coast Delta region and boundaries of the grid net

2 3 The tidal computations

The method, developed by Leendertse (1) has been used for the tidal computations. Some practical remarks are mentioned hereafter.

The vertical tides as measured at the seaward boundaries by pressure meters put on the bottom are used as a boundary condition for the tidal computations. The velocities in the sea, which depend more on local depth conditions than does the vertical tide, are not used as boundary conditions.

In the mouths of the sea-arms the vertical tides measured by means of tidal gauges or the velocities can be considered as boundary conditions. Obviously the components of the velocity perpendicular to the coastline of the islands are zero.

For computational reasons it is desirable to locate the boundaries so that the values of the second order terms in the tidal equations are small in the neighbourhood of the boundary. This is so at the boundary of the strip, but it must be expected that the results of the computation of the velocity vectors near the boundary will be less accurate than in the inner region.

There are however other factors which may cause more serious inaccuracies in the computation. The values of the resistance coefficient and the schematization of the depths. Tests for the accuracy of the computations are necessary.

Tidal computations are carried out for a square net of which the gridsize is 1600 m. This grid is too rough for adequately taking into account depth variations in the neighbourhood of the coast and in the sea-arms. In this region a 400 m square net is used.

(see fig 1) The boundary conditions (vertical tides) of the second net can be computed by means of the results of the coarser net, provided the tide is not influenced by the circumstances near the coast. Otherwise, a separate measuring programme is necessary for the determination of the boundary conditions.

The vertical tides at the various grid points along the seaward and land boundaries are determined by interpolation between the water levels at the locations where the vertical tide has been measured, as follows. The shapes of the measured vertical tides are represented by Fourier series with a basic period of 12h 50 m and the Fourier coefficients of the vertical tides in between are obtained by interpolation. These computations are performed by the computer. The velocities at a great number of locations were also measured in the program e.g. at locations in between the places where the vertical tides were measured. As the vertical and horizontal tides depend on each other a check can be performed by means of the equation of motion applied to the boundary line

$$\frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} + v \frac{\partial u}{\partial y} - \Omega v = -g \frac{\partial h}{\partial x} - g \frac{V u}{C^2 a}, \quad (1)$$

in which the x axis is directed along a part of the boundary, V is the magnitude of the velocity vector of which u and v are the components in x and y directions (counter clock wise) and h is the height of the water level with respect to a datum plane. By replacing the differentials by difference quotients, a relation is found for the differences in water levels between the vertical tides at both sides of the location where the velocity has been measured.

However, the term $v \frac{\partial u}{\partial y}$ could not be determined from the observations. The values of the Chézy coefficient, C, has been checked by means of these computations and from former research in the coastal area. The values of C depend also on the depths, e.g. according to the well-known logarithmic law. However such a relation could not be derived from the observations in the coastal area.

The equation of continuity has also been applied to subregions of the coastal strip. Furthermore the importance of the various terms in equation (1) and the similar equation for the y-direction, has been determined, see Dronkers (2).

After preliminary computations detailed tidal computations were applied to the strip along the coast. The relative changes of the vertical and horizontal tide due to the enclosure of the sea-arms could then be determined.

Calculations were carried out after the Haringvliet was enclosed of which the flood volume was about 300 million m³. The future Europort Harbour was also included in the computations. Obviously the changes are maximal in the intermediate neighbourhood of this region. At a distance of 8 km from the coastline the changes are negligible. In fig 2 the results are shown for maximum flood and ebb. The directions of the velocities are denoted by arrows. A smaller gridsize, 400 m, has been used for these computations (see fig 1).

Computations were also performed for discharges passing through the sluice during the low water period.

During this last case the timestep has been shortened in order to provide the necessary accuracy. For the 1600 m gridsize the time step was 10 minutes and the computed vertical tidal curves appeared

to have a smooth shape. Irregularities in the shape of these curves occurred in the case of a net with 400 m gridsize. Therefore in this case the time step has been taken 2½ minutes. Obviously the propagation time in such a square is about 4 times shorter than in the case of 1600 m gridsize.

