RIJKSWATERSTAAT COMMUNICATIONS

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ELECTRONIC COMPUTATION OF WATER LEVELS IN RIVERS DURING HIGH DISCHARGES

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by

Section River Studies,

Rijkswaterstaat, Directie Bovenrivieren

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DIRECTIE ALGEMENE DIENST VAN DE RIJKSWATERSTAAT

THE HAGUE — NETHERLANDS

Automation of stream lane computation developed jointly by Rijkswaterstaat and the Institute for Applied Mathematics, Delft Technological University.

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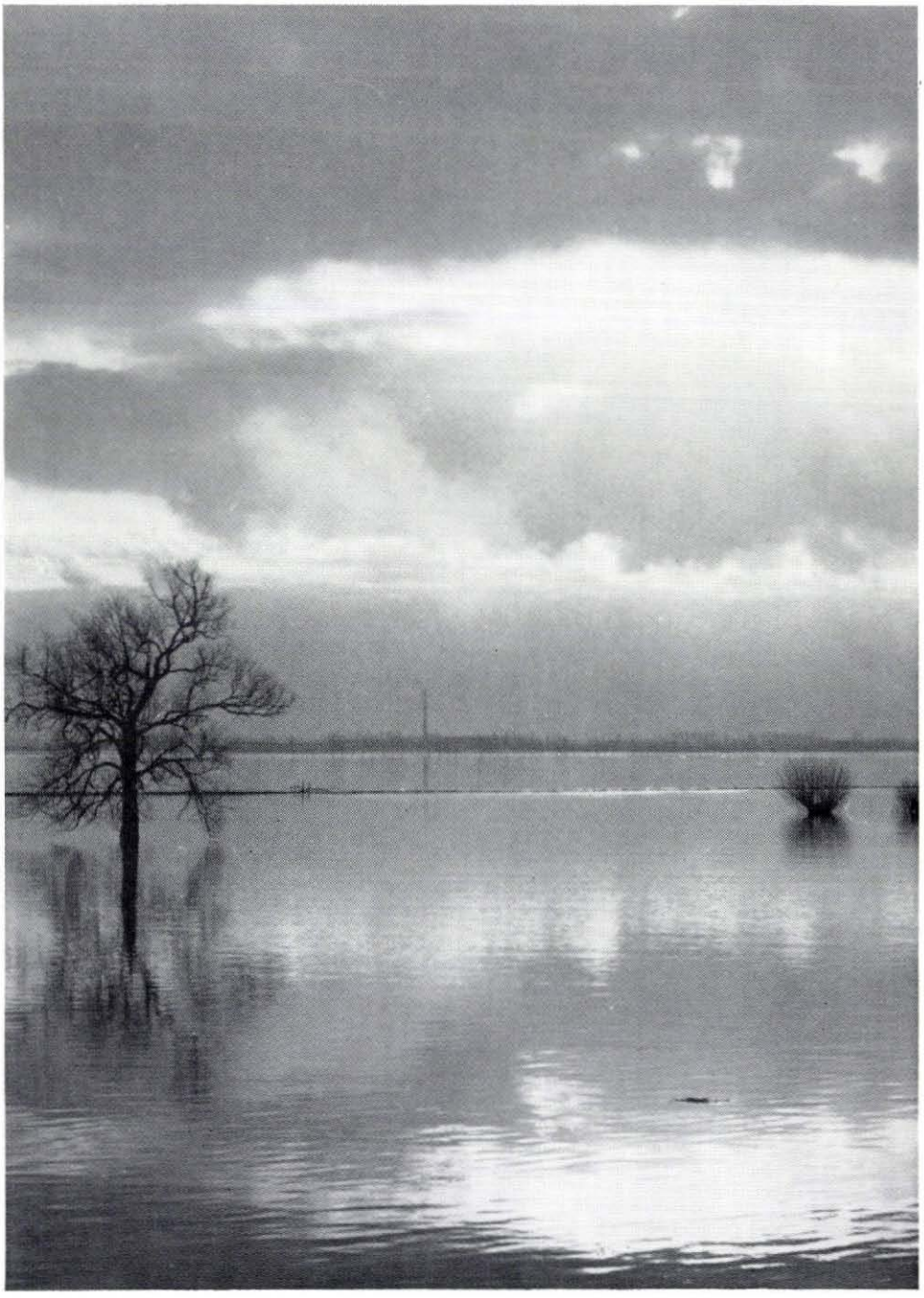
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Silence

Par. 1. Introduction

If we plot the discharges of natural water-courses measured under all manner of circumstances against the water levels measured at the same time, the points obtained will give us what is called the stage-discharge relation curve. But it should be noted that the relation will only be a true one if there is no backwater effect (due, for example, to a dam or a tributary debouching downstream from the measuring point) at the points at which the discharges and levels are measured.

The stage-discharge relation will also change with any changes in the depth of the bed of the water-course, and discharges and levels will then have to be measured at regular intervals to ensure that the stage-discharge relation curve gives a faithful picture of the actual situation.

Once the stage-discharge relation has been established, a glance at the water-level indicator will suffice to enable the Controller to determine the discharge at any particular moment.

Needless to say, the portion of the stage-discharge curve that reproduces nature most truly will be that lying within the range of the most common water levels and discharges. There will be fewer points for the lower and higher discharges, so the curve will be less reliable. There will be very few if any points for extremely low and extremely high discharges, so the curve will be very indefinite indeed if not non-existent, and methods of extrapolation will then have to be resorted to if the Controller is to have even the slightest idea of the water levels he may expect when discharges are extremely low or extremely high.

As a rule, it is more important to have information on extremely high than on extremely low discharges, because extremely high discharges give rise to the problem of protecting the land against flooding. Not only that, but extrapolation for extremely high discharges is more difficult than it is for extremely low discharges, because in the latter contingency the point for zero discharge is fixed.

It was after the disastrous floods on 1st February 1953, when the sea flooded large areas of south-west Holland, that the Delta Project as it is called was cast in its final form; work started on it immediately. The project will be completed by about 1980; the risk of inundation by the sea will then be so slight that the design-level will be exceeded on average only once in 10.000 years.

It need hardly be said that the risk of river dikes being overtopped had also to be weighed. The memory of the inundation of considerable areas along the rivers in 1926 was still vivid, and it was clearly necessary to take steps to enhance the safety of those parts, too.

The problem was approached in two stages. The first was to decide on a design-discharge, i.e. the maximum discharge during which it was desired to live in safety. The second stage consisted in determining the water level at every point along the river system that was likely to accompany the design-discharge.

