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STUDIES FOR INTAKE STRUCTURES OF FLOOD DETENTION BASINS TO BE ESTABLISHED ALONG THE UPPER RHINE

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Abstract

The regulation of the Upper Rhine between the cities Basel and Mannheim, further its subsequent development for energy production by construction of 10 hydropower plants caused a significant diminution of the natural flood retention areas and consequently an enhancement of the flood peaks. In order to counterbalance this effect three types of measures are considered: 1. special operation mode of the power stations, 2. construction of flood retention weirs in the by-passed stretch of the Rhine, 3. establishment of flood-retention basins beyond the flood control dykes (levees).

The author has been commissioned to elaborate design proposals for retention basins to be possibly erected in selected regions. Equations for calculation of the filling/emptying times are presented in the paper.

Besides, the headworks (inlet/outlet structures) of the basins have been tested under the guidance of the author in the Theodor-Rehbock Hydraulic Laboratory in order to obtain appropriate designs. The main requirements are: high discharging efficiency, reasonable openings as regards the movable gates, simple safety measures against scouring with respect to the two-direction flow, last but not least some aspects of environmental and landscape protection. The paper describes the results of the scale model tests and shows the design proposal. The discharge measurements in the model manifested a very good agreement with the equations.

Keywords: flood detention basins, hydraulical computations.

Nomenclature

a = constant in the differential equation (9)
A = area of the flood detention basin, (m^2)
B = free (net) width of the intake, (m)
C = project characteristic; see *Eq.* (4)
C^* = project characteristic related to an intake with an opening of unit width; see *Eq.* (23), (m$^{3/2}$s)

g = gravitational acceleration, (m/s^2)
H = design head related to datum elevation, i.e. max. height of overflooding of the flood plain and the intake floor, respectively, which also equals to the max. storage depth, (m)
= overall discharge loss coefficient (dimensionless)
Q = discharge, (m^3/s)
Q' = discharge, not comprising loss coefficient, (m^3/s)
v = flow velocity (mean value related to the cross-section), (m/s)
t = filling/emptying time, (s)
t_I = time in first phase of filling/emptying, (s)
t_{II} = time in second phase of filling/emptying, (s)
T = entire filling/emptying time, (s)
T_I = time required for the first phase of filling/emptying, (s)
T_{II} = time required for the second phase of filling/emptying, (s)
T^* = specific filling/emptying time, i.e. time required for filling/emptying through an intake of unit width, (s m)
x = variable introduced for solving the differential equation (8)
y = varying height of water level (over the datum elevation intake floor level), (m)

1. Historical Background

When studying the history of recent trends in the regulation and utilization of the River Rhine between the cities Basel and Mannheim, where the River Neckar joins it, four objectives can be differentiated and, accordingly four development phases, though partially overlapping each other, can be identified (*Fig. 1*):

1. *Flood Control.* Correction and stabilization of the main channel to prevent continuous silting up of the multi-branch and extremely meandering river and of its flood plains, having caused a steady rise of the flood levels and catastrophic inundations of the surrounding fields and settlements. The first protection works had been carried out by the famous engineer J. G. TULLA in 1818, who presented a complete plan for the regulation of the above defined 'Upper Rhine' (from Basel to Mannheim) to his Government in 1825. Since that time improvement of flood protection has been, by traditional methods, permanently in progress (channel regulation and alignment correction, lateral bed and bank protection structures, spurs, cuts of bends, blockages of branch channels, dredging and blasting, flood control levels and local dykes), for requirements on safety are evidently much higher at the present than more than a century ago.

