DELFT UNIVERSITY OF TECHNOLOGY

AND

DELFT HYDRAULICS LABORATORY

PUBLICATION No. 92

ON DOMINANT DISCHARGE CONCEPTS FOR RIVERS

BY

A. PRINS AND M. DE VRIES

OCTOBER 1971

SERIES 1 : FLUID MECHANICS.

Group 18 : Hydromechanics on the movements of sediments.

Section 18.66 : Hydromorphology of riverbeds.

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On dominant discharge concepts for rivers

by

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Synopsis

Often the need is felt to schematize a river regime by one single discharge, the Dominant Discharge, and to use this discharge for prediction of changes in a riverstretch as a consequence of riverworks.
This article discusses some methods, as given in literature, to determine the Dominant Discharge and gives a way to compute this discharge by means of the hydrodynamic equations.

Résumé

Souvent il y a besoin de représenter un régime d'une rivière par un seul débit, le Débit Dominant , et d'employer ce débit pour la prédiction des changements d'une extension d'une rivière comme conséquence d'ouvrages de rivières.
Cet article traite quelques méthodes, comme données dans la littérature, pour la détermination du Débit Dominant et donne une manière de calculer ce débit en se servant des équations hydrodynamiques.

1. Introduction

The concept "dominant discharge" (D.D.) has been used widely for attempts to schematize a river regime into one single discharge. Few fundamental studies are reported on these concepts. In principle distinction has to be made between the definition and the determination of the D.D. The objective of the use of D.D. is clear: problems of river morphology are so complex that simplification is a must. It is also clear that the simplified regime can never replace the real regime as far as the reproduction of the morphological characteristics of a river reach are concerned. One aspect hardly treated in literature regards the question how to determine the D.D. for a river reach influenced by future conditions, in other words it concerns the question whether the D.D. (however defined) for an existing reach can be used for the design of the improvement of this reach.

This paper deals with a short resume of some existing concepts of D.D. (chapter 2) and considers the possibility of using the (non-steady) hydrodynamic equations for a realistic approach of the matter.
This approach is a logical consequence of the development of computational methods for the computation of bed-fluctuations under the influence of natural or artificial changes of the conditions for a river reach. The method gives a possibility of the combined use of mathematical and hydraulic river models, taking the advantages of each of those models and mutually compensating the disadvantages of each of those models.

In chapter 3 an example is given together with the results of an experimental check. The limitation in space of this paper naturally causes that only the main lines of the investigation can be reported. The paper summaries the study |1| carried out by the first author with collaboration of the second author.

2. Concepts of dominant discharge

In a recent article Ackers and Charlton |2| define a D.D. as follows:

> " Dominant discharge is the steady flow, probably lying within the range of imposed periodically varying flow that would yield the observed meanderlength, etc....."

Here, contrary to some earlier definitions the D.D. is linked to a selected parameter: the meanderlength. This implies correctly that the D.D. will have a different value for different parameters.

This knowledge together with the definition does not yield much information on the determination of the value of this D.D.

Originally D.D. has been introduced in combination with the regime-"theory" in order to extend the regime-"theory" to those rivers in which discharge and charge vary throughout the year |3| . Intuitively it was assumed that this D.D. corresponded to bankfull-stage and that it could be used for any parameter such as width, depth, slope, etc.

Although the assumption of bankfull-stage made it possible to establish the value of the D.D. for an existing channel by means of observations it was impossible to determine the D.D. for a channel still to be designed. Bankfull-stage has therefore been related to a frequency of duration |4| , a purely statistical approach which made it difficult to establish how far this could be extended to other situations. Moreover there is a contradiction in linking the D.D. to the bankfull-stage as soon as it is accepted that a D.D. differs for different parameters.

Other engineers used a different approach |5| based on the reasoning that the D.D. should be the discharge at which most of the formative work is done and should "therefore" correspond to that stage at which the bulk of the bedload is carried (see fig. 1).