Simple extrapolation of the stage-discharge curve was not considered adequate for

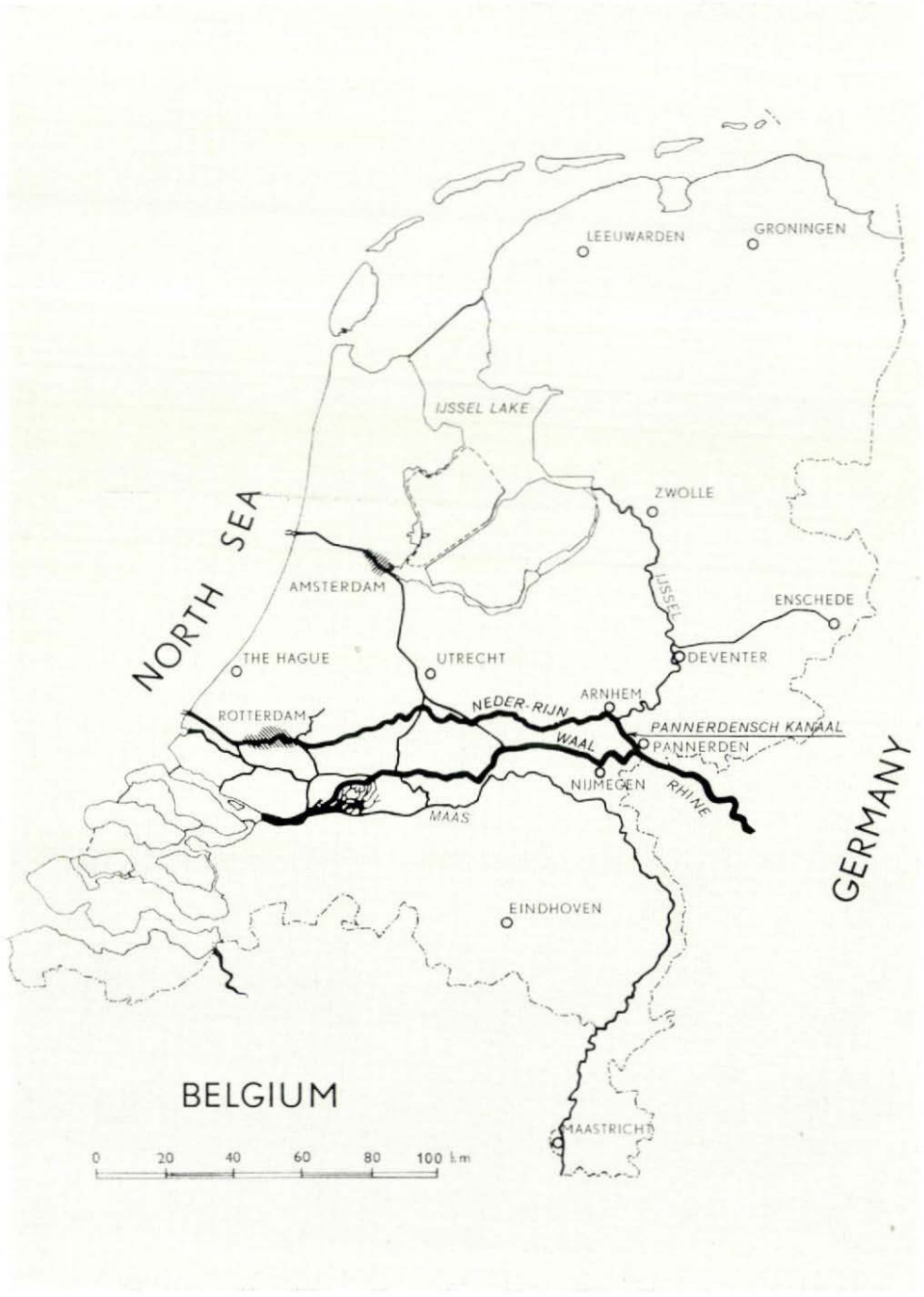


Figure 1. General view

the purpose and it was imperative that methods be devised of obtaining more reliable information on which to base calculations that would give the heights of river dikes capable of preventing inundation.

The method adapted for the effluents of the Rhine that flow across the Netherlands is described in this paper.

Theoretically, it can be used for any system of river effluents.

Par. 2. The river Rhine

The Rhine is fed by a large number of tributaries from its source to the Netherlands frontier. In the Netherlands it discharges through three effluents.

Figure 1 shows the course of the Rhine and its effluents in the Netherlands. Near Pannerden it splits up into the River Waal and the Pannerdensch Kanaal.

Near Arnhem the latter splits up into the Neder-Rijn and the IJssel.

The discharge regime of the Rhine and its effluents can be gathered from the following figures:

	Lowest known discharge in m3/s	Average discharge in m3/s (period: 1951-1960)	Highest known discharge in m3/s
Rhine	630	2140	13500
Waal	490	1490	8250
Pann. Kanaal	130	650	5000
Neder-Rijn	90	390	2700
IJssel	50	260	2300

Figure 2 is a diagrammatic cross section of the Rhine effluents in the Netherlands. When the discharge is mean or low the water runs along the minor bed or "summer bed" as it is called in the Netherlands. The width of the minor bed is kept constant by means of groynes and embankments. The groynes are positioned so that the channel between them will carry the appropriate proportion of the water coming down the Rhine when the discharge is mean or low. The rest of the river bed is separated from the minor bed by what are called summer dikes. The land between the flood-retaining or main dikes and the summer dikes is called the winter bed.

The summer bed and winter bed together constitute the major bed of the river, which carries the water coming down it when the river is in spate.

The river is only in spate sporadically and mainly so in winter. The land between main dikes and summer dikes are referred to as "forelands". The summer dikes serve to prevent flooding of the forelands during minor spates, especially in summer when the

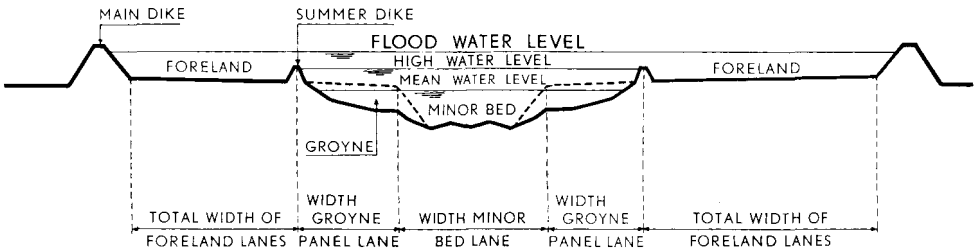


Figure 2. General cross section of river

forelands can be used as pastures for cattle. Moreover, there are dozens of brickyards in the forelands, where the required clay is readily available and only has to be dug out.

There are many other dikes besides summer dikes in the forelands, and roads too, viz. approaches to brickyards and to ferries across the minor bed.

Most of the factories, houses, etc. are situated on raised ground in the winter bed.

Figure 3 shows the configuration of part of the river bed of the Neder-Rijn. It should be noted that over almost the entire length of the Rhine effluents in the Netherlands the areas protected against inundation by the main dikes are level with or actually below the forelands.

Hitherto the height of the main dikes has been determined with reference to the water levels observed during the highest known floods. For the past thirty years the water levels recorded during the greatest known Rhine discharge (in January 1926) have been used as the design basis.

The heights of the dikes were arrived at by adding a safety margin of about 1 m to the basic water levels.

This method is open to a number of objections.