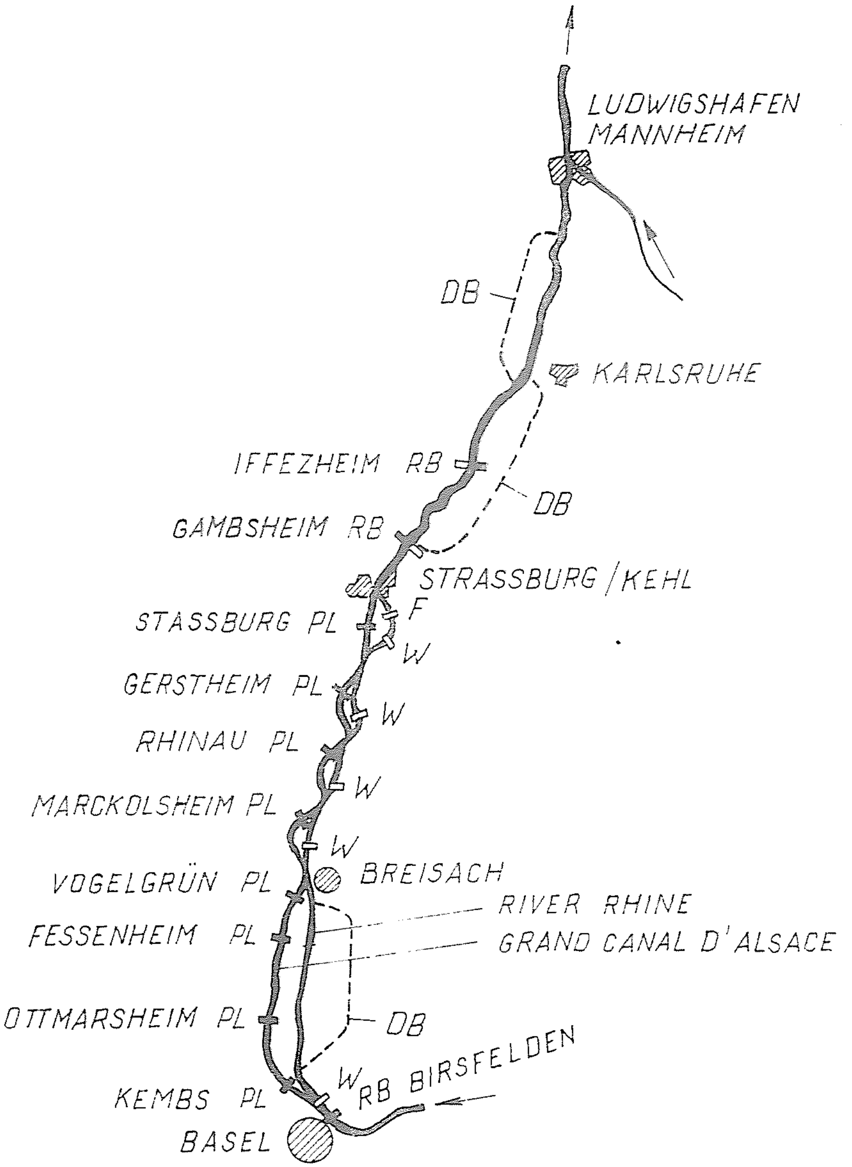

Fig. 1. The Upper Rhine development. Notations: RB = river barrage (weir, power station and ship lock), W = diversion weir, PL = power station and ship lock, F = flood retention and regulating weir at the town Kehl, DB = regions where sites for possible detention basins have been selected

2. *Navigation.* Criteria for regulation of the Rhine bed for navigation purposes are based on several international decrees directing the activity of the 'Central Commission for the Rhine Navigation' established as early as 1816. It was aimed to stabilize and maintain a channel within the mean-flow river bed, where uninterrupted two-way navigation, even for the largest fluvial vessels and later on for pushtows, was rendered possible, viz. related to a pre-determined low-stage water surface profile. An exact hydrological definition of this so-called 'equivalent low-water level' had been conceived by the 'Commission'. The 53 km long diversion canal (Grand Canal d'Alsace) by-passing the Rhine between Basel and Breisach, constructed mainly for hydropower utilization, became simultaneously the international waterway. Also the adjoining canalized stretch ending at the Iffezheim River Barrage provides the necessary nautical depth. Downstrearm of Iffezheim, down to St. Goar, however, the required channel depth can be attained by conventional training measures applied along some sections.

3. *Water Power Utilization.* The potential power of the Upper Rhine has been harnessed by construction of ten stations: four in the Grand Canal d'Alsace, four plants in short by-pass canals and two stream-bed stations.

4. *Restoration.* It became obvious already a few decades ago that the above fluvial projects accomplished for flood alleviation, improvement of navigation and power generation entailed several detrimental effects which must not be tolerated any more, and which, according to present scientific achievements and technological standards, can be counterbalanced or at least significantly reduced by economically feasible measures. Under the title 'restoration' these measures should be assembled. The unfavourable consequences of the Upper-Rhine Development can be briefly listed as follows:

(a) Owing to the regulation the vigorous erosion of the by-passed (abandoned) river bed results in a lowering of the groundwater table, dangerously affecting agricultural lands and forests in the region. This situation can be substantially improved by the establishment of a chain of groundwater controlling sills and weirs (some of them already implemented, others in construction).

(b) Since downstream the last barrage (Iffezheim), due to lack of bed-load supply from upstream, a progressive bed erosion develops, and, since the further canalization of the Rhine has been – at least temporarily – abandoned, the sediment deficiency is supplemented by a continuous supply of gravel excavated from quarries and dropped into the river by special dumping vessels. Up to the present this procedure proved to be efficient, however, in the author's opinion, possible long-term morphological consequences cannot be foreseen yet.

(c) By cutting off the previously inundated vast areas of the innumerable river branches from the freshwater an irreplaceable nature's treasure of fauna and flora is in danger of entire devastation. In order to avoid such irrecoverable ecological losses, the establishment of freshwater supply systems (i.e. intake works crossing the flood levees and distribution canal networks in these 'nature preservation areas') is necessitated.

(d) In consequence of the significant diminution of the natural flood retention areas, caused by the levees and the chain of the barrages, the flood waves are unfavourably distorted: their propagation velocity grows, their peak discharges and peak levels increase. In order to counterbalance these impacts, considerably endangering the densely populated regions and the industrial establishments, three types of measures are considered: firstly, a special operation mode of the power stations, viz. using one part of the pondage capacity for flood retention, based on forecasting; secondly, construction of flood retention weirs in the abandoned stretch of the Rhine; thirdly, establishment of flood-detention basins beyond the levees (*Fig 2*).