Besides the fact that it is difficult to decide at which stage the bulk of the bed-load is carried (e.g. median, mean etc. in fig. 1C), this approach has the same disadvantage as the bankfull-stage as the determination of the D.D. is not based on a selected parameter in which one is interested.

The method can be mathematically expressed.

If the centre of gravity in fig. 1C is taken as an example the dominant depth is:

$$h_o = \frac{\int^T h.S\ dt}{\int^T S\ dt} \qquad (1)$$

NEDECO |6| has tried to use a method more or less similar to the one mentioned above, but based on a more sound physical reasoning.

NEDECO's basic assumptions can be written as:

a. $$\partial S_o / \partial x = 0 \qquad (2)$$

this means that with D.D. steady conditions are prevailing.

b. $B\int^T \partial z/\partial t \; dt = 0$ (3)

This indicates that the channel-bed has reached a dynamic equilibrium or in other words the bedlevel moves around an average value.
If now the bedload equation is written as

$$s = a.v^b \tag{4}$$

where b can be found from the actual bedload-function |7| then

$$\frac{\partial s}{\partial x} = \frac{\partial}{\partial x} \left(a.B^{1-b}.h^{-b}Q^b\right) = (1-b)\frac{S}{B}\frac{\partial B}{\partial x} - b\frac{S}{h}\frac{\partial h}{\partial x} \tag{5}$$

If it is now further assumed that for any discharge

$$\partial h/\partial x = \partial h_o/\partial x \tag{6}$$

it can be found from Eqs. (2), (3), (5) and (6) (if $\partial B/\partial x \neq 0$) that:

$$\frac{b_o-1}{h_o} \cdot h_o = \frac{\int^T (b-1)S \; dt}{\int^T b \; S.h^{-1} dt} \tag{7}$$

or in cases of high transport where b is nearly constant:

$$h_o = \frac{\int^T S \; at}{\int^T S/h \; dt} \tag{8}$$

an expression similar in character to Eq. (1).
The weakness in this derivation lies in (6).
For cases where $\partial B/\partial x \neq 0$ generally $\partial h/\partial x \neq \partial h_o/\partial x$.
An improvement on this method can, however, be made by taking the average

$$\overline{\partial h/\partial x} = \partial h_o/\partial x \tag{9}$$

$\overline{\partial h/\partial x}$ can then be found from measurements and with Eq. (2) and a bedload and discharge equation a D.D. can be determined which would reproduce a $\partial h_o/\partial x$ equal to the parameter $\overline{\partial h/\partial x}$ as found under regime conditions.

A great disadvantage of this method is that although a correct D.D. is found, linked to the parameter $\overline{\partial h/\partial x}$ not much can be done with it as it is only valid for the existing situation for which the measurements have been carried out, but is generally not valid for a future situation. If a D.D. is required for the purposed of establishing the $\overline{\partial h/\partial x}$ in the future, $\overline{\partial h/\partial x}$ will have to be predicted by means of a computation. By means of a hydrodynamic computation $\partial h/\partial x\ (x,t)$ (chapter 3) can be found, (with certain restrictions) and thus also $\overline{\partial h/\partial x}$.
Instead of $\overline{\partial h/\partial x}$ also another parameter could be taken if more apropriate such as velocity, depth or bedlevel.
Note that $\overline{\partial h/\partial x}$ is still a function of x and therefore the D.D. is strictly speaking valid for one cross-section only, which means that in hydraulic models the D.D. should be established for the cross-section considered to be the most important one.

3. Hydrodynamic approach

The idea of schematizing the river-regime into one (or at any rate a few discharges) is attractive for mobile bed river-models as it is usually not easy to obtain sufficient similarity for an entire regime. Moreover technical and financial implications of using the entire regime are avoided by schematizing the regime. It is not obvious to apply the hydrodynamic equations for schematization as the equations for the sand movement are vaguely known. Moreover, in many cases the number of equations is smaller than the number of dependent variables. In the case, however, in which the banks are fixed, four dependent variables: depth (h), flow velocity (v), bedlevel (z) and sediment transport (S) under the influence of the discharge Q(t) are linked by four equations viz. the equations of motion and continuity for water and sand. The well-known equations for the water do not give any difficulty. The equations of continuity for the sediment

$$\partial z/\partial t + \partial s/\partial x = 0 \tag{10}$$

is reasonably accurate if the variation of sediment in suspension can be neglected. As has been shown earlier |8,9| the bedlevel can be computed if it is assumed

$$s = f(v)$$

i.e. if the variations of roughness and mean grain-size can be neglected.

