INFLUENCE OF STORAGE AREAS ON TIDAL WAVE PROPAGATION

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Preface

The study described in this report forms part of the requirements for the Degree of Civil Engineer of the Delft University of Technology.

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Frank R.Rijsberman.

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I.1. Introduction

In river engineering practice it is often required to set up a mathematical model for (part of) a river. A standard computer programme for one-dimensional unsteady flow is often available. The input data for the model are obtained through measurements in the prototype.

It is also necessary to give values for the roughness coefficient. And because this parameter can not be measured the model must always be calibrated.

Difficulties can arise in tidal lowlands where the river banks are very densely vegetated (in tropical areas for instance). In these circumstances it is often much too expensive and time-consuming to acquire accurate measurements of the storage areas outside the flow area. Such extra storage areas can for instance consist of mangrove forest or tidal swamps.

These storage areas influence the tidal wave propagation and for a good representation of the flow it is therefore essential to know the storage areas accurately.

When the storage areas can not be measured the model has to be calibrated on both the roughness coefficient and the extra storage areas. This is a very complex problem and it often proves impossible to obtain an acceptable representation of the flow.

A systematic approach for the calibration would be desirable. But before such an approach can be designed it is necessary to know more about the influence of the storage areas outside the flow area on the propagation of the tidal wave (water levels and discharges).

The aim of this study is to provide information on this subject which could be used to design a more systematic calibration method. Such a method, in turn, should give an indication on the sort and number of measurements that are required to carry out a proper calibration.

I.2 Summary

Choice of the test problem (I.4)

In situations when there are extra storage areas outside the flow area in combination with a dense vegetation these storage areas are not sufficiently measured in practice. This is not done because it would take too much time and money compared with the other measurements.

It was therefore necessary to use synthetic measurements and an artificial river channel for this study. The geometry of the channel is based on the river Mesuji in South-Sumatra (Indonesia).

Criteria (I.5)

The criteria which are used to compare the computations are the differences in phase-lag and amplitude of waterlevel and discharge at several locations in the model.

Composition of the reference computation (II)

In chapter II the development of the reference computation is shown. Several boundary conditions and slightly different geometries are tried to obtain a well interpretable model with realistic dimensions.

Influence of storage area variations (III)

The computations with extra storage areas at different locations and of different magnitude are presented and interpreted in this chapter.

The location of the extra storage area (far from or close to the rivermouth) appears to determine the influence on the tidal wave propagation.

To demonstrate this, first a computation with extra storage area close to the rivermouth and next a computation with extra storage area far from the rivermouth are presented and evaluated in paragraphs III.2 and III.3 respectively.

Furthermore these two (and a few other) computations are evaluated quantitatively in III.4. The influence of the upland discharge on the tidal wave propagation in relation with extra storage areas is considered in III.5

Using the results of III.2 up to and including III.5 some implications for field surveys are deduced in III.6. A procedure is proposed for data collection to set up a numerical model for (part of) a river in tidal lowlands.

I.3 Conclusions

Of course the conclusions are based on the considered river, and especially the percentages given in conclusion 4 can not be used for other rivers. Still it is believed that the trends found here will also hold for other rivers.

1. Location of the extra storage

The location of the extra storage areas along the x-axis determines the influence on the tidal wave propagation.

1.1 Close to the rivermouth (km 32-64):

- the waterlevel amplitudes are unchanged as could be expected.
- the discharge amplitudes are increased downstream of the extra storage area and decreased upstream.
- the waterlevel phase-lags are increased, the increase occurs just downstream of, and in the widened section.
- the discharge phase-lags are increased, the main increase occurs in, and upstream of the widened section.

1.2 Far from the rivermouth (km 96-128):

- the waterlevel amplitudes are damped throughout the model.
- the discharge amplitudes are increased just downstream of the widened section, but decreased close to the rivermouth and upstream of the widened section.
- the waterlevel phase-lags are slightly decreased close to the rivermouth and increased just downstream of, and in the widened section.
- the discharge phase-lags are increased, the main increase occurs in, and upstream of the widened section.

2. Amount of extra storage

An increase of the amount of extra storage does not change the influence on the wave propagation, only the magnitude of the influence is increased.

3. Upland discharge

A modest upland discharge does not change the influence of extra storge on the wave propagation, even the magnitude of this influence remains the same.

4. Percentage-wise

A local extra storage area outside the flow area of some 10% of the total storage area of the four sections (km 0-128) can result in:

- a waterlevel amplitude damping up to 15%.
- a discharge amplitude increase/decrease up to 10%.
- a phase-lag difference of up to half an hour.

5. Calibration strategy

To develop a calibration strategy with respect to both the roughness coefficient and storage areas it would be recommendable to carry out further research concerning the relation between the terms in the equation of motion.

It is thought to be sufficiently clear that the occurrence of extra storage outside the flow area is an important factor. Especially in relation to the discharge measurements.

I.4 Choice of the test problem

For this study one should like to use a situation where the intended problem (extra storage areas outside the flow area) arises but where all parameters are still accurately known, through an intensive measuring campaign. In this ideal location the measured flow and geometry could serve as a reference computation.

The requirements for such an ideal test problem are:

1. Flow area of the cross-profiles as a function of waterlevel (=time) and place.

2. Storage areas outside the flow area as a function of time and place.

3. Roughness coefficients as a function of place.

4. Boundary conditions at the upstream and downstream boundary (and possible internal boundaries).

5. Waterlevels and discharges at several points in the model for a few different periods (springtide, neaptide) for calibration and verification of the model.

A test problem that meets all these requirements could not be found. This was to be expected because in those cases where the problem occurs it is too expensive to accurately measure the extra storage areas. It is therefore necessary to use synthetic measurements and an artificial river channel.

There are two possibilities then. A situation can be selected where through an hydraulic survey most parameters are known. The unknown parameters are estimated and the resulting model is calibrated more or less, using extra waterlevel and or discharge measurements. Such a model could be used in this study as a reference computation.

The other possibility is to use a simple, prismatic channel, not based on prototype measurements. A sine function could be used as a downstream boundary condition for the waterlevel.

The advantage of the first possibility is that the dimensions of geometry and flow are realistic. The advantage of the second possibility is that the results will be more easy interpretable.

The first possibility is chosen here. But after a few trial runs the channel was adapted to a combination of both possibilities. This is described more detailed in chapter II.

I.5 Criteria

Criteria are needed to evaluate the differences between the computations. The influence of the storage areas on the propagation of the tidal wave has to be quantified somehow.

For special purposes, such as for instance an irrigation inlet, there are obvious criteria. For the irrigation inlet a criterion could be the time (per day) that the waterlevel at the inlet exceeds a certain level.

In this case, where the main interest is the wave-propagation in general, the logical criteria are amplitude and phase-lag of the waterlevels and the discharges. This results in 4 criteria per location.

The amplitude is defined here as the difference between the maximum and minimum waterlevel (discharge) in a tidal cycle. The phaselag at km x is defined as the time-difference between the zero passage of the waterlevel (discharge) at the rivermouth (km 0) and km x.

Next to this quantitative analysis, the computations will also be analysed qualitatively, using graphs.

I.6 Computer program FLOWS

FLOWS is a subsystem of ICES, the Integrated Civil Engineering System, for applications in the field of hydraulic engineering (see: Booy, 1978). It is able to compute unsteady flows in hydraulic networks of given dimensions.

FLOWS is used to compute the flow in this study because of its flexibility. Hydraulic networks can be build from a choice of elements like river and canal branches, pipes, culverts, reservoirs etc.. A number of boundary conditions are possible like constant waterlevels or discharges, data-sets and periodic boundaries.

The program uses an implicit scheme because explicit schemes have the disadvantage that a rather strict limit is imposed on the time step, in order to maintain stability. This is especially important for "slow" processes, like flood routing, more than for tidal computations. But FLOWS has to be used for such "slow" processes too.

FLOWS essentially gives a numerical solution for the nonlinear partial differential equations describing the waterflow. These are the equation of motion for long waves in open channels and the continuity equation. They can be written like:

$$\partial Q/\partial t + \partial(Qv)/\partial x + gA \cdot \partial h/\partial x + J(Q,h) = 0 \quad \text{and}$$
$$\partial As/\partial t + \partial Q/\partial x = 0$$

In which Q indicates discharge, V velocity, A the area of the flow cross-section, As the total wetted cross section, and J is the slope due to friction. To solve the equations numerically they are discretized. The discharge and waterlevel in each branch of the hydraulic network are characterized by their values at both ends of the branch.

The discharge values for the branch numbered m, are Q1,m and Q2,m, the waterlevels h1,m and h2,m. To indicate the different time levels a superscript is introduced, + represents time $t+\Delta t$ and - represents time t.