1. It is not clear what degree of security against inundation the method affords the areas along the river.

2. The uniformity of the safety margin (e.g. 1 m) is no guarantee that the riparian regions will be uniformly secure against inundation.

A safety margin of 1 m will afford much greater security against inundation at one spot than at another; it depends on the configuration of the entire river bed. The difference is especially apparent in places where the width of the river bed varies considerably.

An increase in discharges above the highest known discharges at places where the river bed narrows and immediately above these places will cause the water to run over the main dikes much more readily than it will at places where the river is wide. The expressions "characteristic water levels" and "characteristic discharges" in the following paragraphs are used to denote the extreme flood levels or discharges which henceforth will have to be taken as basic data when calculating the heights of main dikes.

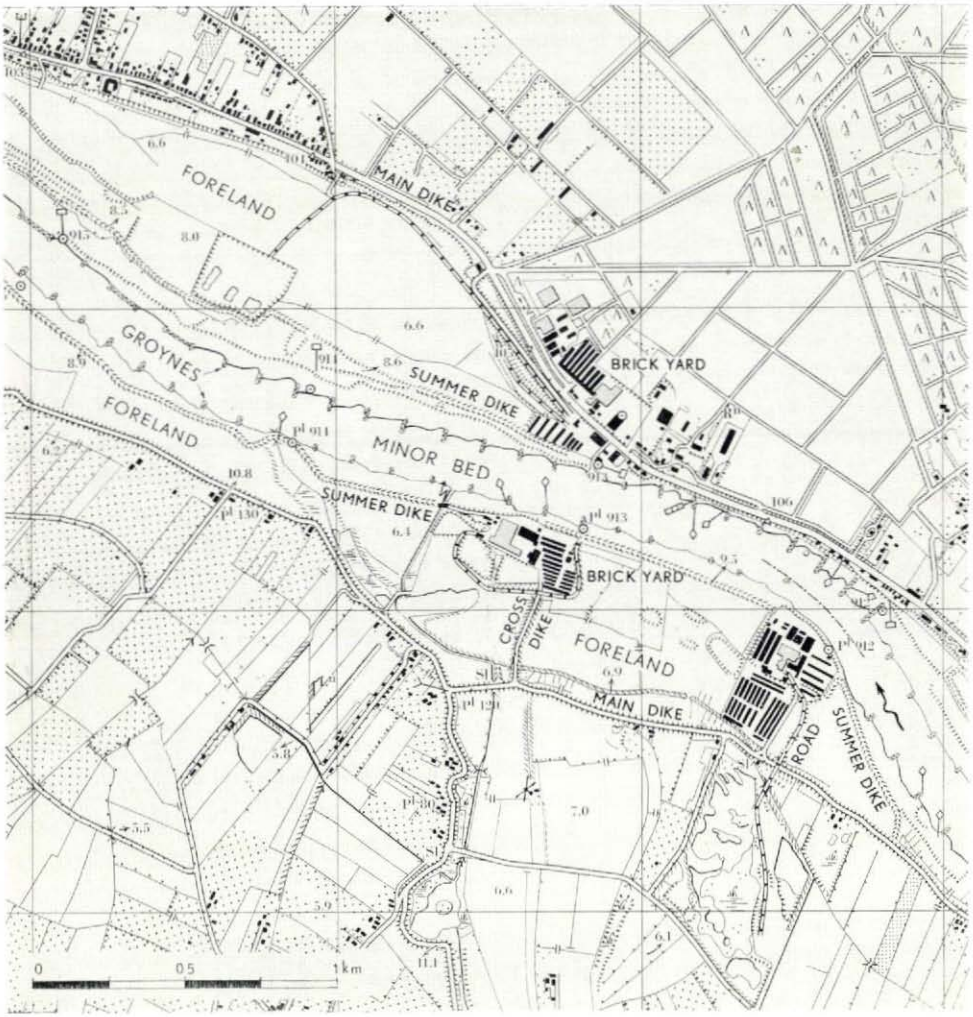
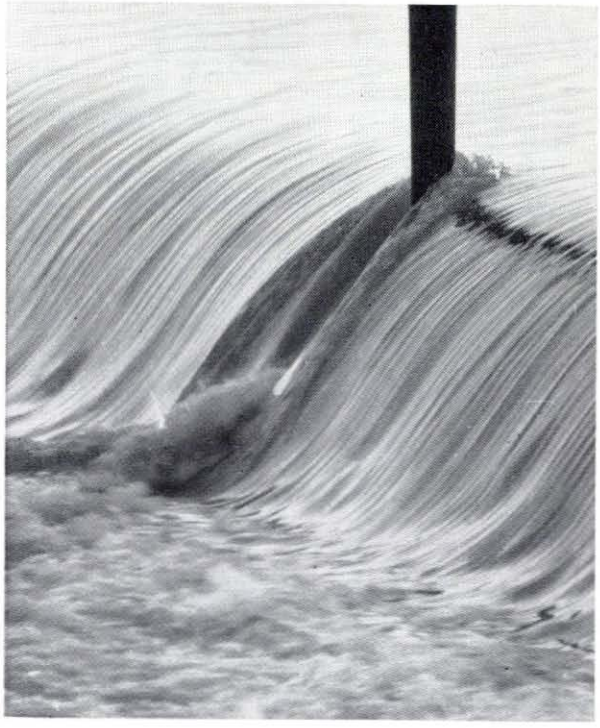


Figure 3. Configuration of a river reach

Par. 3. Determining the characteristic Rhine discharge

The daily figures for Rhine discharges in the period 1901-1950 were used to calculate discharge frequencies. The frequency line was then extrapolated. The extrapolation was done both on logarithmic and on Goodrich paper. The frequency with which a certain Rhine discharge would be reached or exceeded was then read from the extrapolated line. A Rhine discharge of 18,000 m3/s was taken as the basis for the calculations. The extrapolated curve shows that this figure is likely to be exceeded on an average once in 3000 years, which was regarded as an acceptable characteristic discharge.





Flow over an artificially created weir in the winter-bed in order to reduce the discharge through a stream-lane adjacent to the main dike + detail

Par. 4. How characteristic water levels along the Rhine effluents are calculated

Starting from a characteristic Rhine discharge of 18,000 m3/s the first step is to find out how this discharge will be distributed over the various effluents. At first this can only be done by approximation. One way of doing this is to draw separate discharge-frequency curves for each of the branches and then extrapolate them. Discharges for each of the effluents of the same frequency as that taken for the whole of the river Rhine can now be read from the curves.

Using these provisional characteristic discharges, we can now calculate the corresponding water levels in the effluents. We start at the lower end of each of the effluents, where the tidal limits are located. For the IJssel effluent we start at its mouth. We must estimate the water level at each starting point. Any discrepancy between our estimate and reality will lose its significance fairly rapidly as we move upstream and will therefore not affect the outcome of our calculations. Now draw a tentative system of stream "lanes" covering a reach several kilometres long above each starting point. It is assumed that there is no interchange of water between stream lanes except at the nodes, a point dealt with later.