Both approximations have restricted validity. The effect of grain-sorting can in principle be introduced in Eq. (10) as it can be shown easily for each fraction i it holds

$$\partial s_i/\partial x + p_i^{(\prime)}\partial z/\partial t + \delta\partial p_i/\partial x = 0 \qquad (11)$$

if δ is the depth of movement (supposed to be constant, which is consistent with the assumption of constant roughness), and p_i represents the sieve fraction of diameter D_i. Further $p_i^{(\prime)} = p_i$ in the case of sedimentation and $p_i^{(\prime)} \neq p_i$ (and to be given) in the case of erosion. Computation for each portion separately is hampered by an incomplete knowledge of $s_i = f_i(v)$. This problem being a topic of current research forces to neglect grain-sorting for the time being. As has been shown earlier |8,9| , the three celerities (c) govering the problem are linked by

$$c^3 + 2cv^2 + (gh - v^2 + gdf/dv)c - vgdf/dv = 0$$

This cubic equation is plotted in a dimensionless form in figure 2. This figure clearly demonstrates that for practical problems (low Froude numbers, say $F < 0.6$ and relatively low transport intensities s/q) it yields $|c_{1,2}| \ll c_3$.
Thus the bedlevel varies much slower than the waterlevel, which reduces the problem considerably as the watermovement can be assumed quasi-steady if the bed-movement has to be computed. In practice it is attractive not to apply the method of characteristics for the computation of bedlevel- variation. Different methods are available |8| to solve the equations for a given set of boundary conditions.

The mathematical model referred to above can be used to schematize the regime of a river for a hydraulic model. Some characteristics of the two models are:

a. The mathematical model only gives overall-values of h, v, s and z but the variation with time can easily be obtained.
b. The hydraulic model easily gives information of the parameters across the river; variation with time is more difficult to obtain.

As explained in the previous chapter a D.D. can in principle only be established for one parameter and for one cross-section. For rivers in which navigation is important this parameter could be the minimum depth or the highest bed-level during the hydrological cycle. The mathematical model is able to indicate during what period the critical situation for the navigation is present and the hydraulic model can be used to study this problem in detail by applying the D.D.

i.e. the discharge reproducing this critical situation.
It goes without saying that this method can also be used for not (yet) existing circumstances. It therefore is, although not applicable for all riverproblems, thought to be more attractive than the methods briefly outlined in chapter 2.

4. Example.

In this chapter two examples will be given, one of a test-case carried out in a flume, the second concerns a widening in the river Waal one of the Dutch Rhine branches.

The procedure can shortly be described as follows. In both cases a selected parameter (ξ) will be computed as a function of place and time according to the method as given in chapter 3. The time average of this parameter is then determined for a certain cross-section and compared with the results of a number steady flow computations. By means of this comparison it is possible to establish the value of the constant discharge producing the same ξ as found under regime conditions; this by definition is the D.D. for this parameter ξ. The procedure is shown in fig. 3 in which for ξ is taken the bedlevel z. For the <u>test-case</u> the results are compared to the values found in the flume. The example for the actual <u>rivercases</u> has been over-simplified, it is therefore difficult to compare the results with those found in the actual river. The similarity, however, is sufficient to use the D.D. found with some faith for a more detailed hydraulic model.