A weighing factor   is used to influence numerical damping, when needed The partial derivatives of some variable $\emptyset$ are approximated by:

$$\partial\emptyset/\partial t = (\overset{+}{\emptyset}_1 + \overset{+}{\emptyset}_2 - \overset{-}{\emptyset}_1 - \overset{-}{\emptyset}_2)/2\Delta t$$

$$\partial\emptyset/\partial x = \theta\ (\overset{+}{\emptyset}_2 - \overset{+}{\emptyset}_1)/\Delta x + (1-\theta)\ (\overset{-}{\emptyset}_2 - \overset{-}{\emptyset}_1)/\Delta x$$

and a function of $\emptyset$ by:

$$F(\emptyset) = F(\emptyset((x1 + x2)/2,\ t + \theta\Delta t)) =$$

$$= 1/2\ [F(\overset{-}{\emptyset}_1) + F(\overset{-}{\emptyset}_2) + \theta(\partial F/\partial\emptyset)_1 \cdot (\overset{+}{\emptyset}_1 - \overset{-}{\emptyset}_1) + \theta(\partial F/\partial\emptyset)_2 \cdot (\overset{+}{\emptyset}_2 - \overset{-}{\emptyset}_2)]$$

After substituting these approximations into the differential equations one obtains respectively:

$$\frac{\overset{+}{Q}_1 + \overset{+}{Q}_2 - \overset{-}{Q}_1 - \overset{-}{Q}_2}{2\Delta t} + \frac{\overset{-}{V}_2\overset{+}{Q}_2 - \overset{-}{V}_1\overset{+}{Q}_1}{\Delta x} +$$

$$\frac{g(\overline{A1} + \overline{A2})}{2\Delta x} \cdot [\theta\overset{+}{h}_2 - \theta\overset{+}{h}_1 + (1-\theta)(\overset{-}{h}_2 - \overset{-}{h}_1)] +$$

$$1/2\ [J(\overset{-}{Q}_1, \overset{-}{h}_1) + \theta\frac{\partial J}{\partial Q}(\overset{+}{Q}_1 - \overset{-}{Q}_1) + \theta\frac{\partial J}{\partial h}(\overset{+}{h}_1 - \overset{-}{h}_1) +$$

$$+ J(\overset{-}{Q}_2, \overset{-}{h}_2) + \theta\frac{\partial J}{\partial Q}(\overset{+}{Q}_2 - \overset{-}{Q}_2) + \theta\frac{\partial J}{\partial h}(\overset{+}{h}_2 - \overset{-}{h}_2)] = 0$$

and $$\frac{\overset{*}{As1} + \overset{++}{B_1 h_1} + \overset{*}{As2} + \overset{++}{B_2 h_2} - \overset{-}{As1} - \overset{-}{As2}}{2\Delta t} +$$

$$+ \theta\frac{\overset{+}{Q}_2 - \overset{+}{Q}_1}{\Delta x} + (1-\theta)\cdot\frac{\overset{-}{Q}_2 - \overset{-}{Q}_1}{\Delta x} = 0$$

is dat zo als B niet constant is?

Here $As_1^* = As_1^+ - B_1^+ h_1^-$. The scheme is exactly conservative, apart from rounding errors. These equations (2 per branch), plus the continuity equation at the nodes ($\Sigma Q = 0$, $h = h$), and the boundary conditions, form a system of n equations with n unknowns. This system is solved simultaneously through elimination.

Because the system uses an implicit scheme, no instabilities occur. The only limit on the time and space step is then given by the desired accuracy. There are, of course, several sources of inaccuracy, such as the schematization, inaccuracy of measured data, numerical errors etc.. The numerical errors can be limited through limitation of the time and space step.

It appears to be too difficult to find reasonable estimates for the error in the full equations, therefore a trial and error solution is used here. For a first computation a "reasonable" time and space step are estimated (see below). The accuracy of this computation is then checked with another computation, using 0.5 times the space and time step of the first computation. Because of the smaller time and space steps the second computation should be more accurate than the first. When the differences between computation one and two appear to be small, it can be assumed that the values will also not change much for even more accurate computations. When the differences are unacceptably large, smaller time steps should be tried.

The time and space steps for the first computation were in this case estimated with a little help of the Courant-number. The Courant number is a stability criterium for explicit schemes, but it is also an indication for the accuracy when implicit schemes are used. When the Courant number is much larger than 1 the accuracy of the implicit scheme decreases. The Courant number can be written as:

$$C = c \cdot \Delta t / \Delta x$$

in which c is the characteristic celerity.

This characteristic celerity can, for long water waves, be approximated by $\sqrt{g.a}$ (a = waterdepth). Thus, for a constant time and space step, the point in the model with the largest depth determines the largest Courant number.

welke, nog niet genoemd

In this case it is convenient to have a space step of 8 km, because a cross-section profile is available every 8 km. The largest depth in the model is about 10 m, so $c \approx 10$m/s. With a time step of half an hour, the Courant number is:

$$C \approx 10. \frac{1800}{8000} \approx 2.2$$

This seems acceptable. A computation was carried out with Δx = 8 km and Δt = 1800 s, in the second computation Δx = 4 km and Δt = 900 s. The computations were carried out for what is called the "reference computation" in paragraph II.2.

The differences appeared to be acceptably small. It has to be taken into account that the "real" model has a length of 128 km, plus 400 km to obtain a boundary outside the tidal influence (see II.2). The differences in waterlevel are smaller than 2 to 3 mm in the first 144 km of the model and grow in the next 100 km to about 20 mm. The differences in discharge are smaller than about 7 m^3/s in the first 250 km of the model.

dat is nogal wat, oorzaak?

It was concluded that a space step of 8 km and a time step of 1800 s render sufficiently accurate results.

The boundary conditions are described in Chapter II. The correct initial conditions are unknown. Therefore the waterlevels and discharges are set equal to zero at t= 0. After computation during a few periods the waterlevels and discharges become independent of the initial conditions and they are only influenced by the boundary conditions. Two tidal periods appeared to be enough to attain this situation. The third period could be used.

The standard output of the FLOWS system is a printing of the water-levels and discharges at all the nodes of the model, after every time step or a required time interval. Furthermore a rough sketch of some specific waterlevels and discharges as a function of time can be produced, printed at the line printer.

The complete input and output of the reference computation and several other characteristic computations is available in the library of the Fluid Mechanics Group of the Department of Civil Engineering.

II Model and reference computation

II.1 Fully prototype based

In first instance the lower reaches of the river Mesuji, in South Sumatra, Indonesia (fig. 1) were chosen for the test problem. A large part of the East side of Sumatra are swampy lowlands, the Mesuji flows through such an area. In this area the riverbanks are very densely overgrown. The vegetation is often so dense that it is difficult to see where the river stops and the banks start. Because these lowlands are very flat, it is possible that large areas are flooded at high tide (and emptying at low tide), thus influencing the wave propagation.

Because irrigation projects are planned here, a hydraulic survey of the lower reaches of the Mesuji was carried out in 1979 (Main hydraulic survey Mesuji river 1979). Cross-profiles were sounded (using an echo sounder), waterlevels and discharges were measured and a few existant bench marks were leveled to tie the measured waterlevels to a reference level. Because of the bank-vegetation and the lack of manpower it could not be established if and where there were storage areas outside the sounded cross-profiles (in fact only the flow areas of the cross-profiles).

Another difficulty for a mathematical model is that at the upper boundary (130 km upstream of the river mouth) the tidal amplitude is still about 80% of the amplitude at the mouth. It is rather difficult then, to formulate an upper boundary condition for the mathematical model. For the calibration of the model the measured discharges or waterlevels at the upper boundary can be used. But when the model is to be used to predict the water flow when the storage areas in the model are changed, the use of measured values is not possible any more. The change in storage areas will also influence the flow at the upper boundary.

To overcome this problem it was first tried to simulate the upper reaches of the river with a single resistance and a single storage element. When the inertia forces are small enough, the dimensions of this resistance and storage can be approximated from the calibration computation.

Computation 1

The reference level in the model is mean sea level.

The river channel is schematized by the cross-profiles measured in the survey (see Main hydraulic survey Mesuji river, 1979), one profile every eight kilometers. Between these profiles linear interpolation is used. In the reference computations the storage area is equal to the flow area. The storage areas outside the flow area are taken zero for this computation and will be enlarged in later computations.

Because the river banks are steep in the tidal range, the cross-profile can be schematized with vertical banks (see fig. 1A), a stream width and a flow area and hydraulic radius below the

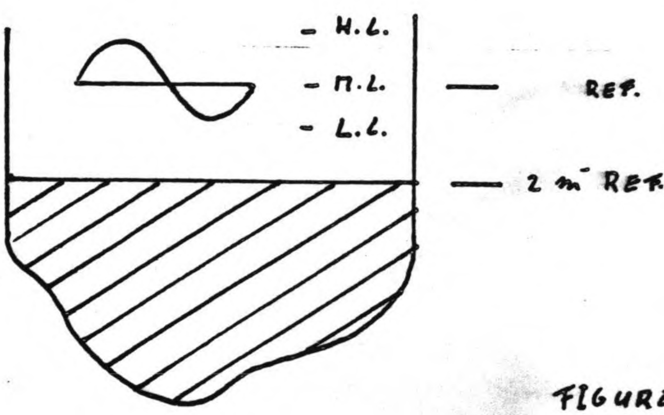

FIGURE 1A

lowest expected level (lined part). This level is chosen 2 m below the reference level. The program computes the total flow area and total hydraulic radius for every timestep from these data. In figure 2 the stream width and flow area and hydraulic radius below REF.-2m are given along the river axis.

From this figure 2 it can be seen that over the first thirty kilometers measured from the river mouth the width decreases sharply and the depth (hydraulic radius) increases (in upstream direction, see fig. 2A). In this part of the river a mud bank is situated. Due to increased salinity the suspended sediment flocculates and the depth decreases. The flow area is more or less constant because of the increased width.