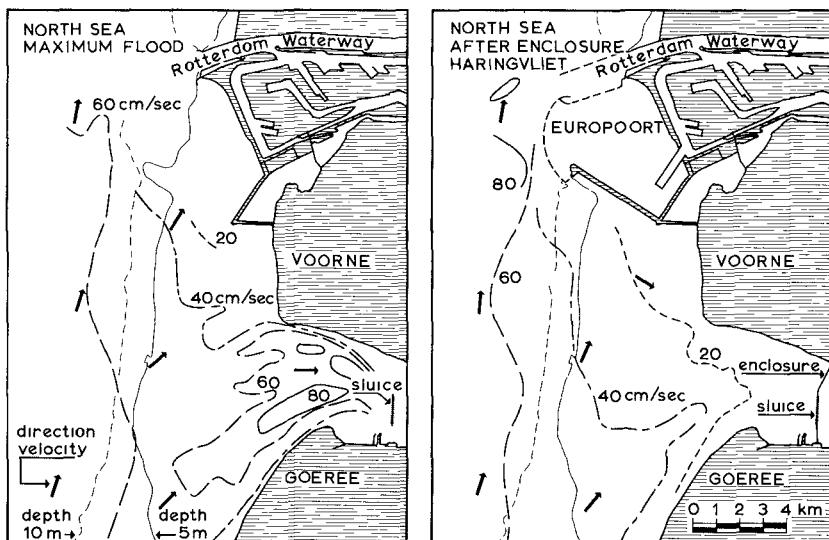


Fig. 2 Lines of equal velocities in the mouth of the Haringvliet

3 Practical wave investigations

For getting insight into the wave characteristics in the coastal area, a great number of wave observation piles are placed in the coastal area. The placing of heavy piles in the seabed is usually a costly affair and therefore the depth at these locations is limited to 10 m. Furthermore a wave instrument is put on a platform, which is at a location where the waterdepth is about 15 m. At some locations observations are taken for several years, maximal 5 years, usually they are observed for a shorter period, minimal one year, and then the piles are relocated. The instrument on the pile (which is an electrical step capacity gauge wave recorder with wireless transmission) does not measure wave heights smaller than 4 cm. The irregular waves and eventually the vertical tide observations are transmitted by radio to a coastal station where they can be observed on a pen recorder. By means of the so-called wave-rider, which is a wave buoy on which instruments are placed, the vertical accelerations of the movement of the buoy are measured. They can be used in the deeper parts of the coastal area. The directions of the waves cannot be measured by the instruments mentioned above. Radar

observations can be used for this purpose

Frequency and exceedance distributions of wave heights, distributions of periods, the wave period by zero crossing method and the energy spectral density function (energy spectrum) are determined for time intervals of about half an hour, during which the mean water level, due to the tide, does not change considerably in comparison to the depth. The significant wave height is one of the most important parameters for the practical applications. The largest and smallest value of the periods to be considered depend on the length of the measuring interval of the waves. Wave periods longer than 30 seconds or shorter than 1 second are eliminated by means of a filter procedure. For modern theories on waves, and in particular the energy spectrum, reference is made to Kinsman (3a) and Benda and Piersol (3b).

The mean wave period in the coastal areas of the Delta is about 5 seconds, although high waves generated by high winds, may have periods up to 10 seconds or more. A maximum wave height of 10 m has been measured at a water depth of 16 m. The significant wave height was about 7 m. Generally speaking the waves in the coastal area and even in a great part of the southern North Sea are of intermediate wave type between a long wave and a deep water wave (see section 4.1).

It has been found by Koele (4) that the distribution of the wave heights (trough to crest) over a sufficiently long measuring interval can approximately be represented by the well-known Rayleigh distribution even in rather shallow water, provided that small disturbances are omitted. In particular cases deviations from this distribution has been found.

From the wave observations in the coastal area of the Delta it has been found that the product of wind velocity and the frequency which occurs at maximum energy density is about 1.8. This is more than mentioned by Freudenthal (5). The definition of the wind velocity also effects the product mentioned above.

Waves may develop higher harmonics during their propagation in shallow water of which periods depend on the period of the original wave developed in deeper water. Consequently more waves of shorter period may be expected in such shallow regions than in deeper water under the same windfield.

It has also been found by Koele and de Bruyn (6) that in the breaker zone the significant wave height of irregular waves does not exceed about 0.5 of the local depth. The breaking of waves also depends on the steepness of the wave. In the example of fig 3, the bound is denoted by $H_s = 0.4a$, in which H_s is the significant wave height and a the depth. Particular circumstances may also have some influence on the coefficient, and therefore values between 0.4 and 0.5 occur.

Regular shaped wave profiles e.g. a trochoïde and a cnoidal wave will break when the wave height is maximal about 0.8 of the local depth. During the propagation in a shallow region the regular wave deforms

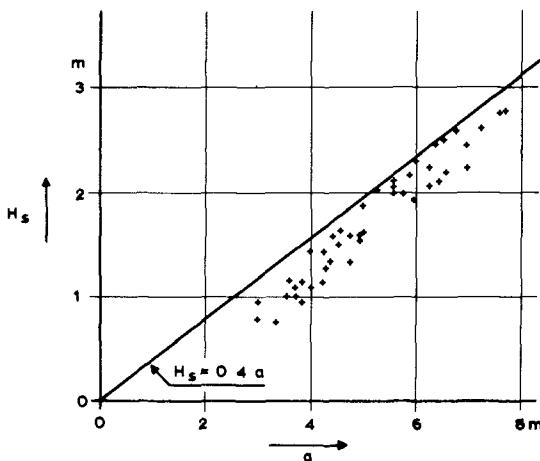


Fig 3 The relation between the significant wave height and the depth

For the statistical evaluation of wave conditions in reference to coastal engineering problems we may refer to Svasek (7). The relation between the significant wave height, H_s , and the area of the energy spectrum m_0 , is $H_s = 4 m_0^{1/2}$. This relation is based on the Rayleigh distribution and has also been verified by means of wave observations. The deviation in the factor 4 is about 0.2.