Flume

The first example concerns a constriction in width in a flume as given in fig. 5. In this flume, which had a bed of bakelite, three regimes have been run as given in fig. 6. For each discharge the amount of sediment fed into the upstream end of the flume corresponded with equilibrium conditions for that discharge and the average slope of the channel bed; this was established by means of some preliminary measurements. From these preliminary measurements it appeared that the Meyer-Peter and Müller bedload function gave good results provided a few modifications were introduced regarding roughness and ripplefactor. As parameter ξ the average bedlevel in the constriction has been taken. In figure 7 the results of the steady flow computation are given, and in table 1 the results of the computation for regime conditions together with the D.D. that follows from the comparison between both results. In fig. 7 also the results of some steady flow measurements are given which show that a reasonable agreement with the computation exist, the D.D. found in this way therefore should give good results; a further proof of this is given by fig. 8 where the results of measurements under regime conditions are given together with the value as found from the D.D.

Table 1

RUN	$\bar{z}$ from transient flow computations in cm	Q_o in l/s
1	25.25	31.5
2	25.5	30.6
3	24.7	33.8

Waal river

In fig. 9 an oversimplified schematization of the river Waal near Nijmegen is given. The jump in bedlevel due to the widening has been computed according to the method indicated in chapter 3. For this purpose an average regime has been assumed; the results of the computations are given in fig. 10. It can be seen that the average jump in bedlevel can be taken as 0.5 m.

In fig. 11 the results of a steady flow computation are given; the jump of 0.5 m corresponds to a discharge of a little more than 1500 m^3/s which therefore can be taken as D.D. for this case.

Symbols

a	a constant
B	channelwidth
B_o	the suffix o indicates the value for D.D.
b	exponent, to be considered as a transport parameter
C	celerity
D	particle diameter
F	Froude number
f( )	function of
g	acceleration of gravity
h	depth of flow
I	slope
j( )	function of
p	sieve fraction by weight
Q	discharge
S	bed-load transport in volume per unit time
s	bed-load transport per unit width and time
T	time interval
t	time co-ordinate
v	flow velocity
x	co-ordinate
z	bedlevel
δ	depth of bedload movement
ζ	a parameter

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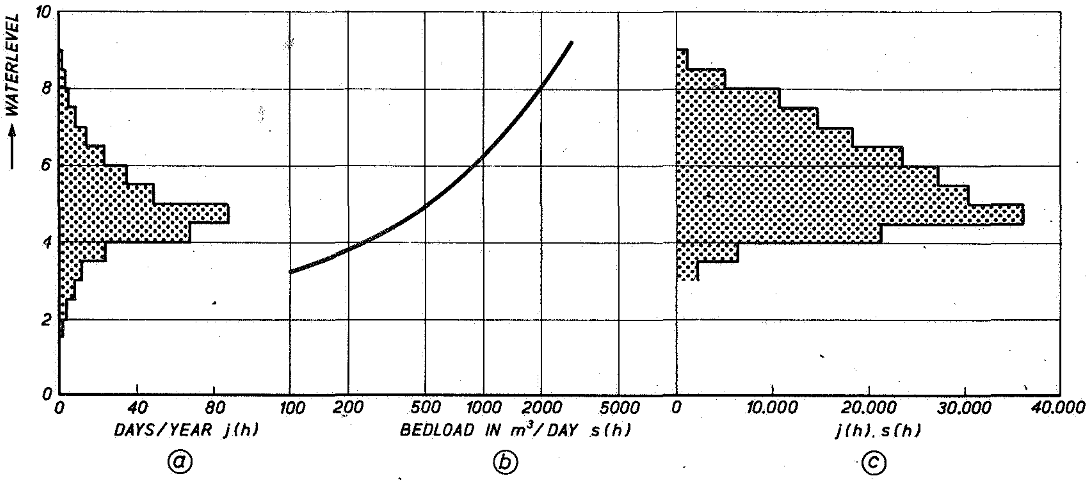