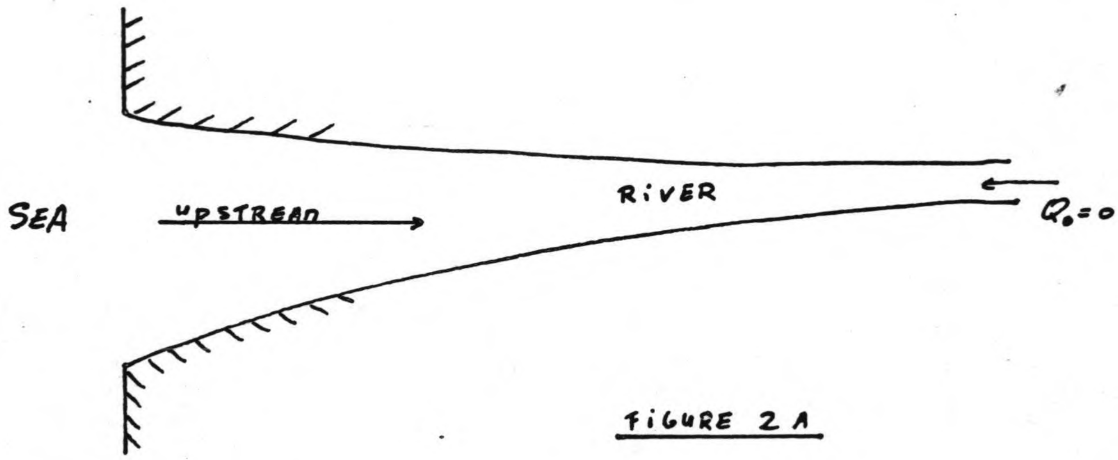

FIGURE 2 A

From 30 km up to 80 - 90 km upstream the depth, width and flow area are more or less constant. From 80 - 90 km upstream the width (and thus flow area) decreases further. The depth decreases from about 120 km upstream. This is not obvious from figure 2 but becomes clear from the survey report.

The upland discharge is taken zero for simplicity in this computation. Its influence will be investigated in later computations. This zero upland discharge will not have been true in reality, but the real upland discharge was small compared to the tidal discharges in these lower reaches (it was a dry season survey).

For the downstream boundary condition the measured waterlevels of October 6, 4 a.m. until October 7, 4a.m. were used. The results of a tidal analysis of the waterlevels at the mouth of the Mesuji show that both diurnal (K1 and O1) and semi-diurnal (M2 and S2) constituents are important.

As a consequence, around springtide the tide is about diurnal but around neaptide the tide is almost semi-diurnal with an important daily inequality (between the two high and the two low tides). It is therefore difficult to use a periodic boundary value. This specific period is chosen because it is almost periodic. It is smoothed so that it is periodic (see fig. 3).

The upstream boundary is chosen below the important branch at 132 km from the rivermouth. The schematized part of the Mesuji is 128 km in length. At this upstream boundary no discharge measurements are available, the measured waterlevels (of the same period as the boundary downstream) are used. They are also smoothed (a little) for periodicity (see fig. 3).

The roughness coefficients for this computation are chosen, not calibrated. Because of the mud the Chézy-roughness value of the downstream part ofmodel is chosen 60 $m^{1/2}/s$ rather arbitrarily. For the upstream part of the model a C-value of 50 $m^{1/2}/s$ is used.

Because all levels and discharges taken zero for t=0 it is needed to compute a few periods to obtain a flow, independent of the intitial conditions in the model. After these periods, the waterlevels at 0, 40, 88 and 128 km from the mouth are depicted in figure 4. The discharges at 0, 64 and 128 km from the rivermouth are given in figure 5.

Boundary condition at the upstream boundary

Using the waterlevels and discharges in the final section at the end of the model it is now tried to dimension a resistance and storage that would reproduce (approximately) the just computed levels and discharges when they were supplemented to the model at the upstream boundary. In this schematization of the upper reaches of the river, the inertia term and convective term of the equation of motion are neglected, the other terms are discretized. The storage element can be visualized as a reservoir, the resistance element as a short, narrow culvert.

With this storage/resistance relation boundary, also called "black-box boundary" (see: PENPAS Users Manual, 1979), the model should reproduce local discharges correctly when the waterlevels at the boundary are correct. The effect of this is that to determine the dimensions of the culvert and reservoir the measured waterlevels can be used as a boundary condition. These measured waterlevels will be called ho, the waterlevel just outside the black box, see fig. 3A.

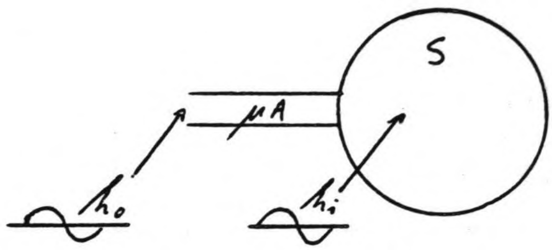

FIGURE 3A

The resistance, the culvert, is in this case characterized by its A value, where A is the flow area and is a discharge coefficient. The storage can be characterized by the storage area S as a function of the waterlevels inside the black box, hi. The discharge through the culvert is Qc.

Since the river part upstream of the boundary is now represented as a single basin filled and emptied by a culvert, a few things can be said about the waterlevel inside the "black box".

1. When Qc=0 $\partial hi/\partial t = 0$ ($\partial hi/\partial Qc = 0$)
2. When Qc=0 ho=hi
3. When Qc is extreme $\partial Qc/\partial hi = 0$

Because ho and Qc (discharge in last section) are known, a ho - Qc curve can be drawn. The curve hi - Qc is unknown. This curve can be estimated to find μA from the equation of motion for the culvert

$$Qc = \mu A \sqrt{2g\,(ho - hi)}$$

and S from the equation of continuity

$$Qc = S \cdot \partial hi/\partial t$$

The method will be illustrated through the computation.

First the ho - Qc curve is drawn, see fig. 7. And here appear the first difficulties. Because in this computation there are more or less two, inequal, periods in one tidal cycle, two curves in one appear. In fact, the model is obviously too simple for this case. Still, more or less out of curiousity, the dimensioning of the black box is carried out as intended, using the outer curve.

Where Qc = 0, hi = ho and the tangent line at the hi - Qc curve is horizontal. Where Qc = maximum the tangent line is vertical. When the curve is symmetrical in the long axis (Qc - axis) a hi - Qc curve can be sketched. Of course the curve is not necessarily symmetrical but this is one way to find an estimate for μA. Afterwards the storage will be approximated in a somewhat more accurate way.

A hoeft niet constant te zijn. (afh. waterdiepte)

Using the two curves an estimate of μA can be found from the relation

$$Qc = +\mu A \sqrt{2g|ho - hi|} \quad ; \quad \mu A = |Qc| \, / \, \sqrt{2g|ho - hi|}$$

?

In this case any value between zero and about 500 m^2 can be found. A value of 200 m^2 is arbitrarily chosen.

Having a value for μA the "real" value for hi can be calculated from ho and Qc, next $\partial hi/\partial t$ and further S from

zelfde voor S

$$Qc = S(\partial hi/\partial t)$$

for a number of different $\partial hi/\partial t$. These S-values can be plotted against ho, which results in a scatter of points. Still some S-ho relation is drawn (see fig. 8). These $\mu A = 200\ m^2$ and S-ho relation are used as an upstream boundary condition for computation 2.

liever hi?

hoe doet Thabet dat?
iets duidelijker
Moeilijkheid in dit geval de dubbele lus, maar die is helemaal niet essentieel gezien de sterke schematisatie

Computation 2

Using the above described resistance- and storage-conditions for an upstream boundary condition, the water flow in the model is calculated. The agreement between computation one and two is not good. The discharges and waterlevels in 120 and 128 km for computation one and two are given in fig. 9. This could be expected. The assumptions are too simple for this case. It might look as if this whole black box method is not very serious, but it is succesfully applied for other cases (see: for instance PENPAS Users Manual, 1979).

In the next computations an upper boundary condition will be created through extension of the river-channel (to realise a physical damping of the tidal wave, without reflection).

?

II.2 Extended upper reaches

To create a boundary condition the channel is extended out of the tidal region. The waterlevels and discharges in these upper reaches have no realistic significance. The only use of the extended channel is to reproduce a water flow in the model (km 0 - 128) that is more or less like in computation 1. It will not be tried to calibrate these "upper reaches" until the flow resembles computation 1 closely. There is no use in doing so, because computation 1 is not calibrated either, and only useful to indicate a realistic waterflow. For instance it is clear that the tidal amplitude at the 128 km cross-profile should be about 70 - 80% of the amplitude at the mouth, and the phase lag should be about 3 - 4 hours.

It is not possible anymore to use a "measured" upper boundary condition, and therefore the measured lower condition is also not very useful anymore. The lower boundary condition is replaced by a sine-function with an amplitude of 0.5 m and a period of 12.5 hours (almost semidiurnal).

The depth, the width and the hydraulic radius in computation 1 were equal to the measured cross-profiles with linear interpolation in between (see fig. 2). For computation 3 the cross-profile data are used, but slightly smoothed (see fig. 10) and extended over 200 km. The depth is decreased rather sharply to damp the tidal wave. Analysing the computed flow with this geometry, this damping appears to be not enough. At 328 km from the rivermouth the tidal amplitude is still about 60% of the tidal amplitude at the rivermouth. Further, the tidal amplitude at 100 km is greater than the amplitude at the rivermouth. This amplification is not unusual when depth and width decrease but might be too much because of a too sharp decrease, for instance in the extended channel.