The author has been commissioned to elaborate a report and present recommendations on the design criteria for retention basins which could be possibly established within selected areas along the lower portion of the Upper Rhine.

2. Required Storage Volume of the Detention Basins

In order to elucidate the influence of canalization and flood protection measures on the flood peaks, and to emphasize the paramount importance of urgent actions an example, related to the last development phase only, should be recorded here: A flood discharge of about 5000 m^3/s having an exceedance probability of 0.5 per cent (i.e. a recurrence interval of 200 years), could have been conveyed between the levees safely in 1955. Nevertheless, at the present, the same rate of flow pertains to a recurrence interval of 40 to 45 years only.

On basis of thorough studies performed by the competent authorities and their scientific institutes the total storage volume necessitated to counterbalance the hazardous increment of flood peaks that could be identified as 240 million m^3, when anticipating, however, an efficient discharge forecasting and the possibility of inflow and outflow control at the intake structures of the basins.

Not only topographical, geological and hydrological conditions but also environmental and political aspects have determined the selection of

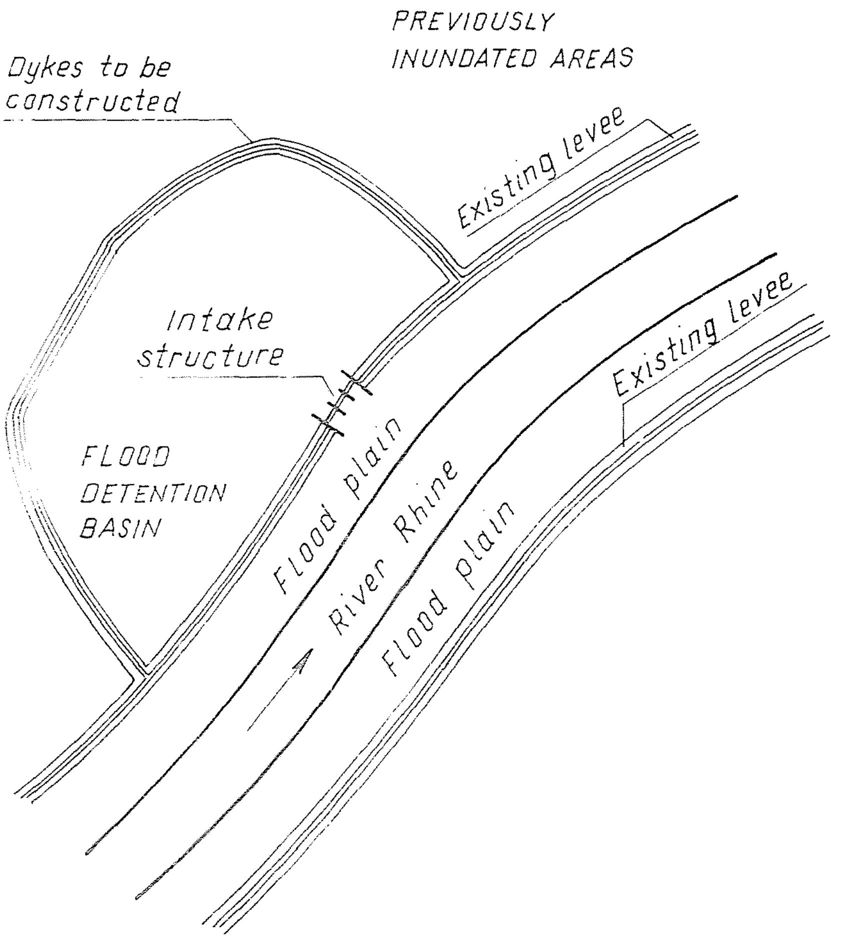

Fig. 2. Sketch of the general layout of a flood detention basin

potential basin sites, which have been finally distributed along the entire Upper Rhine. The author has been commissioned to deal with the basins to be erected possibly along the lower portion of the Upper Rhine and especially along the left-side levees of it. Yet results of these studies, hydromechanical computations and model tests can be, at least in a great part, adapted to planning other basins to be implemented along the upper portion of the Upper Rhine.

Considering the above mentioned aspects, however, with special regard to environmental demands, eight potential basins, providing for a total storage capacity of 66 million m^3, have been selected and delimited. Besides, when selecting, possibilities of storage of building materials and accessibility of the construction pits played important roles. In addition, preliminary plans, approximate cost estimates, and options for the sequence of implementation have been elaborated.

3. Design Criteria for the Intake Structures

Main aspects governing the location and structural solution of an intake are briefly listed as follows:

– In order to attain a high-grade efficiency of the basin for retaining peaks of extreme floods provision for control of both inflow and outflow is indispensable. During periods, namely, when low floods are overtopping the river banks, the structure must be kept closed, otherwise the basin will be partly filled up and only a part of its capacity will be available for the capture of the peak of a shortly subsequent high-flood wave. On the other hand, after a certain subsidence of the flood level the gates of the structure have to be quickly opened to make the basin ready for the reception of the next flood. On the basis of technological and economic aspects the application of full-height tainter gates are recommended.