FIG. 1

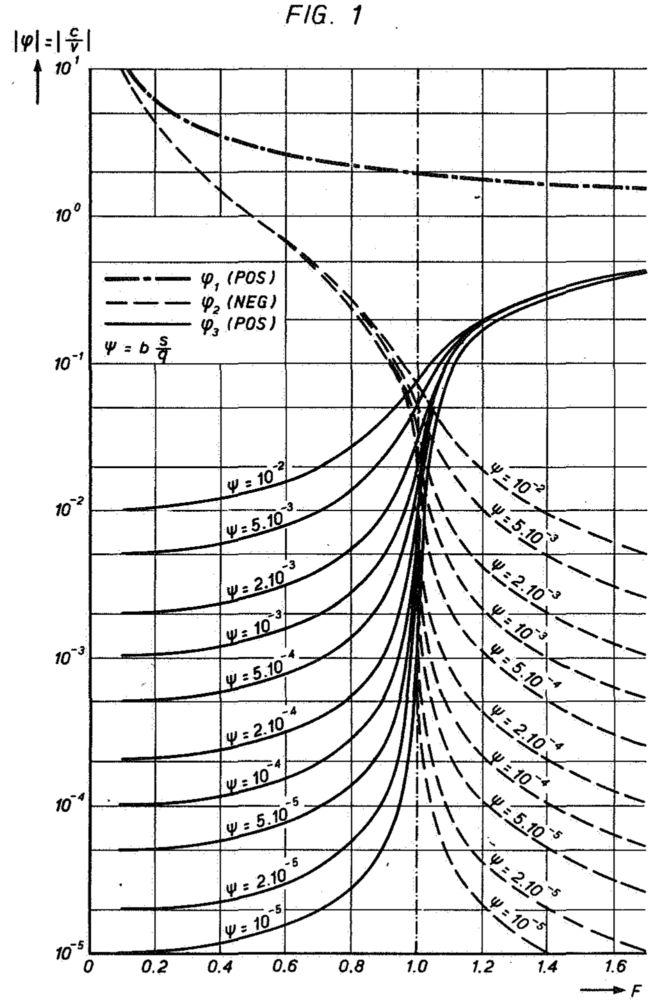

FIG. 2 RELATIVE CELERITIES

CÉLÉRITÉS RÉLATIVES

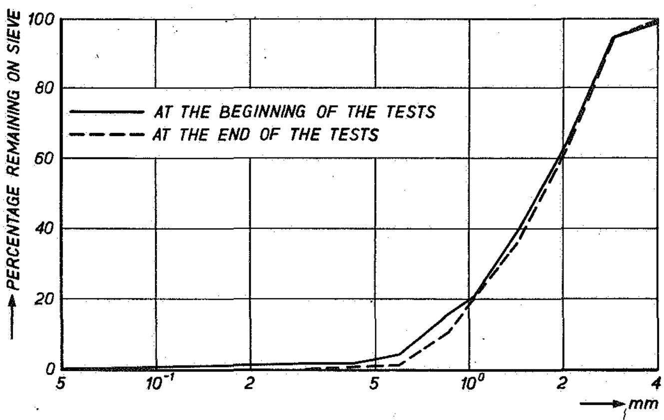

FIG. 4 SIEVING-CURVES OF BEDMATERIAL

COURBES DE TAMISAGE DES MATERIAUX DU LIT

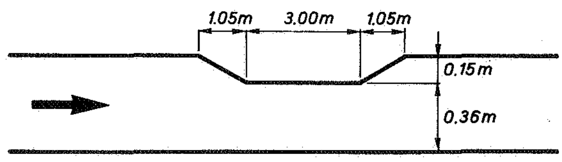

FIG. 5

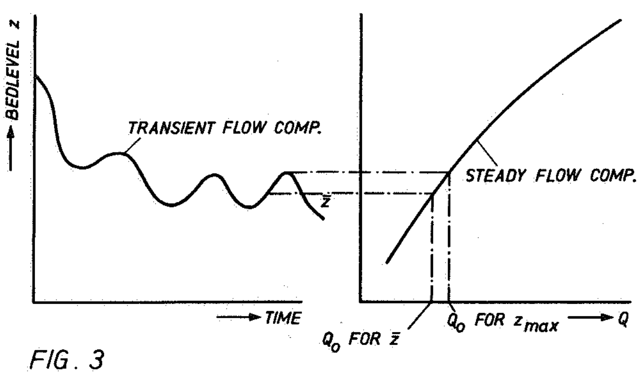

FIG. 3

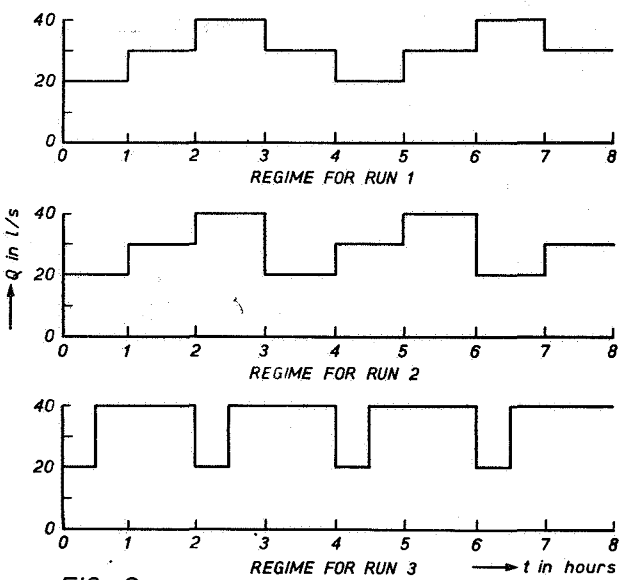

FIG. 6

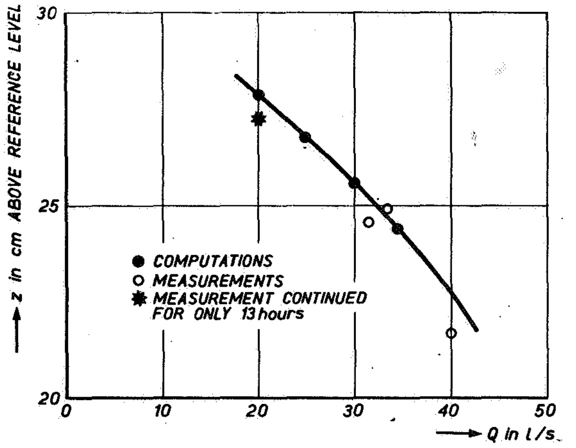