The calculation of the energy spectrum is based on the calculation of the well-known auto-covariance function $R(\tau)$, applied to the wave heights $h(t)$

$$R(\tau) = \frac{1}{P-\tau} \int_0^{P-\tau} h(t) h(t+\tau) dt = \frac{2\pi}{2\pi/30} S'(w) \cos(w\tau) dw,$$

where $S'(w)$ is the energy density and w is the frequency of a wave, while in practical applications the observational period P is usually 30 minutes (= 1800 seconds). The integral is replaced by a summation formula, in which $\Delta t = 0.2$ sec and $R(\tau)$ is calculated for successive values of τ at intervals of 0.2 seconds, from $\tau = 0$ up to $\tau_m = 30$ sec. Thus if the mean wave period equals 5 seconds about 350 wave periods are considered for each observation period of 30 minutes. The energy density $S'(w)$ is determined by

$$S'(w) = \frac{2}{\pi} \int_0^{\tau_m} R(\tau) \cos w\tau d\tau$$

(In practical applications $S(w) = \frac{1}{2} S'(w)$ has been considered)

4 Computation of the refraction diagrams4 1 Formula for the celerity of the wave

The purpose of the computation of wave refraction in the coastal areas of the Delta is to provide data on changes in the local wave characteristics that result from the morphological changes e.g. for the determination of the attack on coastal structures. Numerical refraction calculations on a computer for the case of a regular reference wave have been worked out by G M Griswold (8). The limitation for the accuracy of the wave ray computation is the accuracy of the depths which determine the celerity of the waves, c. A first approximation, based on linear wave theory, is applied

$$c = \frac{gT}{2\pi} \tanh \frac{2a\pi}{cT}, \quad (1)$$

in which T is the wave period and 'a' the depth. This formula holds if $2\pi H/L \ll 1$ and $\pi H/L \ll (2\pi a)^3/L^3$, in which H is the wave height.

For the computation of c from (1) an iteration procedure may be applied (see e.g. Duthler (9)). Here a direct approximate formula for c as a function of depth and period is deduced. This formula is useful for the determination of the accuracy of the refraction computation (see 4.2). The derivation will be included here because the author has not found this method in the literature. Equation (1) may be written in the alternative form

$$u \tanh u = P, \text{ where } u = \frac{2\pi}{cT} a, P = \frac{4\pi^2}{gT^2} a = \frac{4 \cdot 0255}{T^2} a \quad (1a)$$

Let a_T be defined such that for $a < a_T$ the celerity c depends on the depth. For $a > a_T$ the wave propagates as a deep water wave. For deep water waves $\tanh u$ will be approximated by 1, e.g. if $u \geq 2.7$, $\tanh u$ does not depend on depth. Then for $a > a_T$,

$$c = \frac{gT}{2\pi} = 1.56 T \quad (2)$$

From equation (1a) an approximate solution for c can be determined for $a < a_T$ in the following way. After introducing the well-known series for $\tanh u$,

$$\tanh u = u - \frac{2u^3}{3} + \frac{24u^5}{5} - \frac{24 \cdot 17u^7}{7} + \dots,$$

in (1a), which converges for $|u| < \frac{1}{2}\pi$, and after reversing the series of $u \tanh u$ in (1a) in a series of terms of P and by considering three terms of the reversed series, it is found that the approximation for c, c' , is obtained

$$(c')^2 \left(1 + \frac{1}{3}P + \frac{4}{45}P^2\right) = ag \quad (3)$$

This formula has an accuracy of 99% if $u < 1.5$. It appears that after the addition of a term, $0.007 P^3$, a better approximation is obtained, applicable now up to $u \approx 2.7$, with an accuracy of 98%

$$(c')^2 = \frac{ag}{1 + 0.333P + 0.089P^2 + 0.007P^3} \quad (4)$$

Then it follows from (1a) and (4)

$$\frac{c^2}{(c')^2} = \frac{P}{u^2} \left(1 + 0.333P + 0.089P^2 + 0.007P^3\right) \quad (5)$$

The transition depth, a_T , is defined by $a_T = \frac{c'T}{2\pi}$ and $u = 27$ (see (1a)) Then $(c')^2 = 0.37 g a_T$ and $L' = c'T = 2.35 a_T$ (see (4), and $P = 27 \tanh 27 \approx 27$) The line $(c')^2 = 0.37 g a$ is shown in fig 4

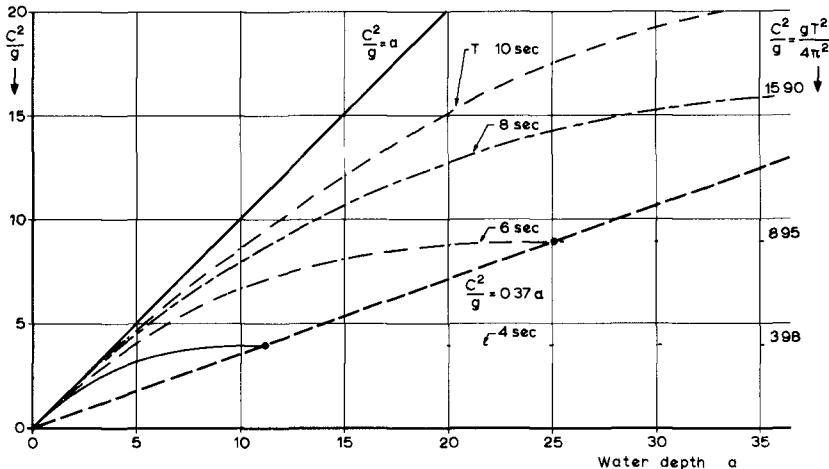


Fig 4 Relation between the celerity of the wave and the depth, according to first order wave theory

In fig 4 the relations (2) and (4) are represented for some periods T. The wave propagates as a deep water wave on the right side of the line, $c^2 = 0.37 a g$. The derivation mentioned above is based on the required accuracy for the determination of c and a_T . The criterion for the boundary between shallow and deep water waves, given in the literature, is $L = 2 a_T$ (see Kindsman (3a))

4.2 The formulae for the computation of the refraction and considerations on the accuracy

Difficulties concerning the accuracy of the computation are often met in the practical applications. The computational method will be discussed in detail in connection with this. During the propagation the wave front will change due to the change of the wave celerity, c .