↓ reflectie!

In computation 4 (next step) the channel is extended another 200 km and the depth-decrease along the x-axis is smoothed (see fig. 11). In figure 12 the maximum and minimum waterlevels are plotted against the river axis. It can be seen that the reflection at the upper boundary (528 km) does not influence the model (0 - 128 km) anymore. There still appears to be reflection and some amplification in the first hundred kilometers which might be caused by the step-like decreases in width.

hoe?

Reference computation

In computation 5 the decrease in width is also smoothed (see fig. 11). There still is some amplification then, apparently caused by the important inertia forces compared with the resistance (see fig. 12). This computation will be used as a reference computation.

ook tegen een geleidelijke versmalling krijg je reflectie?

The waterlevels at 0, 32, 64, 96, 128 and 328 km are depicted in figure 13, the discharges at 0, 32, 64, 96 and 128 km in figure 14. From figure 13 it is clear that after some damping of the waterlevel amplitudes from km 0 - 50, there is some amplification. But this phenomenon is more clear in figure 12. The discharge amplitudes are damped much faster. This can be observed from figure 14, and still better from figure 15. In this figure the minimum and maximum discharges as a function of the x-axis are shown.

When figure 12 and 15 are compared it attracts attention that although the waterlevel variations upstream of about 300 km are still considerable, the discharges are already neglegible.

dwarsprofiel is al factor 10 kleiner, bovendien reflectie → niet zo vreemd

The phase-lag of waterlevels and discharges along the x-axis (with regard to the rivermouth) are shown in figure 16. Depicted is the time difference between the passage of the mean waterlevel (from negative to positive levels) at km 0 and at km x. The same holds for the discharges.

As a result of the very long reach of tidal influence ($L \gtrsim \lambda$) the waterlevels and discharges are not in phase. At the rivermouth the discharge minima/maxima occur about half an hour before the waterlevel minima/maxima. This increases untill about an hour and a half around km 100, and decreases again untill about 0.3 hours at km 200.

To obtain some insight in the momentaneous waterlevel and discharge differences along the x-axis, some momentaneous slope and discharge lines are given in figures 17 and 18.

III Influence of storage area variations

III.1 General

The model which is conceived in par. II.2 is divided in several sections (see fig. 3A). The first 32 km from the rivermouth upstream are called Section 1, the next 32 km are called Section 2, Section 3 and Section 4.

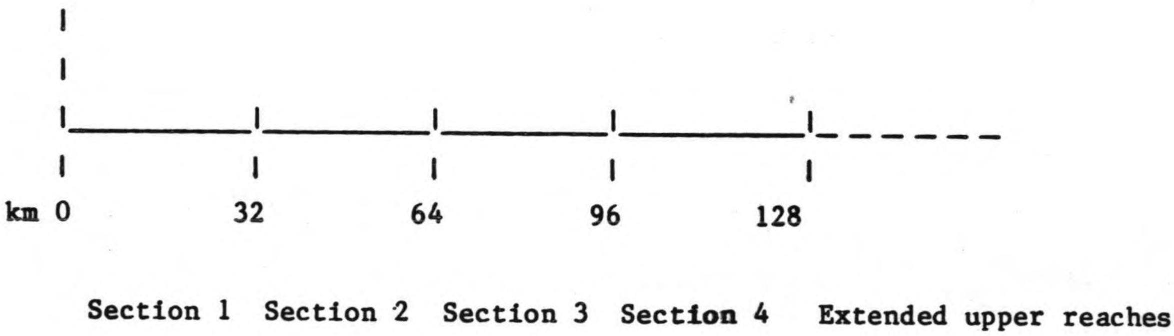

Fig. 3A

This is done because it was soon found that an extra storage area in for instance Section 2 has a significantly different influence on the wave propagation than an extra storage area in Section 4.

The extra storage area is obtained through the introduction of a local storage width exceeding the local flow width. It is assumed that this extra storage width is present for the whole tidal range (which is an important simplification!). This is elucidated in figure 4A.

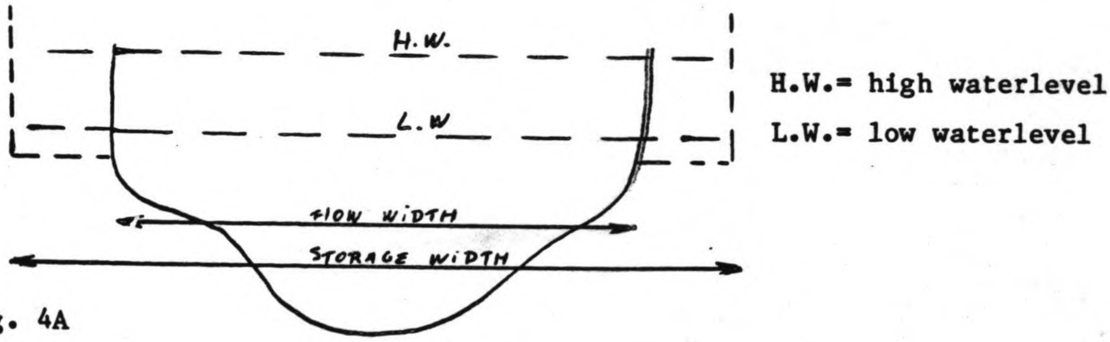

Fig. 4A

For a specific computation the storage width of a section is set equal to 1.1, 1.5 or 3.0 times the flow width. This is shown for Section 2 in figure 5A.

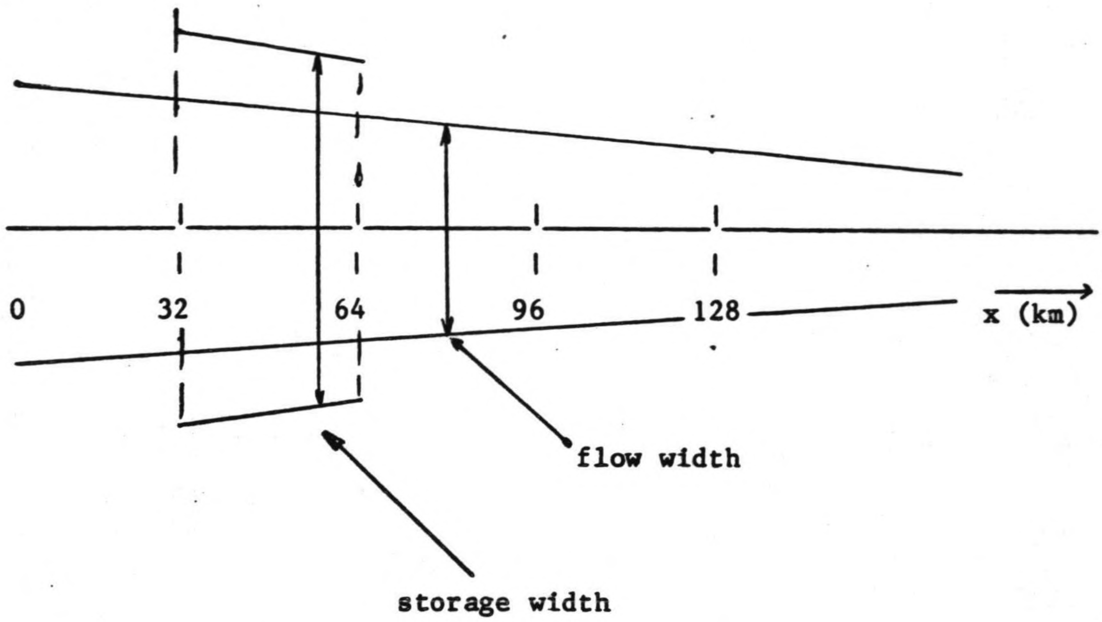

Fig. 5A

As mentioned before, it was learned (by trial and error) that the position of the extra storage with respect to the x-axis decides the way of influence on the wave propagation. When the position is given the measure of extra storage (1.1 or 3.0 times the flow width) does not change the way the wave propagation is influenced. The magnitude of the influence does change of course.

III.2 Extra storage area close to the rivermouth

To illustrate the influence of a storage area close to the rivermouth, a computation is discussed in which the storage width (Bb) of Section 2 is set equal to 1.5 times the flow width (Bs) of Section 2. See figure 6A.

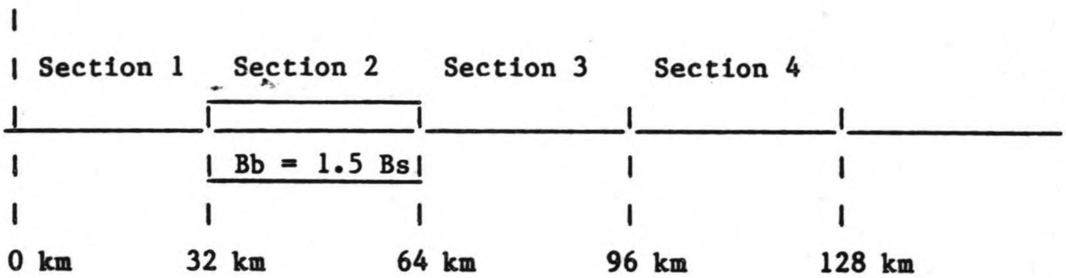

Fig. 6A

The waterlevels and discharges at the section boundaries are given in figures 19 and 20. A better comparison with the reference computation (Bb = Bs) is possible when the amplitude and phase-lag of the waterlevels and discharges are examined. The waterlevel <u>amplitudes</u> along the x-axis are <u>almost the same</u> for this computation as for the reference computation (see fig. 12), therefore they are not shown in an extra figure.