FIG. 7 BEDLEVEL IN NARROWING vs DISCHARGE, STEADY CONDITIONS

NIVEAU DU LIT DANS LE RÉTRÉCISSEMENT vs LE DÉBIT, CONDITIONS PERMANENTES

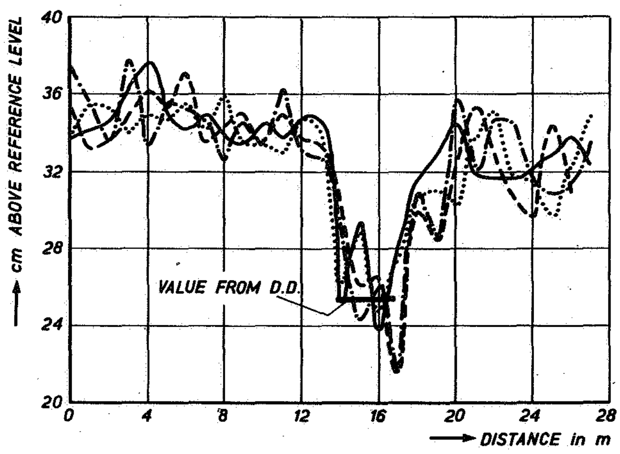

FIG. 8 BEDLEVELS UNDER REGIME CONDITIONS (MEASUREMENTS), RUN 2

NIVEAUX DU LIT SOUS REGIME DU DÉBIT (MESURÉ) ESSAI 2

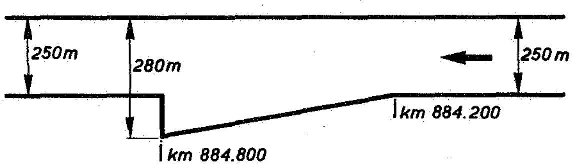

FIG. 9

RUN	$\bar{z}$ FROM TRANSIENT FLOW COMPUTATION IN cm	Q_o in l/s
1	25.25	31.5
2	25.5	30.6
3	24.7	33.8