Let x and y be a perpendicular coordinate system and α the angle of the tangent to the ray at a point (x,y) with the x axis. Furthermore let s be the distance from a fixed point (x_0, y_0) to (x,y) along the ray, and n the distance along the wave front at (x,y) perpendicular to the ray.

The basic formulae for the computation of the wave refraction are

$$\frac{ds}{dt} = c, \quad \frac{\delta\alpha}{\delta t} = -\frac{\delta c}{\delta n}, \quad \text{and} \quad \frac{1}{\rho} = \frac{\delta\alpha}{\delta s} = -\frac{1}{c} \frac{\delta c}{\delta n} \quad (6)$$

in which ρ is the curvature radius

The transformation formulae between the coordinate systems (s, n) and (x, y) are

$$\frac{\delta}{\delta n} = - \sin \alpha \frac{\delta}{\delta x} + \cos \alpha \frac{\delta}{\delta y}, \quad \frac{\delta}{\delta s} = \cos \alpha \frac{\delta}{\delta x} + \sin \alpha \frac{\delta}{\delta y}$$

It follows that α satisfies the first order partial differential equation $\sin \alpha \frac{\delta c}{\delta x} - \cos \alpha \frac{\delta c}{\delta y} = c \cos \alpha \frac{\delta}{\delta x} + c \sin \alpha \frac{\delta}{\delta y} = \frac{c}{\rho}$

Knowing the values of c , $\frac{\delta c}{\delta x}$, $\frac{\delta c}{\delta y}$ to obtain the path of the ray,

$\rho(x, y)$ must be solved in a numerical way

Assume that the path of the ray has been computed up to (x_1, y_1) . Then the point (x_{1+1}, y_{1+1}) of the wave ray can be computed, after time $(\Delta t) = pT$, where $p = 1, 2, \dots$ etc is the number of wave lengths to be considered successively along the wave ray. The following numerical relations are considered

$$\Delta s_1 = c_{1,m} \Delta t, (a), \quad c_{1,m} = \frac{1}{2} (c_1 + c_{1+1}), (b), \quad \Delta \alpha_1 = \left(\frac{1}{p_1} + \frac{1}{p_{1+1}} \right) \frac{\Delta s_1}{2}, (c),$$

$$\frac{c_{1+1}}{p_{1+1}} = \sin(\alpha_1 + \Delta \alpha_1) \frac{\delta c_{1+1}}{\delta x} - \cos(\alpha_1 + \Delta \alpha_1) \frac{\delta c_{1+1}}{\delta y}, (d),$$

$$\Delta x = \Delta s_1 \cos(\alpha_1 + \frac{1}{2} \Delta \alpha_1), \quad \Delta y = \Delta s_1 \sin(\alpha_1 + \frac{1}{2} \Delta \alpha_1), (e), \quad (7)$$

$$\begin{aligned} c_{1+1} &= c_1 + \frac{\delta c_1}{\delta x} \Delta x + \frac{\delta c_1}{\delta y} \Delta y + \frac{1}{2} \frac{\delta^2 c_1}{\delta x^2} (\Delta x)^2 + \frac{\delta^2 c_1}{\delta x \delta y} \Delta x \Delta y + \\ &+ \frac{1}{2} \frac{\delta^2 c_1}{\delta y^2} (\Delta y)^2, (f) \end{aligned}$$

The functions $c_1, \frac{\delta c_1}{\delta x}$ etc occurring in (7, (f)) can be expressed in terms of a , $\frac{\delta a}{\delta x}$, and, $\frac{\delta a}{\delta y}$, by means of (3), if $a < a_T$ (see section 4.1).

For $a > a_T$, $\frac{\delta c_1}{\delta x} = 0$ etc

For $a < a_T$, it may be put

$$\frac{\delta c}{\delta x} = S(a, T) \frac{\delta a}{\delta x} \quad \text{and} \quad \frac{\delta c}{\delta y} = S(a, T) \frac{\delta a}{\delta y} \quad (8)$$

In fig 5 the function $S(a, T)$ is represented as a function of ' a ' for $T = 4, 6$ and 8 seconds. The coefficient S is positive for shallow water. Then $c^2 < 0.37$ ag (see fig 4). For $c^2 = 0.37$ ag, $\frac{\delta c}{\delta x} = 0$ and $S = 0$. In case $c^2 > 0.37$ ag the value of S must be equal to zero. Then the wave is considered as a deep water wave.

The solution of the set of non-linear equations, (7), depends on the variations in the depths, so that the density of the sounding net is a very important factor for the accuracy of the solution. A second important factor is the accuracy of the numerical solution. In this respect great care is necessary.

Assumptions must be made on the values of the derivatives of the celerity, c , and therefore on the depth.