The phase-lag of the waterlevels (with regard to the rivermouth) does change as a result of the extra storage in Section 2. Both the phase-lag development along the x-axis for this computation and the reference computation are depicted in figure 21. From this figure it becomes clear that the phase-lag increases as a result of the extra storage. The increase of the phase-lag (compared with the reference computation) grows from zero at km 0 untill about 0.4 hour at km 50, which is close to the middle of Section 2. The phase-lag increase remains fairly constant upstream of km 50.

Because the waterlevel amplitudes (and of course the tidal period) are unchanged, the $\partial h/\partial t$ are also unchanged. But because of the increasing phase-lag from km 0 untill km 50, the waterlevel falls along the x-axis (Δh over Δx) increase also.

Naturally, the discharges are also changed by the extra storage. The discharge amplitudes are depicted in figure 22 and the discharge phase-lags in figure 21. The discharge amplitudes exceed those of the reference computation from 0 untill about 50 km, with a maximum at the rivermouth. Upstream of km 50 the discharge amplitudes are smaller.

The combination of amplitude and phase-lag differences can be shown in the momentaneous slope and discharge lines along the x-axis. These are given in the figures 23 and 24.

For a better understanding of the problem, the relative importance of the different terms in the equation of motion is interesting. For this aim the equation of motion can be integrated to x:

$$h2 - h1 = \left(-m \frac{\partial Q}{\partial t} - W\, Q|Q| + be\, Q \frac{\partial h}{\partial t} \right) \Delta x$$

The terms in the righthand side of the equation are mean values over the section (x = x2 - x1) but a function of time (momentaneous values). The constants m, w and be are depending on the width, roughness, cross section and hydraulic radius. The first term of the righthand part represents the inertia forces, the second term represents the resistance and the third term is the convective term. In Appendix I the relative importance of these terms is shown through integration of the equation of motion. This is done for the reference computation and for this extra storage computation.

It follows that the influence of the convective term is neglegible. The inertia forces are dominating in those parts of the tidal cycle where the discharges are small. Around the discharge maxima/minima the resistance forces are dominant (resistance is proportional to Q^2).

When the $\partial h/\partial x$ values increase (as a result of the extra storage) the inertia term becomes greater and the resistance even smaller than it already was, for those parts of the tidal cycle where the discharges are small. In other parts the inertia and the resistance term increase.

Studying the figures 19 up to and including 24 some insight is acquired concerning the influence of the extra storage areas. This insight is developed into a line of thought describing the influence of an extra storage area close to the rivermouth.

Influence of an extra storage area close to the rivermouth (Section 2)

1. The waterlevel amplitudes remain practically unchanged in the whole model, as could be expected. Because of this and the unchanged tidal period, the $\partial h/\partial t$ values remain the same also. ?

2. The discharge amplitudes increase downstream of the extra storage area (km 0 - 50), and decrease upstream of km 50.
   meer komberging; reflectie

3. Because of the increased discharges (km 0 - 50) the momentaneous waterlevel differences ($\partial h/\partial x$) are also increased somewhat. But this is relatively limited.

4. The waterlevel amplitudes are not damped, therefore the waterlevel differences along the x-axis ($\partial h/\partial x$) can only become greater as a result of an increasing phase-lag for the waterlevels.
   dit moet ook iets met reflectie (staande golf) te maken hebben

5. In parts of the tidal cycle where the discharges are small, the already dominating inertia term increases. In other parts of the tidal cycle the inertia and the resistance increase (when $\partial h/\partial x$ becomes greater).
   ja, want Q is toegenomen!

III.3 Extra storage area relatively far from the rivermouth

In this paragraph a computation is discussed in which the storage width (Bb) of Section 4 is set equal to 1.5 times the flow width (Bs), see fig. 7A.

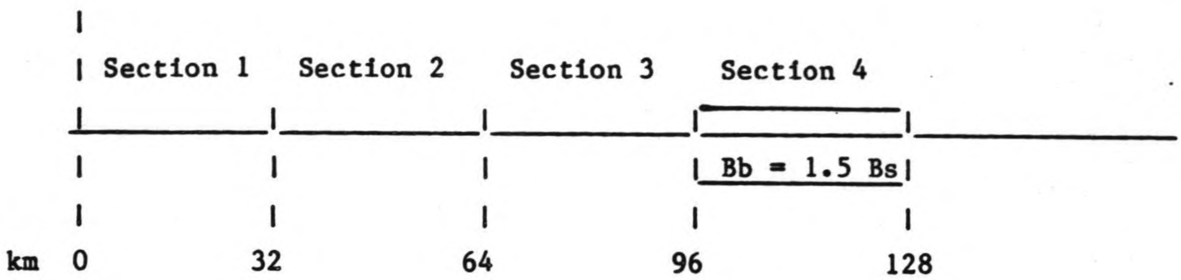

Figure 7A

The waterlevels and discharges of this computation at the section boundaries are presented in the figures 25 and 26. A few things attract attention. The waterlevel amplitudes are much more damped now, and the discharge amplitude at the rivermouth is not increased but decreased. The first of these two points is even more clear in figure 27 where the waterlevel minima/maxima are depicted. Apart from the first twenty kilometers (km 0 - 20) there now is a considerable damping of the waterlevel amplitudes.

The discharge amplitudes for this computation and the reference computation can be compared from figure 28. In contrast with the extra storage close to the rivermouth, the discharge amplitudes now only increase in a part of the model downstream of the extra storage area.

From km 0 untill about km 50 the discharge amplitudes decrease and only from km 50 untill about km 110 the amplitudes increase. Just as in the last paragraph, the discharge amplitudes upstream of the extra storage all decrease.

That the discharges at the rivermouth are smaller now than in the reference computation (in spite of an enlarged storage area) is a result of the decreased storage upstream and downstream of the widened Section 4. The $\partial h/\partial t$ values are decreased there (damped waterlevel amplitudes). This is shown quantitatively in Appendix II through discretized integration of the continuity equation.

The phase-lag differences with regard to the reference computation for both waterlevels and discharges can be seen in figure 29. In this case the waterlevel phase-lag does not increase (with regard to the reference computation) right from the rivermouth. There is even a small decrease from km 35 to km 50. At about km 50 the phase-lag starts to grow faster than in the reference computation. The waterlevel phase-lag difference remains more or less constant upstream of km 110, which is about halfway Section 4.

Like in III.2 there is a constant difference in the discharge phase-lag right from km 0. This difference then increases from about halfway the widened section up to about 20 km after the widened section. Both here and in III.2 the waterlevel phase-lag increase arises downstream of, and in the widened section. The discharge phase-lag increase arises in, and upstream of the widened section.

In figures 30 and 31 some momentaneous slope and discharge lines are given. In this case it is more difficult to give a sound argument describing the phenomenon, there are more factors influencing each other. Nevertheless it will be tried. The point at which the argument is started is not much more than a choice.

<u>Influence of a storage area relatively far from the rivermouth (Section 4)</u>

1. Just downstream of Section 4 the discharges are increased as a result of the enlarged storage area.

2. Because of this, the waterlevel phase-lag has to increase just downstream of Section 4 to produce increased $\partial h/\partial x$ values.

3. Another result of the increased discharges is an increased influence of the resistance. This in turn damps the waterlevel amplitudes.

4. Because of the damped waterlevel amplitudes there is less storage upstream and also downstream of Section 4.

5. Because of the decreased storage upstream and downstream of Section 4 the discharges at the rivermouth decrease. This in spite of the enlarged storage area and increased discharges just downstream of Section 4.

gammel verhaal

maar mijn idee moet je dit globaal kunnen analyseren in termen van lopende golven en terugkaatsing (vierpolen)

III.4 Quantitative influence of storage area variations

To know more about the relative importance of the accuracy of the storage area measurement with reference to the accuracy of waterlevel and discharge measurements it is useful to quantify the storage area influence.

In this paragraph the results of the two computations that were discussed in III.2 and III.3 are given, together with a few other computations.

The storage width of Section 3 (between 2 and 4) is set equal to 1.5 times the flow width in one computation to show the transition between the Sections 2 and 4.

Furthermore, the computations are given in which the storage width of Section 4 is set equal to 1.1 and 3.0 times the flow width. With these computations the effect of an increasing storage width in one section is demonstrated.

An extra computation is presented in which the storage width of the Sections 1, 2, 3 and 4 is set equal to 1.1 times the flow width. The extra storage area in this computation is about equal to the extra storage area in the other computations but divided over 4 sections.

The section division in this paragraph is the same as in the other paragraphs, 0 - 32 km, 32 - 64 km etc.. In this paragraph the amplitudes and phase-lags of waterlevels and discharges at the section boundaries are tabellated.

In table 1 the waterlevel amplitudes at the section boundaries are given for the reference computation and the computations with extra storage in Sections 2, 3 and 4 separately, plus the computation with extra storage in the Sections 1 to 4. As observed in III.2, extra storage in Section 2 gives very little damping of the waterlevel amplitudes, about 2%. Extra storage in Section 4 gives a damping of 10 - 13% except in the first 40 km, where the damping is smaller. Extra storage in the Sections 1, 2, 3 and 4 also has little effect on the waterlevel amplitudes, about 3% damping.