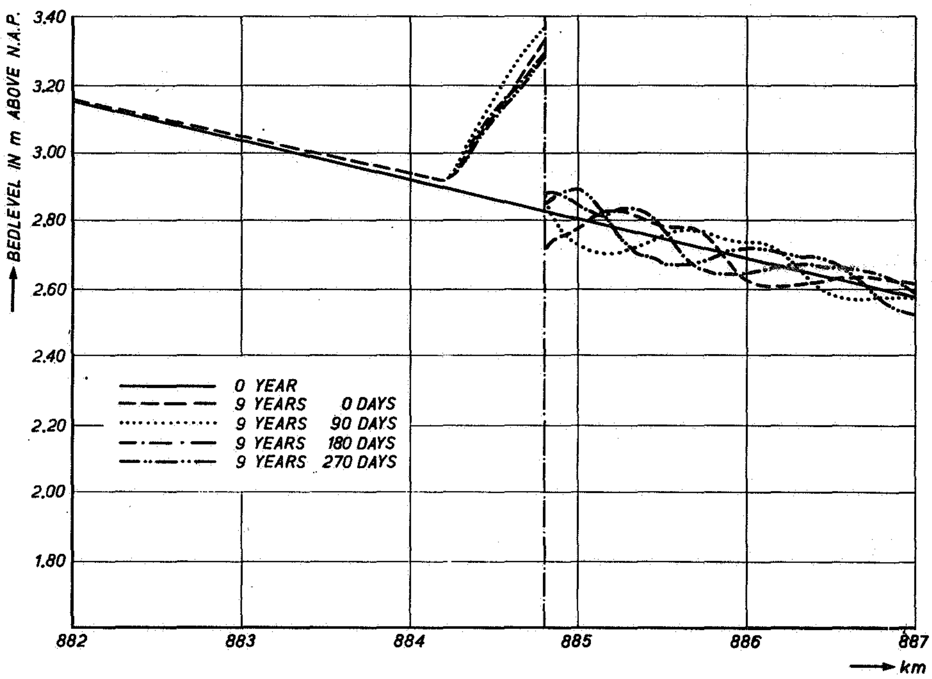

FIG. 10 COMPUTED BEDLEVELS RIVER WAAL NEAR NIJMEGEN

NIVEAUX DU LIT CALCULÉS DE LA RIVIÈRE WAAL PRÈS DE NIJMEGEN

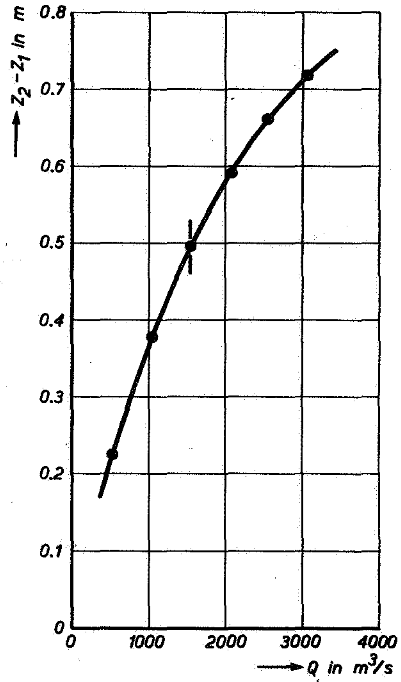

FIG. 11 COMPUTED JUMP IN BEDLEVEL vs DISCHARGE, STEADY CONDITIONS

RESSAUT CALCULÉ vs LE DÉBIT, CONDITIONS PERMANENTES