We start off by estimating the slope of the water level in the minor bed. If figures for bottom level, the heights of smaller dikes in the winter bed and discharge coefficients etc. are available, we can then calculate the discharge through each stream lane corresponding to the estimated slope. We must then check whether the total volume of water flowing across a number of cross sections of the effluent tallies with the tentative characteristic discharge. We then correct the estimated slope of the water level in the minor bed in the light of any discrepancies we discover. We then calculate the discharge along each lane anew, repeating the entire process until the discrepancies between the computed discharges across the afore mentioned cross sections and the estimated characteristic discharge are so small that further calculation would be pointless.

Now, however, we must check whether we have drawn the stream lanes correctly. We do this by working out the water levels in each of the stream lanes. We draw lines connecting the points at which the water levels in the various stream lanes are identical. The system of isometric lines must fit logically into the network of stream lanes. If it does not do so, the stream lane system will have to be modified. We must then start calculating all over again and continue until the outcome is satisfactory. As a rule, and with a little skill, the system of stream lanes will only need slight modification during computation.

Computations have been carried out for the rivers Waal, Neder-Rijn and IJssel in the manner described, going upstream from the lower end and using the tentative characteristic discharge for each effluent. As was to be expected, the computed water levels did not quite tally at the point where the Pannerdensch Kanaal splits up into the Neder-Rijn and the IJssel. To make them do so, the assumed discharge of one effluent had to be increased somewhat and that of the other slightly reduced.



Influence of a fence in the winterbed



Winterbed inundated

Before that could be done the stage discharge relation at the bifurcation for very high discharges had to be determined by means of stream lane computations for the upper reaches of each effluent. The results were used to formulate the said relation (Q – h curve) for the lower end of the Pannerdensch Kanaal for very high discharges in this effluent. Stream lane computations for the Pannerdensch Kanaal were then carried out for various discharges. The same thing was done for the upper part of the river Waal. The computations made it possible to establish for both Rhine effluents the stage discharge relation at the bifurcation for very high discharges. Using the assumed characteristic Upper Rhine discharge, it was now possible to compute finally the characteristic discharges of the Waal and Pannerdensch Kanaal. Next, using the relation between the discharges of Pannerdensch Kanaal, Neder-Rijn and IJssel as determined earlier, it was possible to determine from the characteristic discharge of the Pannerdensch Kanaal the definite characteristic discharges of Neder-Rijn and IJssel.

Finally, the corresponding water levels were computed, either from the results of the earlier stream lane computations or by repeating the computations for the newly found discharges.

Needless to say, computations for the Rhine between the Waal – Pannerdensch Kanaal bifurcation and the Netherlands border had to be carried out, using the assumed characteristic discharge and the characteristic water level found for the bifurcation at Pannerden.

A point that must be borne in mind is that, if there is any erosion of the bed of one of the two branches of a bifurcation, the eroded branch will carry a larger proportion of the total discharge. A sound policy is to determine the portion carried by each effluent when there is a certain estimated erosion of the bed of that effluent under the most unfavourable upstream conditions. This has actually been done for the Rhine effluents.

Assuming there is erosion 1 m deep, the characteristic discharges of the effluents are:

Waal	11,400 m3/s (no erosion	11,250 m3/s)
Pannerdensch Kanaal	7,100 m3/s (no erosion	6,750 m3/s)
Neder-Rijn	4,200 m3/s (no erosion	3,950 m3/s)
IJssel	3,050 m3/s (no erosion	2,800 m3/s)

Par. 5. Data required for stream lane computations

As already stated, a rational system of stream lanes must be drawn up before stream lane computations can be carried out. This calls for a close study of the situation, using river maps and supplementary information concerning the location and heights of groynes, dikes, roads, local raised areas for brickyards, etc.

Stream lanes can be divided into three groups:

1. stream lanes the boundaries of which coincide with those of the minor bed and are located between the ends of the groynes, known as "minor bed lanes";

2. stream lanes running across groynes and groyne panels. (The term "groyne panels" is used here to denote the sections of the bed between the groynes);
3. stream lanes running over the forelands, known as "foreland lanes".

The Chézy formula is used for calculating the discharge of a stream lane. If there are any cross dikes, etc. in the stream lanes, the Chézy calculations must be combined with computations for broad-crested weirs.

Actually, the Chézy formula can only be used when the flow is permanent and uniform. Broad-crested weirs in the stream lanes will mostly produce some backwater effect, which will usually make the flow non-uniform.

All the same, tests have shown that the Chézy formula is quite satisfactory for the flow conditions obtaining when the Rhine effluents in the Netherlands are in spate. Bed levels, the heights of the dikes, the depths of excavations, etc. must be known before a stream lane discharge can be calculated.

The elevation of the minor bed is derived from the latest annual cross-sectional soundings.

There are special records of the heights of groynes and other river works which also contain information on their length, design etc.

The bed levels of the lanes across the forelands can be deduced from the elevation figures on the map and from supplementary levelings and measurements.

Determining the Chézy coefficient

The Chézy coefficient values, hereinafter called "C values" must be known before the Chézy formula can be used to calculate stream lane discharges. Minor bed, groyne panel, and foreland stream lanes all have different C values. It should be noted that C values are not constants. They vary as a function of the values of discharge and water depth.

Of course there are no C values known from measurements for the very high river discharges for which the stream lane computations are being carried out. They can only be derived from the figures of former floods which were measured as accurately as possible.

Supposing that detailed information on the discharge and water level slope along the Rhine effluents is available for a certain flood and supposing that we have obtained, by surveying and measurement, a fair idea of the system of stream lanes, we can calculate the C values by carrying out what may be called "test computations" iteratively. This method of determining C values is very laborious. Besides, the data we have on discharge, water levels and current patterns are seldom accurate enough to enable us to determine the C values for the minor bed, the groyne panel and foreland lanes separately by the method described. Therefore C values that seem reasonable are assumed for the test computations described for foreland and groyne panels. This approximation has no appreciable effect on the results. A more detailed explanation of

the way in which the C values for minor bed, groyne panels and foreland are found for the characteristic high discharges is given below.

C values for foreland lanes

Observations were made of a number of floods in the last few years to obtain more information on the C value for forelands. Measurements carried out in a foreland during floods gave a roughness figure of $k \approx 4$ cm.
However, the velocity of about 1/2 m/s in this particular foreland was very much lower than the velocity that can be expected in forelands during more serious floods. The foreland lanes investigated were relatively smooth. As a rule, forelands are more irregular. Accordingly $k = 6$ to 7 cm was chosen as the basis for the C values for forelands.

The value for the forelands as a function of the water depth is given by the formula $C = 18 \log \frac{12h}{k}$, in which h = water depth and k = bed roughness (Nikuradse).

The C value for forelands during very high discharges is seen to vary between 45 $m^{\frac{1}{2}}/s$ and 55 $m^{\frac{1}{2}}/s$, depending on the water depths over the forelands expected along the various Rhine effluents during very high discharges and bearing field conditions in mind.

The method of determining the C values for forelands described above was adapted both for the test computations of observed floods referred to earlier and for the final computation of the characteristic discharges.