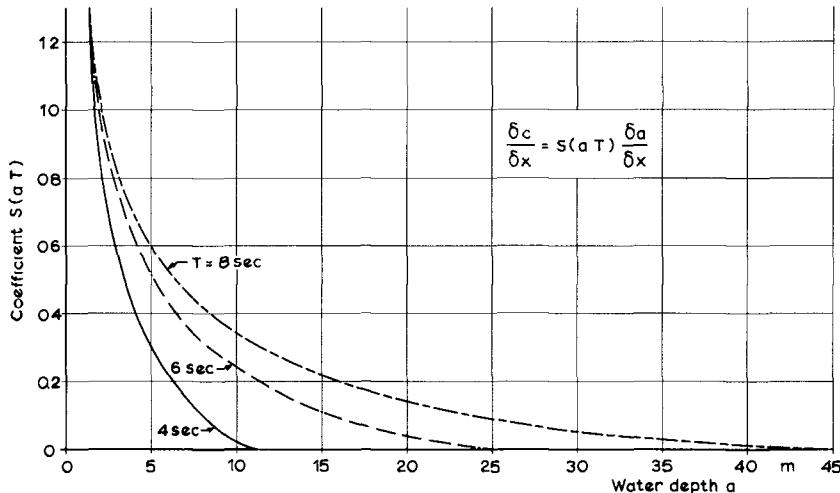


Fig 5 Graphical representation of the coefficient $S(a,T)$ as a function of depth In the practical application is $S(a,T) > 0$ for $c^2 < 0.37$ ag and $S(a,T) = 0$ for $c^2 \geq 0.37$ ag

It is mentioned in the literature that $\frac{\delta a}{\delta x}$, or $\frac{\delta a}{\delta y}$, may not exceed the value, 0.1. A detailed research of the accuracy of the solution shows that this value is too great however for obtaining accurate results. The maximum value, 0.01, is recommended for the slopes $\frac{\delta a}{\delta x}$ and $\frac{\delta a}{\delta y}$.

The following example shows that in case $\frac{\delta a}{\delta x} = \frac{\delta a}{\delta y} = 0.1$, the computed value of Δa_1 is much too large. Let be the value of $\sin(\alpha_1 + \Delta \alpha_1) = 1$. Then it follows from (7 d) and (8) that

$$\frac{c_{1+1}}{\rho_{1+1}} = \frac{\delta c_{1+1}}{\delta x} = S(a,T) \frac{\delta a_{1+1}}{\delta x},$$

and from (7ac), that Δa_1 is of the order of $pT \frac{c}{\rho}$, and $\frac{\delta a}{\delta x} = 0.1$.

If $p = 1$, $T = 5$ sec and the depth is about 2 m, then

$\Delta a_1 \approx 5 \frac{c}{\rho} \approx 5 \frac{\delta a}{\delta x} \approx 0.5$. This value, 0.5, is much too large for an accurate numerical computation of Δa_1 . In the practical applications Δa_1 may not exceed the value 0.05. Otherwise equation (7 c) and the relation $\sin \alpha_1 = \Delta a_1$ etc. are not accurate enough, unless $p < 1$. It is not desirable to consider parts of waves lengths from physical point of view. Therefore the value of p must be an integer.

Moreover restrictions must be made with respect to the values of the second derivatives $\frac{\delta^2 a}{\delta x^2}$ etc. The maximum values of these

second derivatives of 'a' have also to be limited to 0.005/ Δs at the most. In that case the values of the terms which contain the second derivatives in the formula for $\Delta \alpha_1$, e.g. the terms $\frac{1}{2c_{1+1}}(\sin \alpha_1) \frac{\delta^2 c_1}{\delta x^2} \Delta x$ are smaller than 0.025.

The complete formula for $\Delta \alpha_1$, derived from (7 c) and (7 f), is

$$\begin{aligned} \frac{\Delta \alpha_1}{\Delta s} = & \frac{1}{p_1} + \frac{1}{2c_{1+1}} \left\{ \Delta \alpha \left[(\cos \alpha_1) \frac{\delta c_1}{\delta x} + \sin \alpha_1 \frac{\delta c_1}{\delta y} \right] + \right. \\ & + (\Delta x \cos \alpha_1) \frac{\delta^2 c_1}{\delta x^2} - \Delta y \sin \alpha_1 \frac{\delta^2 c_1}{\delta y^2} + (\Delta x \sin \alpha_1 + \Delta y \cos \alpha_1) \frac{\delta^2 c_1}{\delta x \delta y} \left. \right] \\ & + (\Delta x \sin \alpha_1) \frac{\delta^2 c_1}{\delta x^2} - \Delta y \cos \alpha_1 \frac{\delta^2 c_1}{\delta y^2} + (-\Delta x \cos \alpha_1 + \\ & \left. + \Delta y \sin \alpha_1) \frac{\delta^2 c_1}{\delta x \delta y} \right\} \end{aligned} \quad (9)$$

It must be stressed that the accuracy of the refraction computation also depends on the wave length and thus on the period of the wave (see fig 5). The accuracy is greater for shorter waves, and in that case the maximum steepness of the slope of the bottom may be larger than in the case of longer waves.