– The above argumentation reveals that closing and opening have to be performed at specified Rhine water stages. A limitation for the opening schedule arises from the requirement that outflow must not cause a hazardous peak downstream. These limit river stages can be rigidly prescribed, or, for achieving a higher peak-diminution efficiency and safety respectively, they could be 'elastically' handled in connection with a computerized evaluation of a permanent forecasting.

– In order, however, to render possible, under unforeseen circumstances, the filling immediately when the river flow overtops the bank, the floor of the intake structure should be in level with the flood plain. Consequently, the construction of a sill at the intake has to be avoided. Also a sill would reduce the storage capacity.

– The location of the structure built into the levee has to be adjusted to the flood flow pattern so that the inflow flux develops symmetrically, without eddy zones, thus minimizing entrance losses which, accordingly, result in an economical design (min. free opening).

– The design has to meet hydromechanical and structural demands of the two-directional flow; thus, energy dissipation structures and protection measures against scouring, respectively have to be provided for at both sides.

– From environmental point of view an inundation uniformly spreading over the basin terrain is required, in order to avoid the formation of erosion pits and trenches.

4. Calculation of the Filling Time

For obtaining an explicit hydromechanical solution it should be anticipated that during the filling procedure the water level in the river does not change. The initial kinetic energy pertaining to the relatively low approach velocity towards the intake over the flood plain will be neglected. Furthermore, a fast opening of the gates is supposed, so that the river water flows through the completely opened intake into the empty basin. Accordingly, the water, when entering into the intake structure, accelerates up to the critical velocity, thus determining the inflow capacity (*Fig. 3*); thereafter, due to a further acceleration it falls into the basin. The hydraulic phenomenon is in principle equivalent to the overflow of a broad-crested weir.) Since the through-flow occurs at the limit value of the supercritical flow the rising water level in the basin has no backwater effect but until it is flush with the critical water surface within the intake, pertaining to the critical depth: $2H/3$. Consequently, from the Bernoulli equation, a constant filling discharge of

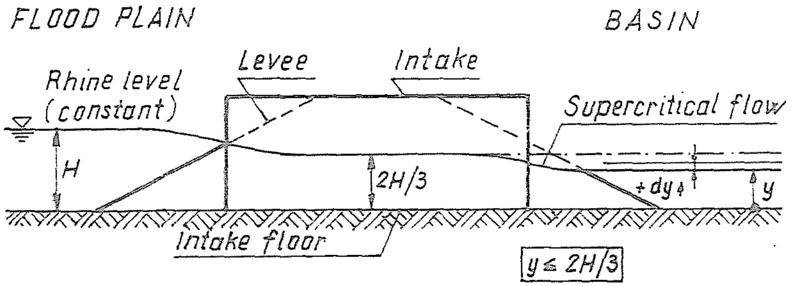

Fig. 3. Water surface profile during the first phase of filling, i.e. up to the limit depth

$$Q = \mu B \frac{2}{3} H \sqrt{2g\frac{H}{3}} = \mu B \sqrt{g} (2H/3)^{3/2} \tag{1}$$

results, where, strictly speaking also, the contraction coefficient μ varies in dependence of the water depth. For a first approach, however, an average coefficient, as a constant value being valid for the entire filling period can be estimated and substituted in *Eq.* (1). Denoting the area of the basin by A the time required for filling it up to a height of y equals to

$$t_I = \frac{Ay}{\mu B \sqrt{g} (2H/3)^{3/2}}, \tag{2}$$

and for the entire first phase, i.e. up to $y = 2H/3$

$$T_I = \frac{\sqrt{3}A}{\mu \sqrt{2g} B \sqrt{H}} = 1.732 \frac{C}{\sqrt{H}} \tag{3}$$

time is needed, introducing the 'project characteristic'

$$C = \frac{A}{\mu\sqrt{2gB}}, \tag{4}$$

which is constant for a selected project.

When the depth in the basin exceeds this limit, i.e. $y > 2H/3$, the through-flow is subcritical and, accordingly, the velocity and the discharge, respectively depends on the water level in the basin (*Fig. 4*):