C values for stream lanes running across groyne panels

We have no figures obtained from tests in the field that would give us some idea of the C values for groyne-panel lanes in the Netherlands Rhine effluents. It is reasonable to suppose that these values will be smaller than those for forelands. The crests of the groynes are virtually level with the forelands. The groynes act as broad-crested weirs. There are deceleration losses below each groyne. Since the groynes are fairly close together as a rule, the lane will offer fairly high resistance. The profile of a groyne-panel stream lane is small compared with that of minor bed and forelands.
The average discharge across the groynes is less than 5% of the total discharge.
The C value for groyne-panel lanes is taken as 40 $m^{\frac{1}{2}}/s$ for test computations and final computations alike.

This assumption cannot be verified without extensive tests. The assumption seems reasonable, since the effect of any error cannot be very great because the groyne-panel lanes account for such a small proportion of the total flow.



Flow over an approach road to a brickyard



Flow over a summerdike

As is pointed out later in the test computations, the C values for the minor bed are calculated after the C values for groyne-panels and forelands have been determined. An error in the assumed C value for the groyne-panels will cause an opposite error in the C value for the minor bed.

C values for the minor bed

The minor bed C values for the characteristic discharges of the various Rhine effluents is usually determined by the following method. We have sufficient data on water levels, stream lane widths and bed levels for the lower and mean discharges to enable us to compute C as a function of discharge or water depth. First of all, we divide each Rhine effluent into stretches of from 5 to 10 km.

The relation between the C value for the minor bed and the water depth for a certain stretch is shown in Figure 4. This curve would have to be extrapolated over a considerable length before it could be used to determine the C values for the minor bed during the characteristic extreme discharge. Reasonable extrapolation is possible within such

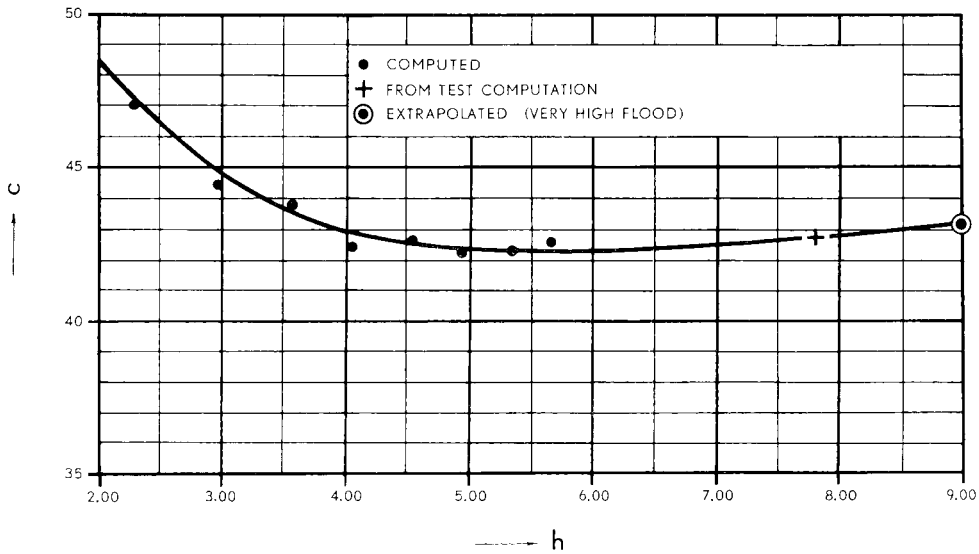


Figure 4. C values as a function of water depth

wide limits that the C values obtained will also vary widely. To secure something more definite, we must determine by means of test computations the C value for one or two spates concerning which we have sufficient data, using:

$$\text{C foreland} = 18 \log \frac{12h}{k}$$

$$\text{C groyne-panels} = 40 \text{ m}^{\frac{1}{2}}/\text{s}$$

We introduce the C values for the minor bed as unknowns and determine them in such a way that the computed water-level curve tallies as nearly as possible with the observed water levels.
The C values for the minor bed during the characteristic high discharges found by extrapolation are seen to vary fairly widely from stretch to stretch.
The values $C_{min} = 40$ m$^{\frac{1}{2}}$/s and $C_{max} = 65$ m$^{\frac{1}{2}}$/s may be regarded as limits.

Cross dikes in the forelands

As already stated there are many dikes and roads in the forelands and they must be allowed for in the computations; we treat them as if they were broad-crested weirs, but before we can do so we must know their coefficients of discharge. The Hydraulics Laboratory at Delft has carried out investigations to determine the resistance of broad-crested weirs as a function of the upper and lower water levels and as a function of the heights of the weirs with respect to the land above and below them.

Although the dimensions of dikes in forelands may differ greatly, only one type of dike can be taken into account in stream-lane computations.
The average crest width was found to be 3 m and the average slope on either side 1 in 4.

The laboratory tests gave a number of graphs showing the relations between $\triangle E_1$, E_1, d and Q/Q_v (see list of Symbols). How to use these graphs and the equation and formula given below is explained in par. 7.

The relation between E_1 and $\triangle E_1$ for $Q/Q_v = 0.8$ for various values of d is shown in Figure 5.

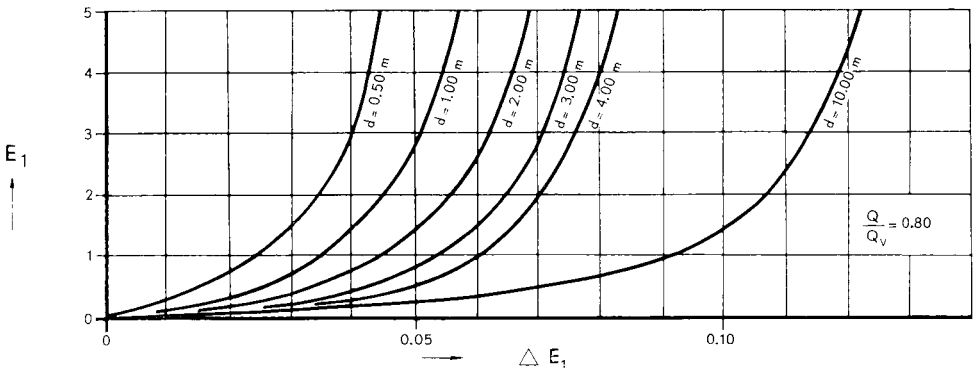


Figure 5. Relation E_1, $\triangle E_1$, Q/Q_v and d

The relation between the head E_1 above the weir and the discharge coefficient (m_v) of the weir during critical flow was determined from the same set of tests enabling us to determine Q_v using the equation

$$Q_v = \frac{2}{3}\sqrt{\frac{2}{3}g}.b.m_v.E_1^{3/2}$$

The flow over the weir was perpendicular throughout the experiments, whereas actually the flow across the majority of the cross dikes in the foreland is oblique. Nevertheless, for theoretical reasons we can ignore the velocity component parallel to the crest of a weir with oblique flow and treat it as one with perpendicular flow[1]). The resulting method can only be used if the weir is long enough to prevent boundary effects from affecting the total discharge to any appreciable extent.