Finally a remark follows on the practical determination of $\frac{\delta a}{\delta x}$ and $\frac{\delta a}{\delta y}$ etc. They are derived from the contour lines of the depth determined in intervals of 0.5 m. The location of the contour lines is usually irregular. Therefore it is often not possible to determine accurate values for the second derivatives. A grid net for the computation must be constructed on the map. The grid size depends on the accuracy of the contour lines. Various procedures can be followed for the numerical presentation of $\frac{\delta a}{\delta x}$, $\frac{\delta a}{\delta y}$ etc., e.g.

$$2 \Delta x \frac{\delta a}{\delta x} = a_{p+1,q+1} + a_{q+1,q} - a_{p,q} - a_{p,q+1},$$

and a similar equation for $\frac{\delta a}{\delta y}$.

It is noted that the points (x_1, y_1) on the wave ray will usually not coincide with the grid points (p, q) of the grid net. The study of the accuracy of the refraction computation will be continued.

5 Considerations on studies of morphological changes

5 1 General remarks

This section deals with the methods to be applied for the determination of the morphological changes along the Delta coast. It was not possible to make satisfactory scientific forecast for these changes because of the uncertainty in the values of the various coefficients in the equations of the phenomena. It is intended that after the closing of the Haringvliet these coefficients will be determined based on extensive measurements in the new circumstances that will prevail along the coast. The sediment transport in the Delta estuaries and coastal regions occurs mainly in suspended form and bed load sediment is negligible. This is the reason that a hydraulic model has not been built for this research. In the preceding sections the basic factors influencing morphological studies have been mentioned. These factors are better defined from a physical and mathematical point of view than are the morphological changes. In this respect distinction must be made between the final shape of the seabed that will result when conditions have stabilized and the speed at which the change takes place. In particular this speed can be approached more theoretically. It is determined by the net sedimentation and erosion and is therefore related to the sand transport. However, simplifications must be accepted, and empirical quantities introduced which have to be evaluated by means of measurements in nature. Consequently only tendencies for the morphological changes can be given. Svasek has discussed these problems for the Delta area in his publications (10) and likewise Terwindt (11).

The concentration of the sand is determined by the currents in combination with the wave motion. The concentration can increase considerably by the wave action, because of the increased turbulence of the water, especially in the case of breaking waves. Tidal currents and wind waves vary with astronomical conditions and meteorological circumstances, especially during stormsurges. Reversely the tidal currents and the wave motion which is statistically distributed, are also affected by the bottom morphology.

The instruments for the measurement of sand in suspension that are currently in use are not accurate enough for all circumstances. Their application is very limited and is only possible during quiet weather. By means of radio-active tracers, the direction and the relative significance of the sand transport can be determined as a mean value over longer time periods. Quantitative evaluation of the sand transport directly related to the fluid motion, and of the net transport, still remains uncertain.

The most important tool for the determination of the net-sand transport is the study of the changes in the shape of the bottom, as obtained from periodically repeated soundings, combined with a knowledge about tidal currents and wave-action. The results of the soundings also give information for the estimation of the final situation of the bottom morphology. Along the Delta coast the soundings are repeated every three months, or at shorter intervals in regions where sand movement is considerable. The study of de Vries (12), on the applicability of tracer techniques for rivers must be mentioned. A discussion on sand transport processes is also given.

5.2 An empirical method for the determination of a cross-section area in the mouth of an enclosed estuary

An important problem is the estimation of the final cross-section area in the mouths of the enclosed estuaries, e.g. in the mouth of the Haringvliet, where the sluice is about 5 km inland and a gully must be maintained for the discharge of the sluices. It has been found that a relation exists between the quotient of the tidal volume (ebb + flood) and the total profile area of the inlet mouth for the various estuaries of the Delta region. This amount divided by the tidal period ($T = 44,700$ sec) determines a velocity \bar{v} over the tide, that appears to have a relation with the morphological conditions. Its value is 55 cm/sec for all the estuaries. The diameter of the sand is of the same order of magnitude for all the mouths, 0.15 mm to 0.2 mm.

It must be expected that in the future silt will be deposited in the mouth of Haringvliet and consequently the value of the scouring velocity will become somewhat higher. By means of laboratory tests it has been estimated as about 70 cm/sec in nature.

The future discharges in the mouth can be determined from the discharges of the sluices and from tidal calculations. Then the future cross-section area can be computed. The time scale of the development must be determined from detailed knowledge about the sand transport and from comparative examples in nature e.g. from the morphological changes after the closure of previous sea arms like Brielse Maas (1951) (see fig 6) and Veerse Gat in the Delta region of which the mean grain-size at the bottom is the same.

The mean value of the depth in the future gullies can also be estimated in an empirical way by comparison with these examples. Then the same relation between mean and maximum depth can be applied to the future gullies of the Haringvliet mouth, provided the widths of the gullies are fixed by banks. The preceding studies for the determination of the final cross-section have been applied by Terwindt in his morphological study of the Haringvliet mouth. Reference is also made to Bruun and Gerritsen (13).