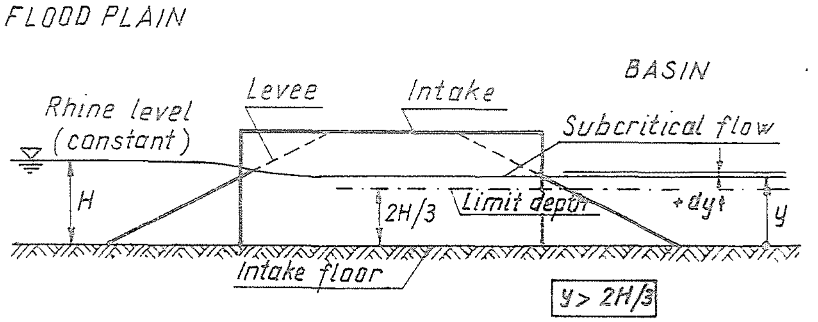

Fig. 4. Water surface profile during the second phase of filling, i.e. when the basin level exceeds the limit depth

$$v = \mu\sqrt{2g(H-y)}, \tag{5}$$

and

$$Q = yB\mu\sqrt{2g(H-y)}. \tag{6}$$

On the basis of the storage equation, $Q\,dt = F\,dy$, the filling time for the second phase is obtained by integration of

$$dt = \frac{A}{yB\mu\sqrt{2g(H-y)}}dy = C\frac{dy}{y\sqrt{H-y}}, \tag{7}$$

between the depths $y = 2H/3$ and $y = H$.

For solving the

$$\int \frac{dy}{y\sqrt{H-y}}$$

integral an appropriate substitution can be found by $x = \sqrt{H-y}$, producing the $y = H - x^2$ and $dy = -2x\,dx$ relations, and thus resulting in the equivalency

$$\int \frac{dy}{y\sqrt{H-y}} = -2\int \frac{dx}{H-x^2}. \tag{8}$$

From the theory of the differential equations it is deducible the solution of *Eq.* (8) and can be expressed as

$$\int \frac{dx}{a^2 - x^2} = \frac{1}{a}\text{Arc th}\,\frac{x}{a} = \frac{1}{2a}\ln\frac{a+x}{a-x}, \tag{9}$$

in the case when $|x| < |a|$, which is satisfied in our case, for $|x| = |\sqrt{H-y}| < |\sqrt{H}|$. Therefore the *Eq.* (8), by resubstituting $a = \sqrt{H}$ and $x = \sqrt{H-y}$, can be re-written as

$$-2\int \frac{dx}{H - x^2} = -\frac{1}{\sqrt{H}}\ln\frac{\sqrt{H}+\sqrt{H-y}}{\sqrt{H}-\sqrt{H-y}}. \tag{10}$$

Expressing it as a definite integral the solution of *Eq.* (7) between the limits $2/3H$ and y, is

$$t_{II} = -\frac{C}{\sqrt{H}}\left[\ln\frac{\sqrt{H}+\sqrt{H-y}}{\sqrt{H}-\sqrt{H-y}} - \ln\frac{\sqrt{H}+\sqrt{H/3}}{\sqrt{H}-\sqrt{H/3}}\right], \tag{11}$$

and by re-arranging and introducing the substitution $\phi = y/H$ the partial filling time during the second phase can be written as

$$t_{II} = \frac{C}{\sqrt{H}}\left[1.32 - \ln\frac{1+\sqrt{1-\phi}}{1-\sqrt{1-\phi}}\right]. \tag{12}$$

The entire length of the second phase is obtained by substituting $\phi = 1$ in *Eq.* (12):

$$T_{II} = 1.32\frac{C}{\sqrt{H}}. \tag{13}$$

When summing up the values from *Eqs.* (3) and (13) the *entire filling time* is

$$T = T_I + T_{II} = 3.05\frac{C}{\sqrt{H}} = 3.05\frac{A}{\mu B\sqrt{2gH}}. \tag{14}$$

A comparison of T_I and T_{II} reveals that 2/3 of the basin is filled up during 57 % of the complete filling period, whereas 43 % of the time is required for filling up the remaining 1/3 of the storage capacity. The scale model tests manifested that, in case of appropriate location and shaping of the structure, value of the average contraction coefficient varies between 0.83–0.95.

When, during the filling operation, remarkable changes in the river water level, i.e. in the height H, have to be anticipated, a step-by-step iteration is required, viz. by applying the adequate differential equations for arbitrarily selected Δy water level increments in the basin.

5. Calculation of the Emptying Time

For the evaluation of the emptying time the same hydromechanical concept is applicable, though, when assuming a constant river water level, the procedure results in much simpler equations. Two cases have to be differentiated.

5.1 The Flood Plain is Not Overflooded

The outflow from the basin can be anticipated as an overflow of a broad-crested weir during the whole emptying period (*Fig. 5*), nevertheless, at continuously sinking headwater level. Thus the varying discharge is:

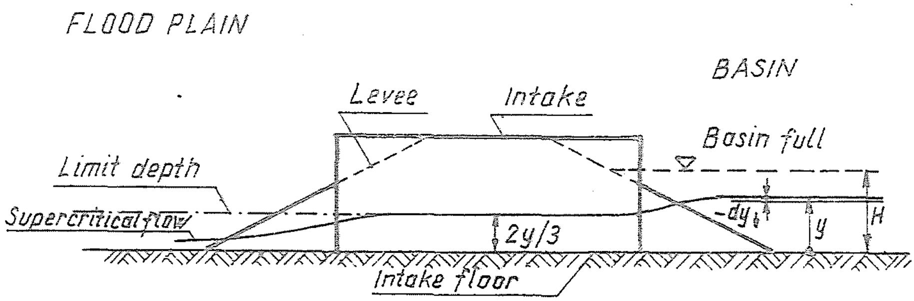

Fig. 5. Water surface profile during the emptying, when the flood plain is not overflooded

$$Q = \mu B \frac{2}{3} y \sqrt{2g\frac{y}{3}} = \mu B \sqrt{g} (2y/3)^{3/2}. \tag{15}$$