Transverse gradient of water surface on bends

It should be noted that the method makes no allowance for the effect of centrifugal force on the water particles on bends. Therefore the level will be higher on the concave side of a bend and lower on the convex side than the level our calculations will give. This point will obviously have to be dealt with separately when calculating the heights of dikes.

Par. 6. The whys and wherefores of the introduction of electronic computation

Stream lane computation is outlined in par. 3.
Since the figuring is done by iterative methods, it is very time-consuming. How much work it involves will be apparent if we consider that the calculations for a stretch of river from 5 to 7 km long calls for the iterative solution of about 60 equations with 60 unknowns.

The need to speed up the computations prompted investigation of the possibility of utilizing an electronic computer for the purpose. After a brief provisional study the Institute for Applied Mathematics in Delft Technological University concluded that digital computers are eminently suitable for handling them.

The calculations are progammed in such a manner as to enable river experts to check the process at any time.

Par. 7. Computations

The method of computation will now be explained point by point, by using the pattern of stream lanes given in Figure 6 which was primarily intended for computing

[1]) Dr. ir. J. C. Schönfeld. Discharge of long and very long weirs. I.A.H.R. The Hague, Netherlands 1955.

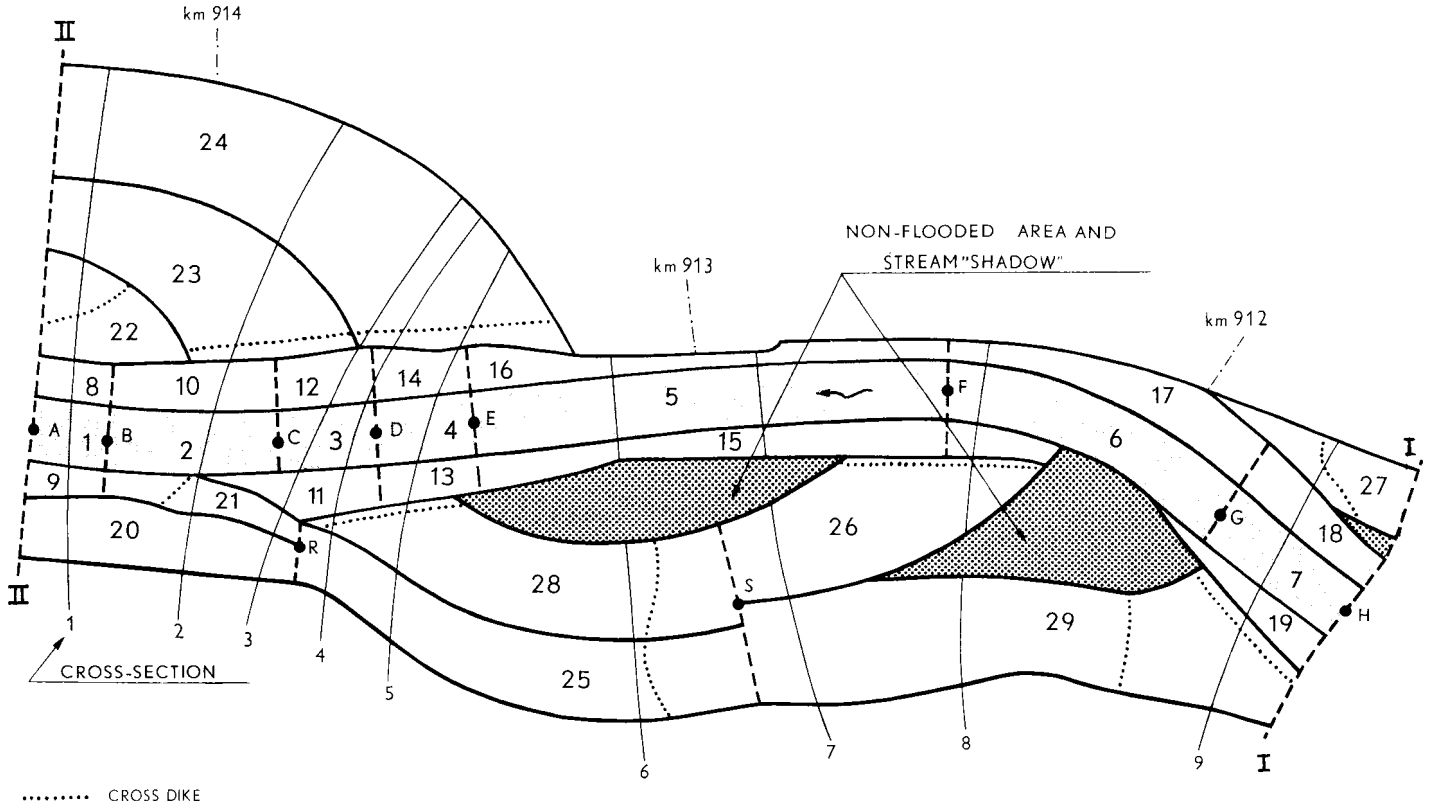


Figure 6. Pattern of stream lanes

the characteristic extreme flood for the stretch represented in Figure 3. Although from 5 to 7 km has proved the best length of stretch for computation, for simplicity's sake the stretch taken for the example is only 3 km long.

1. The pattern of stream lanes is such as takes into account the general situation, cross-dikes, stream-"shadows" of brickyards, etc.

A distinction is made between minor bed lanes, groyne-panel lanes and foreland lanes. The minor-bed lanes are numbered first, starting from the downstream end (Nos. 1 to 7). The groyne-panel lanes are numbered, also from the downstream end (Nos. 8 to 19). The foreland lanes are divided into three categories, viz. with no dike, with one dike and with two dikes.

Category 1 is numbered first, also from the downstream end (No. 20), then the second category (Nos. 21 to 27) and lastly the third category (Nos. 28 and 29).

The lines I-I and II-II are lines of equal water level at the upstream and downstream limits of the stretch, tallying with the water levels at the points H and A in the minor bed. Points A to H are called minor bed "nodes". Each lane is bounded by two nodes. The general situation may lead to the adoption of what is called a "suspended node" i.e. the end of a line of equal water level in the foreland that cannot be linked directly to a point in the minor bed (points R and S). Schematization of the foreland lanes involves limiting their length so that they do not traverse more than two dikes.

2. The depth of the bed, the length and width of lanes and the length and height of cross dikes are punched on special cards and fed into the computer.

3. We know the water level at A from the outcome of the calculations for the preceding stretch and we estimate the levels at the other nodes.

We start from the lowest point and work upstream when dealing with the process as a whole, but work from the highest point downstream when doing the calculations for each separate stream lane.