In table 2 the discharge amplitudes are given for the same computations. The maximum increases and decreases are all about 7-9% except for the computation with the spreaded extra storage. When the extra storage is divided over four sections the effect on the discharge amplitudes is much smaller, only about 2% instead of 7-9%.

The waterlevel and discharge phase-lags are given in the tables 3 and 4. The phase-lag difference with the reference computation grows in all cases to about half an hour. This difference is built up like described in III.2 and III.3. The phase-lag decrease which occurs in the first 50 km of the computation with extra storage in Section 4 is about 0.05 hour.

In the tables 5 and 6 the effect of different amounts of extra storage in section 4 is shown. It attracts attention that the waterlevel amplitude damping in the computation with Bb = 3.0 Bs is more than 30% upstream of about km 50. Such an amount of extra storage is not unthinkable. A stretch of swamp of about 30 km where the total width is about 3 times the width of the river itself is well imaginable (see fig. 8A). From these two tables it becomes clear that the difference in effect of the increasing amounts of extra storage is <u>only a difference in magnitude</u> of the influence on the wave propagation.

ampl. at km	Bb = Bs	Section 2 Bb = 1.5 Bs		Section 3 Bb = 1.5 Bs		Section 4 Bb = 1.5 Bs		Section 1,2,3,4 Bb = 1.1 Bs	
0	1.00	1.00		1.00		1.00		1.00	
32	0.76	0.74	-3%	0.70	- 8%	0.72	- 5%	0.75	-1%
64	0.78	0.77	-1%	0.70	-10%	0.69	-12%	0.75	-4%
96	0.89	0.87	-2%	0.83	- 7%	0.79	-11%	0.86	-3%
128	0.98	0.97	-1%	0.92	- 6%	0.88	-10%	0.96	-2%
160	1.02	1.00	-2%	0.95	- 7%	0.89	-13%	0.99	-3%
192	0.90	0.88	-2%	0.84	- 7%	0.77	-14%	0.88	-2%

Table 1. Waterlevel amplitudes [m] with extra storage at different locations.

ampl. at km	Bb = Bs	Section 2 Bb = 1.5 Bs		Section 3 Bb = 1.5 Bs		Section 4 Bb = 1.5 Bs		Section 1,2,3,4 Bb = 1.1 Bs	
0	2654	2827	+7%	2628	-1%	2466	-7%	2693	+1%
32	2162	2323	+7%	2278	+5%	2087	-3%	2197	+2%
64	1674	1619	-3%	1734	+4%	1732	+4%	1704	+2%
96	1171	1119	-4%	1081	-8%	1275	+9%	1177	+1%
128	716	686	-4%	667	-7%	655	-9%	699	-2%
160	414	400	-3%	378	-8%	378	-8%	405	-2%

Table 2. Discharge amplitudes [m^3/s] with extra storage at different locations.

phase-lag at km	Bb = Bs	Section 2 Bb = 1.5 Bs		Section 3 Bb = 1.5 Bs		Section 4 Bb = 1.5 Bs	
0	0.00	0.00		0.00		0.00	
32	1.00	1.27	+0.27	1.08	+0.08	0.99	-0.01
64	2.11	2.42	+0.31	2.29	+0.18	2.06	-0.05
96	3.00	3.29	+0.29	3.31	+0.31	3.23	+0.23
128	3.63	3.91	+0.28	3.92	+0.29	3.91	+0.28
160	4.16	4.50	+0.34	4.52	+0.36	4.50	+0.34
192	5.00	5.33	+0.33	5.34	+0.34	5.34	+0.34

Table 3. Waterlevel phase-lag [hour] with extra storage at different locations.

phase-lag at km	Bb = Bs	Section 2 Bb = 1.5 Bs		Section 4 Bb = 1.5 Bs	
0	-0.50	-0.34	+0.16	-0.38	+0.12
32	+0.30	+0.42	+0.12	+0.44	+0.14
64	0.98	1.25	+0.27	+1.12	+0.14
96	1.65	1.98	+0.33	1.79	+0.14
128	2.44	2.75	+0.31	2.67	+0.23
160	3.47	3.77	+0.30	3.76	+0.29
192	4.60	4.90	+0.30	4.89	+0.29

Table 4. Discharge phase-lag [hour] with extra storage at different locations.

ampl. at km	Bb = Bs	Section 4 Bb = 1.1 Bs		Section 4 Bb = 1.5 Bs		Section 4 Bb = 3.0 Bs	
0	1.00	1.00		1.00		1.00	
32	0.76	0.74	-3%	0.72	- 5%	0.72	- 5%
64	0.78	0.75	-4%	0.69	-12%	0.54	-31%
96	0.89	0.86	-3%	0.79	-11%	0.59	-34%
128	0.98	0.95	-3%	0.88	-10%	0.67	-31%
160	1.20	0.98	-4%	0.89	-13%	0.70	-31%
192	0.90	0.86	-4%	0.77	-14%	0.63	-30%

Table 5. Waterlevel amplitudes [m] with different amounts of extra storage in Section 4.

ampl. at km	Bb = Bs	Section 4 Bb = 1.1 Bs		Section 4 Bb = 1.5 Bs		Section 4 Bb = 3.0 Bs	
0	2654	2610	-2%	2466	-7%	2095	-21%
32	2162	2149	-1%	2087	-3%	1914	-11%
64	1674	1686	+1%	1732	+4%	1806	+ 8%
96	1171	1196	+2%	1275	+9%	1490	+27%
128	716	703	-2%	655	-9%	551	-23%
160	414	407	-2%	378	-8%	293	-29%

Table 6. Discharge amplitudes [m] with different amounts of extra storage in Section 4.

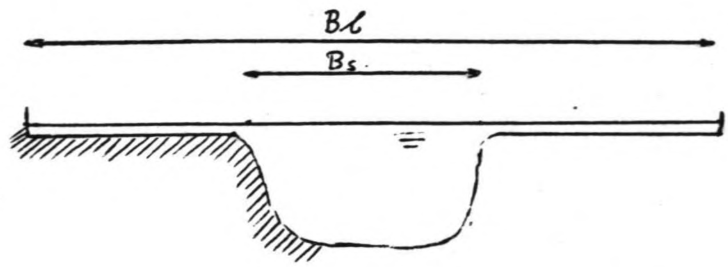

Figure 8a.

Especially when the tables 2 and 6 are compared this is illustrated. When the location of the extra storage is shifted, a discharge amplitude increase (for km 0 and 32) changes to a discharge amplitude decrease. But when the amount of extra storages is increased, no such changes can be found.

It is concluded that a local extra storage area of some 10% of the total storage area of the first four sections can result in a waterlevel amplitude decrease of up to about 15%. And a discharge amplitude increase/decrease is possible up to some 9%, depending on the location along the x-axis. The phase-lag difference grows to about half an hour upstream of the extra storage.

This is considered an important influence on the wave propagation. It may be remembered that it is the aim of this study to examine the importance of the subject.

III.5 Influence of mean upland discharge

The computations in this study are carried out with a zero upland discharge (or fresh water discharge). It is therefore interesting to know if the presence of mean upland discharge changes the way in which an extra storage area influences the wave propagation. Some computations were carried out to investigate this aspect, they are described in this paragraph.

First the waterflow is computed with an upland discharge of 250 m^3/s and a storage width equal to the flow width. This discharge, Qo = 250 m^3/s, replaces the upstream boundary condition Qo = 0 m^3/s. This mean upland (or fresh water) discharge is constant along the x-axis. In the prototype the upland discharge will increase in downstream direction. As a result of the mean upland discharge the physical damping of the tidal wave is increased. Therefore the channel reach from km 128 - 528 (to obtain an upstream boundary condition Qo = 0) could be shortened to 224 km (from km 128 - 352). The value of the upland discharge is chosen rather arbitrarily. It is a rather small discharge but it already significantly influences the mean waterlevels and the waterlevel amplitudes (damping). Because of this significant influence no greater discharges were tried. A mean upland discharge of this magnitude (250 m^3/s) is measured in the Mesuji river in the dry season (see: Main hydraulic survey Mesuji river, 1979).

The waterlevel and discharge amplitudes are depicted in figures 32 and 33. For easy comparison the waterlevel and discharge amplitudes of the reference computation (Qo = 0) are also given in these figures. The mean waterlevels are obviously much affected, but the amplitudes from km 0 up to km 150 remain practically unchanged. The same holds for the discharges.

A computation is then carried out in which the storage width of Section 4 (km 96 - 128) is set equal to 1.5 times the flow width. The resulting amplitudes are also shown in the figures 32 and 33. The influence is very much like the case without upland discharge. A comparison with the figures 27 and 28 renders this clear.

This likeness is further illustrated in the tables 7 and 8. In these tables the waterlevel and discharge amplitudes at the section boundaries are given. All values are from the computations with an upland discharge of 250 m^3/s. Even the quantitative differences as a result of the extra storage are remarkably similar to the values without upland discharge (tables 1 and 2).

Waterlevel Amplitudes			
ampl. at km	Bb = Bs	Section 4 Bb = 1.5 Bs	
0	1.00	1.00	
32	0.74	0.72	- 3%
64	0.75	0.65	-13%
96	0.87	0.75	-14%
128	1.00	0.89	-11%
160	1.01	0.90	-11%
192	0.76	0.68	-11%

Table 7. Waterlevel amplitudes with an upland discharge of 250 m^3/s.