Substituting the discharge into the equation $Q\,dt = -A\,dy$ the

$$dt = -\frac{3\sqrt{3}}{2} C y^{-3/2} dy \tag{16}$$

differential equation is obtained, where C is again the project characteristic introduced by *Eq.* (4). Integration between the limits $y = H$ and y gives:

$$t = 3\sqrt{3}C\left(\frac{1}{\sqrt{y}} - \frac{1}{\sqrt{H}}\right) = 5.20\,C\left(\frac{1}{\sqrt{y}} - \frac{1}{\sqrt{H}}\right). \tag{17}$$

It has to be pointed out, however, that, theoretically an *entire emptying* cannot be accomplished, since *Eq.* (17) yields for $y = 0$ evidently

$T = \infty$. Besides, owing to the increasing influence of the friction along the floor a remarkable retardation comes into being during the last part of the emptying. Consequently, *Eq.* (17) must not be used for the lower values of y.

5.2 The Flood Plain is Inundated

If, during the emptying procedure, at the entrance of the structure a constant water depth y_0 prevails, the outflow at a varying but always critical velocity, as in item 5.1, is maintained until the depth in the basin sinks down to $y = 3y_0/2$; thereafter the outflow is affected by the tailwater (water level over the flood plain). The emptying time for the first phase is obviously presented by *Eq.* (17) by substituting $y = 3y_0/2$, i.e. (*Fig. 6*):

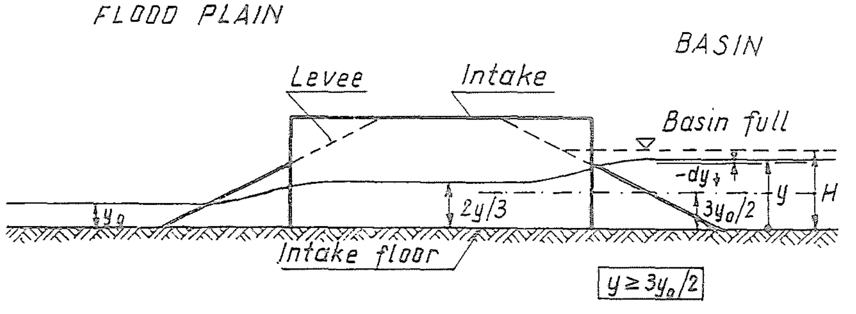

Fig. 6. Water surface profile during the emptying, when the flood plain is overflooded. First phase: the rate of outflow is not affected by the Rhine water level

$$T_I = 3\sqrt{3}C\left(\frac{1}{\sqrt{3y_0/2}} - \frac{1}{\sqrt{H}}\right) = C\left(\frac{4.25}{\sqrt{y_0}} - \frac{5.20}{\sqrt{H}}\right). \tag{18}$$

The emptying time of the second phase is defined by the concept of *Eq.* (7), where instead of the variable y the constant y_0 and in the place of the constant H the changing y have to be substituted. Furthermore, since v has a sinking tendency; dy is replaced by $-dy$. Accordingly (*Fig. 7*)

$$dt = -\frac{A}{\mu B\sqrt{2g}}\frac{dy}{y_0\sqrt{y-y_0}} = -\frac{C}{y_0}\frac{dy}{\sqrt{y-y_0}}, \tag{19}$$

which yields the simple solution between the limits $y = 3y_0/2$ and $y = 0$:

$$t = \frac{2C}{y_0}\left(\sqrt{y_0/2} - \sqrt{y-y_0}\right), \tag{20}$$

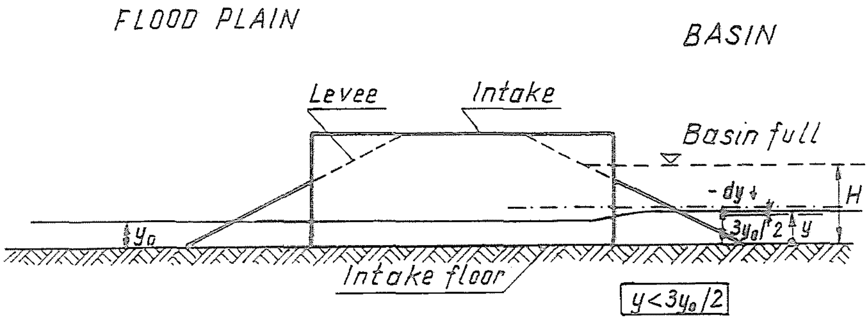

Fig. 7. Water surface profile during the emptying, when the flood plain is overflooded. Second phase: the rate of outflow is retarded by the Rhine water level

respectively between $y = 3y_0/2$ and y_0

$$T_{II} = C\sqrt{\frac{2}{y_0}}. \tag{21}$$

Thus the *entire emptying* time totals to

$$T = T_I + T_{II} = 3\sqrt{3}C\left[\sqrt{\frac{2}{3y_0}} - \frac{1}{\sqrt{H}}\right] + C\sqrt{\frac{2}{y_0}} = C\left[\frac{5.66}{\sqrt{y_0}} - \frac{5.20}{\sqrt{H}}\right]. \tag{22}$$