4. The discharges of minor bed and groyne-panel lanes, and of foreland lanes without dikes can be determined direct by using the Chézy formula, which, for the purpose in hand, has been given the following form:

$$\triangle H = H_y - H_x = \frac{Q^2 L}{B^2 \left(\frac{H_y + H_x}{2} - P \right)^3 . C^2}$$ (See also Figure 7)

5. Direct calculation of the discharge of foreland lanes with cross dikes is impossible because of the extra resistance offered by cross dikes. The first step, therefore is to estimate the discharge. We then use the estimated discharge Q and the other figures for the foreland lane to ascertain from graphs the total loss of head throughout the lane (see Figure 5). Next we must adjust the estimated discharge Q and repeat the calculation until the computed loss of head agrees with the loss derived from the estimated levels at the nodes.

The procedure is described below and illustrated in Figure 7, which also clarifies the symbols.

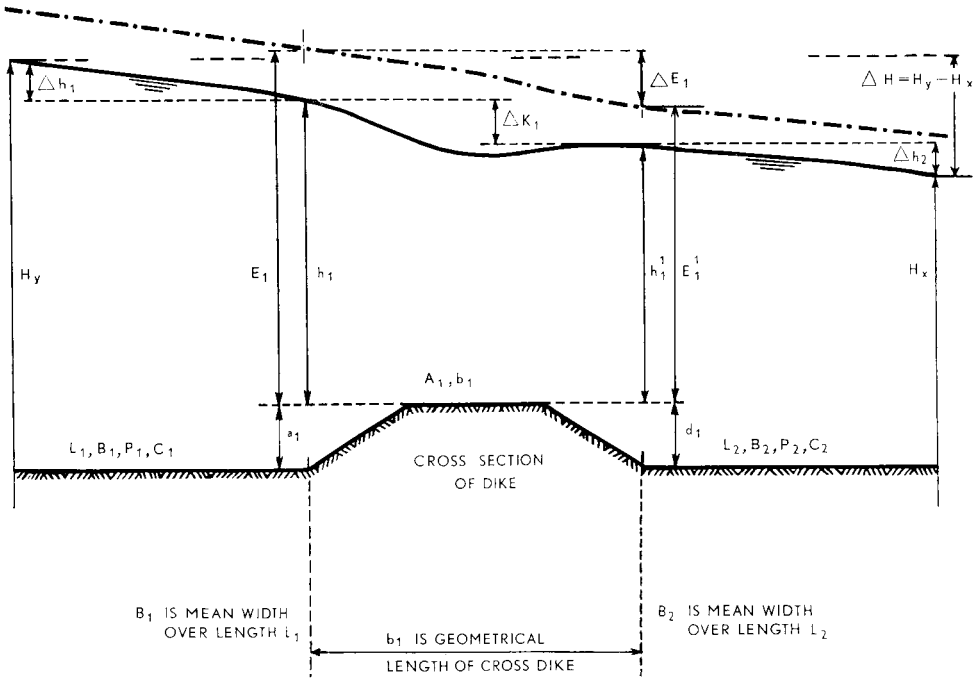


Figure 7. Longitudinal section of foreland stream lane

a. Estimate the discharge Q by using the formula given in point 4. Ignore the resistance of the cross dikes for the moment.
b. As stated above, we work downstream when doing the calculations for separate lanes. Using estimated Q, we then calculate the loss of head in the section above the first cross dike by solving:

$$\triangle h_1 = \frac{Q^2 L_1}{B_1^2 (H_y - P_1 - \frac{1}{2} \triangle h_1)^3 \cdot C_1^2}$$

c. We then calculate the water depth above the crest of the first cross dike by solving:

$$h_1 = H_y - A_1 - \triangle h_1$$

d. Next we calculate the head above the crest (with a correction for oblique flow) at the point named in c by solving:

$$E_1 = h_1 + \frac{\alpha v_1^2}{2g} = h_1 + \frac{\alpha Q^2}{2g \cdot b_1^2 (h_1 + a_1)^2}$$

e. We determine the discharge coefficient m_v for critical flow over the cross dike as a function of E_1, using the equation given in par. 4.
f. We can ascertain the critical discharge Q_v for head E_1 by solving:

$$Q_v = \frac{2}{3}\sqrt{\frac{2}{3}g}.b_1.m_v.E_1^{3/2}$$

g. We can now calculate the ratio Q/Q_v, i.e. the ratio between estimated discharge and critical discharge over the cross dike.
h. If the result is $Q/Q_v > 1{,}0$, the discharge has been overestimated and we must start calculating again at point b, with the Q_v from the equation in f as the new estimated discharge.
If $Q/Q_v \leqslant 1{,}0$ we must ascertain the loss of head $\triangle E_1$ over the cross dike as a function of Q/Q_v, E_1 and d_1, from the graphs (Figure 5).
i. We can determine the water level relative to the crest at a point just below the cross dike by solving:

$$h_1^1 = E_1 - \triangle E_1 - \frac{\alpha Q^2}{2gb_1^2 (h_1^1 + d_1)^2}$$

j. Now find the loss of head over the first cross dike by solving:

$$\triangle K_1 = h_1 - h_1^1$$

k. Then we must find the loss of head over the stretch of foreland covered by the lane below the first cross dike by solving:

$$\triangle h_2 = \frac{Q^2 L_2}{B_2^2 (H_y - \triangle h_1 - \triangle K_1 - \frac{1}{2}\triangle h_2 - P_2)^3 . C_2^2}$$

There is only one cross dike in the lane in the figure. In practice there may be two and theoretically a number.
In such cases we must continue the procedure until we have found $\triangle h_3 \ldots . \triangle h_{n+1}$ and $\triangle K_2 \ldots . \triangle K_n$, n being the number of cross dikes in the lane.
l. Lastly, we check the outcome by calculating the difference between the loss of head derived from the levels at the nodes and the total loss of head calculated; for this we use the term

$$(H_y - H_x) - (\Sigma\triangle h_n + \Sigma\triangle K_n)$$

If the difference is zero or negligible we estimated the discharge correctly.
If necessary, we must make a second estimate of Q and do the calculation over again.
The method of estimation adopted will depend on whether the flow across one or more of the cross dikes is critical or non-critical. A simple method known in mathematics as "regula falsi" will enable us to calculate the discharges along all the foreland lanes for the assumed drop in water levels.
6. The water levels at the suspended nodes are estimated and the estimate is corrected by the computer in such a manner that

$$Q_{20} + Q_{21} = Q_{25}$$
$$\text{and } Q_{25} + Q_{28} = Q_{26} + Q_{29}$$

7. To satisfy to the continuity condition, the same quantity of water must pass each of the cross sections 1 to 9 in Figure 6. This is evidently the total discharge Q_T. For cross section 1 we have:

$$Q_{20} + Q_9 + Q_1 + Q_8 + Q_{22} + Q_{23} + Q_{24} = Q_T$$

As a rule, the sum of the computed lane discharges will not equal the total discharge and the lane discharge will have to be corrected by using a certain equalizing factor. This factor is defined as the derivative from Q to $\triangle$ H, which can be gathered from $\triangle H = R \cdot Q^2$ in which R is the resistance.

The computations are carried out on the assumption that the changes are so small that the resistence R will not need modification at first.