Discharge Amplitudes		
Bb = Bs	Section 4 Bb = 1.5 Bs	
2599	2413	- 7%
2153	2083	- 3%
1698	1763	+ 4%
1199	1330	+11%
746	698	- 6%
375	334	-11%
194	177	- 9%

Table 8. Discharge amplitudes with an upland discharge of 250 m^3/s.

It is concluded that a (modest) upland discharge does not change the way in which wave propagation is influenced by storage areas outside the flow area. Even the magnitude of the influence remains the same.

III.6 Some implications for field surveys

From the preceding paragraphs some implications for hydraulic field surveys can be deduced. In swampy, dense grown (tropical) areas, when it is not possible to measure the storage width, it is not very usefull to carry out accurate discharge measurements to set up a numerical model.

?

Waterlevel measurements are more easy to carry out, and not as expensive as discharge measurements. Besides, waterlevel measurements are needed anyway and can be used to check the discharge measurements. It is therefore often recommendable not to economize on the number of waterlevel measurements.

Some idea of the magnitude of the error in discharge and waterlevel values as a result of inaccurately known storage areas can be got from this study.

A possible procedure to avoid useless measurements can be described as follows. First a reconnaissance survey of the area to be modelled should be carried out (might even be possible from literature). In this survey some data should be gathered about the river geometry. This could consist of width, depth and slope along the x-axis and the location and importance of tributaries.
Furthermore the possible existance of storage areas outside the flow area should be investigated. When possible the location and magnitude of these extra storage should be recorded.

Of course it is necessary to measure waterlevels as a function of time, at least at the mouth and at some more places farther upstream (bifurcations). When possible, a provisional discharge measurement could be carried out to get some idea about the runoff (the mean upland discharge).

With these data a provisional numerical model could be set up. Such a model would not accurately reproduce the waterflow in the prototype, of course. But it could be used to investigate the sensitivity of that specific geometry and tidal circumstances for extra storage areas at the location and of the magnitude that were expected in the reconnaissance survey.

When these extra storage areas appear to give differences that are too important to neglect there are a few possibilities. It is not advisable to just measure the discharges and waterlevels in the normal way and then try to calibrate both roughness and extra storage. As can be concluded from the preceding paragraphs, the influence of extra storage areas can be rather complicated, even if some important simplifications are applied here.

One solution is to invest some extra funds and try and measure the storage width as a function of time and place. This seems to be more profitable than measuring extra discharge and waterlevel values. Still, when it is decided that it is too difficult to measure the extra storage, it could be tried to carry out extra discharges and waterlevels and decide the extra storage in calibration.

When this is tried, it would be very usefull to have a calibration-strategy. A strategy from which the measurements that are needed for the calibration can be decided before the main survey. At this subject further research seems advisable. A clear insight into the connection between inertia, relative resistance and storage, and their influence on the phenomenon is needed. Especially for the situation with extra storage far upstream, which is rather complicated.

In those cases where the model has to be calibrated on the roughness only, a surplus of waterlevel information should prove to be sufficient An important tool in the analysis can be the integration of the equation of motion, as shown in Appendix I.

But when both roughness and storage areas have to be calibrated, extra information about the discharge differences along the x-axis are needed. Thus extra discharge measurements. A tool which can render a good service in this case is the integration of the equation of continuity as shown in Appendix II. In essence this method uses the relation

$$\Delta Q = \frac{\partial h}{\partial t} \cdot \int_{x}^{x+\Delta x} Bb \, dx$$

When the discharge difference over the section x, and the momentaneous $\partial h/\partial t$ value are known, the storage area in the section $\int_{x}^{x+\Delta x} Bb \, dx$ follows.

It is important that the chosen $\frac{\partial h}{\partial t}$ value is representative for the section ($\Delta x = x2 - x1$).

V References

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Appendix I

Relative importance of the terms in the equation of motion

The equation of motion will be integrated numerically in this Appendix. This is done for both the reference computation and the computation with Bb = 1.5 Bs in Section 2 at two points in the model and for a few characteristic moments in the tidal cycle. The result is not a complete picture of the mutual connection between the different terms, but some insight in the relative importance of the different terms for this case is gained.

The equation of motion can be integrated to x:

$$h2 - h1 = \left(-m.(\partial Q/\partial t) - w\ Q|Q| + be\ Q\ (\partial h/\partial t)\right)\Delta x$$

The $\frac{\partial Q}{\partial t}$ and $\frac{\partial h}{\partial t}$ are representative mean values for the section x (but a function of t). The m, w and be in the above given expression are mean values over Δx (but a function of time) depending on the width, roughness and hydraulic radius. They are explained in the computation.

Since the h and Q values as functions of time and place are known from the computer computations, the accuracy of this integration can be deduced from the agreement between the right- and left-hand side of the equation.

The calculation is first carried out using the h and Q values of km 0 and 8. These are given in the figures 34 and 35. The waterlevel values are unchanged for the computation with the extra storage. The changed Q-values are shifted (on paper) 20 mm to the right. The mean cross-section (at km 4) is sketched in fig. I.1.

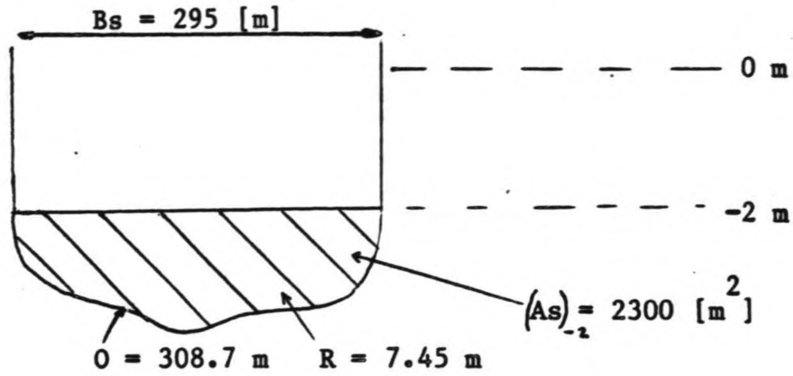

Figure I.1

roughness : $C = 50\ m^{1/2}/s$

total wet cross-section: $As = 2300 + 295\ (2 + hg(t))$

total hydraulic radius : $R = As\ /(308.7 + 2(2 + hg(t))$

In the following the computation is carried out for t = 0, t = 2 hour and t = 2.5 hour. First the values of the wet cross section and hydraulic radius are calculated, they are given in table 9. the integration itself is shown in table 10.

TIME [hour]	Bb = Bs As [m]	Bb = Bs R [m]	Bb = 1.5 Bs As [m]	Bb = 1.5 Bs R [m]
0	2881.2	9.22	2881.2	9.22
2.0	3003.6	9.58	3003.6	9.58
2.5	3020.7	9.63	3020.7	9.58

Table 9. Hydraulic radius and total wet cross section at km 4.

When the discharge is small, the inertia term is dominating the other two. For large discharges the resistance term is dominating. The convective term is neglegible. The accuracy of the calculation is good, the difference between the columns 3 and 23 in table 10 is not more than a few millimeters.

As a result of the extra storage, the difference between the waterlevels at km 0 and 8 increases 3 - 12 mm. When the discharge is small (t = 0) this extra fall is realised through an increase in the inertia term in spite of a decrease in the resistance term. This decrease in resistance is a result of the increased phase-lag. For larger discharges the fall-increase is realised through a larger inertia as well as a larger resistance term.

Some more calculations are carried out using the discharges and waterlevels of km 88 and 96. These are shown in the figures 36 and 37. The waterlevels are changed (with extra storage) at this place (mainly a phase-lag difference), therefore they are also shifted 20 mm to the right on the paper. The mean cross-section (at km 82) is sketched in fig. I.2.

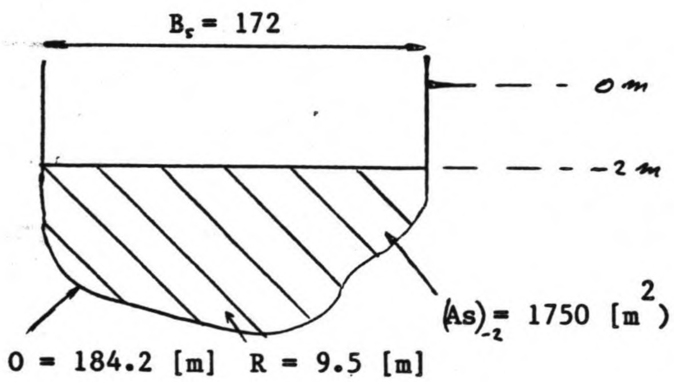

Figure I.2

roughness : $C = 50\ m^{1/2}/s$

total wet cross section: $As = 1750 + 172\ (2 + hg(t))$

total hydraulic radius : $R = As/(308.7 + 2.(2 + hg\ (t)))$.