When a changing water level over the flood plain has to be considered the computation can be accomplished by a step-by-step iteration as it has been indicated in Chapter 4. It should be mentioned here again that the overall discharge loss coefficient μ varies and, according to local conditions and features of the design may considerably differ from the values prevailing at filling.

6. Determination of the Net Width of the Intake Structure

It should be borne in mind that the planning task can be conceived in a way that the allowable, maximum filling and/or emptying times are prescribed, and the necessary minimum free width of the structure has to be determined. Since, evidently in all phases, the times are inversely proportional to the free (net) width B of the intake, instead of C, a factor related to a unit-width intake

$$C^* = \frac{A}{\mu\sqrt{2g}} \tag{23}$$

can be replaced in every formula, and, consequently, instead of the times T their specific values T^* (sm) can be obtained. When denoting the permissible filling/emptying time by T_0, the required net width is

$$B = \frac{T^*}{T_0}. \tag{24}$$

A final remark has to be made to the calculations: it is obviously an approximation that the area of the basin is considered as being independent of the height. In most cases this assumption is practically acceptable. If not, the increase of the area based on topographical surveys can easily be taken into account in a step-by-step computation.

7. Computation with Varying Loss Coefficients

If results derived from model tests and/or from prototype investigations are available, the μ coefficients can be evaluated by comparing these discharges and times respectively with the values obtained from the corresponding equations when substituting $\mu = 1$.

In the case that the overall loss coefficient, at least with a satisfactory approximation, can be expressed in function of the water depth, i.e. $\mu = f(y)$, the partial or total filling/emptying times can be gained by solving the

$$t = \int \pm \frac{A \quad dy}{f(y) \cdot Q'(y)} \tag{25}$$

differential equation, where Q' indicates the discharge without losses.

Though several useful relationships concerning the loss and contraction coefficients can be found in the literature, at least for idealized conditions, an improved evaluation or a reliable mathematical formulation of the overall loss coefficient for the case in question still need further systematical model studies and, as far as possible, in situ observations, too.

By means of an appropriate computer program, naturally, every expression of *Eq.* (25) can easily be solved, even for the most sophisticated assumptions as regards $\mu = f(y)$.

8. General Arrangement of an Intake Structure

Several basin sites and various alternative locations of their intakes have been studied and a few general layouts, though differing from each other only in some details have been elaborated. Nevertheless, a display of these alternatives should be dispensed with here, since these details are obviously of no importance, when bearing in mind that this report aims the explanation of the main hydraulic and structural aspects of the schemes only. Therefore, it seems to be appropriate to present here a prototype arrangement which, with more or less adaption to the specific local conditions, can satisfy all requirements listed in Chapter 3. The design is based on calculations and scale model tests and deserves a concise description (*Fig. 8*).

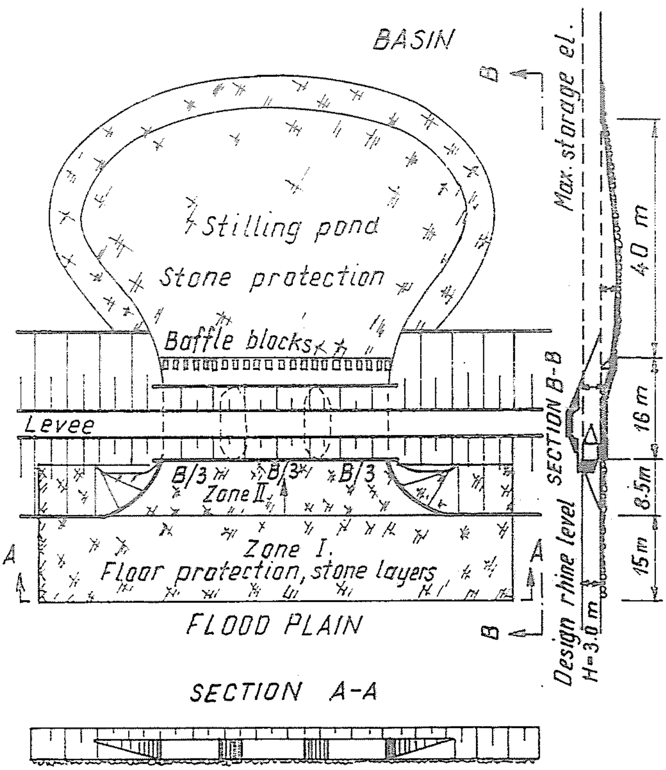

Fig. 8. Prototype design for intake structures

The so-called 'design head', i.e. the max. flood height above the intake floor had been assumed as $H = 3$ m for dimensioning the structure and for studying the filling/emptying operations. This equals, evidently, to the maximum possible storage depth in the basin.