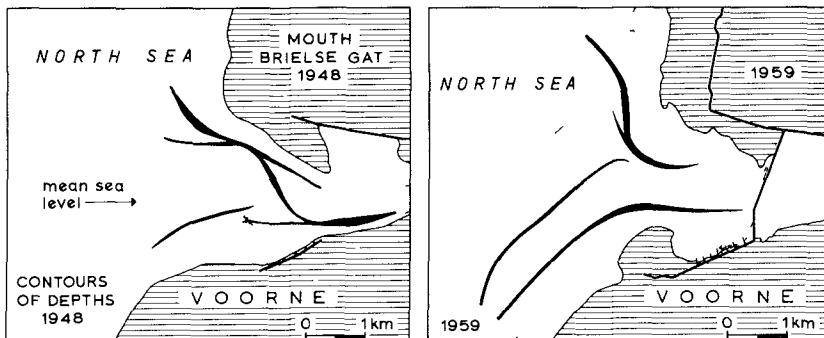


Fig. 6 Changes in the contours of depths due to the enclosing of the Brielse Maas

Extensive shoals have been formed with gullies at both sides after the enclosing of this sea-arm

5 3 A more theoretical method for the determination of the sand transport in a river

For the computation of the sediment transport due to the tidal currents in the Delta region, various sediment transport formulae can be used. They are however generally based on laboratory tests, trials in nature are limited. Various bed-load formulae are summarized in the book of A J Raudkivi (14). The formulae for the computation of suspended sediment movement, which is related to diffusion processes, are still more limited. The vertical distribution of the velocity in turbulent water is one of the determining factors, however the formulae for such distributions in nature are also more or less based on an empirical basis.

The various quantities which are defined in the formulae under idealized circumstances must be redefined for the more irregular conditions in nature in an empirical way by adapting the formulae to the observations obtained from measurements in nature. After that the formula can be applied for different conditions and engineering purposes e.g. higher upland river discharges, or changes in the profiles of the river. Then the formula may give indications for changes in scour and sedimentation. The method has been applied by Morra (15) in the tidal region of the Delta, where the sand is fine, 0.15 mm to 0.2 mm. The calculation of the suspended sand transport is based on a formula for the distribution of the suspended sand concentration in the vertical, combined with the formula of Kalinske (16). The major feature of the method will be described.

The exchange of sand over the vertical is determined by the equilibrium condition per unit of width,

$$cw = \epsilon \frac{dw}{dh}, \quad (1)$$

in which w is the local sediment concentration (the volume of sand per unit of volume water), h is the height above the bottom, c is the fall velocity of particles in the fluid at rest, and ϵ is an exchange coefficient. It is assumed to be equal to the momentum transfer coefficient in the relation for the shear stress τ ,

$\tau = \rho \epsilon \frac{dv}{dh}$, in which v is the velocity at height h , determined according to the parabolic formula

$$v = v_1 h^{1/q}, \quad (2)$$

and ρ is the density, q is of the order 5 to 7. Then

$$\frac{dv}{dh} = \frac{1}{q} v_1 h^{(1-q)/q} \quad (3)$$

It is well-known that the shear stress τ at the height h is defined by $\tau = \rho g a (1-z)$, in which $z = h/a$, a is the total depth, and i is the slope of the water surface in the case of steady flow. In the case of unsteady flow it is the difference in head, determined by the friction term, $i = v_m^2/C^2 a$, (4), in which v_m is the mean velocity at time t in a cross-section, and C is the coefficient of Chezy. After integration and some calculation it is found from (1) and (3) that the vertical distribution of the sand concentration is

$$\frac{w_z}{w_{z_0}} \approx \left[\frac{(1-z)^{z^{1/q}} e^{-(q+1)z^{1/q}}}{(1-z_0)^{z_0^{1/q}} e^{-(q+1)z_0^{1/q}}} \right]^{p_c}, \quad (5)$$

in which $z_0 = h_0/a$, and h_0 is a reference height above the bottom. For $h_0 = 0$, the formula is

$$\frac{w_z}{w_0} = \left[(1-z)^{z^{1/q}} e^{-(q+1)z^{1/q}} \right]^{p_c}, \quad (6)$$

in which $p_c = \frac{c C^2}{g q v_m}$, v_m is the mean velocity over the vertical, defined by $\frac{q}{q+1} v_1 a^{1/q}$ (see (4)), and w_0 is the suspended sand concentration near the bottom.

The formula of Kalinske determines the relation between w_0 and the bottom material. A brief explanation of this formula follows. Let v' be the velocity component in the vertical at the height h above the bottom and c the fall velocity in still water of a particle, and let the statistical distribution of v' be determined by the formula of Gausz. Then the formula

$$\Delta F(c) \int_c^\infty (v' - c) \frac{1}{2((v')^2)^{\frac{1}{2}}} \exp \frac{-v'^2}{2((v')^2)^{\frac{1}{2}}} dv', \quad (7)$$

in which $\Delta F(c)$ is an interval with equal diameters of bottom material with fall velocity c (expressed in percentages), determines the transport of sand from the bottom into suspension in course of time. The horizontal velocity component may not change considerably.

In the equilibrium situation the transport in vertical direction per unit of time equals the transport downwards, $c w_0$.