The computation is (just as at km 0-8) carried out for t = 0, t = 2 [hour] and t = 2.5 [hour]. The results are shown in tables 11 and 10. It can be seen that the differences between the columns 3 and 23 are increased slightly up to 6 mm.

	t	h_1 km 0	h_2 km 8	h_g	Δh_g over Δt = 3600 sec	$\frac{\Delta h_g}{3600}$ in 10^{-4} m sec^{-1}	Q_1 km 0	Q_2 km 8	$\bar{Q}$	$\Delta\bar{Q}$ over Δt = 3600 sec	$\frac{\Delta\bar{Q}}{3600}$ in m^3 sec^{-2}	FACTOREN (ZIE BIJL.) m in 10^{-4} m^{-3} sec^2	w in 10^{-10} m^{-6} sec^2	b_e in 10^{-4} m^{-4} sec^2	B in m	TERMEN BEW. VERG. m $\frac{\Delta\bar{Q}}{3600}$ in 10^{-4}	w $\bar{Q}/\bar{Q}/$ in 10^{-4}	b_e $\bar{Q}$ $\frac{\Delta hg}{3600}$ in 10^{-4}	BEREKENING VERVAL IN m traagh.-term + Δx ⑯	weerst.-term + Δx ⑰	Bern.-term Δx ⑱	TOTAAL Δh = $h_1 - h_2$	controle $h_2 = h_1 - \Delta h$	CONT. VERG. B $\frac{\Delta hg}{3600}$	ΔQ = −Δx ㉔ in m^3 sec^{-1}	controle $Q_2 = Q_1 - \Delta Q$
	U. MIN	in m t.o.v. N.A.P.					in m^3 sec^{-1}																			
	①	②	③	④	⑤	⑥	⑦	⑧	⑨	⑩	⑪	⑫	⑬	⑭	⑮	⑯	⑰	⑱	⑲	⑳	㉑	㉒	㉓	㉔	㉕	㉖
Bl=Bs	0.00	0,000	-0,061	-0,030	+0,240	0,667	360	200	280	720	0,200	0,354	0,0523	0,0724	295	0,071	0,004	+0,001	+0,057	+0,003	-0,001	0,059	-0,059	0,0197	157,4	202,6
Bl=1,5B, km 32-64	0.00	0,000	-0,064	-0,032	+0,240	0,667	250	90	170	780	0,217	0,354	0,0523	0,0724	295	0,077	0,001	+0,001	0,062	+0,001	-0,001	0,062	-0,062	0,0197	157,4	92,6
Bl=Bs	2.00	0,422	0,352	0,387	+0,140	0,389	1331	1236	1284	157,5	0,0438	0,339	0,0452	0,0667	295	0,015	0,075	0	0,012	+0,060	0	0,072	0,350	0,0115	91,8	1239,2
Bl=1,5B, km 32-64	2.00	0,422	0,340	0,381	+0,140	0,389	1406	1310	1358	218,5	0,0607	0,339	0,0452	0,0667	295	0,021	0,083	0	0,017	0,066	0	0,083	0,339	0,0115	91,8	1314,2
Bl=Bs	2.50	0,476	0,416	0,446	+0,084	0,247	1349	1288	1319	-9,0	-0,0025	0,337	0,0455	0,0639	295	-0,001	0,079	0,002	-0,001	0,063	-0,002	0,060	0,416	0,0072	58,3	1290,7
Bl=1,5B, km 32-64	2.50	0,476	0,404	0,440	+0,084	0,247	1445	1385	1415	28,5	-0,0079	0,337	0,0453	0,0639	295	+0,003	0,091	0,002	+0,002	0,072	-0,002	0,072	0,404	0,0072	38,3	1386,7
		km 88	km 96				km 88	km 96																		
Bl=Bs	0.00	-0,420	-0,424	-0,422	-0,044	-0,121	-426	-407	-417	+117	0,0325	0,504	0,0707	0,0838	172	+0,016	-0,016	0	+0,013	-0,013	0	0	-0,420	-0,0021	-16,6	-409,4
Bl=1,5B, km 32-64	0.00	-0,379	-0,379	-0,379	-0,071	-0,196	-449	-420	-435	+121	0,0336	0,502	0,0707	0,0838	172	+0,017	-0,017	+0,001	+0,013	-0,011	-0,001	+0,001	-0,378	-0,0034	-27,0	-422,0
Bl=Bs	2.00	-0,190	-0,249	-0,220	+0,250	+0,692	+263	+156	+210	+466	0,1294	0,496	0,0674	0,0824	172	+0,064	+0,004	+0,001	+0,031	+0,003	-0,001	+0,033	-0,243	+0,0119	+95,2	167,8
Bl=1,5B, km 32-64	2.00	-0,258	-0,310	-0,284	+0,210	+0,583	+89	+6	+43	+434	0,1206	0,498	0,0677	0,0830	172	+0,060	0	0	+0,048	0	0	+0,048	-0,286	+0,0093	+74,0	15,0
Bl=Bs	2.50	-0,063	-0,114	-0,089	+0,240	+0,661	+482	+387	+435	+355	0,0986	0,490	0,0831	0,0816	172	+0,046	+0,011	+0,002	+0,038	+0,004	-0,002	+0,045	0,108	0,0114	+91,2	391,?
Bl=1,5B, km 32-64	2.50	-0,131	-0,187	-0,159	+0,250	+0,686	+342	+237	+290	+434	0,1206	0,493	0,0821	0,0821	172	+0,059	+0,007	+0,002	+0,047	+0,006	-0,002	+0,051	0,182	0,0118	+94,4	247,6

DE BEWEGINGSVERG. : $\frac{h_1 - h_2}{\Delta x} = -m \frac{\Delta\bar{Q}}{\Delta t} - w\,\bar{Q}/\bar{Q}/ + b_e\,\bar{Q}\,\frac{\Delta hg}{\Delta t}$

DE CONTINUITEITS VERG. : $\frac{Q_1 - Q_2}{\Delta x} = -B\frac{\Delta hg}{\Delta t}$

WAARIN: $m = \frac{1}{g A_s}$, $w = \frac{1}{C^2 A_s^2 R}$, $b_e = \frac{2B}{g A_s^2}$ EN B GEMIDDELDEN ZIJN OVER Δ X.

T.H. DELFT AFDELING W & W
LEERSTOEL VLOEISTOF MECHANICA

APPENDIX I
TABLE 10

Berekend door: F.R Rysbirman
Datum :

The explanation of the differences in the several terms is not as straight forward here, as it was at km 0-8. The mutual connection between the terms would probably become clear from an analytical solution, the harmonic method for instance, but this is not possible with this geometry. Extra research on this topic may be recommendable.

TIME [hour]	Bb = Bs As [m]	Bb = Bs R [m]	Bb = 1.5 Bs As [m]	Bb = 1.5 Bs R [m]
0	2021.4	10.79	2028.8	10.82
2.0	2056.2	10.95	2045.2	10.90
2.5	2078.7	11.06	2066.7	11.00

Table 11. Hydraulic radius and total wet cross section at km 92.

Appendix II

Integration of the continuity equation

The continuity equation can be written as

$$\partial Q / \partial x + Bb\, \partial h / \partial t = 0$$

A numerical approximation of the equation can be obtained when the equation is integrated to x:

$$[Q(x,t) - Q(x + \Delta x,t)] = - \Delta x \cdot \frac{Bb[h(x,t + \Delta t) - h(x,t)]}{\Delta t}$$

In this way the difference in discharge between two points along the x-axis can be checked. This simple method is used to show the effect of extra storage in Section 4 on the maximum discharge in the rivermouth. The difference in discharge between km 0 and km 192 (where the discharge is small) is calculated here from the computed waterlevel changes between km 0 and km 192. This difference is then compared with the computed value to see if the method is accurate enough.

The discharge difference is first calculated for the situation where Bb = Bs and then for Bb = 1.5 Bs from km 96 - 128.

The 192 km riverlength is divided in 6 sections of 32 km each. Per section one mean width is used and one mean h/ t value.

$$Q(o,t) - Q(192,t) = (32\ km/3600\ s).[B16.(h(16,t + 1800\ s) - h(16,t - 1800) + \\ + \ldots\ldots + B176.(h(176,t + 1800) - h(176,t - 1800)]$$

Reference computation, Bb = Bs

$Q(o,t) = +1349\ [m^3/s]$ $Q(192,t) = -141\ [m^3/s]$

B16 = 278 [m] B 80 = 189 [m] B144 = 100 [m]

B48 = 233 [m] B112 = 144 [m] B176 = 56 [m]

1349 + 141 =	(32000/3600).	278	. (+0.405 − 0.286) =	294.1
+	"	. 233	. (+0.226 − 0.069) =	325.2
+	"	. 189	. (+0.074 + 0.132) =	346.1
+	"	. 144	. (−0.082 + 0.356) =	350.7
+	"	. 100	. (−0.304 + 0.482) =	158.2
+	"	. 56	. (−0.454 + 0.464) =	5.0
$1490\ m^3/s$				$1484.3\ m^3/s$

It is concluded that the accuracy is sufficient.

Extra storage, Bb = 1.5 Bs from km 96 - 128

$Q(o,t) = +1259\ [m^3/s]$ $Q(192,t) = -132\ m^3/s$

Only B112 is changed from 144 [m] to 216 [m]

1259 + 132 =	(32000/3600).	278	. (+0.410 − 0.298) =	276.8
+	"	. 233	. (+0.230 − 0.091) =	287.9
+	"	. 189	. (+0.055 + 0.125) =	302.4
+	"	. 216	. (−0.133 + 0.352) =	420.5
+	"	. 100	. (−0.326 + 0.438) =	99.6
+	"	. 56	. (−0.413 + 0.400) =	− 6.5
$1391\ [m^3/s)$				$1380.7\ [m^3/s]$

It becomes clear that, although the storage in the widened section is clearly increased, the storage in the other sections decreases so much that the result is a smaller maximum discharge in the rivermouth.