General designs have been elaborated, however, in a sketchy form only, viz. for intakes comprising two or three gated openings of 10 m free width each.

In order to avoid scouring a two-zone protection within an area marked out on basis of the model tests in front of the intake, i.e. on the river plain, is recommended. (Stone layers placed over sand/gravel filter, average stone diameter 40 cm in zone I and 80 cm in zone II). The adjoining portions of the levee slopes have to be protected, too.

The intake structure is a multi-opening gated sluice. Its roof is a continuation of the levee crest. Owing to the two-direction flow the pier heads have similar shaping at their both ends. Sheet piles at both ends of the floor control seepage; their depth, evidently, depends on local geological conditions. The tainter gates must be operated against pressures and flow from both directions. At the basin side, i.e. at the end of the sloping bottom slab a row of baffle blocks of 1.5 m height serves the dissipation of the greatest part of the kinetic energy.

The adjoining stilling pond of 2.5 m depth has to meet a twofold demand: on the one hand, it has to contribute to a further quieting of the inflowing water masses and, on the other hand, it has to distribute the water so that the overtopping of the pond bank and the spreading out into the basin occurs fairly uniformly along its long periphery. The pond and a stripe along its bank has to be protected by stone layers and rip-rap, respectively.

It must not be left out of consideration that seepage and consequently erosion must not come into existence between the outer walls of the structure and the earth bodies of the adjoining levees. The measures against lateral seepage, however, will be selected according to local specific conditions (compacting, application of seals along the walls, etc). In order to maintain a high-grade safety against levee break utmost attention has to be given to the execution of these details.

Since it can be anticipated that the basins will be very irregularly and at long intervals used only, the detailed designs, further the directives for the maintenance and surveillance have to be elaborated in such a way that readiness for instant gate operation will be secured.

The area of the 8 flood-plain basins selected for possible implementation varies between 1 and 5 hectares (1 to 5 million m^2).

9. Scale Model Studies

It was not desirable at all to simulate the retention basin, and, accordingly, fairly great models of the structures had been built in the great hall of the Theodor-Rehbock Hydraulic Laboratory, viz. a scale of 1:25 had been selected. Evidently, the tests were performed on the basis of the similarity law of Froude. In a flat basin, on the one side of the structure, the flow from the Rhine over the flood plain towards the intake was simulated. The basin was supplied from and drained by a circumferential flume, viz. through perforated walls surrounding its periphery. By various partial blockages of this wall it had been rendered possible to simulate the boundary conditions. The water level in the trough, joining the structure on the other side and representing a small portion of the retention basin, was regulated by tainter gates. This trough was filled up with fine-grain gravel for rendering possible tests on scouring.

The studies included the following measurements, observations and evaluation:

- Filling and emptying discharges have been measured under steady-flow conditions, i.e. at constant river and basin water elevations. From discharge data obtained from measurements carried out at several combinations of the water levels the filling/emptying times had been calculated.
- A comparison of the measured discharges with the computed ones (by means of the equations derived in Chapters 5 and 6) yielded the overall (representative) loss coefficients, which enabled the planning of other basins by a theoretical approach, model studies dispensed with.
- Experiments for a favourable shaping of the pier heads and the juncture between abutments and the levee slopes, for enhancement of the discharge capacity (i.e. for maximizing μ). Consequently, eddy zones and vortices had to be avoided.
- Velocity pattern in front of the intake and, especially, the velocities measured next the bottom indicated the necessity of protection measures and, accordingly, two zones of protection by heavy stone layers have been recommended.
- Experiments in order to find a simple and a fairly inexpensive solution of energy dissipation, which finally resulted in a proposal for application of a series of baffle blocks and a stilling pool adjoining the sloping floor of the structure. It was also important to obtain an arrangement which, on the one hand, provides for an efficient energy dissipation at higher heads, though, on the other hand, does not exert a backwater effect and, consequently, a retardation of flow during the first phase of filling, i.e. at low basin stages.
- Inflow into and outflow from the stilling pool and the basin respectively, in order to determine the necessary area and depth of the stilling pool, and the required protection measures.
- A fairly uniform overtopping of the crest of the stilling basin and a proper spreading of the flux toward the basin, especially at low basin stages, has been endeavoured.

9. Results

Theoretical and model studies yielded a few individual designs complying with local conditions and a prototype design.

Studies, discussions, field investigations and inspections, and last but not least, a permanent and close co-operation of experts representing various disciplines, revealed the implication of the environmental aspects and rendered possible to set up further criteria for the options and priorities from among the various alternatives.

Results of these studies can be, at least partly, adapted to the planning and design of other detention basins which will be very probably required for increasing the safety against floods along the Upper Rhine.

During accomplishment of these studies and during the discussions with experts of the competent authorities it has been clarified what type of hydrological investigations were required to elaborate properly defined directives for a safe and efficient operation of the detention basins.