It may be put for turbulent flow $((v')^2)^{\frac{1}{2}} = k (g a l)^{\frac{1}{2}}$, (8) in which k is an empirical constant. The transport of sand from the bottom, $c w_0$, must be proportional to the expression (7). After the transformation, $v' = k(2g a l)^{\frac{1}{2}} u$, it is found

$$c w_0 = A c \Delta F(c) \int_{u(c)}^\infty \left(\frac{u}{u(c)} - 1 \right) e^{-\frac{u^2}{2}} du, \quad (9)$$

in which A is the proportional factor and $u(c) = c/k(2g a l)^{\frac{1}{2}}$. The equation (9) may be rewritten in the form

$$\frac{w_0}{\Delta F(c)} = \frac{A}{(2\pi)^{\frac{1}{2}}} \left\{ \frac{1}{2 u(c)} e^{-\frac{u^2(c)}{2}} - \frac{1}{2} (1 - \frac{2}{\pi^{\frac{1}{2}}} \int_0^{u(c)} e^{-\frac{u^2}{2}} du) \right\}, \quad (10)$$

A and k are empirical constants in (8) and (10). They depend on the local circumstances.

The following remark must be made for the application of the equation (10) in a tidal river. At a location in a tidal region the value of u , which is determined by the friction term is usually different from the slope of the water surface caused by the tide at time t . This slope equals the algebraic sum of the friction term

and the inertia terms

It must be assumed in the practical applications that the changes of the tidal velocities are small enough, so that the equilibrium condition of the sand transport, defined above, is satisfied more or less

The bottom material can be considered as homogeneous in tidal relations with a standard grain diameter of d_{60} for this special purpose. In that case $\Delta F(c) = 100$. The practical determination of the empirical parameter, p_c , in formulae (5), and (6) is as follows. The relation between w_z and w_{z_0} is found from the measurements of the sand concentration w_z in the vertical, and the exponent q of the parabolic formula (2) is determined from velocity measurements. Then the exponent p_c in formula (5) can be computed. The value of p_c varies in the tidal regions of the Delta area between 0.25 and 2.5.

It is evident that due to the great variations of the sand transport in the course of time, a great number of measurements must be taken. Then the least square rule can be applied for the best fit of the sand concentration in the vertical.

After that w_0 can be calculated from (6) and introduced into the left member of equation (10). The values of A and k can be determined such that equation (10) is satisfied. A trial and error procedure is usually necessary. It is difficult to obtain reliable values for A and k .

It is also possible to determine A and k from the results of the measurements mentioned in Kalinske (10). However Kalinske applies the logarithmic formula for the vertical velocity distribution, and ϵ_m , being the mean value for the exchange coefficient over the vertical. Kalinske found by this procedure $A \approx 39$ and $k \approx 0.75$.

5.4 Sand transport in the coastal area

The morphological conditions at the coast of the Delta region will change in the future as a consequence of the enclosing of the Haringvliet and the works which are in execution. The future situation depends on the new equilibrium of the sand transports, due to the combined effect of currents and waves. The new equilibrium in the channels, and seaward of the breaker zone is determined considerably by the changes in the tidal currents and in particular by the velocity components towards the coast line. A complicated sand transport process exists in the region of the sand banks, in front of the coast line between the existing gullies. Here the sand transport depends on the combined effect of currents and waves. Breaking waves produce very high sand concentrations in these regions. The combined effect has been studied by Bijker (17) on the basis of laboratory tests. He determines the combined shear stress at the bottom of currents and waves in the two dimensional case. Vertical velocity components and accelerations are not considered in his study.

In the breaker zone where tidal currents are negligible a well-known procedure is to assume that the total long-shore transport rate is proportional to the loss of long-shore energy flux per unit of length. Reference is made to Bijker and Svašek (18).

The basis of all the morphological studies are the tidal and refraction computations which determine the changes in the velocity

components and the directions of the waves. These morphological studies are continued. An example of the changes in the coastal area is shown in fig 7. Some contours of depths in 1960 and 1969 are represented. It appears that the 2 and 5 m lines near the coast changed considerably, due to the wave action and the changes in the currents.

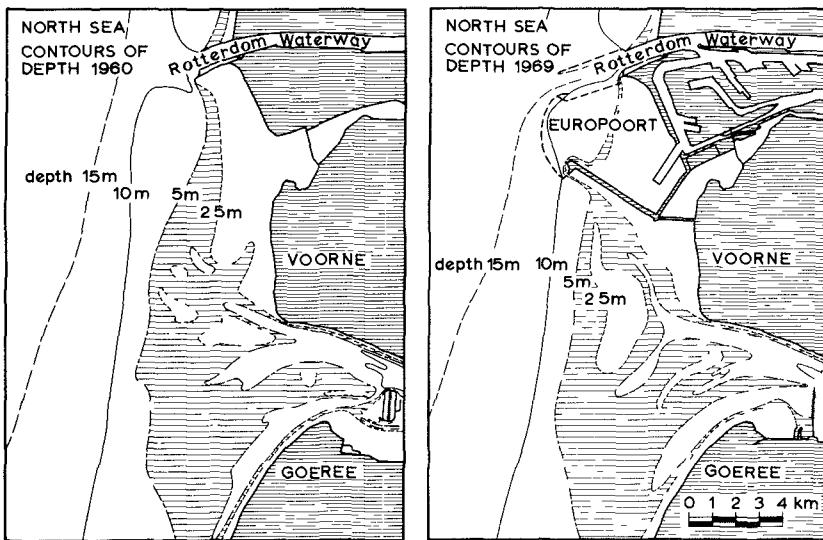


Fig 7 A comparison between the contours of depths in 1960 and 1969 in the mouth of Haringvliet and near Europoort

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