The quadratic equation does not apply when there are cross dikes in the lanes. An error is acceptable if the flow over the cross dikes is quite non-critical. If the flow is almost or entirely critical ($Q \geqslant 0.9\ Q_V$), the equalizing factor is taken to be zero.

The derivative $\dfrac{dQ}{d(\triangle H)} = \dfrac{1}{2R \cdot Q} = \dfrac{Q}{2\triangle H}$

The equalizing of lane discharges being a matter of proportion, the equalizing factor can simply be taken as $f = \dfrac{Q}{\triangle H}$

Each lane has its own Q, $\triangle$ H, and therefore it also has its own f. The correction of the computed discharges of H for various lanes is $\triangle Q = \dfrac{f}{\Sigma f}\ (Q_T - Q_C)$, in which $Q_T - Q_C$ is the difference between Q and the sum of the computed discharges of the lanes transversed by the section concerned.

8. The water levels at the nodes will seldom be found to tally with the equalized discharges when all the lane discharges have been equalized. The loss of head in the minor bed lanes can be determined from the equalized discharges of the minor bed lanes direct, by using the equation in point 4.

The water levels at the minor bed nodes are then known. The levels at the suspended nodes have to be adjusted again to fulfil the condition that

$$Q_{20} + Q_{21} = Q_{25}$$
$$\text{and } Q_{25} + Q_{28} = Q_{26} + Q_{29}$$

9. The new water levels at the nodes are now known, so we can determine the discharges of all the lanes anew and ascertain their equalizing factors.

10. If necessary, we must repeat the process several times, until the results of the computation without equalization approximate the sums of the discharges in the cross sections and the total discharge.

11. We then equalize for the last time and get the lane discharges, the water levels at the nodes and the water levels above and below each cross dike. The computer will give us these figures.



Water flowing over a cross dike from the right to the left

12. The Rivers Department uses these figures to draw isometric water level lines with which to check the first draft of the stream lane pattern. If the lines in this pattern link up or nearly do so, the computation of the stretch under consideration may be regarded as completed. If not, we must redraft the pattern, readjust the data and do all the calculations over again. In practice one or two repetitions will very likely be enough.

Some 300 km of river had to be dealt with in the manner described to ascertain the characteristic extreme flood.

The methods used to determine the C values for the extreme flood are explained in par. 5. Evidently, these calculations must be done before those for the flood itself.

When we know the water levels at the minor bed nodes and when we have decided on the C values we shall assume for the groyne panels and foreland, we can determine all the lane discharges in the manner explained in the foregoing except those of the foreland lanes adjacent to suspended nodes. The water levels at these nodes have to be determined to fulfil the condition

$$Q_{20} + Q_{21} = Q_{25}$$
$$\text{and } Q_{25} + Q_{28} = Q_{26} + Q_{29}$$

The discharge of minor bed lanes is determined in each cross section by subtracting from the total discharge the sum of the discharges of the lanes traversed by the section, viz.:

$$Q_1 = Q_T - Q_{20} - Q_9 - Q_8 - Q_{22} - Q_{23} - Q_{24}$$

The C value of the minor bed can now be determined by using the modified Chézy formula. In this type of computation, too, the isometric water level lines are used to check the acceptability of the draft stream-lane pattern.

We shall not do these "test computations" for the entire river, but since we shall be doing test computations for some critical reaches near the bifurcations for more than one flood, the aggregate will cover about 200 km of river.

Par. 8. Final remarks

As a rule, the computerizing of a certain calculation calls for a critical approach to the formula on which the calculation is based. Certain approximations may have been introduced or adjustments carried out to make the process also amenable to treatment by convential "longhand" methods.

As stated in the Introduction, the computation method described was developed to make it possible to compute the water level at any spot in the river where the characteristic design discharge happens to obtain.

Since the height of the crest of the main dikes is based on the computed water levels, the River Board reponsible for a certain effluent must of necessity see to it that no project is carried out in any part of the river bed if it is likely to cause the water level to rise.

Whenever a project is proposed which would cause the water level to rise, efforts must be made to provide some compensation, for instance, by widening the rest of the river bed by lowering forelands, eliminating cross dikes, etc.

First the compensation required is estimated, then the water levels for the characteristic design discharge under the new circumstances (viz. the proposed project and the compensation estimated) are calculated and the resulting water levels compared with the known water levels for the discharge being considered under present conditions (no project and no compensation).

If an agreement is not reached, the compensation must be re-estimated and the water levels re-calculated. In practice two estimations and two calculations will usually result in agreement being reached.

The River Board will only sanction the execution of a proposed project if the compensation calculated will actually be forthcoming.

If sufficient compensation cannot be provided, the River Board will either refuse to sanction the execution of the proposed project or will, in exceptional cases, make it conditional on heightening the main dikes to such a point as will ensure that the risk of their being overtopped is not increased.

Proposals that projects be carried out on the bed of a Rhine effluent are fairly frequent; consequently, the calculations described have also to be done fairly frequently. As a rule, the calculations will be done on a computer, but from time to time they will be done "longhand" to ensure that the technique of convential "longhand" calculating is not lost and to maintain the engineers' ability to judge the calculations done by computers.

List of symbols

E_1 upstream head in m in relation to height of crest of weir or cross dike

E_1^1 downstream head in m in relation to height of crest of weir or cross dike

$\triangle E_1, \triangle E_2 \ldots . \triangle E_n$ loss of head in m over the 1st, 2nd, nth weirs or cross dikes

Q discharge along stream lane in m3/s

Q_T actual total discharge in m3/s

Q_C computed total discharge in m3/s

$\triangle Q$ correction of computed lane discharge in m3/s

$$\triangle Q = \frac{f}{\Sigma f}(Q_T - Q_C)$$

Q_v the maximum discharge possible for the upstream head E_1 in m3/s

m_v discharge coefficient of weir for critical flow over crest

$a(a_1)$ height of weir in m in relation to level of land above it

$d(d_1)$ height of weir in m in relation to level of land below it

$b(b_1)$ length of weir in m

$A(A_1)$ height of weir in m above datum level

v_1 velocity in m/s above the weir

f equalizing factor ($f = \frac{Q}{\triangle H}$) in m2/s

g acceleration of gravity in m/s^2

α coefficient, dependent on velocity distribution

$\triangle K_1, \triangle K_2 \ldots . \triangle K_n$ loss of head in m over the 1st, 2nd, nth weirs

$h_1, h_2 \ldots . h_n$ water depth in m above the 1st, 2nd, nth weirs in a foreland lane in relation to height of crest

$\triangle h_1, \triangle h_2 \ldots . \triangle h_n$ loss of head in m over foreland lane above the 1st, 2nd, nth weirs

$\triangle H$ total loss of head in m in stream lane

H_y, H_x upstream and downstream water levels of stream lane, in m above datum level

$\frac{H_y + H_x}{2}$ mean water level of lane in m above datum level

$L(L_1)$ length of lane in m

$B(B_1)$ mean width of lane in m

$P(P_1)$ mean terrain level in m above datum level

$C(C_1)$ Chézy coefficient in m$^{\frac{1}{2}}$/s

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